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Performance of Protective Perimeter Walls SubJECTED TO Explosions in Reducing the Blast Resultants on Buildings

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A thesis submitted in partial fulfillment of the requirements for the degree in Doctor of Philosophy

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PERFORMANCE OF PROTECTIVE PERIMETER WALLS SUBJECT TO EXPLOSIONS IN REDUCING THE BLAST RESULTANTS ON BUILDINGS

(Thesis format: Integrated Article)

by

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Graduate Program in Engineering Science
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A thesis submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy

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Abstract

The existing methods to predict the blast effects on structures located behind blast walls are based on plane rigid walls and the results of small-scale studies, which are not validated by the results of full-scale experiments. Hence, they are only valid for a very limited range of scaled parameters. Also, they do not account for the impact of non-spherical charges or for different ignition points; they also put their emphasis on mid-and far-field overpressures.

The current thesis investigates blast wave propagations created by cuboid charges and their effects on structural members at close-in and nearby ranges. First, a preliminary numerical study using the ProSAir finite element program is performed to investigate the effectiveness of a plane and canopied rigid blast wall, which is subjected to a close-in explosion. It was found that, when a canopy was used with the wall, the wall was more effective in reducing the blast wave resultants on the building located behind the wall.

As a part of this study, a High Speed Data Acquisition System was developed along with an in-house LabVIEW program to test the response of reinforced concrete and reinforced masonry walls, which were subjected to blast loading, and to measure the blast wave parameters.

Half-scale blast experiments are conducted to investigate the effectiveness of reinforced concrete perimeter walls of different shapes in reducing the blast wave resultants along the height of a target building located behind a blast wall a wall as well as to determine the maximum damage that the wall could suffer without fragmentation. The effectiveness of reinforced concrete and walls coupled with a reduction in the street elevation adjacent to a wall’s perimeter in reducing the blast wave resultants along the height of a target building located behind the wall was also investigated. It was found that changing the shape of the RC walls or erection of a plane blast wall combined with the lowering of adjacent street elevation could markedly reduce the blast wave resultants on structures located behind the walls.

Full-scale blast experiments were also conducted to investigate the effectiveness of reinforced canopied walls as well as the effectiveness of double fully-grouted reinforced masonry walls infilled with polyurethane foam. The walls were tested both with and without aluminum foam retrofitting and tested using both close-in and nearby explosions. In addition,
the reduction level of the blast wave resultants along the height of a target building located behind these walls was investigated. It was found that in addition to adding a canopy to the top of a reinforced concrete wall, or using a double reinforced masonry wall infilled with polyurethane foam, and retrofitting the walls with aluminum foam reduced the blast wave resultants on structures located behind these walls.
Keywords: Reinforced concrete wall, fully grouted double reinforced masonry wall, Canopy wall, Full-Scale blast tests, Half-Scale blast tests, Aluminum foam, and Data Acquisition system.
Co-Authorship Statement

This thesis has been prepared and written in accordance to the rules for a sandwich thesis format required by the faculty of graduate studies at Western University. All the analytical and the experimental work were performed by Fada Alsubaei. Chapter 2 and 3 of this thesis were published in conferences. Chapters 4-6 of this thesis will be submitted to scholarly journals as manuscripts co-authored by Fada Alsubaei and Ashraf El Damatty.
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<tr>
<td>A</td>
<td>area</td>
</tr>
<tr>
<td>$A_S$</td>
<td>area of reinforcement</td>
</tr>
<tr>
<td>$A_P$</td>
<td>adjustment factor for peak pressure</td>
</tr>
<tr>
<td>$A_I$</td>
<td>Adjustment factor for impulse</td>
</tr>
<tr>
<td>$A_v$</td>
<td>total area of stirrups</td>
</tr>
<tr>
<td>ANFO</td>
<td>Ammonium nitrate and diesel fuel oil</td>
</tr>
<tr>
<td>a</td>
<td>acceleration</td>
</tr>
<tr>
<td>B1</td>
<td>Target building one</td>
</tr>
<tr>
<td>B2</td>
<td>Target building two</td>
</tr>
<tr>
<td>B3</td>
<td>Target building three</td>
</tr>
<tr>
<td>b</td>
<td>Width of wall</td>
</tr>
<tr>
<td>CF</td>
<td>Charge factor</td>
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<tr>
<td>D</td>
<td>Standoff distance from the blast wall to the target building</td>
</tr>
<tr>
<td>$d_c$</td>
<td>Distance between the centroids of the compression and tension reinforcement</td>
</tr>
<tr>
<td>E</td>
<td>Modulus of elasticity</td>
</tr>
<tr>
<td>$f_{cu}$</td>
<td>Static ultimate compressive strength of concrete at 28 days</td>
</tr>
<tr>
<td>$f_{dc}$</td>
<td>Dynamic design stress for concrete</td>
</tr>
<tr>
<td>$f_{ds}$</td>
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Dynamic ultimate stress of reinforcement

Dynamic yield stress of reinforcement

Static ultimate stress of reinforcement

Static yield stress of reinforcement

Height of the target building

Height of burst

Blast wall height

Positive Impulse

Moment of inertia

Equivalent elastic unit stiffness

Load-mass factor

Street elevation

Unit mass

Point on the target building

Peak pressure

Positive Reflected pressure

Incident pressure

Ultimate resistance
R  Standoff distance

RMW  Reinforced masonry concrete

S  Scale factor

SGF  Strain gage on the longitudinal reinforcement on the front side of the wall

SGB  Strain gage on the longitudinal reinforcement on the back side of the wall

t_d  Duration of positive phase of blast pressure

t_m  Time at which maximum deflection occurs

\nu_c  Ultimate shear stress permitted in an unreinforced concrete web

\nu_c  Ultimate shear stress

V  volume

v  velocity

W  Charge weight

X  dimension

X_m  Maximum deflection

X_E  Equivalent elastic deflection

\theta  Angle of incidence

\delta_{max}  Maximum member displacement

\delta_y  Yield displacement
\( \mu \)  Ductility ratio

\( \rho_s \)  Reinforcement ratio

\( \sigma_{p1} \)  Plateau stress of aluminum foam

\( \varepsilon_D \)  Densification strain of aluminum foam
Chapter 1

1.1 Introduction

In recent years, explosive loads have received considerable attention due to a variety of accidental or intentional events that have adversely affected important structures and their occupants all over the world. Conventional structures normally are not designed to resist blast loads. And, because the magnitude of design loads is significantly smaller than those produced by most explosions, conventional structures are susceptible to damage from such attacks. As a result, the design of structures, with respect to their ability to withstand shock and blast wave impacts, has received increased attention. The explosive threats can be detonated within close range of a target, producing high pressure loads and flying debris that can cause severe damage to personnel and structures. One of the most significant attacks was that which took place in Saudi Arabia in the city of Al-Khobar in June 1996 (Crowder et al., 2004). The bomb caused an extensive damage to one of the building towers.

Hence, the mechanics of an explosion must be fully understood before accurate explosion data can be gathered and analyzed and the nature of impacts predicted. A detonation is the sudden release of energy due to a chemical reaction on a large scale. This energy increases the temperature of the surrounding air up to 3000 °C and then creates high pressure gas shock waves that radiate with initial pressures of around $3 \times 10^7 \text{ Pa}$ at the source. These waves radiate from the origin of the explosion at speeds of around 7000 m/s (American Society of Civil Engineers, 1997). A blast phenomenon takes place over a very short period (milliseconds). For a ground-level explosive device, the pressure wave will be instantly reflected and strengthened by the ground surface and will travel away from the source in the form of a hemispherical wave front if there are no obstructions in its path. When the blast wave hits a structure, it will be reflected and magnified by the structure, according to the angle of incidence between the direction of the wave's movement and the type of surface of the structure. The peak overpressure (the pressure above normal atmospheric pressure) and the duration of the overpressure vary according to the distance from the explosive device. The magnitude of these parameters also
depends on the type of explosive materials from which the bomb is made; the size of the bomb is usually given in terms of an equivalent weight in Trinitrotoluene (TNT).

The response of concrete structures to blast loading depends on the nature of the blast, the geometric and the dynamic structural characteristics involved, and the material’s dynamic response characteristics. The loading regime is characterized by using the Hopkinson’s standoff distance, \( Z \), defined as the ratio of the standoff distance to the cube root of the charge weight, and is classified as close-in, near field, or far field, depending on the value of \( Z \), according to Smith et al. (2009). In the far-field design range, blast-generated pressure loads can be considered uniform with respect to the structure and a basic single degree of freedom approximate analysis is often implemented. In the close-in design range, however, blast pressures are non-uniform and the pressure magnitudes can be very high (U.S. Department of Defense [DoD] 2008). A close-in detonation is often considered to exist when the scaled distance is less than 1.2 m/kg\(^{1/3}\). Following the detonation of a high explosive at a short scaled distance from a concrete wall, a shock wave is generated. Part of the shock wave that strikes the wall surface is then transmitted to the concrete, resulting in a compressive wave. When the transmitted shock wave reaches the back surface, it reflects, resulting in a tensile wave. If the tensile stress on the back face of the wall is greater than the dynamic concrete tensile resistance, the concrete will fragment, i.e., spall (DoD, 2008).

Blast loads can be generated and applied to structures by a pressure load using a ConWep computer program or other detonation simulation methods. ConWep is based on a collection of conventional effects calculations as presented in TM5-855-1 (U.S. Department of the Army, 1998); Hyde 2005; United States Army Corps of Engineers). ConWep is a standalone program and is restricted to be used by the U.S. Military but it is also implemented in the LS-DYNA FE program, which is available to researchers. In this method, the blast pressure is applied directly to the front surface of the lagrangian structure. This approach can be used for analysis before the failure of a structural surface. An eroding technique is used in this approach to eliminate the damage elements from the structure to avoid element distortion. The blast load continues to act on the uneroded elements; therefore, the simulation using this method underestimates the damage to the structure because the blast load is not transmitted to the concrete core after the spalling of
the concrete surface. This is a one of the disadvantages of this method. Nevertheless, this method is relatively simple and it requires for the input only the charge weight as the equivalent of TNT weight and its position relative to the structure according to Simoens et al. (2011). As a consequence, the computational cost is less than that incurred in other simulation approaches. Another disadvantage of this method is that it does not account for reflections from the ground to the structure. Hence, the shadowing of the blast wave due to the presence of a blast wall or other adjacent structures cannot be taken into consideration in this approach. Also it does not take into consideration the effect of differences in charge shape or the ignition point, or the orientation of the charge to the structure.

There is another numerical approach that can be used to model an explosion and its effect on a structure, which involves the modeling of the air and the explosive with a multi-material-arbitrary Lagrangian-Eulrian (MM-ALE) formulation in which a proper equation of state is assigned to the materials and a burn model controls the explosive’s detonation behaviour. In this method, the structure is treated as a Lagrangian and Fluid-Structure-Interaction (FSI) according to Hallquist et al. (2012) and is used for communication between Lagrangian and the ALE domains. This method is appropriate where the charge and the structure are at close-range according to Yi et al. (2013). There are several advantages of this method over the pressure load method that is used in the ConWep software: (1) it accounts for the fact that the blast wave load continues to act on the structure after the elements erode from the structural surface; (2) it facilitates the prediction of the reflection and diffraction of the blast wave, and (3) it accounts for the effect of the charge shape and its ignition point. The main disadvantage of this method is the large size of the air blast model that usually needs to be included to mitigate the boundary effects. The large domain increases the computation time.

More and more attention has been recently drawn to the design and retrofit of critical military and civil buildings and to the related infrastructure with respect to protecting them against an explosive attack. Because the events of the past few years have greatly heightened the awareness of structural designers to the threat of terrorists using explosive devices, protecting buildings from the threat of terrorist activities has become one of the
most critical challenges for structural engineers today. The goal here is to develop new and improved methods of protecting critical structures, the related infrastructure, and the surrounding built environment without resorting to the use of massive amounts of military-style reinforced concrete. Moreover, it is important that these designs result in blast-proof systems that are both effective and cost efficient. The conflict for designers of equipment and technologies in this role is to provide an adequate level of protection without creating a sense of people living in a bunker or having people think that their routine or lifestyle is compromised.

The need and requirements for blast resistance in the construction industry have evolved in recent years. The design of blast resistant structures requires knowledge of both blast loadings and the behaviour of structures under these loadings. Due to the difficulties related to the nature of explosions and the severity effects of this kind of threat on structure, full-scale blast tests are required to fully understand the behaviour of structures under this kind of loading. The majority of the previous experimental results are related to explosive charges of a spherical shape. Hence, there is a lack of a sufficient number of test results for other charge shapes. Therefore, there are some difficulties in precisely defining the ignition point and the most likely impact of non-spherical charges on structures.

Blast walls can be constructed to shield a building or other structure from blast waves by providing a reflective surface for the blast, and/or by limiting the size of the load on the structure of interest to some fraction of the predicted reflected load. Blast walls can erect around structures as a first line of defense. They serve as a physical barrier against a forced entry by a vehicle. The walls would work be better, if its anchored to the ground and to each other. They would also provide good protection in a multi-hit environment against both blast and ballistic threats. Unfortunately, the effectiveness of such a wall is highly dependent on the height and thickness of the wall and the distance from the rear of the protective wall to the structure or building to be protected. A protective wall would be required to resist the full reflected blast load. Thus, if overloaded, the wall itself could become a source of significantly hazardous debris.
Optimal blast load mitigation can be achieved by using rigid, non-destructible barriers. Using a barrier’s mass instead of its strength to attenuate blast loads is a lower cost method of achieving the same objective, but this approach generally requires excessive amounts of mass and therefore very thick (and also architecturally unacceptable) barriers. Therefore, it is worth considering the possibility that blast walls could also be relatively lightweight and still offer a certain degree of protection because the high level of deformation of such a wall could absorb a significant amount of the blast wave energy from a threat weapon. Thus, depending on the design of the wall deployed, a combination of energy reflection and energy absorption could mitigate the blast wave loading developed behind a wall. It is important to realize, however, that partial (or complete) wall failure should not produce fragments that could in themselves cause damage to the building asset. However, only a limited amount of information is available on the nature of the blast loading environment behind a blast wall and more accurate methods of predicting the effectiveness of these walls are needed.

Moreover, different wall configurations will alter the flow of a blast over a barrier wall and could affect the size of the load reaching the structure. Moreover, the blast environment behind the wall may be further altered by placing a canopy near the top of the wall. The use of a canopy assumes that the blast wave would be re-directed away from the protected target. While the canopy could be blown away by the force of the explosion, it would remain in place long enough to reduce the magnitude of the blast wave diffracting over the wall. Hence, the effectiveness of the use of a canopy is also investigated in this study.

A perimeter wall is constructed in this study to increase the standoff distance between an explosion and the building to be protected and to reduce the blast resultants at the structure. The cost is very little compared to that of strengthening the building with a protective material. In addition to erecting a blast wall around a building, the street layout can play important role in mitigating the effects of blast loads on buildings. In order to quantify the degree of protection to be provided, the pressures behind a blast wall along the height of the protected building are compared with the pressures obtained in the absence of a blast wall.
Over the years, a great deal of effort has been directed toward the study of the blast resistance of reinforced concrete and masonry walls. However, the present study will focus on the use of more advanced materials and on new reinforced concrete and reinforced masonry wall configurations in order that more effective damage mitigating systems may be developed. An effective means by which the blast effects on a RC wall could be mitigated is to retrofit such walls (barriers) with aluminum foam. In this study, full-scale experimental tests were utilized to investigate the effectiveness of using retrofitting/mitigating systems of aluminum foam cladding to protect RC and RMW walls that were to be subjected to blast loads.

Metallic foams are newly developed as lightweight materials with excellent energy absorption capacity. Due to their excellent energy absorption capacity, metallic foams, for example, have been considered for use as sacrificial claddings to protect structures, or structural components, against blast loads. In the event of an explosion, the incident overpressures can be reduced to a much lower level by adding foam cladding if it is properly designed.

The published literature revealed that the majority of the experimental studies were performed using small-scale models that were subjected to either non uniform, spherical, or cylindrical charges while the experiments conducted for this thesis involved full-scale models and cuboid charge shapes. Moreover, most of the studies focused on solid blast walls while only a few were focused on reinforced concrete plane walls. In this study, the effectiveness of reinforced concrete walls of a T-Shape both with and without the protection of aluminum foam was examined. The effects of adding aluminum foam to improve the resistance of a concrete wall as well as to reduce the blast resultants along the height of the building located behind the wall were investigated in this study.

Most existing masonry walls have little strength to withstand blast loads and the unreinforced masonry (URM) walls have a low resistance against out-of-plane blast loadings due to their low flexural capacity and their brittle mode of failure. The majority of the previous studies focused on a brick of reduced-scale unreinforced masonry walls using numerical analyses of the charges detonated either on or above unpaved ground.
This means that some energy would be absorbed by the ground. In this study, the charges were considered to be detonated above rigid ground and, hence, it was magnified by the reflection off the rigid ground. The goal of this study is to investigate a full-scale double reinforced hollow concrete block masonry walls under blast loads and to examine the use of aluminum foam of different layers as a protective material. The pressure distributions along the height of building located behind the wall were also investigated.

1.2 Statement of Problem

Structural engineers today need guidance on how to design structures to withstand various types of terrorist attacks. A better understanding of what an explosion is and how it can affect the structural performance of a building is necessary for developing effective physical security methods. In this thesis, some of the fundamental concepts of weapon effects and blast loadings on buildings and on their components are outlined. While the issue of blast-hardening structures has been an on-going topic with the military services, the relevant design documents are restricted to official use only. Hence, a very limited body of design documentation currently exists, which can provide engineers with the technical data necessary to design civil structures for enhanced physical security. The professional skills required to provide blast resistant consulting services include: knowledge of structural dynamics, knowledge of the physical properties of explosive detonations and a general knowledge of physical security practices. And the simplified analytical techniques can be used as an engineering tool for obtaining conservative estimates of the blast effects on buildings.

To date, based on the information available in the open literature, most research programs have focused on understanding the effectiveness of plane rigid walls using small-scale models to reduce the blast wave resultants on a protected building located behind them. However, there are fewer studies that have experimentally and numerically investigated the resistance capacity of reinforced concrete and masonry walls to blast loading. Experiments and calculations based on shaped explosions with reflections are relatively rare in literature. Moreover, to the author’s knowledge, limited or no testing has been performed to investigate the influence of RC walls of different shapes or the influence of
street elevation coupled with a blast wall in reducing the blast wave resultants on a target building located behind the wall has not been investigated.

Knowledge of blast characteristics is critical to accurately predicting blast effects especially at close range. There are several basic elements that have a significant impact on pressure and impulse values of a blast wave that is propagated in different directions in air, especially close to an explosion. These elements include such dimensions as: charge shape, ignition point, type of explosive and structural geometry as well as the orientation of the charge and the structure. The majority of the previous experimental results, however, are related to explosive charges of a spherical shape. And there is a lack of a sufficient number of test results for other charge shapes. Therefore, there are some difficulties related to precisely defining the ignition point or the impact of non-spherical charges on structures. In the majority of the previous studies, the charges were placed on unpaved ground, which means that part of the energy associated with the surface burst is lost through the ground and in digging a crater. In the current study, the charges were placed above rigid ground, which resulted in a fully-reflected blast wave that reinforced the blast loading. Thus the full effect of the load was taken into account.

In a blast event, the two parameters that most directly influenced the blast environment are the bomb’s charge weight and the stand-off distance. Blast loading and its effects on a structure are influenced by a number of factors including charge weight ($W$), location of the blast (or stand-off distance), geometrical configuration and orientation of the structure (or direction of the blast). Structural responses will also differ according to the way these factors are combined with each other. Hence, the potential threat of an explosion is random in nature. Therefore, the analysis of bomb attack becomes complex and it is necessary to identify the influence of each factor in relation to the most likely event when assessing the vulnerability of structures. An explosion can be caused by different kinds of explosives with various stand-off distances (distance between target and the blast source). Therefore, scaling laws have been introduced to identify or evaluate the properties of blast waves.

In a design or analysis procedure, normally the dynamic loading (pressure and impulse) is
first determined by these two parameters, and then the response of the structure can be analyzed. In conjunction with the measurement of the pressure and impulse, a widely used way to assess structural damage is to measure the Pressure-Impulse curves corresponding to different damage levels (P-I curves). A P-I curve can be obtained from a simple Single Degree-of-Freedom (SDOF) analysis, complicated numerical simulations, or field blast tests. Recently, a range-to-effect chart (FEMA 428, 2008) was also being used to indicate the distance (or standoff) from which a bomb of a given size will produce a given effect. This type of chart can be used to directly display the blast response of a building component at different levels of protection. TM5-1300 (1990) published by US Departments of the Army, the Navy and the Air force is the best known source in literature offering several methods for establishing blast load parameters. The documentation includes step-by-step analyses and design procedures for structures to protect them from the effects of explosions.

The analysis method in TM5-1300 (1990) can be used to approximately estimate structural responses and damage. But the method is mainly based on the SDOF approach, which is relatively straightforward and also provides an overall assessment of a structure's response to blast loads. However, using the SDOF approach may yield unreliable predictions because it cannot capture the couplings between the shear and flexural failure modes, or the localized failure that occurs within the structure. Moreover, the damage criterion, which is based on the displacement response of the SDOF model, is very difficult to accurately define. The SDOF and an advanced FEM are available for the designers to build the structures subjected to blast loading. However, the challenge lies in using them with a high degree of confidence. The SDOF approach can be used for far range blast loading only (scaled distance $Z \geq 3$) where the blast loading is uniform; this approach cannot be used to determine close-in effects as stated in the ASCE 59-11 standards (2011). To obtain a more accurate prediction of the likely response of structures or of structure members, a more detailed analysis than that provided by the SDOF approach is often necessary.

Some standards (DoD 2002) and design methods (TM5-1300 1990; DoD 2004) were developed to analyze structural responses to blast loads, and to assess structural
performance with respect to blast weight and standoff distance. For example, the DoD (2002) sets minimum standoff distances based on the required level of protection. If these minimum stand-off distances are met, conventional construction techniques can be used with some modifications. However, if the minimum stand-off distances cannot be achieved, the building must be hardened to obtain the required level of protection. The standoff distance given in DoD (2002) provides a quick assessment of structural safety for blast loads. However, the primary issue here is that damage levels are not clearly defined. It is not clear if ‘damage’ means collapse of the structure, or collapse of the infilled walls, or the development of cracks in the infilled walls, or some other type of structural or structural component failure.

A simple approach for protecting important structures against blast loads is to improve the perimeter security by means of blast-proof perimeter walls that act as barriers against blast waves. However, research on blast wave propagation around a blast wall and on the blast environment behind the wall is very limited.

Blast loads in simple geometries can be predicted using empirical or semi-empirical methods, such as those presented in the Manuals (UFC 340-01, and UFC 340-02). These methods can be used to estimate the blast loading on an isolated structure when there is no barrier. However, there is no direct method that can be used to estimate the blast resultants on building façade if there is a blast wall between the explosive center and the structure.

The existing methodologies for determining the effectiveness of blast walls, such as those presented in (TM5-853-3) coupled with UFC 340-02, use an adjustment factor approach (the ratio of pressure or impulse in the case of the presence of a blast wall to the pressure or impulse in the case of where there is no wall) to calculate the reduction in pressure or impulse behind a plane wall. But the TM5-853-3 manual is restricted to be used by only the U.S. military and it is valid over only a very limited range of scaled parameters (wall height, charge-to-wall standoff distance, wall-to-structure-standoff distance) and is based on small-scale models of plane rigid walls, which have never been validated with full-scale experiments. The empirical design charts assume either a spherical (free-air) burst
or hemispherical (surface) burst. The formulations in these manuals are based on a spherical charge being initiated at its center with an emphasis on mid-field and far-field overpressures.

Making accurate blast load predictions, however, is difficult because they do not take into account the variations in charge shape, charge orientation or the point of detonation of the charge. Since the charge shapes selected by the terrorist organizations are unknown and could be non-spherical (e.g., cylindrical, or cuboid) and these shapes could have different orientations and detonation points within the explosives, which will produce different distributions of the overpressures than a spherical charge of the same size—especially in the immediate vicinity of the detonation. Also the manuals do not take into consideration the reflection due to the vicinity of other structures, the street elevation, or the structure’s geometry. The assessment of air blasts from charges in the free field is well understood while the variation in blast resultants along the building façade protected by a blast wall is not fully understood. Alternatively, the experiments or the FE codes such as LS-DYNA can be used to assess the overpressure of the detonations of the non-spherical charges and the reflection due to the geometry of the structure or that of adjacent structures. However, the methodology used in this study is similar to those used in these manuals. The reflected pressures and impulses along the height of the building behind each wall were investigated.

1.3 Research Objectives

The specific objectives of this research are as follows:

1. To develop a high speed data acquisition system along with an in-house written LabVIEW programs to test the response of reinforced concrete and reinforced masonry walls subjected to blast loading as well as to measure the blast wave parameters.

2. To investigate the effectiveness of reinforced concrete walls with different shapes in reducing the blast wave resultants along the height of target building behind the wall as well as to determine the maximum damage that the wall can suffer without fragmentation.
3. To investigate the effectiveness of utilizing reinforced concrete walls coupled with decreasing the street elevation adjacent to the perimeter surrounding the protected building in reducing the blast wave resultants along the height of target building located behind the wall.

4. To investigate the response of the reinforced canopy walls retrofitted both with and without different layers of aluminum foam to a close-in explosion and to study the reduction of the blast wave resultants along the height of the target building located behind the wall.

5. To investigate the response of double fully-grouted reinforced masonry walls both with and without different layers of aluminum foam and infilled with polyurethane foam subjected to nearby explosions and to study the reduction of the blast wave resultants along the height of target building located behind the wall.

6. To review the available literature on the effectiveness of aluminum foam as a protection material and to find the proper mechanical properties that can be used in the current study.

1.4 Scope of Research

The experimental program in this research involves five phases. The first phase was to develop a field blast test site and a high speed data acquisition system along with in-house written LabVIEW programs for data collection and the analysis of structures subjected to blast loading and to measure the blast parameters as well as the response of the structural members subjected to blast loading. To conduct successful blast field tests, it is very important to understand and to select the appropriate measurement devices. It is also necessary to understand the mechanism of the explosive material. Before developing the blast field measurement system, an overview of field blast testing using a high speed data acquisition system used by numerous researchers worldwide was obtained to assist in building the system used in this study. Several experiments were conducted in order to understand the mechanism of the ANFO charges with different sizes.
To check the system, several blast tests were conducted at the Special Security Forces Range, Riyadh, Saudi Arabia using specific charge sizes at different standoff distances. These tests required an examination of many different elements for their successful completion. One element consisted of the pre-blast preparation. This included finding an adequate blast site, installing the specimen, setting up the data acquisition system, setting up the high-speed cameras, and preparing the blast components. Proper pre-blast preparation insured the successful acquisition of the data regarding the air blast pressures and insured that the high-speed cameras captured the blasts. Other responsibilities included post-blast duties such as the clean-up and data analysis. The components of the field blast test and the instruments developed in this study using the high speed data acquisition system, which included the software, signal conditioner, blast wave transducers, accelerometers, strain gages and coaxial cables that were connected to the test specimens and then to the high speed data acquisition system as well as the procedure for field blast tests utilized in this study are herein described.

In phase two, six half-scale walls of different shapes were constructed; the walls measured 2 m wide, 1 m high, and 0.2 thick. A total of 96 shots were carried out with different sizes of ANFO charges. Each wall was subjected to 12 varied successive charges and each charge was placed in a cuboid non-fragmenting wooden container and placed on a wooden table with its height adjusted to position the center of the explosive at the mid height of the specimen. A reinforced concrete slab was constructed in front of the blast wall and a replaceable steel plate with a thickness of 20 mm was positioned on the slab, and laid under the charge to avoid excessive damage to the slab. This setup provided a fully-reflective surface. The experiments were conducted to study the wave propagation and distribution of the blast loadings on protected buildings located behind the walls as well as to determine the maximum damage that the wall could suffer without fragmenting.

In phase three, three half-scale walls at three different street elevations were constructed; the wall measured 2 m wide, 1 m high, and 0.2 thick. The street elevations were 0 m, 0.5 m and 1 m. A total of 48 shots were carried out with different sizes of ANFO charges. Each wall was subjected to 12 varied successive charges and each charge was placed in a
cuboid non-fragmenting wooden container and placed on a wooden table with its height adjusted to position the center of the explosive at the mid height of the specimen. The experiments were conducted to study the wave propagation and distribution of the blast loading on protected buildings located behind the walls as well as to determine the maximum damage that the wall could suffer without fragmenting.

In phase four, full-scale walls with canopies at their top both with and without aluminum foam protection were constructed. The wall measured 4 m wide, 2 m high, and 0.4 thick. A total of 5 shots were carried out with ANFO charges. Each wall was subjected to a single charge and each charge was placed in a cuboid non-fragmenting wooden container and placed on wooden table with its height adjusted to position the center of the explosive at the mid height of the specimen. The goal here was to investigate the response of a reinforced concrete wall with a canopy at the top both with and without the protection of different layers of the aluminum foam and subjected to close-in explosions generated by ANFO charges and to study the reduction of the blast wave along the height of the target building located behind the blast wall.

In phase five, full-scale double fully-grouted reinforced masonry walls both with and without aluminum foam protection were constructed; the wall measured 4 m wide, 2 m high, and 0.4 thick. A total of 4 shots were carried out with different sizes of ANFO charges. Each wall was subjected to a single charge and each charge was placed in a cuboid non-fragmenting wooden container and placed on a wooden table with its height adjusted to position the center of the explosive at the mid height of the specimen to study the response of double fully-grouted reinforced masonry walls both with and without the protection of different layers of aluminum foam and utilizing polyurethane foam to infill the cavities in the walls. The walls were subjected to nearby explosions generated by ANFO charges. The reduction of the blast wave along the height of the target building behind the blast wall was studied.

1.5 Organization of Thesis

This thesis consists of eight chapters and each chapter addresses one particular topic of the study as presented hereafter in an “Integrated-Article Format” following the
guidelines described in Western University-School of Postdoctoral and Graduate Studies (SPGS) General Thesis Regulations. A summary of each chapter is described as follows.

Chapter 1 contains an introduction, statement of the problem, the research objectives and the scope of this research.

Chapter 2 gives a review of the available literature on the effectiveness of the existing approaches to the evaluation of the effect of blast loads on structures as well as on the existing methods to predict the blast effects on structures located behind blast walls. A preliminary numerical study using the Propagation of Shocks in Air (ProSAir) program was conducted to investigate the effectiveness of plane and canopied rigid blast walls subjected to a close-in explosion.

Chapter 3 presents a comprehensive review of the efficiency of aluminum foam sandwich composites when used as a structural protection layer against blast wave loads and highlights those areas where research is lacking.

Chapter 4 discusses the development of the field blast testing site and the development of the high speed data acquisition system and the in-house written LabVIEW programs for data collection and analysis as well as the proper techniques for mounting the sensors, which could measure the response of the structures subjected to blast loading and the blast wave parameters. The components of the field blast tests and of the instruments developed in this study using a high speed data acquisition system that includes the software; the signal conditioner; the blast wave transducers, and the accelerometers as well as the strain gages and coaxial cables, which are connected to the test specimens. The high speed data camera and the procedure for field blast tests utilized in this study are also described. The experimental results of the blast resultants when there is no wall in front of the target building are compared with the A.T.BLAST program to check the results. Several experiments were conducted to examine the mechanisms of the ANFO charges of different sizes.

Chapter 5 presents the field blast testing using half-scale models of reinforced concrete walls of different cross sectional shapes subjected to shaped close-in explosions of a
variety of sizes to study the wave propagation and distribution of the blast loading on protected buildings located behind the walls as well as to determine the maximum damage that the wall could suffer without the production of fragments. The efficiency of the reinforced blast walls in reducing the reflected pressures and impulses is demonstrated by utilizing the ratio (reduction factor) of the measured reflected pressures, or impulses to the case where there is no wall between the charge and the target building.

In the case where there is no blast wall between the charge and the target building, the experimental results of the reflected pressure and impulses are compared with the numerical values obtained by the A.T.BLAST program. As a base line for this study, a plane RC blast wall designed based on a UFC-340-02 and it was designed structure to resist the blast load at a scaled distance of 0.93 m/kg$^{1/3}$. The safe scaled distance used as one of the parameters in some regulations (U.S. DoD, 2004 and ASCE, 1997) in assessing structural safety with respect to resisting blast loads and the values given in these regulations are for conventional structures. The safe scaled distances used in the current study are compared with the values given in these regulations. Also, the damage level for the RC blast walls utilized in the current study is compared to the qualitative damage indicator provided in ASCE/SEI 59-11 (2011).

Chapter 6 presents the field blast testing of half-scale models of a combination of erected plane reinforced concrete walls and a decreasing street elevation adjacent to the wall surrounding the protected building, which are subjected to a variety of sizes of shaped close-in explosions to study the wave propagation and the distribution of blast loadings on the protected building located behind the wall. Field blast testing was also used to determine the maximum damage that a wall can suffer without fragmentation. The efficiency of the reinforced blast walls in reducing the reflected pressures and impulses is demonstrated by utilizing the ratio (reduction factor) of the measured reflected pressures, or impulse, where there is no wall between the charge and the target building. Where there is no blast wall between the charge and the target building, the experimental results of the reflected pressures and impulses are compared with the numerical values obtained by the A.T.BLAST program. As a base line for this study, a plane RC blast wall is designed based on UFC-340-02: Structure to resist the blast load at scaled distance 0.93
m/kg$^{1/3}$. The safe scaled distance provided there is used as one of the parameters in some regulations (U.S. DoD, 2004 and ASCE, 1997) in assessing a building’s structural safety to resist blast loads; the values provided in these regulations are for conventional structures and the scaled distances in the current study are compared with those given in these regulations. The damage level for the RC blast walls utilized in the current study is compared with the qualitative damage indicator provided in ASCE/SEI 59-11 (2011).

Chapter 7 presents the field blast testing of full-scale models to investigate the response of a reinforced concrete wall with a canopy at the top as well as both with and without the protection of different layers of aluminum foam and subjected to a close-in explosion generated by ANFO charges. The level of damages for each wall is visually observed and compared qualitatively with the damage classification in accordance to the ASCE 59-11 (ASCE 2011) standard as well as using the performance (rotation) limits; the damage level to each wall in the current study is classified in accordance with the code. In addition, the safe scaled distance used in this study is the same as that used in some regulations such as (US DoD, 2004 and ASCE, 1997) as one parameter in assessing the ability of a building to resist an airblast load. The safe scaled distance in the current study was compared with those specified in the regulations and in the previous studies. Moreover, the reduction of the blast wave along the height of the target building located behind the blast wall is investigated. The blast wave resultants along the height of the building in the absence of a wall were measured and the results were compared with those given in the A.T.BLAST program to check the experimental results.

Chapter 8 presents the field blast testing of full-scale models to investigate the response of double, fully-grouted reinforced masonry walls both with and without the protection of different layers of the aluminum foam and utilizing polyurethane foam to infill the cavities of the walls, which are subjected to close-in explosions generated by an ANFO charge. The level of damage to each wall was visually observed and compared qualitatively to the damage classification given in the ASCE 59-11 (ASCE 2011) standards. Also the performance (rotation) limits of each wall in the current study were classified according to the damage levels provided in the code. In addition, the safe scaled distance used in some regulations such as (US DoD, 2004 and ASCE, 1997) was
used as a parameter in assessing the structural safety to resist airblast loads. The safe scaled distance in the current study was compared with those specified by the regulations and in the previous studies. Moreover, the reduction in the size of the blast wave created along the height of the target building located behind the blast wall was investigated.
1.6 References


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Chapter 2

2 Review of Literature - Effectiveness of Blast Walls Designed for Blast Shielding

2.1 Introduction

A blast barrier wall is a perimeter wall that is placed around a building, facility, or compound. The wall mitigates the level of the blast loading that propagates to the asset that is to be protected. It also provides an effective means of forcing a standoff from any blast threat because it ensures that any vehicles containing explosives cannot get any closer to the asset than the perimeter blast wall. In the event of an explosion, a high level of energy from the explosive is transferred through the air in the form of a high-density shock wave. This shock wave propagates through the atmosphere shocking the air with which it comes in contact along the way. When the shock wave impacts a rigid surface, the band of high-density air that defines the shock wave is compressed against the rigid surface as it is forced to reflect off the surface. This reflection creates the peak reflected pressure load on the structure. Blast loads have a peak pressure that decays exponentially over very short time duration (Baker, 1973) compared to that of common dynamic loading events such as wind loads. Blast pressure has a very high load, and is applied to a structure in a matter of only milliseconds. As the blast wave reflects away from the structure, it generates a vacuum that causes the air pressure to drop below that of the ambient air pressure. This is called the negative phase of a blast. The area under the pressure–time history curve is called the impulse and is a measure of the energy imparted to the structural components. These terms are illustrated in Figure 2.1.
The erection of blast barrier walls around the perimeter of a facility changes the way that a blast load impacts a structure. As illustrated in Figure 2.1, the initial shock wave is reflected off the barrier wall. It is evident that blast wave propagation is not a linear vector motion. The high energy from the explosion compresses the air, which then expands in every direction possible. This means that the blast wave propagates to the top of the blast wall and that a small vortex is created at the top of the blast wall similar to fluid flows. The air then expands in every direction once it is freed of the top of the wall. Referring to point A in Figure 2.1, this point receives a blast pressure from the incident wave propagating in a straight line that runs from the top of the barrier as well as a second peak that originates from the wave that is propagated back to the ground and reflected back to the structure. The strength of this process is compounded when a structure is present. Depending upon the configuration, if a structure is very close to a blast wall, the number of spikes in pressure can be two, three or more numerous. The blast wave created will be reflected off the front of the building back to the blast wall; then it will be reflected by the back surface of the wall and propagated again toward the building causing several pulses. As the barrier-to-structure standoff is increased, the structural loads begin to more closely resemble the loads in a free-field blast propagation configuration. This pattern of wave interaction introduces a nonlinear problem that requires enhanced tools for analysis.

Figure 2.1: Typical Free-Field Blast Loading on Structure
2.2 The Effectiveness of Blast Walls

There are several factors to be considered when examining the effectiveness of the blast walls. For example, the positioning of the blast wall relative to both the likely threat location and the asset being protected is important in order to ensure that the destructive effects of the blast behind the wall are effectively mitigated. Behind the wall, a significant reduction in blast wave resultants (as compared to those, which would be obtained in the absence of a wall) will be observed in a region extending up to five or six wall heights behind the wall and over a vertical distance of three wall heights above the ground Rose et al. (1995). If the asset to be protected must be close behind the wall (because of the physical constraints of the asset’s location, for example) this zone offers the best blast resultant mitigation. Of course, if the location allows for a greater distance between the wall and the asset, advantage should be taken of this increased standoff and the attenuation reduction in blast resultants.

Another factor to be considered is that different wall configurations will alter the flow of the blast over the barrier wall and can affect the size of the load reaching the structure. The blast environment behind the wall may be further altered by placing a canopy near the top of the wall. The use of a canopy assumes that the blast wave would be re-directed away from the protected target asset. While the canopy could be blown away by the force
of the explosion, it would remain in place long enough to reduce the magnitude of the blast wave diffracting over the wall. Hence, the effectiveness of the use of a canopy will be considered in this paper.

### 2.3 Approaches to the Evaluation of the Effect of Blast Loads on Structures

There are three possible approaches to the evaluation of the effect of blast loads on structures. Experimentation is the first approach and is the quickest way of obtaining accurate information concerning any given blast loading situation. However, there are problems associated with explosive experimentation and shock wave measurement due to the danger to the equipment from unforeseen loads/fragments and the lack of repeatability associated with small scale events of short duration. The experiments are also expensive to conduct and are both time and labour intensive. These problems are, however, compensated for by the fact that shock waves retain their characteristic shape and behavior at virtually any practically achievable scale. These facts mean that experimentation especially that at full-scale is still the primary method of advancing knowledge of the subject area.

The second approach is to use the results from fundamental or generic experiments combined with analytical methods to develop new prediction techniques, which form either a subset of the original data (empirical methods), or extend the applicability of the data by extrapolation or manipulation in justifiable ways (semi-empirical methods). However, most of the empirical approaches are limited by the extent of the underlying experimental database. The semi-empirical approaches are essentially correlations with experimental data. These basic techniques are very useful for preliminary designs and in addressing other basic problems. These methods are based on empirical data from explosions in free air, however. Hence, they cannot represent reflections from and interactions with structures. Simplified load prediction techniques that use available equations and curves are useful for resolving relatively large-standoff problems, but they typically do not provide good results for small-standoff problems because the accuracy of the empirical data decreases at close ranges. The data used to develop the chart and equation-based load prediction methods are from actual field measurements taken during
blast testing. Data from very close-in blasts have been, and still remain, very difficult to measure due to the extremely high temperatures and pressures, which are created, in the immediate vicinity of a blast from a high explosive. Therefore, the chart- and equation-based procedures may not be adequate to predict blast loads for close-in blasts.

The third approach is the use of high-fidelity modeling, which is accurate and serves as the more-commonly employed approach. The drawback of such computational modeling is that it requires a highly trained professional and it can take a long time to analyze varying configurations of barrier height and standoffs. The numerical (or first-principle) methods are based on mathematical equations, which describe the basic laws of physics governing a problem. These principles include the conservation of mass, momentum, and energy. In addition, the physical behaviour of materials is described by constitutive relationships. These models are commonly termed computational fluid dynamic (CFD) models. The simulation approach assumes that there is a skilled engineer using internally developed, commercial, or government-owned software that is based on first-principle physics. Popular software commonly used in this approach includes LS-DYNA (2006), AUTODYN (Century Dynamics, 2003), Air3D (Rose T. A., 2003), and ProSAir (2012).

The downside to this approach is the logistics of performing such simulations. If an evaluation of an existing facility is required, then there is only one simulation to run and this might be a good approach for an experienced computational modeller. However, if there are multiple cases to be considered (i.e., differing standoffs, charge sizes, and blast wall heights) then a modeling approach becomes very complicated. Computational models can require many processors, up to several thousand depending on the computational domain size and software chosen. The simulations can take hours, days, or weeks to run to completion. If there are large problems that have to be compiled on a large-scale supercomputer, queue times can drive the simulation time out even further. Studies of the ability to use a much coarser mesh for computational assessments performed on computers without large memory or processor capabilities have shown errors of as much as 50% (Lohner, Baum, and Rice, 2004). Accurate results are attainable, but require a skilled modeller and time for the simulations to be compiled and post-processed.
2.4 Prediction of Airblast Loads on Single Structures

The blast loads on the vertical exterior wall of a building are calculated based on the input equivalent TNT charge weight \( W \), the charge location relative to the building, and the assumption of a relevant blast wave propagation model. There are generally two blast environments that could be considered in this situation: (1) a spherical air blast; and (2) a hemispherical surface blast. Nearly all exterior bomb threats on architectural targets can be modelled using the surface burst model. In this model, a charge is located either on or very near to the ground surface. The wave of the explosion is reflected from the ground and reinforces the energy of the blast wave propagating it through the air. If the ground were to provide a perfectly rigid surface, approximately half of the bomb energy would be reflected from the ground effectively doubling the blast wave intensity. Since the ground is not a perfect reflector, however, some of the energy (about 20%) is lost in forming a crater and producing the ground shock.

Many empirical formulae for predicting peak pressures in the air are available in the literature. Most of them were obtained from the field blast tests of UFC 3-340-01 (2002). Since then a number of analytical methods for predicting blast loading have been developed. These analytical procedures are presented in several technical design manuals and reports. The UFC 3-340-02 (2008) manual is one of the most widely used publications available to both military and civilian sectors for designing structures to provide protection against the blast effects of an explosion. The design curves presented in the manual give the blast wave parameters as a function of scaled distance for three burst environments: free air burst; air burst; and surface burst. The UFC 3-340-02 (2008) shows how to calculate the enhancement of pressure in the front of and the reduction of pressure to the side and rear of a structure. A safety factor of 20% should be applied to the charge weight before calculating the blast wave parameters.

For points with zero angle of incidence, Figure 2-15 (UFC 3-340-02, 2008) is used for calculating the shock wave parameters in the case of a surface burst of a hemispherical TNT charge while Fig. 2-17 is used for the free-air burst spherical charge and the point of detonation is assumed to be the middle of the sphere. For the points where the angle of
incidence is greater than zero, Figures 2-193 and 2-194 are used and these curves provide the reflected pressures as a function of the incident pressure and the angle of incidence for incident shock strengths up to 34.5 MPa (5000 psi). The above Figures show the blast resultants as a function of scaled distance where the scaled distance \((Z = R/\sqrt{W})\) is a function of the mass of the explosive, \(W\), and the distance between the target and the point of detonation, \(R\). However, it contains no information about the magnitude of the blast pressure if there is an obstacle between the building and the detonation point.

The angle of incidence of a point on a surface is the angle between the outward normal and the direct vector from the explosive charge to that point. It is well known that the angle of incidence is one of the factors that generally affect the blast load on structural components. For a given scaled standoff, \(Z = R/W^{1/3}\), the pressure measured on a large rigid surface with an angle of incidence equal to zero degrees \((\theta = 0)\) is the fully reflected pressure \(P_r\) at that scaled standoff while the pressure measured at a point on a surface that has an angle of incidence of 90 degrees (i.e., it is parallel to the direction of the blast wave propagation) is the incident or side-on pressure \(P_{so}\) at the given scaled standoff distance. The impulse applied to a surface being the integral of the pressure – time history is also affected by the angle of incidence. The strength of the impulse is generally increased from its free-field value if the angle of incidence is less than 90 degrees.

If the angle of incidence is less than 45 degrees, the use of a fully reflected peak pressure and impulse can be justified by analyzing the reflected pressure – angle of incidence relationship shown in Figure 2-193 in the UFC 3-340-02 (2008) manual. In Figure 2-193, the reflected peak pressure is the product of the side-on pressure and the reflection factor shown on the vertical axis of the figure.

The UFC 3-340-01 (2002) manual provides procedures for the design and analysis of protective structures subjected to the effects of conventional weapons. The manual also provides closed-form equations to generate the predicted airblast pressure – time histories. This manual can also be used to evaluate the effects of blast loadings on multi-storey buildings. Load time histories for buildings and building components located at some height above the ground can be calculated according to the methodology presented
in this manual. One of the limitations of this simplified method lies in its neglecting of the true physics of the blast wave – structure interaction phenomena in that it assumes that the load – time history is applied to all parts of the surface simultaneously. This assumption provides a poor approximation of close-in blast effects.

Blast pressure can be measured by using empirical formulae. CONWEP (Hyde, 1992) is a program that uses the equations developed by Kingery-Bulmash (1984) and the curves found in UFC 3-340-02 (2008) and UFC 3-340-01 (2002), which are based on Kingery-Bulmash (1984) equations to predict the nature of an air blast from a spherical air burst and from hemispherical surface bursts. It is based on explosion tests using charge weights that vary from less than 1 kg to over 400,000 kg. These equations are widely accepted as engineering predictions for determining free-field pressures and loads on structures. However, CONWEP (Hyde, 1992) cannot deal with the problems of complex geometries such as those produced when blast pressures are reflected off several surfaces before reaching a building. These equations can also be found in the UFC 3-340-01 (2002) in graphical form only. Unlike as in the UFC 3-340-01 (2002), where an approximate equivalent triangular pulse is proposed to represent the decay of the incident and reflected pressures, CONWEP takes a more realistic approach, assuming an exponential decay of the pressure with time.

A.T.BLAST (2006) is another program, which can be used to calculate blast loading parameters from an open-air hemispherical explosion based on the distance from a device. The program allows the user to enter the weight of the explosive charge, a reflection angle, minimum and maximum ranges to the charge and a calculation interval. From this information, A.T.BLAST calculates the shock front velocity, the time of arrival, the pressure, the impulse and the equivalent linear load duration.

Wu and Hao (2005) performed a numerical analysis to investigate the variations in the air blast forces along a structure height. Based on the numerical data they developed an empirical formula for surface explosions to determine the pattern of air blast forces that develop along a rigid wall. The normal peak reflected pressure at the bottom of a rigid wall was first derived by measuring the best-fitted relationship between the peak reflected
pressure and the peak free air pressure; then they developed an empirical formula to estimate the pressure distribution along the structural height. They found that if the explosive center was very close to the structure then the pressure along the height of the structure was non uniform.

In the above procedures, it is assumed that there are no obstacles between the explosive device and the target. However, if a blast wall were to be erected to protect either personal or the structures behind it, the damaging effect of the actual blast load would be significantly reduced for some distance behind the blast wall. Procedures to predict the blast loads on the structures behind a protective blast wall are discussed below.

2.5 Prediction of Airblast Loads on Structures behind a Protective Blast Wall

Finding the solution to the prediction of the effect of blast loads on structures is greatly complicated by the presence of a blast barrier wall. In this case, the blast pressures and impulses are reduced due to the greater distance that the shock front must travel to propagate over the height of the blast wall. This configuration presents a non-linear problem. Depending on the specific configuration of the charge weight, $W$, the charge to barrier standoff, $L$, the charge to structure standoff, $D$, and the barrier height, $H$, there are multiple reflections of the blast wave that are introduced to the problem.

For the pressure behind blast walls, there are some unique empirical procedures to be used for the determination of the blast environment behind protective walls. Each has its own region of applicability and validity. The first among these is presented in the paper by Beyer (1986), which provides graphical information for a relatively small number of geometries.

Rose et al. (1995) carried out a programme of research in which detailed measurements of the blast environment were made behind a 1/10th scale plane vertical barrier. Pressure-time histories were measured in a grid of locations behind the wall and the results were presented both graphically and in the form of contour plots of peak pressures and scaled impulses. For the set of parameters studied in that work, it was concluded that
in the region between three and six wall heights behind a blast wall and over a vertical
distance of three wall heights above the ground, pressures and impulses may be reduced
by no more than 60% and 80%, respectively, of those produced without a wall. This work
demonstrated that a robust, plane, non-deforming wall produces a significant mitigation
of pressures and impulses out to about six wall heights behind the wall. For greater
distances, although mitigation does occur, it does so to a lesser extent.

for calculating the reduction in the pressures and impulses that are created behind a blast
wall, but the manual is restricted to official use only and has a very limited availability to
the non-military engineering community. The formulations in the manual are based upon
small-scale tests conducted at the U.S. Army Engineer Waterways Experiment station
(WES) in the 1980s (Dove et al., 1989). These formulations were never validated by full-
scale experiments. A simple methodology is included in the manual to predict the pattern
of the blast forces created along the height of the building behind the blast wall. Two
adjustment factors for peak reflected pressure $A_P$ and for impulse $A_I$ are introduced. They
are defined as the ratio of the peak reflected pressure and the ratio of the positive impulse
on a building surface estimated both with and without a blast wall in front of the building,
respectively. That is, $A_P = \frac{P_{\text{(with barrier)}}}{P_{\text{(no barrier)}}}$ and $A_I = \frac{I_{\text{(with barrier)}}}{I_{\text{(no barrier)}}}$,
where $P_{\text{(no barrier)}}$ and $I_{\text{(no barrier)}}$ are the maximum pressures and
impulses created at the ground level on the building surface in the no barrier case,
respectively. The adjustment factors are obtained from the blast load adjustment curves
appearing in the manual as a function of the parameters charge weight, blast wall height,
distance from the location of the charge to the blast wall, distance from the charge
location to the protected building and the height at which the reflected pressures and
impulses are measured on the building. To use these curves, one first needs to generate
the value of the pressures and impulses that would be expected to be created at the
location of interest if there were no perimeter wall. Typically, the no barrier blast loads
can be estimated using simplified analytical relationships or design charts UFC 3-340-02
(2008) and they may also be evaluated using a computer program such as CONWEP or
A.T.BLAST in which these calculations are automated.
The scaled wall height is calculated as the height of the wall divided by the cube root of the charge weight \( (H_m/W^{1/3}) \) and it is used in one of the few references, which attempts to predict blast wall effectiveness, the TM-5-583-3 (1986), Security Engineering Manual. The test data on which this manual's algorithm is based are in the range of 0.32 m/Kg\(^{1/3}\) and 0.52 m/Kg\(^{1/3}\). Scaled wall heights coming out of the existing data are to be investigated. The majority of the research to date has been performed without the use of structures, meaning that the methods cannot accurately capture the multiple increases in pressure that occur in small barrier-to-structure standoff situations.

The empirical method, which is based on a collection of experimental data, is easy to use. But the accuracy of this method depends on the quality of the test data available. Although analytical methods can perform quick and reliable analyses, it is sometimes not possible to obtain analytical solutions due to the complexity of the problems. However, the finite element method, which is widely used in practical engineering, provides explicit and direct results. The blast wave behaviour can be predicted from first principles using such numerical tools as, AUTODYN, LS-DYNA, ProSAir and others. Such tools solve the governing fluid dynamics equations and can be used to simulate three dimensional blast wave propagation including multiple reflections, and rarefaction. In addition, Computational Fluid Dynamics (CFD) techniques can be used to capture the effects of the blast focusing due to the level of confinement and blast shielding provided by barriers or other buildings. The numerical methods used to simulate the blast effects problem typically are based upon a finite element method with an explicit time integration scheme that allows the engineer to estimate the size of the blast loading.

Zhou and Hao (2008) carried out numerical simulations using AUTODYN to study the effectiveness of blast barriers for blast reduction. They found that the erection of a barrier between an explosion and a building can reduce the peak reflected pressures, and impulses, which are created on the surface of a building, also delay the arrival time of the blast wave. However, the time delay caused by the barrier is longer when the gauge point height, \( H \), is lower. The duration of the positive phase for the with barrier case is longer than that for the no barrier case. Based on the numerical data, they derived an approximate formula to estimate the reflected pressure-time history for a structure located
behind a barrier. Their formula when used together with other available empirical methods for the no barrier case (such as UFC 3-340-02 (2008)) provides a simple estimation of blast loading on building structures located behind a blast barrier. In their study, the rise time was assumed to be zero to be consistent with most empirical relationships used in blast analyses and designs, such as, UFC 3-340-02 (2008).

Two modification factors for peak reflected pressure $A_P$ and for impulse $A_I$ were introduced and they were as the same as used in the U.S. Army security Engineering Manual TM-5-583-3 (1986). To simplify the estimation of the modification factors along the building height, approximate piece-wise linear distribution was assumed and only four points could be determined. Hence, the paper fails to validate the strength of the model in any meaningful way. In a couple of case studies a new model, a CFD model, and a free-field model, are compared graphically for making estimations of the pressures/impulses over the centerline of a building face; however, in both cases, the comparison was for only one bomb–barrier–building configuration. So there is no quantitative assessment of performance/validity.

### 2.6 Effective Perimeter Wall Shape to Mitigate Blast Loads

The blast environment behind the blast wall may by further suppressed by locating a canopy on the top of the wall as in Figure 2.3b. This concept assumes that the canopy would re-focus the shock waves in a safe direction. While the canopy could shatter, and be blown away by the force of the explosion, it would probably remain in place long enough to mitigate the power of the shock waves spilling over the wall. A preliminary numerical simulations using ProSAir (2012) finite element software were performed only to examine the concept of changing the blast wall shape in mitigating the blast loads on buildings. In the simulation, both the blast wall and the building structure are assumed to be rigid.

A schematic of the geometric configuration for blast wall protection with a building behind it is shown in Figure 2.3. A half scale model for the plain blast wall shown in Figure 2.3a is investigated. The TNT charge is assumed to be hemispherical and to be detonated at 0.5 m above the ground level. The TNT charge weight, $W$, used in the
simulations is 10 kg. The height of the building, HB, is 6 m; the distance between the blast wall and the building, D, is 2 m, and the height of the blast wall, hw, is 1 m. The distance from the blast wall to the charge, R, is 2 m. The barrier thickness of 200 mm is presented. The width of the barrier and the structure is assumed to be of infinite longitude. However, only 2 m is considered in the 3D model. In the case where there is no wall, the distance from the charge to the protected building is R+t+D. It should be noted that only half of the barrier and structure is included in the model owing to the need for symmetry. Therefore, the model implies a 1 m long barrier and structure. Five target points are vertically located along the center line of the target building at heights of 0 m (T1), 0.75 m (T2), 2.25 m (T3), 3.75 m (T4), and 5.25 m (T5).

The width of each side of the canopy located on the top of the wall is varied (0.15 m, 0.20 m, and 0.30 m).
More than 5 cases are calculated in the present study. Only some typical results are shown in this section. Figure 2.4 shows the comparisons of the reflected pressure-time history at two different gauge points both for the no barrier and for the with barrier cases. From this Figure, it can be seen that: (1) not only the peak-reflected pressure is reduced but also the arrival time is delayed when there is a barrier; (2) the time delay caused by the barrier is longer when the gauge point height, H, is lower; (3) the duration of the positive phase for the with barrier case is longer than that for the no barrier case; (4) there may be two pulses due to the blast reflection (Figure 2.4b); When the reflected blast wave from the front of the building reaches the back of the barrier, it will be reflected by the back surface and propagates again to the building, causing the second pulse; (5) the peak pressure, impulse and the duration of the negative phase for both the with and the without barrier case are nearly the same. Therefore, the negative phase can be assumed to be the same as that in the no barrier case. The positive phase is the main concern of the present study.
To allow for an easy estimation of the effects of blast pressures and impulses on structures behind a barrier, two modification factors for peak reflected pressure $A_P$ and for impulse $A_I$ are introduced and they are the same as in the U.S. Army security Engineering Manual TM-5-583-3 (1986). These modification factors, together with the

**Figure 2.4: Reflected pressure time histories at lower and upper target heights**

![Graph showing reflected pressure time histories](image-url)
use of other available empirical methods such as either the A.T.BLAST program or the UFC 3-340-02 (2008) when introduced without a blast wall between the explosive and the structure, facilitate an easy estimation of the peak-reflected pressures and impulses on structures behind a blast wall.

In the simulation, both the maximum reflected pressures and the impulses are calculated. Comparisons of the peak reflected pressure distributions and the impulse distributions over the height of the building both with and without a blast wall are shown in Figure 2.5. In this figure, HB = 6 m, hw = 1 m, L = 2 m, W = 10 kg, D = 2. The vertical axis is the height of the building, and the horizontal axes are the modification factors $AF_P$ (Figure 2.5a) and $AF_I$ (Figure 2.5b). It should be noted that when a canopy is used the barrier is more effective in reducing the peak-reflected pressure and also the strength of the impulse even though the duration of the positive phase of the blast pressure is longer.
Figure 2.5: Adjustment factors for peak reflected pressures and impulses verses building height.
2.7 Summary

In this chapter, the existing approaches and methods used to predict the effects of blast loads on structures both in the presence or the absence of the blast wall located in front of a protected building are outlined and discussed. The existing engineering-level techniques for calculating the blast effects on buildings are based on the assumption that a building experiences a blast load when it is isolated in an open space. Historical evidences suggest that the actual blast loads can either be reduced due to shadowing by intervening buildings or can be enhanced due to the presence of other buildings or perimeter walls in the immediate vicinity.

It has been shown that the numerical techniques including Lagrangian, Eulerian, Euler-FCT, ALE, and finite element modelling should be used for the accurate prediction of the effect blast loads on commercial and public buildings. The experimentation approach at full scale is still the primary method of advancing knowledge of the subject area. The presented results achieved for the blast wall simulation with different shapes and their comparison to the results achieved with no blast wall of evaluating loads on buildings have demonstrated the importance of accounting for the perimeter walls when determining the effect of blast loads on buildings. However, the availability of precise data on the effectiveness and type of blast walls is limited.

A numerical study is conducted to investigate the effectiveness of blast wall. As the shock wave reaches the blast wall, a portion of the shock wave energy is reflected. The barrier also causes wave diffraction, and the diffraction process greatly weakens the shock wave energy located behind the barrier. As a result, the maximum pressure may occur at a point higher than the blast wall on the building, although the reflection of the blast wave from the ground intensifies the blast pressure and thus results in a high peak reflected pressure at the building base. When a building height is low, a relatively large portion of the energy will propagate over the top of the building. This makes the pressure and impulse strength at the building base relatively low. The concept of changing the
shape of the blast wall is more effective in reducing the strength of the peak reflected pressure and the impulse.

2.8 Acknowledgment

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3.1 Introduction

In recent years, blast protection and energy absorption structures has been the subject of considerable interest from both the military and civil industries. The protection of humans as well as the structures - particularly critical infrastructures - is an important and challenging issue. The blast protection materials are commonly manufactured in the form of sandwiches, which have been recognized for several decades for their excellent energy absorbing capabilities. The sandwich structure shown in Figure 3.1 consists of a lightweight core material and two sheeting materials, one on the front and one on the back face of the core, which are made from the same material and in the same thickness. The core material is usually made from some type of foam due to its low weight. The components of the sandwich material are bonded together using either adhesives or mechanical fasteners. The primarily role of the face sheets is to resist the in-plane and lateral loads.

Closed-cell foams are becoming increasingly important because they have good energy absorption capabilities as well as good thermal and acoustic protection properties. After reaching their yield stress levels, these materials provide a region of constant stress for increasing strain until the material is completely compacted. The energy needed to crush the material is proportional to the area under the stress-strain curve. Because foams have this “plateau” region, they absorb a considerable amount of energy relative to their low density.
Aluminum foams are being used more frequently in energy absorbing structures. In the theoretical analysis of aluminum foams for blast amelioration by Ashby et al. (2000), it was shown that the foam was exploited as an energy absorber by mounting a heavy buffer plate in front of it. The blast impulse first accelerated the buffer plate and the kinetic energy expended on the plate was dissipated by the foam. This implies that a thicker and heavier plate would have less acceleration; hence, less kinetic energy would have been dissipated by the foam. The foam should possess a plateau stress level just below that of the pressure that the structure can support. It should be noted that the density and the thickness of the aluminum foam play significant roles in determining their energy absorption characteristics. Increasing the density and the thickness of the aluminum foam increases the energy absorbing capabilities of the aluminum foam plate. Recently, blast tests on aluminum foam that protected the reinforced concrete (RC) structural members have been conducted and it was found that aluminum foam was very effective in absorbing blast energy. This paper presents a review of experimental and FE research on retrofitting structures with aluminum foam against blast wave loads.

### 3.2 Mechanical properties of aluminum foam

Generally speaking, most foam properties are a function of the foam density. This means that the collapse load for blast-protective sacrificial layers made of aluminum foam is easily specified by the selection of the proper foam density. The stress-strain behavior of
aluminum foam can be represented as consisting of three stages. In the first, there is linear elastic for small strains, which is controlled by three different types of strains, i.e. the bending of cell edges, the compression of gases trapped in the cells, and ultimately, the stretching of the cell walls. This stage terminates when a critical level of stress is reached; this critical stress level remains almost constant over a large range of strains (stage 2). In the second stage, the stress plateau ($\sigma_{p1}$) represents cell collapse and the strain is not recoverable in this region. This property is an important aspect of foams when they are used for energy absorption applications. At this stage, it can be observed that the stress plateau is serrated, which is due to the brittle nature of the foam. The serrations correspond to fractures in the cell walls. In the third stage, densification of the foam takes place as a result of the compaction of the cells, and the stress increases rapidly with increases in the amount of strain. The strain at which the densification starts is known as the ‘densification strain’ ($\epsilon_D$) as shown in Figure 3.2. In the case of uniaxial tension, the foam exhibits brittle fractures and failure strain. The tensile strength of aluminum foam is less than its compression strength due to different failure mechanisms, i.e. a local fracture in tension and a buckling under compression (1975). In the case of a uniaxial shear, the failure strain of the aluminum foam was approximately 0.002 (2004). In a study of the dependence of the material distribution on the modulus of aluminum foams (1998), it was found that the material distribution did not have a significant effect on the modulus.
3.3 Aluminum foam performance

Experimental, numerical and analytical studies of the effect of blasts on aluminum foam both in the presence and absence of a buffer plate were performed by Hanssen et al. (2002). The plastic compression of the aluminum foam consumes kinetic energy, which eventually halts the progression of the shock wave produced by the blast. It was observed that both the cover plate and the foam attained a double/concave curvature when subjected to blast loading. The final depth of the deformation of the foam panel relative to its edges was termed “dishing”. On the basis of the work performed by Liang et al. (1999), Hanssen et al. (2002) reported that the foam and cover plate could sustain a higher blast load because of “dishing”.

Hanssen et al. (2002) carried out field tests with explosives in order to investigate the behaviour of aluminum foam panels as sacrificial layers subjected to explosion-induced blast wave loadings. A ballistic pendulum was used to calculate the energy and impulse transfer from a close-range blast. Their experimental setup also included a thin aluminum cover plate that was added to protect the aluminum foam. They reported increases in the energy and impulse transfer due to the addition of the panels. This was an unexpected result since it was assumed that the addition of the foam panels would reduce both the
amount of energy and the size of the impulse transferred to the pendulum by absorbing some of the energy as a result of their crushing. They suggested that the increase in energy and impulse transfer, when foam-covered panels were added to the pendulum, could be due to the continuous change of the shape of the initial panel surface to a double-curved shape. The impact resistance and energy absorption of sandwich composites with aluminum foam and polymer matrix fiber-reinforced face sheets was studied by Vaidya et al. (2003). They reported that the amount of energy absorption and degree of failure strongly depended on the type (e.g. S2-glass, Kevlar, carbon and E-glass) and property of the face sheets. Face sheets made of S2-glass and Kevlar were seen to possess superior impact resistance and energy absorption when compared to carbon and E-glass fiber reinforcements. The face sheets with higher compressive strength and heavier tow (such as Kevlar and S2-glass) were more effective in spreading the blast load over a larger area of the underlying core. Blast load and damage progression on an aluminum foam sandwich composite plate with S2-glass face sheets have been simulated using LS-DYNA (1998).

Sriram et al. (2006) examined the modeling of aluminum foam sandwich composites subjected to blast loads using LS-DYNA software. The sandwich composite was designed using laminated face sheets (S2 glass/epoxy) and an aluminum foam core. A blast load was applied using the CONWEP [19] blast equations (*LOAD BLAST) in LS-DYNA. They discussed the blast response of constituent S2 glass/epoxy face sheets, and the closed cell aluminum foam core as well as the sandwich composite plate.

Aluminum foam without cover sheets with dimensions of 0.3 m×0.3 m×0.0159 m was modeled. It was restricted for all degrees of freedoms along the corners. The foam was modeled using material 126 with a co-rotational element formulation 0. A blast load of 5 MPa was applied to the foam. The elements along the corners eroded and shear mode failure occurred along the corners. The foam along the corners underwent shear or tensile failure because the (SSEF) and the (TSEF) of the foam was small, i.e., 0.003 and 0.002, respectively.
The sandwich plate was modeled using two S2-glass/epoxy face sheets on either side of the foam core. The thickness of the face sheet was 0.0015 mm and the sandwich plate had a similar dimension to that of the aluminum foam. The sandwich composite was restricted for all degrees of freedom along the corners.

The study by Hanssen et al. (2002) revealed that a cover plate in front of the foam resulted in ‘dishing’ (double/concave curvature) under blast loading. The study concluded that because of this dishing, the foam could sustain a higher blast load. Dishing was also observed in the simulation conducted in this study.

Feng et al. (2008) carried out experimental investigations to study the resistant behaviour and energy absorbing performance of the square sandwich panels under blast loading. A four-cable ballistic pendulum system was employed to measure the impulse delivered to the pendulum/specimen. The frames were clamped on the front face of the pendulum, and the charge was fixed in front of the centre of the specimen using an iron wire with a constant standoff distance of 200 mm. The specimens used in the tests consisted of two identical face-sheets and a core of aluminum foam. The face-sheets were made of aluminum alloy and had two different thicknesses of 0.8 mm and 1.0 mm, respectively. The aluminum foam cores had two relative densities of 6% and 10%. The cores were cut into 300 mm × 300 mm plates with two different thicknesses of 20 mm and 30 mm. The panels were peripherally clamped between two square steel frames. After the tests, the specimens showed that the front face-sheets had attained an inwardly curved dishing deformation, and the back face sheet was deformed outwardly. The core exhibited progressive crushing damage, and a cavity between the front face and the crushed foam core developed, which was essentially a core fracture, rather than a debonding of the interface.

In the simulation, the face-sheets were meshed using the Belytschko-Tsay shell elements. The foam core was meshed into the eight-node brick (solid) elements. The explosive charge used in the tests had a cylindrical shape. Eight-node brick (solid) elements with the ALE (Arbitrary Lagrange Euler) formulation Hallquist J.O. (1998) were adopted for the explosive cylinder.
The face-sheets of the specimens used in the tests were made of aluminum alloy and were modeled with the Type 3 material (*MAT_PLASTIC_KINEMATIC) in LS-DYNA. The material type 63 (*MAT_CRUSHABLE_FOAM) in LS-DYNA was used to model the aluminum foams. Material 63 assumes that the Young’s modulus of the foam is constant. The material Type 8 (*MAT_HIGH_EXPLOSIVE_BURN) in LS-DYNA was used to describe the material property of the TNT charge. It allowed for modeling the detonation of a high explosive using three parameters: mass density of charge, detonation velocity, and Chapman-Jouget pressure. Likewise, an equation of state, which is called the Jones-Wilkins-Lee (JWL) equation, should be defined along with the explosive burn material model.

The results showed that there were a very good agreement between the experiment and the computational prediction, and thus indicated that the foam behaviour had been accurately characterized by the material model.

A parametric study has been conducted to investigate the energy absorbing behaviour of the blast loaded square sandwich panels, which included the time history of plastic dissipation in the face-sheets and core as well as a partitioning of the plastic energy absorbed by the different component parts of the panels; the effect of the panel configurations was also analyzed. During the interaction between the explosion product and the structure, the explosion energy was transferred to the sandwich panel, and then dissipated by the panel as it deformed. It was concluded that the foam core provided a major contribution to the energy dissipation and that the thinner face-sheets could raise the total internal energy while a denser and thicker core could increase its portion of energy dissipation.

Çagri (2008) investigated the blast performance and energy absorption capability of closed-cell aluminum foam based on lightweight sandwich structures by using a coupled experimental and numerical technique to determine the effect of face and core materials on the blast response. A finite element modeling of sandwich structures subjected to blast loadings were performed for different core and face thicknesses and face materials in order to investigate their effects on the blast load mitigation. Fifty by one hundred cm flat
rectangular sandwich panels were clamped at all four edges and modeled as a quarter model using appropriate boundary conditions and were subjected to 10 kg and 0.5 kg blast loadings with a 30 cm constant standoff distance. Blast loading was assumed as an air blast load. The ConWep function, which was developed by the US Army in 1991 (1993), was used to apply the blast loading. The commercial explicit finite element code LS-DYNA 971 was used. Three components were created as back face, core, and front face. The front face plate was referred to as the blast face because the wave front of the blast pressure directly strikes this face. The mechanical properties of the aluminum foam were 430 kg/m3 the density, 69 Gpa was the elastic modulus, and the Poisson’s ratio was 0.285.

As the core thickness increased, the total internal energy of the panel increased as the energy absorption capability of the foam layer increased. Hence, the sandwich structure became more effective as the core thickness increased. A numerical simulation of the results also showed that the permanent deflection increased as the thickness of the foam core increased. During the blast loading of the sandwich structures, the surface faces resisted the bending moment of the longitudinal compressive and tensile stresses while the foam core carried the transverse shear force.

The type of face material used had little effect on the amount of internal energy transferred. Due to this fact, another parameter of the deflection of the plate’s center as a function of time for a specific face material was investigated. The type of face material was found to have a significant effect on the amount of plate deflection. Based on the amount of plate deflections observed, one can conclude that the steel-steel face combination was the best face material combination whereas the Ti-Al face combination was the worst.

Lightweight aluminum foam core sandwich structures have been found to be very effective in energy absorption under blast loading. The results were consistent with previous studies, which showed that as the core thickness increased, the total internal energy of the panel increased due to the energy absorption capability of the foam layer. Numerical simulations showed that 6.3 and 7.2 cm thick foam interlayers were the most
efficient foam thicknesses for a 9 cm sandwich plate as protection against a 10 kg TNT blast load at a standoff distance of 30 cm. Another important conclusion with respect to the same blast threat, i.e., that of 10 kg of TNT, was that AISI 4340 Steel was the most effective face material to use.

3.4 Aluminum foam retrofitted RC members

Schenker et al. (2005) conducted experimental and numerical investigations in order to examine the capabilities of aluminum foam in mitigating the effect of blast waves acting on reinforced concrete (RC) beams and plates. The aluminum foam was used as a sacrificial layer. The aluminum-foam plates were glued to the concrete slabs on the side facing the blast wave. In addition, a very thin steel cover plate was glued to the front of the aluminum foam, which was facing the blast in order to prevent the rupturing of the closed cells (pores) of the foam. Impact tests were conducted by means of a ballistic pendulum on full-scale RC beams and full-scale RC plates and blast field tests on RC plates. The experimental results were used to calibrate the numerical analysis parameters and to validate the analysis using the commercial FE code LSDYNA. In addition, they determined the efficiency of aluminum foam as a structural protection layer against blasts.

Two full-scale high explosive field tests were conducted in order to study the effectiveness of the aluminum foam as a protection layer. In the first test, two 1.2 x 1.3 m RC plates having a width of 0.2 m were exposed to a blast wave that was generated by the explosion of a 100 kg TNT at a distance of 10 m. In the second test, two 1.4 x 3.2 m (B-100 RC) the plates having a width of 0.2 m were placed at a distance of 21 m from a TNT charge of 900 kg. In both cases, one of the RC plates was protected with aluminum foam and the other was not. The results revealed that while the unprotected RC plates had many cracks on their back side, the aluminum foam-protected RC plates had very few cracks. Quantitatively speaking, whereas the maximal strain on the steel rebars in the unprotected plate reached 4,000 µstrain, its maximal value in the foam-protected plate was 2,000 µstrain. Similarly, the maximal measured accelerations at the center of the unprotected and protected plates were 500 and 300 g, respectively. The results clearly
demonstrated the ability of aluminum foam to reduce the damage to structures exposed to explosion-generated blast waves

The experiments provided very clear results for the strains and accelerations of the RC beams and the plates, and also for the forces acting between the pendulum and the RC beams and plates. Since the numerical models did not account for cracks, they were calibrated using the experimental results by reducing the stiffness of the RC beams and the plates until they predicted dynamic behaviours similar to those recorded and observed in the experiments. The calibrated numerical model provided very similar results for the impact behaviour. The calculated strains within the RC beams and plates were not the same as those measured in the experiments, and this phenomenon is still under investigation.

Schenker et al. (2008) conducted full-scale field explosion tests on protected and unprotected concrete slabs to check the ability of aluminum foams to mitigate the blast wave loads. Two types of concrete slabs were examined: commonly reinforced B-30 (regular) and fiber reinforced B-100 (strong). The concrete slabs were supported by a heavy pre-cast concrete rigid structure. Two 3-m span slabs of each of these two concrete types, one with full or partial protection, and one without any protection of the aluminum foam layers were exposed to a blast of a nearly 1000 kg hemispherical TNT charge at a distance of about 20 m. The aluminum foam layers were applied in two ways: the first consisted of two or four layers and the second consisted of two layers that were covered by a thin steel plate.

For the B-100 and the B-30 concrete slabs, it was found that the back of the unprotected concrete slab suffered much more damage than the back of the protected one. The acceleration of the protected slab at the center was significantly less than that of the unprotected slab. The velocity-time histories were calculated at the center of the slabs by integrating the measured acceleration-time histories. The velocities reached by the protected concrete slab were significantly lower than those acquired by the unprotected concrete slab. The displacement-time histories at the center of the concrete slabs were calculated by using a double integration of the measured acceleration-time histories. It
was found that the displacements reached by the protected concrete slabs were smaller than those reached by the unprotected concrete slabs. In comparing the ratio between the acceleration of the protected to the unprotected slabs, it seemed that the aluminum foam was more effective for the “regular” case. This was because the regular plate was more flexible and thus it was more sensitive to energy dissipation. As was expected, the maximum displacement of the “strong” concrete was significantly less than the maximum displacement of the “regular” concrete.

Two types of numerical simulations were carried out for the regular B-30 unprotected RC slabs. The explosive charge was represented as a hemisphere of a hot and dense gas initially having the density and specific internal energy of a TNT high explosive. The structural response of the concrete slab was carried out by assuming that the B-30 material was elastic and had no rebars. However, to compensate for the absence of the rebars in the model, the concrete’s strength under tension was increased appropriately. The pressure profile was also calculated and was found to be similar to the measured profile. The pressure load in the model was a simplified profile, while the impulse in the model was the same as in the test. A simply supported boundary condition was defined at the edge of the concrete slabs. The energy from the blast was latent in the concrete slab and was transferred to the support structure as kinetic and strain energy during the blast process.

3.4.1 Aluminum foam retrofitted unreinforced masonry wall (URM)

Su and Wu (2008) conducted numerical studies to investigate the effectiveness of the retrofitting/ mitigating systems aluminum foam cladding on URM walls subjected to blast loads. Discussions were held regarding the efficiency of the mitigation of the blast effects on the URM walls using different kinds, thickness, and densities of aluminum foams.

The Drucker-Prager strength model was used to model the behaviour of bricks and mortar of masonry structures. The aluminum foam was modeled using a nonlinear elastoplastic material model. The steel skin for the aluminum was simulated using an
elastoplastic model. The interfaces between the masonry and the steel face of the aluminum foam were simulated using a layer of interface elements with a thickness of 1 mm. The material models for the masonry, the steel cover and the aluminum foam, were coded into a finite element program LS-DYNA3D to perform the numerical calculations of response and damage to the 2500 mm × 2500 mm × 110 mm URM wall both with and without retrofitting under airblast loads. Airblast loads acting on the surfaces of the URM wall and the aluminum foam protected URM wall at different scaled distances with different charge masses were estimated using the TM5 manual (1999). A triangular pressure function with a peak pressure of $P_{r,max}$ and duration of $t_d$ was assumed in the analysis. The pressure was uniformly applied over the incident face of both the URM walls and the aluminum foam protected URM walls. The strain rate effects on the material properties were not considered since the blast loads used were relatively small. A series of numerical simulations with a scaled distance increment of 0.01 m/kg$^{1/3}$ were carried out. The critical scaled distance to prevent the failure of the URM wall was found to be 9.0 m/kg$^{1/3}$.

It was observed that the retrofitted URM wall with aluminum foam sheets on both sides can absorb blast energy more than 14 times as effectively as the URM wall, indicating that aluminum foam is very effective in mitigating blast effects on a URM wall.

Parametric studies were carried out to investigate the influence of the thickness and density of aluminum foam sheets in the mitigation of blast effects on URM walls. It was found that the greater the thickness, the smaller the response. Or, in other words, the more effectively it mitigated blast effects on the URM wall. It was found that the higher the density, the smaller the response, i.e., the more effectively it mitigated the blast effects on the URM wall.

3.5 Summary and Conclusion

This Chapter has presented a review of experimental and FE research on retrofitting structures with aluminum foam against blast loads with areas which require further research highlighted. A very limited amount of research has been conducted on the blast resistance of aluminum foam strengthened reinforced concrete (RC) members as well as
that of masonry and concrete walls. Existing research has indicated that aluminum foam can effectively mitigate the blast effects on structures and absorb blast energy. It was found that by increasing the density of the aluminum foam or by using thicker foam increased the energy-absorbing capabilities of the aluminum foam plate. The presence of cover sheets (buffer plates) on the top and the bottom of the aluminum foam had the potential to improve its energy-absorbing capability. The blast impulse first accelerated the buffer plate and the kinetic energy acquired by the plate was dissipated by the foam. This implies that a thicker and heavier plate would have less acceleration; hence, less kinetic energy must dissipated by the foam.

The aluminum foam modifies the response of the RC members and of the unreinforced masonry walls. However, to reach a definitive conclusion as to its efficiency for practical purposes, more numerical and experimental research is needed to better define and measure the contribution of aluminum foam in reducing part of the blast-induced load on a structure and, as a consequence, reducing the actual load exerted by the blast on a structure.

In most of the previous studies, however, only the properties of foam cladding and blast loads have been considered; whereas the protected main structure itself has been neglected in the analysis by assuming that the foam cladding was fixed to a rigid reaction wall. To design appropriate sacrificial foam claddings and to evaluate the protective effect of such cladding, it is important to take all of the three components, i.e., the blast load, the foam cladding, and the structure, into consideration.

To make the best use of a study, it is essential that all of the details of loading, the geometrical and material properties, and the structural responses as well as the failure modes are reported. When making comparisons with experimental results, it is preferable to have actual measured pressures at different positions because the loading can always contain some uncertainties arising from a non-perfect detonation.

3.6 Acknowledgment
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3.7 References


Chapter 4

4 Development of Blast Field Measurement System

4.1 Introduction

Experimental blast field testing of a structure is rarely practised in Saudi Arabia. In this study, there was a challenge to design a blast field measurement system to test the response of concrete members subjected to blast loading. Nevertheless, it is well known that blast field tests more closely reflect reality and that the results could be used to calibrate data collected from laboratory experiments. To conduct successful blast field tests, it is very important to both select the appropriate measurement devices and to understand the mechanism of the explosive material being used. Before developing the blast field measurement system used here, an overview of blast field testing experiments using the high speed data acquisition systems used by various researchers worldwide was conducted to assist in developing system that would be suitable for this study.

Here several blast tests were conducted at the Special Security Forces-Site (SSF-Site), which is located north of Riyadh, city, Saudi Arabia using specific charge sizes at fixed standoff distances. These tests required careful attention to many different elements for their successful completion. For example, the pre-blast preparation included: finding an adequate blast site, installing the specimen, setting up the data acquisition system, setting up high-speed cameras, and preparing the blast components. Proper pre-blast preparation insured that the data from the blast pressures would be successfully acquired and that the high speed cameras would capture the blasts. Other responsibilities included post-blast duties such as clean up and data analysis.

The main objective of this study then is to describe the various components of the blast field testing. These include both the procedure and the instruments developed using high speed data acquisition system. They include: the software, a signal conditioner, blast wave transducers, accelerometers as well as the strain gages and the coaxial cables that are connected to the test specimens and the high speed camera. Figure 4.1 shows the components involved in the field blast testing program.
4.2 Blast Field Site

The blast field site was constructed and developed for this study to conduct the experiments with an actual explosive device of about 125 kg. Three protected bunkers were constructed at safe distances: one to secure the data acquisition device and, the other to protect the operators. The blast site was located on the Special Security Forces Range of the Ministry of Interior north of Riyadh city, Saudi Arabia. The site provided an ideal location for blast testing and offered an area isolated from any residential areas. This site was safe and secure and the explosive engineers were well practiced in planning and executing assignments of all sizes using a variety of sizes for the charges. Hence, any possible problems that may arise due to the public's concern about the noise or danger from the explosion generally were minimized. Although isolated, the blast site was equipped with the electrical power that is necessary for the pre-blast setup, the data acquisition system, and the high-speed cameras. Figure 4.2 shows the blast field test site.
4.3 Blast Equipment

In this study, it was necessary to understand the unique effect of each of the blast components, which were not covered in previous studies such as composition of the explosive material, the blast booster, and blasting cap. The blast equipment, which consisted of all the equipment required to conduct a scientifically controlled blast test is discussed later in this study. The explosive material used was ammonium nitrate/fuel oil (ANFO). An electric switch was used to ignite and detonate a blasting cap, which then detonated a blast booster consisting of a high powered explosive. The booster detonated the ANFO creating an air blast pressure loading. A data acquisition system then recorded the resulting air blast pressures.
4.3.1 Explosive Charges

In this research program, a combination of ammonium nitrate (AN) and diesel fuel oil (FO) (ANFO) was used as the explosive. ANFO is widely used high powered explosive that is used extensively throughout the world for mining, agriculture, and construction purposes. It is also a very common terrorist weapon because it can causes extensive structural damage and serious injuries. Another advantage is that ANFO can be formed into small pellets that are easily transported. Since ANFO is a relatively stable substance, a high-grade explosive is required to detonate it. Therefore it does not have to be handled with the same caution as other high explosives. The main disadvantage of ANFO is that they are not waterproof and also have low bulk strength. ANFO is non-ideal explosive, its performance or release of energy is affected by many factors such as the mixing ratio of the AN to the FO, the particle size, the moisture content and the degree of confinement. It has been found that a mixing ratio of 5.7% FO and 94.3% AN by weight provides the maximum blast explosive energy (Johansson and Persson, 1970). Table 4.1 lists the properties of bulk ANFO.

<table>
<thead>
<tr>
<th>Specification</th>
<th>Measure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk density, kg/m$^3$</td>
<td>850</td>
</tr>
<tr>
<td>Explosive energy, KJ/Kg</td>
<td>3717</td>
</tr>
<tr>
<td>Detonation velocity, m/s</td>
<td>3990</td>
</tr>
<tr>
<td>Volume of reaction products, L/Kg</td>
<td>4757</td>
</tr>
<tr>
<td>Equivalent weight for pressure</td>
<td>0.82</td>
</tr>
<tr>
<td>Equivalent weight for impulse</td>
<td>0.82</td>
</tr>
</tbody>
</table>

The shape, the type and the orientation as well as the booster quantity and the initiation point of the charge can cause a significant increase in the blast loading. In this study, ANFO with a cubic shape initiated at the top was considered. The boosters that have been used in this study were (PETN-based) explosive composition shaped into two cylinders with 10 and 20 gm and 400 gm Pentolite. The first one was used in the half scale tests and the second used in the full scale tests.
In this study, the charge was placed on a wooden table with its height adjusted to position the center of the explosive at the mid height of the specimen. The height of the blast above the ground was first taken as being in the full scale of 1 m because the explosive was assumed to be detonated in a vehicle and, second in the half scale of 0.5 m. In order to calculate the blast wave pressure-time history from a conventional explosion, two critical factors should be considered, the charge weight and the standoff distance. For safety considerations, all explosion facilities were provided and operated by SSF personnel. Figure 4.3 shows the preparation of the blast equipment (ANFO, Booster, and Blasting Cap.)

4.3.2 TNT Equivalency

Blasts can be caused by using different explosive material. According to the type of the explosive material, the detonation rate or velocity, the effectiveness, the blast density, and level of heat production are determined. For blast resistance design, TNT is commonly used as the reference explosive. While other types of explosives are usually transformed to an equivalent weight of TNT. The majority of empirical data generated from the start of when high explosives were first studied has been for TNT. Due to the large number of empirical studies based on the use of this explosive compared with other explosives, TNT has been adopted as a benchmark high explosive. Hence, TNT equivalency is used in the majority of research on blast effects to relate the energy output of common explosives to that of TNT.
When the high explosive is other than TNT, the equivalent energy is obtained by using the charge factor (CF). This will form the value of the scaled distance \( Z = R/W^{1/3} \). The charge factor is equal to the actual mass of the charge/mass of the TNT equivalent. The equivalency of explosive material compared to TNT is dependent on many factors e.g. the material shape (flat, square), the explosive quantity, the degree of explosive confinement, the nature of the source and the pressure range (HNDM, 1977, Doering 1949). Table 4.2 summarizes the averaged free-air equivalent weights for commonly used explosives based on their peak pressure and impulse (ASCE 1999).

**Table 4.2: Average Free-Air Equivalent Weights (ASCE, 1999).**

<table>
<thead>
<tr>
<th>Explosive</th>
<th>Equivalent Weights, Pressure</th>
<th>Equivalent Weights, Impulse</th>
<th>Pressure Range, Psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>ANFO</td>
<td>0.82</td>
<td>0.82</td>
<td>1-100</td>
</tr>
<tr>
<td>C-4</td>
<td>1.37</td>
<td>1.19</td>
<td>10-100</td>
</tr>
<tr>
<td>PETN</td>
<td>1.27</td>
<td>-</td>
<td>5-100</td>
</tr>
<tr>
<td>PENTOLITE</td>
<td>1.42</td>
<td>1</td>
<td>5-100</td>
</tr>
<tr>
<td>TNT</td>
<td>1</td>
<td>1</td>
<td>Standard</td>
</tr>
</tbody>
</table>

TNT equivalents can also be determined by using the comparative values of the energy output for different explosive material relative to that of TNT charges of similar size (TM5-885-1).

### 4.3.3 High Speed Data Acquisition System and Instrumentation Plan

In order to undertake even a fundamental analysis of the mechanisms of load transfer accurate test facility instrumentation is required to measure the blast force, the displacement, the acceleration and the strain. The proper selection and use of such instrumentation is essential to accurately characterize a blast. Sensors are frequently subject to artifacts in their measurements caused by their resolution, mounting, and/or positioning. For this study an extensive investigation has been done by visiting different laboratories and reviewing the literature to select the appropriate sensors and the data acquisition system. The proper installation of the test specimen constitutes the first basic step in conducting a successful test blast. A high speed data acquisition and analysis
system was developed, which implemented hardware from National Instruments as well as in-house written LabVIEW programs that are utilized for data collection and analysis.

The system was powered by a generator and an uninterruptable power supply as a back-up. The CompactRIO system was connected to 20 pressure sensors and 4 accelerometer sensors via a signal conditioner in between using shielded low noise coaxial cabling up to 50 m. The CompactRIO system was networked with a PC located in the control room using Ethernet. The LabVIEW program was developed to gather data as well as to check the system modules, which show the data in graphs for selectable period. After making the system settings and testing the connection with the sensors, the system was ready to start acquiring data from the tests by clicking start on the specified button in the graphical window.

4.3.4 High Speed Data Acquisition System

The measurement of the shock wave, displacement, accelerations and strains is an important element with regards to tracking the performance, or the structural response to the blast loading. These parameters were measured using a high speed data acquisition system that included National Instruments data recording hardware and LabVIEW software for data collection and analysis, a signal conditioner, sensors transducers and cables which were connected to the test specimen. The sampling rate was up to 100 ks/s/channel.

A National Instruments CompactRIO data acquisition (DAQ) system controls the acquisitions of data from all sensors and is configured for 32 differential input channels utilizing 16 bit sampling channels. It has a built in memory storage capacity of 16 GB. It is configured to record eight strain channels, 20 pressure channels, and four accelerometer channels. ICP 4-20 mA signal conditioning devices are used to amplify the signals received from the sensors. Figure 4.4 provides a schematic of the system’s components. The data acquisition system and instrumentation were situated in a protected location approximately 25 m from the blast testing location.
All of the aforementioned instrumentation was triggered from the control room by the control computer to provide synchronization of the Data Acquisition System (DAS) and the camera with the explosive. For long distances between the control computer in the control room and the CompactRIO system, a fiber optic cable was used.

### 4.3.5 LabVIEW Software

LabVIEW is a graphical programming language that is easily implemented for acquiring, monitoring control systems, and conducting the data analysis. Two programs were written using LabVIEW. The data acquisition program was developed for recording experimental data in a consistent and easily analyzed format. The software collects the data, applies sensor calibrations, and records important experimental parameters. Programs like Microsoft Excel can be used for graphing data, but graphing with LabVIEW is often faster, especially when large amounts of data are being analyzed. Also, Microsoft Excel 2007 is limited to 32000 data points per series in a chart (Microsoft Corporation, 2011), and the processing time can be extensive. Although LabVIEW can
provide faster graphing, some programming is required. However, this can be advantageous since the program can be adapted to specific user requirements. For this study, a data analysis program was written in LabVIEW for rapid data viewing and analysis.

The pressure–time history was measured using pressure sensors and the impulse (area under the pressure–time history) was automatically calculated and displayed. The accelerations were measured using the accelerometers. The velocity–time histories were calculated by integrating the measured acceleration–time histories. The displacement–time histories were calculated by using a double integration of the measured acceleration–time histories.

The data acquisition unit was trigged first using the graphical window located in the control computer which had been programmed by the LabVIEW program at the firing location where the DAQ was designed with five second delay time and a five second acquiring time. Figure 4.5 shows a screenshot of the main window of the program.

![Graphical Window Used to Check and Trigger the High Speed Data Acquisition System](image)

Figure 4.5: The Graphical Window Used to Check and Trigger the High Speed Data Acquisition System
While the sensors were continuously acquiring information once powered, they required a trigger signal to start the recording process. The instrumentation gathered five seconds of data during a test but the analysis required milliseconds of data. The second step was to trigger the charge using a switch connected to both the firing cable and the camera cable so that they worked simultaneously. Experimental data were stored in log files located on the CompactRIO system. All of the data was then downloaded automatically to the computer in the control room after each test. The data could be viewed using the LabVIEW analysis software or it could be outputted to an Excel spreadsheet for the analysis.

4.4 Free Field Pressure Transducer

Free field blast pressure were measured using PCB™ pressure pencil probes ICP 137A. They were located at specific distances from the centre of the explosive; a single BNC connector was connected to the free field pressure transducer by a coaxial cable, which was connected to the signal conditioner, and short cables connected the signal conditioner to the CompactRIO. The probes were positioned at a specified height from the ground. For incident pressure (side-on) measurements, the direction of the propagation of the shock wave front was parallel to the gage sensing surface. The sensor had the ability to capture high frequencies of up to 500 kHz. The free field pressure transducer had a total length of 16.0 in. (406 mm) with a sensing element, which was located 6.20 in. (157 mm) from the tip of the pressure transducer as shown in Figure 4.6. The free field transducers were installed on tripods located along a line parallel to the direction of the blast wave.
4.5 Reflected Pressure Transducers:

Reflected pressures were measured using a piezoelectric pressure sensor screwed onto the non-responding columns. Their direction was perpendicular to the blast wave. Sensors taken from PCB Piezotronic model 102B, 102B03, and 102B04 were utilized and the sensors were installed on a non-respond column, which represented the target buildings to measure the reflected pressure created along the height of the rigid column. An important feature of the PCB transducers was that they were integral-electronics piezoelectric (IEPE) gages, which converted the output from the transducer to a low impedance signal. Thus, by eliminating the noise effects and permitting the use of inexpensive cables that do not require noise treatment. Each transducer was calibrated for the expected pressure produced at its location based on the results of the models used in the Blast Effects Computer (Department of Defence Explosives Safety Board, 2001). Model 102B can accurately measure pressures ranging from 1 to 5000 psi; Model 102B03 can accurately measure pressures ranging up to 10000 psi; Model 102B04 can accurately measure pressures ranging from 0.2 to 1000 psi. The signal was taken along the 50 m long coaxial cable to the bunker and recorded by a DAS.

The SSF staff mounted four PCB piezoelectric reflected pressure transducers along the height of each non-responding column, which were labelled as P1 to P4. The sensor direction was normal to the explosives as shown in Figure 4.7. These gages were coated
with silicone rubber to minimize the effects of heat radiation during the explosion. The pressure transducers were screwed into the target column to minimize the disturbance in the pressure readings. This step ensured proper reflected pressure readings from the reflected pressure transducers. The pressure gages were mounted inside a machined nylon and O-ring steel plate mount and screwed to the non-responding column. The nylon mount was used in an attempt to reduce any high frequency vibration.

Three steel non-responding columns were used which designed to ensure that the flow of the shock wave would be normal to the gages and that minimal aerodynamic interference would be encountered in the vicinity of the gages. The non-responding round steel columns represented the target building at three different standoff distances. The columns were designed to minimize the gage vibrations or “ringing”. The steel columns used for this series of experiments were manufactured from galvanized pipe. The columns were designed to provide a height of 6 m and were anchored to a concrete foundation. These steel columns were used in an array of up to three columns with four gages on each column and these were aligned to minimize shrouding, or interference, from adjacent gages. Washers ensured a tight connection to the steel plate and a single nut secured the reflected pressure transducers as shown in Figure 4.7. A bayonet navel connector (BNC) connected the reflected pressure transducers to the coaxial cables. The reflected pressure transducer mounting system had three parts. The outer portion consisted of a mounting disk. The fabricated circular Teflon mold connected the mounting disk to the reflected pressure transducer as shown in Figure 4.7. The gages were screwed onto a steel mount and then mounted onto a pipe that was flush-mounted to the column section. (The pipe was cast into the column sections during construction). The transducers were then connected to the DAS by coaxial cables, which were inspected for any kinking that could have damaged the cable or the insulation or produced noise and disturbance in the pressure readings.
4.6 Accelerometers

A uniaxial shock accelerometers model 350A13 Piezoelectric ICP® was selected and used in this study. It has a maximum measurable shock of 10,000g. Figure 4.8 shows the sensor and its installation where the sensor is attached by mounting it to the back side of the wall and connected to the data acquisition system module.

A 350A13 High Amplitude ICP® Shock Accelerometers was used, which was specifically designed to withstand and measure extreme, high amplitude, short-duration, and transient accelerations.
4.7 Strain Gage Installation on Reinforcing Bars

A strain gages manufactured by Vishay CEA-06-125UN-350 strain gages were installed on the vertical reinforcing bars to measure the deformation inside the test specimen after each blast test. The gages have a resistance of 350 Ω with a gage factor of 2.11.

It is important to note that the rebars should be straightened prior to installing the strain gages. And in order to protect the steel strain gages from damage due to the concrete pouring and moisture, a coating with an epoxy layer was provided. Shielded vinyl lead wire cables (3 mm diameter 3 cores) were used to connect the strain gages to the data acquisition system. Figure 4.9 shows the installation of the strain gages. Each strain gage was given a designation to indicate its location according to the particular arrangement, i.e. strain gage 1 was designated as SGF-1. Prior to testing and after each installation step, the gage resistance was checked for proper resistance using a digital voltmeter as shown in Figure 4.9.

![Figure 4.9: Strain Gauge Installation](image)

4.8 High-Speed Digital Video Cameras

During testing a computer-controlled high speed camera was used to capture the blasts. It was the Memrecam GX-3 high-speed camera colour version with a 4GB memory which is capable of taking 2910 frames per second with a resolution of 1024 x 1280 pixels and up to 600,000 fps at a lower resolution. The SSF staff placed the high-speed camera at a safe distance from the explosion point. The camera was used to record the entire blast
setup and to show the blast wave hitting each wall and how the pressure waves moved through the air. The camera was connected to the firing switch, which was to be activated once. And a Windows PC was used to setup and monitor the high-speed camera in the control room where the camera was connected via an Ethernet cable to the laptop in the control room. An example of the kind of output gathered is shown in Figure 4.10.

Figure 4.10: Photo Shot of the Blast Wave and Camera Setup

4.9 Test Procedures

The data acquisition system consisted of PCB Piezoelectric reflected pressure transducers, a PCB Piezoelectric free field pressure transducer, a signal conditioning device, coaxial cables, a PC with LabVIEW Software, and an uninterruptible power supply. The DAS was connected to an uninterruptible power supply ensuring continuous power during data acquisition. The SSF staff setup the data acquisition system at a safe distance from the blast arena. The setup included configuring the PC and the "LabVIEW Software" and connecting the CompactRIO device to the computer in the control room an
Ethernet communication cable. Long, 50 m coaxial cables connected the reflected air blast pressure transducers and the free field pressure transducer to the signal conditioner. Short cables then connected the signal conditioner to the CompactRIO device while the accelerometers and the strains were connected to the CompactRIO device. The CompactRIO device was in turn connected to the PC in the control room. The PCB Piezoelectric free field pressure transducer was mounted on a stake located next to the test specimen. The PCB Piezoelectric reflected pressure transducers were affixed vertically to the non-respond columns at four different heights above the ground. The test consequence is illustrated in Figure 4.11.

The data acquisition system was then tested after the complete installation of all of the system components. Testing the system first involved initializing the system. The sensors connections could be checked from the signal conditioner first and then the modules connections as well as the memory space could be checked from the control computer in the control room.

Explosives delivered the blast components to the field test. The SSF staff placed the explosives at their normal location relative to the test specimen. The blast director then

![Figure 4.11: Test Consequence](image-url)
placed a blasting cap and a booster in the explosives. The SSF provided hard hats and ear plugs to all of the individuals present at the blast site and checked to verify that all of the spectators and staff members maintained a safe distance from the blast arena. Upon verification of all safety precautions, the data acquisition system was triggered first using the control computer in the control room then the blast director detonated the explosives. The high speed camera was connected to the firing device by the same switch in the control room while the camera was connected via an Ethernet cable to the control room. The LabVIEW Software collected the air blast pressure-time histories in conjunction with the DAS.

Immediately following the blast, the SSF staff saved the air blast pressure-time histories to the hard drive in the DAS as well as to the computer in the control room to prevent any potential loss of the air blast pressure-time histories due to an unforeseen system failure. Subsequent to the blast, the test specimen was visually inspected. After the completion of all the necessary inspections, the SSF staff secured all of the blast equipment and cleaned up the blast site. Next, the air blast pressure-time histories were analyzed.

### 4.10 Prediction of Blast Load

Prior to each trail, calculations were performed using A.T.BLAST software to estimate the blast loading (the peak air blast pressure, the positive phase duration, and the positive phase impulse) caused by an explosive material at a fixed standoff distance. This software was developed by Applied Research Associates (ARA) Inc. The United States Department of Defence provided charts relating the blast wave parameters to the scaled distance (TM5-1300, 1990) and these parameters are presented numerically in the A.T.BLAST software. Users of the software can input a specific charge size and standoff distance as well as the angle of incidence and receive the blast loading from an open-air hemispherical explosion that is based on a spherical charge initiated at the center.

### 4.11 Calibration and Analysis:

The instrumentation was calibrated and checked before conducting the test. This phase was necessary to validate the theoretical pressure values obtained from the empirical equations with respect to real experimental values. It was also important to calibrate and
test the instrumentation before proceeding with the entire test series. The blasts predicted by A.T.BLAST are presented and compared to the measured blast pressures as shown in this section. Free field tests were conducted using ANFO charges of various sizes at three different standoff distances. The blast field testing measurement results were successfully obtained using the high speed data acquisition system. The results showed that the pressure wave followed the classical shape of the blast wave pressure history. Figure 4.12 shows one of the reflected pressure-time histories and impulse-time histories recorded after the detonation of a charge of ANFO. The profile was captured by one of the reflected pressure gages mounted on the non-respond column and the resulting profile shows the typical features of a pressure profile induced by the detonation of conventional explosives, i.e. zero rise time, exponential decay, and positive and negative pressure phases.

Figure 4.12: Screenshot of the Output
A comparison of the pressures and impulses obtained from the experiment and the numerical values obtained by A.T.BLAST for a variety charges at two standoff distances is presented in Table 4.3 to Table 4.14. The results showed a good agreement between the measured and the numerical values for the reflected pressure especially for the longest standoff distance. The difference in the reflected pressure was between 13% and 48% at the shortest distance while the difference in the reflected pressure was between 2% and 48% at the longest distance. The program overestimated the value of the impulses where the percentage difference in the impulse was between 40% and 90%. It should be noted that the shape, type and orientation as well as the booster quantity and the charge initiation point can lead to a significant increase in the blast loading. Moreover, ANFO is a non-ideal explosive because its performance, or the release of energy is affected by many factors such as the mixing ratio of AN to FO, the particle size, the moisture content and the degree of confinement. In this study, ANFO with a cubic shape initiated at the top has been considered. The A.T.BLAST program was based on spherical charges being initiated at the center of the charge. The height of the charge burst (HOB) was 0.5 m.

Table 4.3: Comparison of Numerical and Measured Reflected Pressures for Varied ANFO Charges at 8.2 m Standoff Distance

<table>
<thead>
<tr>
<th>SHOT</th>
<th>1</th>
</tr>
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<tbody>
<tr>
<td>CHARGE (W), Kg</td>
<td>2.44</td>
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<tr>
<td>HOB , m</td>
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</tr>
<tr>
<td>Total distance , m</td>
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<tr>
<td>Reflected Pressure, KPa</td>
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<td>P1</td>
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<td>47.71</td>
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Table 4.4: Comparison of Numerical Reflected Pressures for Varied ANFO Charges at 8.2 m Standoff Distance

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<td>Reflected Pressure, KPa</td>
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Table 4.5: Comparison of Numerical and Measured Reflected Pressures for Varied ANFO charges at 8.2 m standoff distance

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<td>P3</td>
<td>168.86</td>
</tr>
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<td>P4</td>
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Table 4.6: Comparison of Numerical and Measured Reflected Impulses for Varied ANFO charges at 8.2 m standoff distance

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<tr>
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<td>118.46</td>
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<td>P2</td>
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<td>P4</td>
<td>94.740</td>
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Table 4.7: Comparison of Numerical and Measured Reflected Impulses for Varied ANFO charges at 8.2 m standoff distance

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<td>Reflected Impulse, KPa-ms</td>
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Table 4.8: Comparison of Numerical and Measured Reflected Impulses for Varied ANFO charges at 8.2 m standoff distance

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Table 4.9: Comparison of Numerical and Measured Reflected Pressures for Varied ANFO charges at 11.85 m standoff distance

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<td>P2</td>
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Table 4.10: Comparison of Numerical and Measured Reflected Pressures for Varied ANFO charges at 11.85 m standoff distance

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<tr>
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<tr>
<td>Reflected Pressure, KPa</td>
<td>P1</td>
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<td></td>
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Table 4.11: Comparison of Numerical and Measured Reflected Pressures for Varied ANFO charges at 11.85 m standoff distance

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Table 4.12: Comparison of Numerical and Measured Reflected Impulses for Varied ANFO charges at 11.85 m standoff distance

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Table 4.13: Comparison of Numerical and Measured Reflected Impulses for Varied ANFO charges at 11.85 m standoff distance

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Table 4.14: Comparison of Numerical and Measured Reflected Impulses for Varied ANFO charges at 11.85 m standoff distance

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<tbody>
<tr>
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<tr>
<td>HOB , m</td>
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<td>P2</td>
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<td>P3</td>
<td>252.56</td>
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<td>237.33</td>
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4.12 Conclusion

A blast site was constructed and specifically developed for this study. A new data acquisition and analysis system for real experiments were developed in conjunction with LabVIEW-based software. The techniques for the proper mounting of the piezoelectric sensors were also worked out. (The methods of collecting and analyzing shock wave data has been discussed previously). Operational procedures for the data acquisition and analysis programs are outlined in this study. Sensor information including specifications and practical observations are covered to give a concise overview of the key operational aspects. The successful execution of the experiments provided the initial “framework” for conducting the series of tests discussed in this study.

AFNO charges with a cuboid shape were utilized in this study. And the boosters that were used were of a PETN-based explosive composition shaped into 10 gm and 20 gm cylinders and Pentolite of 400 gm. The small ones were used in the half scale tests and the larger one was used in the full scale tests.

A comparison of the measured and numerical reflected pressures and the impulses of various ANFO charges at the two standoff distances are presented here. The blast field testing measurement results were successfully obtained using the previously described high speed data acquisition system. The results showed that the pressure wave follows the classical shape of the blast wave pressure history. The results showed a good agreement between the measured and the numerical values for the reflected pressure while the values obtained by the A.T.BLAST program overestimated the impulse.

4.13 Acknowledgment

The authors gratefully acknowledge the financial support provided by the Saudi Government.
4.14 References


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Chapter 5

5 Effective Shape of RC Blast Wall Subjected to Close-In Explosion for Blast Pressure Mitigation: Experimental Tests on Half-Scale

5.1 Introduction

Damage to structural assets, loss of life and social panic are factors that have to be minimized if the threat of terrorist action cannot be stopped. For many years, structures have undergone blast loads due to large-scale blasts. These large-scale blasts were caused by devices ranging from terrorist devices and conventional explosive charges to nuclear weapons. Designing the structures to be fully blast resistant is not a realistic or economical option; however, current engineering and architectural knowledge can enhance new and existing buildings to mitigate the effects of an explosion.

The hardening and retrofitting of an existing building to minimize the damage from blast loads is a very expensive procedure and is very difficult to achieve in a dense urban environment. An alternative method of protection is to construct a perimeter wall around and at some distance from the building reinforced concrete walls are generally utilized to secure buildings and to deflect blast loads away from the areas behind them. The use of barrier walls as a mitigation strategy is an important area of current research yet the number of studies currently found in the literature is relatively sparse.

Blast resistant walls are used to protect a building, or the surrounding areas from blast damage. Such blast walls will provide a stand-off distance to protect the structure from an external explosion. Moreover, they will act as an obstacle between the blast source and the building in the direction of the blast wave propagation. Therefore, some of the explosive energy will be reflected back, and the distribution of the blast pressure on the structure behind the barrier will be changed and the peak pressure will be reduced.

When a barrier is placed between a structure and an explosive charge, the barrier interferes with the propagation of shock waves towards the structure. In this case the load
on the structure is affected by the weight of the explosives, the distance between the explosives and the barrier wall, the distance between the barrier wall and the structure, and the height of the barrier wall. If the barrier wall is too far away from the structure, the shock will re-form and there will be little decrease in the size of the loads reaching the structure. Similarly, if the explosive charge is too far away from the barrier wall, the barrier wall will be ineffective.

Optimal blast load mitigation can be achieved by erecting rigid, non-destructible barriers. Using a barrier’s mass instead of its strength to attenuate blast loads is a lower cost method of achieving the same objective, but this approach generally requires excessive amounts of mass and therefore very thick (and also architecturally unacceptable) barriers. Therefore, it is worth considering the possibility that blast walls could also be relatively lightweight and still offers a certain degree of protection because the high level of deformation of such a wall could absorb a significant amount of the blast wave energy originating from the threat weapon. Thus, depending on the design of the wall deployed, a combination of energy reflection and energy absorption would mitigate the blast wave loading developed behind the wall. It is important to realise, however, that partial (or complete) wall failure of such a wall should not produce fragments that could in themselves cause damage to the building asset. However, only a limited amount of information is available on the nature of the blast loading environment behind a blast wall. And more accurate methods of predicting the effectiveness of these walls are needed.

There are several factors to be considered when designing a blast wall. For example, the positioning of the blast wall relative to both the likely threat location and the asset being protected is important in order to ensure that there is an effective mitigation of the blast resultants behind the wall. The threat face should be positioned relatively close to the most likely location of the explosive threat. If the asset to be protected must be close behind a wall (because of the physical constraints of the asset’s location, for example) this zone offers the best blast resultant mitigation. Of course, if the location allows for a
greater distance between the wall and the asset, advantage should be taken of this increased standoff distance and the attendant reduction in blast resultants.

Another factor is that different wall configurations will alter the flow of the blast over the barrier wall and could affect the size of the load reaching the structure. The blast environment behind the wall may be further altered by placing a canopy near the top of the wall. The use of a canopy assumes that the blast wave would be re-directed away from the protected target. While the canopy could be blown away by the force of the explosion, it would remain in place long enough to reduce the magnitude of the blast wave diffracting over the wall. Hence, the effectiveness of the use of a canopy was also considered in this study.

A simple approach to protecting important structures from blast loads is to improve a building’s perimeter security by means of blast proof perimeter walls acting as barriers against blast waves. However, research on blast wave propagation around a blast wall and the blast environment behind the wall is very limited. The TM5-1300 manual (UFC 3-340-02 Manual, 2008) shows how to calculate the enhancement pressure in the front and the reduction pressure to the side and rear of a structure. However, it contains no information about the magnitude of the blast pressure if there is an obstacle between the building and the detonation point.

The U.S. Army Security Engineering Manual (TM5-853-3) coupled with UFC 340-02 uses the adjustment factor approach (the ratio of pressure, or impulse, with the presence of a blast wall to the pressure, or impulse, without a wall) to calculate the reduction in pressure or impulse behind a plane wall. But the TM5-853-3 manual is restricted to use by only the U.S. military. Moreover, the formulations in this manual are based on small scale tests conducted at the U.S. Army Engineer Waterways Experiment Station in the 1980s (Dove, Hamilton, Coltharp, 1989). And the formulations were never validated with Full-Scale experiments. The formulations in these manuals are for spherical charges initiated at their center with an emphasis on mid-field and far-field overpressures. Since the charge shapes selected by terrorist organizations will be unknown and could be non-
spherical (cylindrical or cuboid), and these shapes could have different orientations and detonation points within the explosives, they will produce distributions of the overpressures that are different from those of a spherical charge of the same size. This is especially true in the immediate vicinity of the actual detonation. Also the manual does not consider the blast reflection due to the vicinity of other structures, the street elevation or the structure’s geometry. Hence, these manuals are not suitable for use in this study. However, the results of the experiments, or the FE codes such as LS-DYNA, can be used to assess the overpressure of the detonations of the non-spherical charges and the blast wave reflection due to the geometry of the structure or that of the adjacent structures.

Another problem concerns the accuracy of empirical equations proposed that diminishes as the standoff distance of the explosive reduces. Moreover, the aforementioned equations were generated from experimental studies with a limited range of test conditions (i.e. a blast on single obstacle in an open flat terrain). To deal with a wider range of blasts with more specific structural layouts and configurations, additional field tests are needed. Such studies may produce new design concepts for implementation in new structures and/or feasible economic of retrofit methods that can be applied to existing structures.

5.2 Importance of Research

The majority of the governmental and residential buildings in Saudi Arabia are surrounded by perimeter walls constructed by either RC walls or unreinforced masonry walls. Most of these walls have a height of 2 m and are located 2 m to 4 m from the street and because increasing the height of the walls is not allowed. Hence, another method is needed to reflect as much as possible of the blast wave and hence reduce the strength of the impact along the full height of the building to be a protected.

To date, based on the information provided in the available literature, most research programs have focused on understanding the effectiveness of plane rigid walls using small scales to reduce the blast wave resultants on a building located behind it. However, there are few studies that have experimentally, or numerically, investigated the strength
of the resistance of concrete and masonry structures to blast loading. Moreover, very limited, or no testing has been performed on RC blast walls with different shapes, which have been subjected to a blast from a shaped explosive. Also the manner in which pressures vary with time and position along the height and around the edges of these types of members was not well understood.

To understand how a blast pressure wave interacts with a barrier wall, one must first understand dynamic of a free-field blast pressure wave. Hence, the supersonic detonation within a high explosive forms gases, which undergo violent expansion. This expansion causes the surrounding layer of air to compress and form a blast wave. The blast wave which, follows the detonation shock wave, produces a high pressure wave front that expands out from the explosive charge. The wave is followed by a negative pressure trough, which forms before the air resumes its natural equilibrium at atmospheric pressure (Johansson and Persson, 1970 and Smith and Hetherington, 1994).

The primary goal here is simply to determine the best means of protecting the occupants of a building located behind an RC wall. Explosive attacks produce a significant amount of fragmentation, which in turn leads to significant human casualties. Therefore, by reducing the amount of fragmentation of a structure that results from various blast loads, the amount of harm to the occupants can also be reduced. The primary steps in this work are: (1) determining the degree of blast pressure as a function of the location and magnitude of the explosion; (2) determining how the structure would react to the blast pressure; (3) determining how the blast waves would be reflected with different configurations of the RC walls, and investigating the phenomenology of the blast wave propagation.

In this present study, 2 m, 6 m, and 9.65 m from the barrier wall to the building are considered as well as 2 m distance from the explosive to the barrier wall is considered. The charge weights are varied from 0.45 kg to 14 kg.

The selection of the wall type depends on the:

- Weight of the explosives
Blast walls subjected to blast loads are designed to ensure adequate shear strength so that flexure is the controlling mode of failure. In flexure, reinforced concrete structural components that are properly detailed possess good ductility, while in shear, failures can occur in a brittle component. Thus, it is desirable to have flexure be the controlling mode of failure. However, there is little research on blast-wave propagation along the facades of protected buildings located behind blast walls.

Half-scale experiments are carried out here to study the effect of erecting reinforced concrete blast wall in changing the blast wave propagation and distribution of blast pressure on protected buildings located behind the wall. Phase II of this research aimed to characterize the structural loads acting on the target building subjected to the detonation of a close-in explosive. In general terms, this research was intentionally done to shed light on the performance of existing buildings when subjected to blast loads arising from close-in explosions, and particularly to understand the effectiveness of erecting a perimeter wall around important structures.

The results of this study will indicate how and to what extent wall shape will affect the amount of pressure created by the detonation of explosives in the region selected. Hence, the results of these experimental tests will provide a better understanding of how the façade of a target building, which is located behind deferent blast walls, would be affected by blast pressures originating from a particular charge weight, standoff distance, and distance between the charge and the building’s position. The results of the pressures along the height of the protected building are compared to those obtained in the absence of the perimeter wall. It is expected that this research will contribute to the existing literature and hopefully lead to the introduction of new design guideline recommendations for significant new buildings in Saudi Arabia.
5.3 Pressure behind Blast Wall Literature Review

The shock front, which a charge produces, travels along a contact surface and expands outward from the charge into the atmosphere. In a free-field application, this blast wave propagates along the surface until it is no longer supersonic. It behaves this way until a structure, such as a barrier wall, is introduced. The blast wave from a charge located at a particular standoff distance impacts the barrier wall, which, due to its size and construction, does not move. This configuration causes the blast wave to diffract over the barrier wall as illustrated in Figure 5.1. The wave is reduced for some distance behind the barrier wall before that distance becomes so large that the pressures are no longer affected by the wall (Smith and Hetherington, 1994 and Remennikov and Rose, 2007). The area where the wall can reduce the blast pressures is defined as the shadow area. The extent of this shadow area defines the effectiveness of the barrier wall with respect to pressure reduction. This is the effect of blast barrier walls that was investigated by the research leading to this thesis.

Figure 5.1: Blast Wave Diffraction over a Barrier Wall (Remennikov and Rose, 2007)

The effect of barrier walls on blast pressures has been studied prior to this investigation. The Army Corps of Engineers’ Geotechnical and Structures Laboratory published a
report on the research conducted to date on a blast table using Composition-4 (C-4) explosive charges, and various wall heights and standoff distances (Rickman and Murrell, 2004). The USACE investigated how the pressures produced by an explosive charge were affected by a blast barrier wall and the effectiveness of the ConWep program in predicting the pressure reduction caused by the barrier wall.

The ConWep software was used to calculate the blast effects of conventional weapons, and in the case of the USACE report, it was found that the ConWep software could effectively calculate blast pressures and pressure reduction caused by a barrier wall. ACE also concluded that the maximum effect of the barrier wall was produced at the shortest standoff distance and with the greatest wall height.

While ConWep, the conventional weapons effects software created and maintained by the USACE Protective Design Center, was not used for this study; USACE suggests that it could be used to predict the peak pressures produced over a barrier wall. ConWep is based on charts appearing in Army Technical Manual (TM) 5-855-1, Fundamentals of Protective Design for Conventional Weapons, which contains information on the structural response to conventional weapons. Another technical manual, TM 5-853-3, Security Engineering Final Design, contains the details of methods to be used for calculating pressures and impulses created behind a barrier wall; however, the circulation of these technical manuals is restricted and both have limited availability (Remennikov and Rose, 2007). The formulations applied in this manual are based on small scale tests conducted at U.S. Army Engineer Waterways Experiment Station in the 1980s (Dove, Hamilton, Coltharp, 1989). However, these formulations were never validated with Full-Scale experiments. The formulations applied in these manuals are for spherical charges initiated at its center with an emphasis on mid-field and far-field overpressures. No consideration is given in the manual to the reflection due to the vicinity of other structures, the structural geometry, or the charge shape.

Zhou and Hao (2006) also described numerically the blast loading of structures located behind a barrier wall. Their study introduced formulae based on empirical results of pressures being exerted on a rigid building located behind a barrier to predict the peak
reflected pressure and impulse. These formulas can be used together with TM-1300, Structures to Resist the Effects of Accidental Explosions, to estimate the impulse and the pressure on buildings located behind a barrier wall.

Several barrier configuration studies of blast propagation on protected structure have been published. One of these was on an experiment conducted by Chapman et al. (1995a) on one-tenth scale target structures protected by blast walls from the detonation of high explosive charges placed slightly above the ground level. The various effects of several different geometrical parameters on resulting blast pressure were investigated. The parameters observed were as follows: height of burst of the charge (HOB), height of the target, height of the blast wall, the stand-off distance between the charge and the blast wall, and the distance between the blast wall and the structure. It was discovered that as the distance between the charge and the blast wall and the height of the blast wall increased, the impact of the blast waves on a structure was attenuated by the presence of the blast wall resulting in smaller pressures being observed. However, the response to the variation in the structure's height was unclear.

Other small scale experiments on the blast barriers' functionality as a means of protection for structure were also carried out by Bogosian and Piepenburg (2002) and Rickman et al. (2006). Bogosian and Piepenburg (2002) focused merely on the type and width of the barrier while Rickman et al. (2006) reported solely on the effectiveness of on steel barrier walls of several heights placed at certain distances between the structure and the charge. These experiments showed that such blast barriers, even of a modest height, could significantly reduce the reflected pressure on the structure and provide a significant amount of shielding for a structure.

Zhou and Hao (2007) performed numerical simulations to study the effectiveness of plane blast barriers for blast load reduction. It was found that a barrier erected between an explosion and a building could not only reduce the peak reflected pressure and impulse on the surface of the building, but also it could delay the arrival time of the blast wave. The effectiveness of a blast barrier in reducing the blast pressure on structures located behind a barrier depends not only on the barrier height, the distance between the
explosion centre and barrier, the distance between the barrier and structure, but also on the structure’s height. Based on their numerical results, approximate formulae have been derived to estimate the reflected pressure-time history on a structure located behind a barrier. Those formulae, used together with other available empirical methods (such as TM5-1300) provide a simple and reliable means of estimating blast loading on building located structures behind a blast barrier.

The aforementioned experimental studies involved field tests of a limited number. And the study constraints could be due to limitations in terms of the time and budget available. Moreover, most of the studies presented in the literature did not provide full information regarding the dimensions of the structures. That might be related to the need for confidentiality of blast data common to studies conducted for military purposes.

5.4 Design Guidelines

5.4.1 Safe scaled distance

Scaled distance, which is used in some regulations such as those appearing in the U.S. Dept. of Defense US DoD (2004) guidelines, was used as the base parameter in assessing the structural safety of buildings in resisting airblast loads; whereas other guidelines such as those of ASCE (1997) were used as source of pressure–impulse (P–I) diagrams to estimate structural damage against blast loads. The U.S. DoD (2004) specifies a safe scaled distance of $4.46 \, m/kg^{1/3}$ for unstrengthened buildings to ensure that a building is not destroyed. The scaled distances given in these regulations were usually obtained from field blasting tests conducted on scaled structural models, or on low-rise residential structures. The downside of these guidelines is that the effects of the use of various structural materials and different configurations are not taken into consideration, in spite of the fact that they affect structural performance significantly. Moreover, the definition of structural damage is vague in these guidelines.
The ASCE (1997) P–I diagrams indicate that the scaled distance suitable to prevent structural collapse against 1,000 kg TNT blast loads is $4.65m/kg^{1/3}$. However, these codes do not explicitly define the structural types and their compositions (masonry structure, masonry infilled RC frame, or a steel structure). They also do not indicate if the structure is a low-rise, medium-rise or high-rise building. Moreover, the explicit descriptions of the damage scenario for the structures under blast loads at different scaled distances are not given in the codes. Also, the code definition of a structure approaching destruction, or “structural collapse”, is not clear. It could be the total collapse of the structure, or the collapse of the infilled masonry walls, or the partial collapse of masonry walls.

The range to effects chart provided by FEMA 428 (2008) shows a generic relationship between the weight of the explosive threat and its distance to an occupied building. These generic charts are for conventional construction. However, these distances are so site-specific that the generic charts provide little more than general guidance in the absence of more reliable site-specific information. Based on the information provided in the chart, the onset of column failure is associated with stand-off distances in the order of tens of feet.

Zhou and Hao (2008) conducted a numerical analysis of a typical unretrofitted reinforced concrete wall, which was subjected to different blast loads. The objective of their study was to obtain a range-to-effect chart similar to that of another research body in (FEMA 428, 2008) for RC walls. According to FEMA 428 (2008), the explosive type of a charge used in a terrorist bombing can be divided into the following four subcategories: luggage bomb, automobile bomb, vans bomb and truck bomb. According to the numerical results presented, critical curves and analytical formulae, which were related to different damage conditions, were proposed from which safe stand-off distances for different terrorist bombing scenarios were determined. The critical charge weight to stand-off distance relationships are shown in Figure 5.2. The three curves used are only suitable for standoff distances ranging between 2 m and 20 m. The threshold line for the failure of concrete columns of FEMA (2008) was also given for comparison. These results were obtained
based on the particular type of RC wall analysed in their study. Hence, more analyses are needed on RC walls of different dimensions, different reinforcement ratios, and under different boundary conditions in order to derive a more general formula that can be used for a quick assessment of a RC wall’s resistance to blast loads.

Figure 5.2: Critical Charge Weight-Standoff Distance for Different Damage Level (Zhou and Hao, 2008)

Further research conducted by Wu and Hao (2007a) was done to fill in this gap for concrete constructions. Wu and Hao (2007a) developed an improved approach based on the U.S. DoD Code, which defined various performance levels, including collapse. Besides charge weight and standoff distance, structural materials and configurations are also two important parameters. However, some tests conducted by Baylot et al. (2005) showed that by increasing the charge weight or decreasing the stand-off distance other types of damage could be observed in addition to building collapse. These included cracks, catastrophic breaching as well as the production of both low and high velocity debris. Therefore, the development of new guidelines covering major damage levels for retrofitted RC and masonry walls is necessary.
The numerical results presented by Wu and Hao (2007) allow for a quick assessment of the safety of RC frame structures with masonry infilled walls with respect to explosions at different scaled distances. Their results showed that, when the scaled distance is less than about $1.82 \frac{m}{kg^{1/3}}$, the collapse of low-rise structures can be expected, whereas medium-rise structures will collapse when the scaled distance is less than about $1.18 \frac{m}{kg^{1/3}}$. The safe stand-off distance, then, to prevent excessive damage to a low-rise structure against a blast load is $4.50 \frac{m}{kg^{1/3}}$, whereas it is $5.60 \frac{m}{kg^{1/3}}$ for medium-rise structures. In their study, the type of explosion considered was a surface explosion. And the study was limited to examining only the damage to RC frames with masonry infilled walls.

To achieve a more general understanding of blast effects on structures, further study is needed to more accurately predict structural performance after various strengthening measures, and the use of other construction materials have been examined. A study to predict the degree of fragmentation from infilled masonry walls and the amount of glazing necessary for personnel protection is also needed. Therefore, the development of guidelines covering major damage levels for retrofitted masonry walls is necessary, but due to a lack of experimental data, more research is required to achieve this goal.

### 5.4.2 Blast Performance Criteria

Blast performance of reinforced concrete structures is typically measured as a function of maximum displacement $\delta_{max}$. Three such criteria exist. One, the support rotation $\theta$, which can be calculated based on the maximum rotation of the supports of a flexure member corresponding to the maximum member deflection and is typically reported in degrees. Two, the ductility ratio, $\mu = \delta_{max}/\delta_y$ which is the ratio of the maximum member displacement to the member yield displacement $\delta_y$. And three, the deflection ratio $\delta_{max}/L$, which is the ratio of the maximum member displacement to span length, $L$ and is typically reported as a percentage. UFC 03-340-02 (2008) suggests that blast resistant structures should be designed according to the degree of support rotation, while ASCE (1999) suggests that performance should be measured in terms of the deflection
ratio. These performance criteria are summarized in Table 5.1. Those criteria defined in UFC 03-340-2 may be considered conservative for design purposes, while those described by ASCE are empirical and are less suited for design purposes (ASCE, 1999).

Table 5.1: Blast Performance Criteria for Reinforced Concrete Flexure Members (ASCE, 1999)

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<tr>
<td></td>
<td>Displacement Ratio, $\frac{\delta}{L}$</td>
<td>Support Rotation, $\theta$</td>
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</tr>
<tr>
<td>Light/Low</td>
<td>4%</td>
<td>20$''$</td>
<td></td>
</tr>
<tr>
<td>Moderate/Medium</td>
<td>8%</td>
<td>60$''$</td>
<td></td>
</tr>
<tr>
<td>Severe/High</td>
<td>15%</td>
<td>120$''$</td>
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* Reinforcing Ratio, $\rho > 0.5$ %/face

Depending upon the magnitudes of the blast output and the amount of permissible deformations, either one of the following three types of reinforced concrete cross sections can be utilized in the design or analysis of blast resistant concrete walls as discussed in UFC 03-340-02 (2008):

Type I - The concrete is effective in resisting moment. The concrete cover over the reinforcement on both surfaces of the element remains intact.

Type II - The concrete is crushed and is not effective in resisting moment. Compression reinforcement equal to the tension reinforcement is required to resist moment. The concrete cover over the reinforcement on both surfaces of the element remains intact.

Type III - The concrete cover over the reinforcement on both surfaces of the element is completely disengaged. Equal tension and compression reinforcement which is properly tied together is required to resist moment.

Elements designed using the full cross section (Type I) are usually encountered in those structures or portions of structures designed to resist the blast output at the far field design range. This type of cross section is utilized in elements with maximum deflections corresponding to support rotations of less than 2 degrees. The maximum strength of an element is obtained from a Type I cross section. Type I elements may be reinforced on
either one or both faces. However, due to rebound forces, reinforcement is required on both faces of an element. Crushing of the concrete cover over the compression reinforcement occurs in elements that undergo support rotations greater than 2 degrees. This failure results in a transfer of the compression stresses from the concrete to the compression reinforcement which, in turn, results in a loss of strength. Sufficient compression reinforcement must be available to fully develop the tension steel (tension and compression reinforcement must be equal). Elements which sustain the crushing of the concrete without any disengagement of the concrete cover are encountered in structures at the far design range when the maximum deflection conforms to support rotations greater than 2 degrees but less than 6 degrees. Although the ultimate strength of elements with Type III cross sections is no less than that of elements with Type II cross sections, the overall capacity to resist the blast output is reduced. The spalling of the concrete cover over both layers of reinforcement, caused by either the direct transmission of high pressures through the element at the close-in range or the large deflections at the far range, produces a loss of capacity due to the reduction in the concrete mass.

The ultimate dynamic strength of reinforced concrete sections may be calculated in accordance with the ultimate strength design methods of the American Concrete Institute Standard Building Code Requirements for Reinforced Concrete (ACI Building Code). The safety or reliability of the protective structure is inherent in the establishment of the magnitude of the blast output for the donor charge, and in the criteria specified for deflection, support rotation, and fragment velocity. Because limited tests have been conducted to determine the response of lightweight concrete elements designed for close-in and far-distance design ranges, the pertinent formulae for this type of concrete are not included in the manual. Light-weight concrete may be utilized, but the reduction in mass from conventionally weighted concrete must be accounted for in the design to maintain the blast resistant capacity of a structure.

Consequence upon the calculation of blast loads, damage levels may be estimated by explosive analysis and Engineering analysis. Table 5.2 illustrate the incident pressures at which damage may occur.
Table 5.2: Damage Approximations (UFC 03-340-02, 2002)

<table>
<thead>
<tr>
<th>Damage</th>
<th>Incident Overpressure (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Typical window glass breakage</td>
<td>0.15 - 0.22</td>
</tr>
<tr>
<td>Minor Damage to some buildings</td>
<td>0.50 - 1.10</td>
</tr>
<tr>
<td>Panels of sheet metal buckled</td>
<td>1.10 - 1.80</td>
</tr>
<tr>
<td>Failure of concrete block walls</td>
<td>1.80 - 2.90</td>
</tr>
<tr>
<td>Collapse of wood framed buildings</td>
<td>Over 5.0</td>
</tr>
<tr>
<td>Serious damage to steel framed building</td>
<td>4.0 - 7.0</td>
</tr>
<tr>
<td>Severe damage to reinforced concrete structures</td>
<td>6.0 - 9.0</td>
</tr>
<tr>
<td>Portable total destruction of most buildings</td>
<td>10 - 12</td>
</tr>
</tbody>
</table>

5.4.3 Cantilevered Concrete Blast Wall Design

In this section, consideration is given to the design of a cantilevered concrete blast wall to resist the blast threat scenario of 80 kg TNT at a 4 m standoff distance. The design was carried out by following the design example given in Cormie et al. (2009). The height of the concrete blast wall was 2 m. It was symmetrically reinforced using grade 460 steel for flexural reinforcement and grade 250 steel for shear reinforcement. The reinforcement ratio of 0.5% (ρ_s) was chosen for the concrete blast wall and the concrete grade was 40 MPa. The density of the concrete was assumed to be 2400 kg/m³. The peak reflected impulse on the concrete blast wall obtained from A.T.BLAST was 4217.4 KPa-msec. The dynamic design stress for the flexural reinforcement was provided by Cormie et al. (2009) as,

\[
f_d = f_{dy} + (f_{du} - f_{dy})/4
\]

\[
= 1.2 f_y + (1.05 f_u - 1.2 f_y)/4
\]

\[
= 1.2 \times 460 + (1.05 \times 550 - 1.2 \times 460)/4
\]

\[
= 560 \text{ MPa}
\]

The ultimate resistance of the concrete blast wall was,

\[
r_u = \frac{2\rho_s f_{ds} d_c^2}{H^2}
\]
\[ \frac{2 \times 0.005 \times 560 \times 10^{6} d_{c}^{2}}{2^{2}} = 1400 \times 10^{3} d_{c}^{2} N/m^{2} \] (5.2)

For protection Category 2, the maximum displacement of the cantilevered concrete blast wall was limited to,

\[ X_{m} = H \tan 4^\circ = 2 \tan 4^\circ = 0.14 \text{ m} \] (5.3)

The Young's modulus of the concrete and the steel was 28 GPa and 200 GPa, respectively. The ratio of Young's modulus of steel to concrete was 6.67. From Figure 5.8 in Mays and Smith (1995), the second moment of area of the concrete blast wall was,

\[ I = 0.023 b d_{c}^{3} \]

The elastic flexural stiffness of the concrete blast wall was,

\[ K_{E} = \frac{8EI}{H^{4}} \]

\[ = \frac{8 \times 30 \times 10^{9} \times 0.023 \times 1 \times d_{c}^{3}}{2^{4}} \] (5.4)

\[ = 34.5 \times 10^{7} d_{c}^{3} N/m^{2}/m \]

The maximum elastic displacement was,

\[ X_{E} = \frac{r_{u}}{K_{E}} \]

\[ = \frac{1400 \times 10^{3} \times d_{c}^{2}}{34.5 \times 10^{7} d_{c}^{3}} \] (5.5)

\[ = \frac{40.6 \times 10^{-4}}{d_{c}} m \]
Hence, the basic impulse equation that can be used to determine the required concrete depth,

\[ I = \frac{2K_{LM}m}{2K_{LM}m} = r_u \left( X_m - \frac{X_E}{2} \right) \]  

(5.6)

Where I is the impulse on the concrete blast wall, \( K_{LM} \) is the load mass factor, \( m \) is the mass per unit area of the wall, and \( X_m \) is the maximum displacement of the wall. The load mass factor \( (K_{LM}) \) for the cantilevered wall subjected to uniform loading is 0.66.

For \( i = 4217 \) N-s/m\(^2\) we obtain,

\[ 4217^2 = 2 \times 0.66 \times 2400d_c \times 1400 \times 10^3 d_c^2 \times \left( 0.14 - \frac{40.6 \times 10^{-4}}{2d_c} \right) \]  

(5.7)

And when simplified gives,

\[ 621 \times 10^6 d_c^3 - 9 \times 10^6 d_c^2 = 17.8 \times 10^6 \]

Solving the above equation gives \( d_c = 0.31 \)

The flexural reinforcement required on each face was,

\[ A_s = 0.005 \times 1000 \times 310 = 1550 \text{ mm}^2/m \]  

(5.8)

Therefore, the use of T20 bars at 200 mm centers on each face. Hence, the overall section thickness using a 40 mm cover was,

\[ T_C = 40 + 8 + 310 + 8 + 40 = 406 \text{ mm}, \text{ Therefore 400 mm is chosen.} \]  

(5.9)

\[ t_m = i/r_u \]  

(5.10)

Now, \( r_u = 1400 \times 10^3 \times 0.31^2 = 143360 \text{ N/m}^2 \), hence

\[ t_m = \frac{4217}{143360} = 0.0294 \text{ s} = 29.4 \text{ ms} \text{ Which gives} \]

\[ t_m/t_d = 29.4/8.62 = 3.42 \geq 3 \]  

(5.11)
Therefore, the impulsive loading design is valid.

The ultimate shear stress at distance $d_c = 0.31\text{m}$ from the support was given as,

$$v_u = \frac{r_u(H-d_c)}{d_c} \text{ (Table B.5, Mays)}$$

$$= \frac{143360(2-0.32)}{0.32} = 752640 \frac{N}{mm^2} = 0.75 \frac{N}{mm^2}$$

The shear capacity of the concrete is determined based on Canadian building code.

$$\frac{100A_s}{bd_c} = 0.5 , f_{cu} = 40 \frac{N}{mm^2} \quad (5.13)$$

Therefore, $v_c = 0.68 \frac{N}{mm^2}$

The design shear stress was chosen from the maximum between $(v_u - v_c) = 0.07 \frac{N}{mm^2} \times (0.85v_c = 0.58 \frac{N}{mm^2})$. \quad (5.14)

The dynamic yield stress of grade 250 shear reinforcement is,

$$f_{ds} = f_{dy} = 1.1 \times 250 = 275 \frac{N}{mm^2} \quad (5.15)$$

The required area of stirrups of width $b = 150 \text{mm}$ and spacing ($s$) of $300 \text{mm}$ is,

$$A_v = \frac{(v_u - v_c)bs}{f_{ds}}$$

$$A_v = \frac{0.58 \times 150 \times 300}{275} = 95 \text{mm}^2 \quad (5.16)$$

The shear reinforcement provided is R10 stirrups at 300 mm centers ($78.6 mm^2$). (The design of the diagonal bars at the support is ignored in this study). The details of the reinforcement arrangement for the designed concrete blast wall are illustrated in Figure 5.3.
Figure 5.3: Reinforcement Detail in the Concrete Blast Wall Design to Resist Blast Loading of 80 kg at a 4 m Standoff Distance

5.5 Blast Scaling Laws

The similarity method has been discussed by Harris and Sabin (1999), in their book. There are four components required of similitude requirements for similarity method to relate a reduced scale model to the prototype (full scale) structure, i.e. dimensional and similarity analyses, geometric as well as loading and resistance conditions.

Hopkinson (1915)-Cranz (1926), or cube rooting scaling, is a common and useful way to describe blast wave properties. Blast wave scaling applies when two explosive charges of similar geometry and type, but of different sizes, are detonated in the same atmosphere. A brief summary of the relationship between the scale-model (model dimension=scale factor S times full-scale dimension) and the full-scale parameters is given in Table 5.3.

Table 5.3: Scaling Relationship

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Full-Scale Value</th>
<th>Scale-Model Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dimension</td>
<td>x</td>
<td>SX</td>
</tr>
<tr>
<td>Area</td>
<td>A</td>
<td>$S^2A$</td>
</tr>
<tr>
<td>Volume</td>
<td>V</td>
<td>$S^3V$</td>
</tr>
<tr>
<td>(Mass)</td>
<td>W</td>
<td>$S^3W$</td>
</tr>
<tr>
<td>Charge standoff</td>
<td>R</td>
<td>SR</td>
</tr>
<tr>
<td>Scaled standoff</td>
<td>$Z=R/W^{1/3}$</td>
<td>$Z=SR/SW^{1/3}$</td>
</tr>
<tr>
<td>Pressure</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>Loading duration</td>
<td>$t_d$</td>
<td>$St_d$</td>
</tr>
<tr>
<td>Impulse/unit area</td>
<td>I</td>
<td>SI</td>
</tr>
<tr>
<td>Velocity</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>Acceleration</td>
<td>a</td>
<td>a/s</td>
</tr>
</tbody>
</table>
To illustrate the blast scaling laws, consider a spherical charge as shown in Figure 5.4, where the distance from the charge center to a point of interest is \( R \) and the charge diameter is \( d \). The explosive mass to be detonated is \( W \), which is proportional to \( d^3 \). The distance can then be scaled from factor \( S \) according to

\[
S = \frac{R_2}{R_1} = \frac{d_2}{d_1} = \frac{\sqrt[3]{W_2}}{\sqrt[3]{W_1}}
\]

This equation may be rewritten to relate each position to the other according to

\[
\frac{R_1}{\sqrt[3]{W_1}} = \frac{R_2}{\sqrt[3]{W_2}}
\]

There is also a constant expression relating the standoff distance, \( R \), to its own weight, \( W \), between point A and B in Figure 5.4 This is known as the scaled distance, \( Z \) and its unit is, \( m/kg^{1/3} \) for explosives, which is written in general form as

\[
Z = \frac{R}{\sqrt[3]{W}}
\]

Where \( R \) is the standoff distance from the center of the explosive source to the target and \( W \) is the mass of the explosive. Although \( Z \) is called a scaled distance, this is incorrect because it is really the parameter of the dimension length divided by the cube root of the weight. However, the cube root of the weight is proportional to the cube root of the volume and thus to a characteristics size of \( D \). The scaled distance is thus proportional to a non-dimensional distance \( R/D \).

Figure 5.4 shows a schematic comparing the pressure time histories arising from two charge size standoff scenarios. Since \( W^{1/3} \) is directly proportional to the charge distance \( d \), both scenarios possess the same scaled distance, and both produce self-similar blast waves that can be scaled with respect to one another. In Figure 5.4 the same overpressure is achieved at positions A and B.
To illustrate Hopkinson’s scaling numerically, assume a charge weight of 1000 kg at a distance of 100 m from a specific target. The pressure produced by this scenario is the same as that produced by a charge weight of 125 kg at a distance of 50 m, and is the same as that produced by a charge weight of 1 kg at a distance of 10 m. However, for each of the above scenarios, the duration of the positive phase is different, and hence the magnitude of the impulse is different. The magnitude of the impulse increases with each increase in the charge weight.

### 5.6 Experimental Program

In this experiment, half scale RC cantilever walls were tested against a close in explosion. And the performance of the RC walls with different cross-sectional shapes was investigated. The design of the reinforced concrete walls and of the non-responding columns was chosen to represent the target buildings as well as the construction of the specimens; the test matrix and the test setup are presents in this section. All of the instrumentation, construction, and testing was conducted on the firing range at the SSF test site in Riyadh.
5.6.1 Test Specimen

A total of six half-scale explosive field tests were conducted in order to discover the most effective shapes for blast mitigating barrier walls of several cross-sectional shapes. In addition to a plane vertical wall, one of a T-shape and one of Y-shape all with and without an opening were tested. The scale model RC wall represented a full-scale wall 4 m wide, 2 m high, and of a 0.4 m thickness. Accordingly, the half-scale model measured 2 m wide, 1 m high and 0.2 m thick was tested in this study. A ready mix concrete was used to construct the test specimens and since the wall specimen was a half scale model, the maximum aggregate size scaled down from 20 mm to 10 mm. Normal concrete was used which had a compressive strength of 40 MPa and a 10 mm slump.

The wall was reinforced on both the front and rear faces with 10 mm diameter steel at 200 mm spacing center to center in both directions and with a clear concrete cover on all sides that was 20 mm thick. Standard deformed Grade 60 reinforcing bars with strength of 420 MPa were specified for all reinforcements. Running reinforcing steel along both faces of the wall was necessary due to dynamic action effects in which these structures might rebound due to loading reversal. The shear reinforcement provided was 10 mm @ 200 mm.

The foundation was not the focus of attention as long as it could impart fixity at the base of the wall. The construction and curing of all the walls and footings were carried out at the SSF main site. The foundation of each wall was first cast. Then the required amount of longitudinal reinforcement, which projected from the foundation, was introduced to ensure the continuity of the wall’s longitudinal reinforcement at the wall’s base, and to provide adequate resistance against direct shear failure when acted upon by the blast. Figure 5.5 shows the formwork and of the reinforcing process.
Figure 5.5: Construction of RC Wall and Reinforcement and Formwork

The second stage was to construct the walls as shown in Figure 5.6. The walls were cast with a concrete pump truck and consolidated with a hand vibrator, as shown in Figure 5.6. The walls were cured 28 days before the actual field tests and cubic samples were taken as shown in Figure 5.6 to verify the compressive strength.
A heavy duty crane and a trailer truck were needed to lift and transport the specimens to the test site. Hooks were installed in the specimens to facilitate their transportation. Figure 5.7 shows the transportation, installation, and positioning of the specimens.

Figure 5.6: Process of Wall Construction and Wall Casting and Wall Finishing
5.6.2 Test Matrix

Six half-scale blast walls of different shapes were erected in front of the target building and were tested using various close-in charge weights and a constant standoff distance to the blast wall. The walls measured at 2000 mm in width, 1000 mm in height, and had a thickness of 200 mm. The first objective was to observe each wall’s performance; for this purpose each wall was subjected to twelve successive shots, which increased from 0.45 kg to 14 kg until the wall began to fail. The standoff distance from the charge center to each wall was kept constant at 2 m. The explosive material used in all cases was ANFO, a mixture of ammonium nitrates and fuel oil. Table 5.4 shows the test matrix. Figure 5.8 and Figure 5.9 illustrated the dimensions of the walls.
The second objective of the tests was to assess the effectiveness of wall shapes to reduce the structural loads on the target building located behind a blast wall. The reflected pressures on the target building at four different heights both in the presence and the absence of a blast wall were measured that resulted from the detonation of a high explosive charge. First, total twelve shots were fired at the target building with there was no barrier wall between the explosive charge and the target building to serve as a baseline for the barrier wall study. Then a blast wall was erected between the explosive charge and the structure, and twelve shots were fired at the wall. Hopkinson or cube root scaling was applied to those physical dimensions. The scaled dimensions provided in terms of the actual models were divided by the cube root of the model’s explosive yield (m/kg$^{1/3}$), and were referenced to the equivalent explosive mass of TNT. The scaled parameters were varied so as to expand considerably the range of parameters addressed in TM5-853-3, as shown in Table 5.5.

### Table 5.4: Test Matrix.

<table>
<thead>
<tr>
<th>Wall No.</th>
<th>Wall Description</th>
<th>Charge Size, kg</th>
<th>Standoff, m</th>
<th>Scaled Distance, m/kg$^{1/3}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>PWS</td>
<td>Plane vertical wall solid</td>
<td>0.45 - 14</td>
<td>2</td>
<td>0.89-2.79</td>
</tr>
<tr>
<td>PWO</td>
<td>Plane vertical wall with opening</td>
<td>0.45 - 14</td>
<td>2</td>
<td>0.89-2.79</td>
</tr>
<tr>
<td>YWS</td>
<td>Y- vertical wall solid</td>
<td>0.45 - 14</td>
<td>2</td>
<td>0.89-2.79</td>
</tr>
<tr>
<td>YWO</td>
<td>Y- vertical wall with opening</td>
<td>0.45 - 14</td>
<td>2</td>
<td>0.89-2.79</td>
</tr>
<tr>
<td>TWS</td>
<td>T- vertical wall solid</td>
<td>0.45 - 14</td>
<td>2</td>
<td>0.89-2.79</td>
</tr>
<tr>
<td>TWO</td>
<td>T- vertical wall with opening</td>
<td>0.45 - 14</td>
<td>2</td>
<td>0.89-2.79</td>
</tr>
</tbody>
</table>

### Table 5.5: Range of Scaled Parameters Addressed

<table>
<thead>
<tr>
<th>Scale parameter</th>
<th>TM5-853-3 m/kg$^{1/3}$</th>
<th>Small-scale experiments, Rickman (2006), m/kg$^{1/3}$</th>
<th>Present study, Half scale experiments m/kg$^{1/3}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height of burst</td>
<td>-</td>
<td>0.00</td>
<td>0.20-0.68</td>
</tr>
<tr>
<td>Barrier wall height</td>
<td>0.32-0.52</td>
<td>0.29-1.50</td>
<td>0.41-1.35</td>
</tr>
<tr>
<td>Distance, charge to wall, R</td>
<td>0.20-1.20</td>
<td>0.072-6.90</td>
<td>0.89-2.79</td>
</tr>
<tr>
<td>Distance, wall to structure, D</td>
<td>0.40-8.00</td>
<td>0.24-11.90</td>
<td>0.87-16.24</td>
</tr>
</tbody>
</table>
Figure 5.8: Schematic of Blast Walls (Elevation View)
Figure 5.9: Schematic of Blast Walls (Front and Side view)
5.6.3 Test Setup

The test program included six-half scale blast walls of different shapes. Each wall was situated in front of non-respond round steel pipes mounted on a reinforced concrete foundation (i.e., essentially non-responding columns) represents the target buildings. Three columns were used and positioned at three different distances from the blast wall. This test setup allowed all of the three non-responding columns to be loaded simultaneously with the same explosive charge, which increased the accuracy of the comparison between the loadings on the three columns. The height of each column was 6 m and thus represented a half-scale model. The first column distance from the blast wall was 2 m; the second one was 6 m, and the third one was 9.65 m. Four reflected pressure transducers were embedded in a vertical array on the centre line of the target column at heights of 0.75 m, 2.25 m, 3.75 m, and 5.25 m to measure the reflected pressure both in the presence and in the absence of a blast wall placed in front of the target building.

On the test site, the foundation was buried in a pit, which was excavated to similarly position each wall; then each specimen was aligned using a lifting crane to its final position after which the excavation was backfilled and compacted as shown in Figure 5.10. Concrete block wings walls were erected around the RC wall in both sides to increase the width of the wall and to prevent the air blast pressure from wrapping around the wall.
A reinforced concrete slab was constructed in front of the blast wall and a replaceable steel plate with a thickness of 20 mm was positioned on the slab, which was laid under the charge to avoid excessive damage to the slab. This setup provided a fully reflective surface. The test setup is illustrated in Figure 5.11. Figure 5.12 shows the test site prior to one of the detonations.

**Figure 5.10: The Process to Position the Blast Wall**
Model-2 (Plane Blast Wall Solid)

Figure 5.11: Schematic of Blast Wall and Parameters Test Setup (Elevation View)

P1 = 0.75 m, P2 = 2.25 m, P3 = 3.75 m, P4 = 5.25 m

Figure 5.12: Test Site Prior One of the Detonations
Ammonium Nitrate – fuel oil mixture (ANFO) was used as the explosive material, and non-fragmenting cube wood containers of different dimensions were used based on the charge weight. Each charge was positioned 2 m away from the test wall on the front face and it was placed on a wooden table with the height being adjusted to position the center of the explosive at the mid height of the wall. Since this was a half-scale model, the charge was elevated to 0.5 m above the ground. The initiation point was fixed at the top of the charge. The charge was boosted by a suitable quantity of PETN explosive. Figure 5.13 illustrates the preparation of the ANFO charge.

![Preparing the ANFO charge](image.jpg)

**Figure 5.13: Preparing the ANFO charge**

The experimental program described herein was established with the aim of providing as wide a range of geometrical conditions for protecting structures from blast loads as possible. Variations in the distance from the blast wall to a target building and in the charge weight, \( W \), were examined, while the wall height, the charge to wall distance, the HOB, and the transducer configuration were kept constant.

The shock wave parameters were measured using a high speed data acquisition system which includes National Instruments data recording hardware and LabVIEW software, for the data collection and analysis, and a signal conditioner as well as sensors transducers and cables that are connected to the test specimen. The sampling rate was up
to 100 ks/s/channel. Reflected pressures were measured using a piezoelectric pressure sensor screwed onto the non-responding columns and their orientation was perpendicular to the blast wave. Sensors from PCB Piezotronics models 102B, 102B03, and 102B04 were utilized and the sensors were installed on non-responding columns that represented the target buildings to determine the reflected pressure along the height of the rigid column.

Blast wave characteristics, including reflected pressures were recorded. And the post-blast damage and the mode of failure of the blast walls were observed. Experimental observations were used to evaluate the wall’s performance and to determine the capacity and failure limit states of a concrete wall. In the case where there was no the wall, the blast characteristics were compared with those obtained using A.T.BLAST software to validate the results obtained from the experiments.

5.7 Test Results and Discussion

5.7.1 Post-Blast observations

Based on the analysis of the data collected during each test and upon the close inspection of the post-blast condition of each specimen, the following observations can be made. Shear cracking patterns were observed in all of the specimens on the front surface of the wall. Figure 5.14 shows the light shear cracking at the base of the plane wall both with and without an opening at scaled distance of 0.86 m/kg$^{1/3}$. There was no significant observable wall deformation or sign of local failure in the plane walls.
In the case of the presence of the canopy at the top of the wall there was significant deformations at the connection of the front face of the wall as shown in Figure 5.15. The test results indicate that the plane wall suffered less damage compared to the canopied wall when subjected to a blast at a scaled distance of 0.86 m/kg\(^{1/3}\). It was observed that the combination of the energy reflection and the energy absorption by the canopy on the wall mitigated the strength of the impact blast wave loading developed behind the wall because the canopy still remained in place and reduced the magnitude of the blast wave diffracting over the wall. Hence, the effectiveness of the use of the canopy is a significant finding for this study. Moreover, the existence of an opening in the wall can increase the reduction of the blast wave carried behind the wall. It should be noted, too, that damage to a perimeter wall is considered to be acceptable if a high level of protection is needed for a target building.
5.7.2 Observations and Comparison with Results of Other Authors

Previous studies focused on the effect of the blast wave propagation on rigid blast walls using small scale models. But experiments and calculations using shaped explosions with reflections are relatively rare in the scientific literature. Moreover, to the best of the author’s knowledge the effect of the erection of RC walls along with the introduction of different charge shapes on reducing the blast wave resultants on a target building located behind a wall was not investigated.

A cantilevered plane blast wall was designed to function as a base line in this study. The design was based on that provided in the UFC 340-02: Structure to Resist the Accidental Explosion manual. The blast wall was designed to resist a blast load with a scaled distance of 0.93 m/kg\(^{1/3}\).

The safe scaled distance set as the parameter in some regulations (US DoD, 2004 and ASCE, 1997) were used in assessing the structure’s ability to resist airblast loads. The values used in the manuals were for conventional structures without any blast hardening. In this study, the safe scaled standoff distance from the charge to the wall was 0.86 m/kg\(^{1/3}\).
m/kg$^{1/3}$ for the plane wall and 0.99 m/kg$^{1/3}$ for the canopied wall; these values were much smaller compared to the scaled distances of 4.46 m/kg$^{1/3}$ and 4.65 m/kg$^{1/3}$ specified in the U.S. DoD (2004) and ASCE (1997) manuals, respectively. It should be noted, too, that the effects of the use of different structural materials and the configurations are not taken into consideration in these guidelines. Moreover, the definition of structural damage is not clear in these guidelines.

Maximizing the standoff distance between a charge and a building is often the most effective way to reduce the blast effects on target structures. However, if the standoff distance is limited, the presence of a blast wall between the building and the assumed location of an explosion may be beneficial. The effectiveness of a blast wall is also highly dependent on the charge type and size as well as the distances between the explosives, the walls, and the buildings. Previous test data as stated in the ASCE/SEI 59-11 standard (2011) and addressed in the TM5-853-3 manual suggest that blast walls with heights of 0.3 m/kg$^{1/3}$ to 0.8 m/kg$^{1/3}$ will reduce the reflected pressure and the reflected impulse on building surfaces if they are located within a scaled distance of 1.2 m/kg$^{1/3}$ from the explosive and a scaled distance between 0.4 m/kg$^{1/3}$ and 8.0 m/kg$^{1/3}$ from the building. It is important to note that hardening is often more economical at a scaled distance of 4.0 m/kg$^{1/3}$. All of these values are referenced with respect to the TNT equivalent size of the explosive. The blast wall’s scaled height used in this current study was 0.44 m/kg$^{1/3}$ to 1.39 m/kg$^{1/3}$, the scaled distance from the charge to the blast wall was 0.86 m/kg$^{1/3}$ to 2.79 m/kg$^{1/3}$, and the scaled distance from the blast wall to the building was 0.86 m/kg$^{1/3}$ to 16.24 m/kg$^{1/3}$.

The U.S. Army Security Manual TM5-853-3 (DoD 1994) and UFC 340-02 use the adjustment factor approach (the ratio of pressure or impulse in case of the presence of a blast wall to the pressure or impulse in case of the absence of a blast wall) to calculate the reduction in pressure or impulse created behind a plane wall. However, the use of the TM5-853-3 manual is restricted to be used by only the U.S. military. The formulations used in this manual also are based on small scale tests as well as on formulations that were never validated with Full-Scale experiments. Moreover, the formulations used in
these manuals are for spherical charges initiated at the charge center with an emphasis on mid-field and far-field overpressures.

It is important to note here that shaped charges with different orientations and detonation points within the explosives will produce different distributions of the overpressures than will a spherical charge of the same size, especially in the immediate vicinity of the detonation. Also the manuals not consider the blast wave’s reflection due to the vicinity of other structures as well as the structural geometry. Hence, these manuals are not suitable to be used in this study. Alternatively, the results of the experiments or the FE codes, developed therein such as LS-DYNA can be used to assess the strength of the overpressure of the detonations of non-spherical charges and the degree of reflection due to the geometry of the structure or that of the adjacent structures.

The recently published American Society of Civil Engineers standards [ASCE/SEI 59-11 (ASCE 2011)], which are applied to the design and analysis of structures subjected to blast loadings use qualitative damage indicators to classify the performance of different structural components and the expected level of damage. Four damage levels are classified in this standard, Superficial (visible permanent damage is unlikely); Moderate (permanent damage might be visible but is repairable and component failure is unlikely); Heavy (significant damage, mostly non-repairable and component failure remains unlikely), and Hazardous (component fails). According to the qualitative damage descriptions appearing in this standard, the damage to the plane walls both with and without openings with a scaled distance of 0.86 m/kg$^{1/3}$ could be classified as Superficial as none of the walls exhibited permanent damage. For the canopy walls and at the same scaled distance (0.86 m/kg$^{1/3}$), the significant damage occurred at the top of the wall would be classified it under the Heavy Damage Category. Since the blast wall was erected to protect the target building located behind it, this level of damage to the canopy wall is considered to be acceptable because the combination of the energy reflection and energy absorption by the canopy walls can mitigate the blast wave loading developed behind the wall and the canopy will still remain in place and reduce the magnitude of the blast wave diffracting over the wall.
FEMA 428 (2008) charts show a generic interaction between the weight of an explosive threat and its distance from an occupied building. These generic charts are intended for use in conventional construction. However, these distances are so site-specific that the generic charts provide little more than a general guideline in the absence of more reliable site-specific information. Based on the information provided in the chart, the onset of column failure is associated with stand-off distances on the order of tens of feet. This type of chart can be utilized to demonstrate the blast response of structural members at diverse levels of protection. The maximum charge used in the current study was 14 kg and the standoff distance from the charge to the blast wall was 2 m and these values are out of the range of those given on the FEMA chart.

5.7.3 Pressure profile

Figure 5.16 shows the typical pressure-time histories captured by one of the reflected pressure gages after the detonation of 6.46 kg of ANFO at an 8.2 m standoff distance and an angle of incidence of 12.1°. Figure 5.16 also shows the typical impulse-time history which is calculated as the integration of the area of the pressure-time histories. They show the typical features of a pressure profile induced by the detonations of conventional explosive, i.e. zero time, exponential decay, positive phase, and negative phase. The measured peak reflected pressure is shown as 67.75 KPa compared to the 106.3 KPa obtained from the A.T.BLAST (difference is 36 %) and the corresponding impulse is 52.25 KPa-ms compared to 225.4 KPa-ms obtained from the A.T.BALST (difference is 77 %); these values were obtained where there was no blast wall between the charge and the target building.
Figure 5.16: Reflected Pressure and impulse Profiles Recorded by Pressure Transducer

The results of the experiments are compared with the numerical results obtained by the A.T.BLAST software and a comparison was made where there was no wall between the charge and the target building. The results were obtained at four points along the height of the target building. Tables 5.6- 5.7 show the comparison of the experiments and the numerical results for the reflected pressure and the impulse, respectively. The experimental results showed a good agreement between the measured and the numerical values for the reflected pressure while the program overestimated the reflected impulses.

Table 5.6: Reflected Pressure Comparison of the Computed and the Experiments

<table>
<thead>
<tr>
<th></th>
<th>R, m</th>
<th>θ</th>
<th>W, Kg</th>
<th>A.T.BLAST, kPa</th>
<th>EXPERIMENT, kPa</th>
<th>Difference %</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>8.20</td>
<td>1.750</td>
<td>14</td>
<td>200.60</td>
<td>148.86</td>
<td>26</td>
</tr>
<tr>
<td>P2</td>
<td>8.39</td>
<td>12.05</td>
<td>14</td>
<td>189.75</td>
<td>131.20</td>
<td>31</td>
</tr>
<tr>
<td>P3</td>
<td>8.82</td>
<td>21.62</td>
<td>14</td>
<td>168.86</td>
<td>120.04</td>
<td>29</td>
</tr>
<tr>
<td>P4</td>
<td>9.48</td>
<td>30.10</td>
<td>14</td>
<td>142.80</td>
<td>124.11</td>
<td>13</td>
</tr>
</tbody>
</table>
Table 5.7: Reflected Impulse Comparison of the Computed and the Experiments

<table>
<thead>
<tr>
<th></th>
<th>R, m</th>
<th>θ</th>
<th>W, Kg</th>
<th>A.T.BLAST, KPa-ms</th>
<th>EXPERIMENT, KPa-ms</th>
<th>Difference %</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>8.20</td>
<td>1.75</td>
<td>14</td>
<td>405.36</td>
<td>114.2</td>
<td>72</td>
</tr>
<tr>
<td>P2</td>
<td>8.39</td>
<td>12.05</td>
<td>14</td>
<td>389.10</td>
<td>99.15</td>
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<tr>
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<td>8.82</td>
<td>21.62</td>
<td>14</td>
<td>355.92</td>
<td>52.20</td>
<td>85</td>
</tr>
<tr>
<td>P4</td>
<td>9.48</td>
<td>30.10</td>
<td>14</td>
<td>314.80</td>
<td>109.56</td>
<td>65</td>
</tr>
</tbody>
</table>

To allow for an easy estimation and comparison of the blast pressure and impulse on the structures located behind a blast wall, adjustment factors for a peak reflected pressure of AF_p and for an impulse of AF_i were introduced. The adjustment factors represent the ratio of the pressure and impulse occurring behind the blast wall to its original value without the blast wall. Thus, an adjustment factor of 0.4 represents a 60% reduction in pressure and impulse occurring behind a wall compared to a no-wall configuration.

To study the effectiveness of a reinforced concrete blast wall on the blast pressure reduction on buildings of different wall shapes, half scale experiments corresponding to different standoff distances from the charge to the building were carried out and the results were compared. Typical ratios of the peak reflected pressure and impulse distribution along the height of different buildings are shown in Tables 8 to 13, respectively. The adjustment factors for both the peak-reflected pressure and the impulse are generally smaller, especially with the YWO-Shape.

Table 5.8: The Adjustment Factors (AF_p) at Standoff Distance 4.2 m

<table>
<thead>
<tr>
<th>Case #</th>
<th>Wall shape</th>
<th>AF (P1)</th>
<th>AF (P2)</th>
<th>AF (P3)</th>
<th>AF (P4)</th>
<th>AF (P1)</th>
<th>AF (P2)</th>
<th>AF (P3)</th>
<th>AF (P4)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>1</td>
<td>PWS</td>
<td>*</td>
<td>0.48</td>
<td>0.57</td>
<td>*</td>
<td>*</td>
<td>0.43</td>
<td>0.80</td>
<td>*</td>
</tr>
<tr>
<td>2</td>
<td>PWO</td>
<td>*</td>
<td>*</td>
<td>0.42</td>
<td>1.36</td>
<td>*</td>
<td>*</td>
<td>0.53</td>
<td>1.76</td>
</tr>
<tr>
<td>3</td>
<td>TWS</td>
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<td>*</td>
<td>0.38</td>
<td>1.57</td>
<td>*</td>
<td>*</td>
<td>0.64</td>
<td>0.59</td>
</tr>
<tr>
<td>4</td>
<td>TWO</td>
<td>0.35</td>
<td>*</td>
<td>0.50</td>
<td>2.25</td>
<td>*</td>
<td>*</td>
<td>0.61</td>
<td>1.69</td>
</tr>
<tr>
<td>5</td>
<td>YWS</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>2.17</td>
<td>*</td>
<td>*</td>
<td>0.47</td>
<td>1.55</td>
</tr>
<tr>
<td>6</td>
<td>YWO</td>
<td>0.28</td>
<td>*</td>
<td>0.72</td>
<td>0.89</td>
<td>*</td>
<td>*</td>
<td>0.45</td>
<td>0.86</td>
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</table>

*Instrumented gages failed
Table 5.9: The Adjustment Factors ($AF_p$) at Standoff Distance 8.2 m

<table>
<thead>
<tr>
<th>Charge (kg)</th>
<th>6.46</th>
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</tr>
</thead>
<tbody>
<tr>
<td>Case #</td>
<td>Wall shape</td>
<td>$AF$ (P1)</td>
</tr>
<tr>
<td>0</td>
<td>NW</td>
<td>1</td>
</tr>
<tr>
<td>1</td>
<td>PWS</td>
<td>*</td>
</tr>
<tr>
<td>2</td>
<td>PWO</td>
<td>0.81</td>
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<tr>
<td>3</td>
<td>TWS</td>
<td>0.56</td>
</tr>
<tr>
<td>4</td>
<td>TWO</td>
<td>0.82</td>
</tr>
<tr>
<td>5</td>
<td>YWS</td>
<td>0.61</td>
</tr>
<tr>
<td>6</td>
<td>YWO</td>
<td>0.61</td>
</tr>
</tbody>
</table>

Table 5.10: The Adjustment Factors ($AF_p$) at Standoff Distance 11.85 m

<table>
<thead>
<tr>
<th>Charge (kg)</th>
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<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case #</td>
<td>Wall shape</td>
<td>$AF$ (P1)</td>
</tr>
<tr>
<td>0</td>
<td>NW</td>
<td>1</td>
</tr>
<tr>
<td>1</td>
<td>PWS</td>
<td>0.83</td>
</tr>
<tr>
<td>2</td>
<td>PWO</td>
<td>0.66</td>
</tr>
<tr>
<td>3</td>
<td>TWS</td>
<td>0.86</td>
</tr>
<tr>
<td>4</td>
<td>TWO</td>
<td>0.80</td>
</tr>
<tr>
<td>5</td>
<td>YWS</td>
<td>0.66</td>
</tr>
<tr>
<td>6</td>
<td>YWO</td>
<td>0.60</td>
</tr>
</tbody>
</table>

Table 5.11: The Adjustment Factors ($AF_I$) at Standoff Distance 4.2 m

<table>
<thead>
<tr>
<th>Charge (kg)</th>
<th>6.46</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case #</td>
<td>Wall shape</td>
<td>$AF$ (P1)</td>
</tr>
<tr>
<td>0</td>
<td>NW</td>
<td>1</td>
</tr>
<tr>
<td>1</td>
<td>PWS</td>
<td>*</td>
</tr>
<tr>
<td>2</td>
<td>PWO</td>
<td>*</td>
</tr>
<tr>
<td>3</td>
<td>TWS</td>
<td>0.58</td>
</tr>
<tr>
<td>4</td>
<td>TWO</td>
<td>*</td>
</tr>
<tr>
<td>5</td>
<td>YWS</td>
<td>*</td>
</tr>
<tr>
<td>6</td>
<td>YWO</td>
<td>*</td>
</tr>
</tbody>
</table>

*Instrumented gages failed
Table 5.12: The Adjustment Factors (AF) at Standoff Distance 8.2 m

<table>
<thead>
<tr>
<th>Charge (kg)</th>
<th>6.46</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case #</td>
<td>Wall shape</td>
<td>AF (P1)</td>
</tr>
<tr>
<td>0</td>
<td>NW</td>
<td>1</td>
</tr>
<tr>
<td>1</td>
<td>PWS</td>
<td>*</td>
</tr>
<tr>
<td>2</td>
<td>PWO</td>
<td>0.61</td>
</tr>
<tr>
<td>3</td>
<td>TWS</td>
<td>0.78</td>
</tr>
<tr>
<td>4</td>
<td>TWO</td>
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</tr>
<tr>
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<td>YWS</td>
<td>0.69</td>
</tr>
<tr>
<td>6</td>
<td>YWO</td>
<td>0.59</td>
</tr>
</tbody>
</table>

Table 5.13: The Adjustment Factors (AF) at Standoff Distance 11.85 m

<table>
<thead>
<tr>
<th>Charge (kg)</th>
<th>6.46</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case #</td>
<td>Wall shape</td>
<td>AF (P1)</td>
</tr>
<tr>
<td>0</td>
<td>NW</td>
<td>1</td>
</tr>
<tr>
<td>1</td>
<td>PWS</td>
<td>0.83</td>
</tr>
<tr>
<td>2</td>
<td>PWO</td>
<td>0.59</td>
</tr>
<tr>
<td>3</td>
<td>TWS</td>
<td>0.98</td>
</tr>
<tr>
<td>4</td>
<td>TWO</td>
<td>0.18</td>
</tr>
<tr>
<td>5</td>
<td>YWS</td>
<td>1</td>
</tr>
<tr>
<td>6</td>
<td>YWO</td>
<td>0.51</td>
</tr>
</tbody>
</table>

The goal of the experiment was to measure the effectiveness of a reinforced concrete wall and assess its ability to reduce the structural loads on the rigid building face behind the barrier wall. Four reflective pressure gages were placed on the structure along the centerline of the structure at heights of 0.75 m (Target 1), 2.25 m (Target 2), 3.75 m (Target 3), and 5.25 m (Target 4). During the experiment, all data produced were captured. (There were some gages failures). The explosive threats were used 6.46 kg and 10.05 kg ANFO charges. The charge-to-barrier standoff distance was 2 m, and the charge-to-structure standoff distances were 4.2, 8.2 and 11.85 m.

Analysis of the test data generated allowed several general trends to be readily observed. First the largest reduction factors were found to be toward the lower part of the structure. As the standoff distances increased, there was often a kink point, which was found slightly above the base of the structure.
It can be noted that for the close range of standoff distance of 4.2 m, the blast wall significantly reduced the reflected pressure on the building surface. This effect was most significant for the lower part of the building and was less significant for the upper part of the building. A similar trend can be observed for the standoff distances of 8.2 m and 11.85 m.

### 5.8 Summary and Conclusions

In this study, half-scale experiments were carried out on RC blast walls of different shapes to study the effectiveness of a blast wall for blast wave reduction. A cuboid shaped explosive was used for all of the experiments. The charge was elevated 0.5 m above ground level and detonated above rigid ground. An ANFO charge was used and detonated at the top.

The experimental results for the measured reflected pressures and the computed corresponding impulses at the four different heights along the face of the target building were compared with the numerical values obtained by the A.T.BLAST software where there was there is no wall between the charge and the target building. The results showed good agreement for the reflected pressures with differences of between 13 % - 31 % while the A.T.BLAST software overestimated the reflected impulses with differences of between 65 % - 85 %. It should be noted that the formulation used in the A.T.BLAST software is based on the use of a spherical charge detonated at its center and the emphasis is put on mid-field and far-field overpressures while the charge used in the current study was based on cuboid charge detonated at the top of the charge.

It should be notated that the shape, the type and the orientation as well as the booster quantity and the initiation point of the charge can lead to a significant increase in the blast loading especially for the close-in range. Moreover, ANFO is a non-ideal explosive because its performance or release of energy is affected by many factors such as the mixing ratio of (AN) to (FO), the particle size, the moisture content and the degree of confinement.
As a base line for this study, a plane RC blast wall was designed based on the UFC-340-02 manual: The structure was designed to resist the blast load at a scaled distance of 0.93 m/kg\(^{1/3}\). The safe scaled distance used as the parameter in some regulations (U.S. DoD, 2004 and ASCE, 1997) in assessing a structure’s ability to resist blast loads and the values given in these regulations are for the conventional structures. It was found in this study that the safe scaled distance for plane blast walls was 0.86 m/kg\(^{1/3}\) while the safe scaled distance for canopied walls was 0.99 m/kg\(^{1/3}\). These values were much smaller compared to the 4.46 m/kg\(^{1/3}\) and 4.65 m/kg\(^{1/3}\) as specified in the U.S. DoD, 2004 and ASCE, 1997 manuals, respectively. It should be noted that these regulations do not clarify the effects of differences in the structural materials, the configurations or in the definition of the structural damage.

The damage level for the RC blast walls utilized in the current study was compared with the qualitative damage indicator provided in ASCE/SEI 59-11 (2011). The plane blast walls suffered less damage compared to the canopied walls at a scaled distance of 0.86 m/kg\(^{1/3}\). The damage to the plane blast wall can be classified as Superficial and that to the canopied walls classified under the Heavy Damage Category. Since the purpose of the blast wall is to protect the target building behind it, this level of damage is considered to be acceptable in the canopied wall because the combination of the energy reflection and the energy absorption by the canopied walls can mitigate the blast wave loading developed behind the wall and the canopy can still remain in place and reduce the magnitude of the blast wave diffracting over the wall.

However, the U.S. Army Security Manual TM5-853-3 (DoD 1994) and the UFC 340-02 are based on small scale experiments and the results were never validated in full-scale experiments. Also they can only be used for a rigid plane blast wall, with spherical charges being initiated at the center of the charge. Moreover, the TM5-853-3 manual is restricted to be used by only the U.S. military. The manuals also do not consider the shape of the explosive or the reflection due to the vicinity of other structures or the structure’s geometry. Alternatively, the results of the experiments or the FE codes such as LS-DYNA can be used to assess the overpressure of the detonations of the non-spherical
charges and the reflection due to the geometry of the structure or to that of the adjacent structures. The methodology used in this study was similar to that used in these manuals.

The efficiency of blast walls in reducing reflected pressure was clearly demonstrated by comparing the ratio of the measured reflected pressure or impulse with a blast wall in place to that for the corresponding charge-to-structure distance in the case where no wall is present. This ratio, or reduction factor, provides a direct measure of the effectiveness of the blast wall. The experimental results have demonstrated that changing the shape of the reinforced concrete wall can effectively reduce a blast load and therefore, protect structures from an external explosion. The maximum effectiveness of a blast wall (smallest values of the reduction factor) was obtained with a wall to the structure distance of 8.2 m. The T-Shape and the YWO-Shape were found to be the most effective shapes for mitigating the blast resultants.

5.9 Acknowledgment

The authors gratefully acknowledge the financial support provided by the Saudi Government.
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Chapter 6

6  Street Elevation Effects in Reducing Blast Resultants on Front Face of Buildings Surrounded by Perimeter Wall: Half-Scale Experiments

6.1  Introduction

In the past few decades, the world has experienced a large number of attacks using explosion by various terrorist organizations around the world. The threat generated by the explosions can have devastating effects for both the structures and their occupants. With the rising threat of terrorism, protecting critical structures and the infrastructures against explosion attacks has become a critical issue. It is also one of the greatest challenges for structural engineers because the hardening of an existing building to minimize damage caused by blast waves is very expensive and difficult to achieve. A simple approach to providing protection to structures and other assets from the effect of blast wave from an external explosion is to construct blast walls around the perimeter of the structure. However, research on blast wave propagation around a blast wall and the blast environment behind the wall is very limited.

One solution presented to date is to construct a perimeter wall around a structure to increase the standoff distance between the explosion and the building to be protected in order to reduce the blast resultants at the structure. However, architects do not agree that a building should be hidden behind a wall and prefer to find another solution. Aspects other than the strengthening of buildings can help increase their level of protection. The cost is very little compared to strengthening the building with protection material. In addition to erecting a blast wall around a building, the street layout can play an important role in mitigating the effects of blast loads on buildings. For example, the street level could be lowered where a truck carrying explosives could be parked. Hence, the street layout alone could significantly reduce the blast resultants on buildings. In order to quantify the level of protection provided, the pressures behind a blast wall along the
height of the protected building are compared with the pressures obtained in the absence of blast walls.

Blast loads in simple geometries can be predicted using empirical or semi-empirical methods, such as those presented in the Manuals (UFC 340-01, UFC 340-02). These methods can be used to estimate the blast loading on an isolated structure when there is no barrier. However, there is no direct method that can be used to estimate the blast resultants on a building façade if there is a blast wall between the explosive center and the structure. Remenniko (2003) presents a review of the existing methods for predicting blast effects on buildings. The existing methodologies for determining the effectiveness of blast walls, such as those presented in (TM5-853-3) are valid over only a very limited range of scaled parameters (wall height, charge-to-wall stand-off distance, wall-to-structure stand-off distance).

The existing empirical design charts assume either a spherical (free-air) burst or hemispherical (surface) burst and do not consider variations in charge shape, charge orientation or the point of detonation of the charge. In the empirical design charts, the point of detonation is assumed to be at the middle of the sphere charge. While the characteristics of air blasts from charges in the free field are well understood, the variations in blast resultants along a building façade protected by a blast wall are not fully understood.

One purpose of this study is to investigate the influence of street elevation on the blast loads on a target building and to demonstrate the importance of considering such effects. The selected configuration is a relatively narrow street width. The layouts of the experiments were chosen to represent a half-scale of a realistic threat where charges of up to a 91.84 kg TNT equivalent charge are detonated in the middle of a street. The building models are constructed from reinforced concrete.

6.2 Literature Review

There have been a number of studies on the performance of blast walls over the years (Smith, 2010). However, the influence of various parameters in selecting the blast wall
and its location relative to the explosion threat and the building it is protecting is still not fully understood. There are limited studies on the effect of street layout. The effects of the confinement produced by surrounding buildings on blast wave resultants for a high explosive detonation was studied by Smith et al. (2001). The study conducted using small scale models and the buildings were constructed as a block from reinforced concrete beams. They found that the presence of the street buildings increased the pressure and the impulse experienced by the building by a factor of up to about four. Whalen et al. (2001) studied blast wave propagation in a different configuration of city streets and their models were built from steel plates. Small-scale experiments were carried out. The results of their experiments demonstrated the blast enhancing effect of different street configurations and the blast resultants on building, which were increased by the enhancement of the charges detonated in the street.

Smith and Rose (2002) performed experimental and numerical investigation of small scale straight streets to study the effect of confinement due to the street width the building height on the positive and negative phase blast wave impulses on the building facades located along the street. The selected configurations were representative of relatively narrow and wide street widths, together with relatively low to very high building heights. The experiments were performed at a 1/40th scale. The explosive charge was 15.6 g of TNT equivalent and the charge was detonated in the middle of the street. The models used were constructed from reinforced concrete beams. The results of the experiments indicated that street width and building height influence the magnitude of the positive phase impulse. A scaled street width of $w/W^{1/3} > 4.8 \text{ m/kg}^{1/3}$ is sufficiently to ensure that the loading on one side of the street is not affected by reflections from buildings on the other side.

Rose and Smith (2003) experimentally and numerically studied the influence of street junctions on the characteristics of a blast that propagates into the streets beyond a junction. The study concentrated on crossroads, T-intersection and 90° streets bend with the physical and numerical models configured to be infinitely long and relatively narrow. It was found that the distance of the charge from the junction influences the extent to
which the blast diffracts at the junction. The greater the distance of the charge from the
junction, the greater the degree of diffraction that occurs at the junction, as opposed to the
reflection and transmission back down the street in which the charge is located. It was
observed also that the intensity of the blast propagation round a 90° street bend was
similar to that which propagates a long a straight street.

6.3 Statement of Problem

The war against terrorism that Saudi Arabia is leading today is a factor that makes this
country a target for the terrorist groups especially in today’s turbulent world. However,
the majority of the existing building perimeter walls in Saudi Arabia are constructed from
either reinforced concrete or hollow concrete blocks and are not designed to resist blast
loadings. One of the most significant attacks that took place in Saudi Arabia was in the
city of Al-Khobar in June 1996. A 13.7 m long and 10.7 deep crater was formed by a
2268 kg TNT bomb, which caused extensive damage to one of the city’s building towers
as shown in Figure 6.1.

![Al-Khobar Tower in June 1996.](image)

Figure 6.1: Al-Khobar Tower in June 1996.
Currently, some security barriers such as jersey barriers have been used at suitable distances from the building as a temporary protection. These barriers were installed temporarily and were to be removed when the level of risk fell or until more permanent blast walls could be designed and constructed. However, these temporary barriers have been in place for considerable time and may never be removed or replace in the foreseeable future. It is a well-known and significant fact that strength of blast loads decreases rapidly with distance. Therefore, the construction of a blast wall is one of the first considerations when attempting to mitigate the damage from blast loads. One problem in the design of a blast wall is determining where to site the wall relative to the building to be protected. In this study, a combination of utilizing a blast wall and decreasing the street level adjacent to the wall around the building to be protected is proposed as shown in Figure 6.2. To understand the behaviour of blast wave propagation, half-scale experiments are required.

Figure 6.2: Building Protection Approach Proposed in this Study

6.4 Experimental Program

In this experiment, half scale reinforced concrete cantilevered walls are tested against a close-in explosion. And the performance of the RC walls coupled with the effect of a lowered street elevation is investigated. The design of the reinforced concrete walls and
the non-responding columns which represents the target buildings, the construction of the specimens, the test matrix and the test setup are all presented in this section. All of the instrumentation, construction, and testing were conducted on the firing range at SSF test site in Riyadh City, Saudi Arabia.

6.4.1 Test Specimen

A total of four half-scale explosive field tests were conducted in order to test the effectiveness of a blast wall coupled with a change in street elevation in reducing the blast wave resultants along the height of the building to be protected. The scale model RC wall represents a full-scale wall 4 m wide, 2 m high, and 0.4 m thick. Accordingly, the half-scale model used measured 2 m wide, 1 m high and 0.2 m thick. A ready mix concrete was used to construct the test specimens and because the wall specimen was a half-scale model, the maximum aggregate size was scaled down from 20 mm to 10 mm. A normal concrete was used with compressive strength of 40 MPa and a 10 mm slump. The wall was reinforced on both the front and rear faces with 10 mm diameter steel with 200 mm spacing center to center both direction and a clear concrete cover on all sides, which was 20 mm thick and 10 mm stirrups spacing 200 mm were used. Standard deformed Grade 60 reinforcing bars with strength of 420 MPa are specified for all reinforcements. Insuring reinforcing steel inside both faces of the wall is necessary due to dynamic action affects in which these structures might rebound due to loading reversal.

The foundation was not the focus of attention as long as it was able to impart fixity at the base of the wall. The construction and curing of all walls and footings were carried out at the SSF main site. The foundation of each wall was first cast. The required amount of longitudinal reinforcement, which projected from the foundation to ensure continuity of the wall’s longitudinal reinforcement at the wall base, was used to ensure that there would be adequate resistance against direct shear failure when acted on by the blast. Figure 6.3 shows the formwork and the reinforcing process.
The second stage was to construct the walls as shown in Figure 6.4. The walls were cast with a concrete pump truck and consolidated with a hand vibrator. The walls were cured 28 days before the actual field tests and cubic samples were taken to verify the compressive strength. The average compressive strength is 37 MPa. Reinforced concrete slabs (2 x 2 m) were constructed and used to represent the street.

Figure 6.4: Wall Construction (a) Wall casting (b) Wall Finished
A heavy duty crane and a trailer truck were needed to lift and transport the specimens to the test site. Hooks were installed in the specimens to facilitate their transportation. Figure 6.5 shows the transportation and installation of the specimens.

![Figure 6.5: (a) Transportation (b) Installation and Positioning of Specimens](image)

### 6.4.2 Test Matrix

Four half-scale blast walls with different street elevation in front of target building were tested using various charge weights close in and at a constant standoff distance to the blast wall. The wall measured 2000 mm wide, 1000 mm high, and had a thickness of 200 mm. Because the first objective was to observe the wall’s performance, each wall was subjected to twelve successive shots, which increased from 0.45 kg to 15.052 kg. The standoff distance from the charge center to the wall was kept constant at 3 m. The explosive material used in all cases was ANFO, a mixture of ammonium nitrates and fuel oil. Table 6.1 shows the test matrix.

<table>
<thead>
<tr>
<th>Wall No.</th>
<th>Wall Description</th>
<th>Charge Size, kg</th>
<th>Standoff, m</th>
</tr>
</thead>
<tbody>
<tr>
<td>NW-0.0</td>
<td>No wall</td>
<td>0.45 - 15.052</td>
<td>3</td>
</tr>
<tr>
<td>WL-0.0</td>
<td>Plane vertical wall +street elevation – 0.0</td>
<td>0.45 - 15.052</td>
<td>3</td>
</tr>
<tr>
<td>WL-0.5</td>
<td>Plane vertical wall +street elevation – 0.5</td>
<td>0.45 - 15.052</td>
<td>3</td>
</tr>
<tr>
<td>WL-1.0</td>
<td>Plane vertical wall +street elevation – 1.0</td>
<td>0.45 - 15.052</td>
<td>3</td>
</tr>
</tbody>
</table>
The second objective of the tests was to assess the effectiveness of the blast wall when coupled with street elevation to reduce the structural loads to the target building located behind the blast wall. The reflected pressures on a target building at four different heights both with and without the blast wall were measured from the detonation of a high explosive charge. A total of twelve shots were fired in which there was no barrier wall between the explosive charge and the target structure to serve as a baseline for the barrier wall study as shown in Table 6.1. Then a blast wall was erected between the explosive charge and the structure and twelve shots were fired at the wall. The charge mass, the structure used the height of the burst (HOB), the charge to wall standoff distance, the wall height and the wall to structure standoff distance for each test are presented in Table 6.2. Hopkinson or cube root scaling was applied to those physical dimensions. The scaled dimensions provided in terms of the actual models divided by the cube root of the model explosive yield (m/kg\(^{1/3}\)), and referenced to the equivalent explosive mass of TNT. The scaled parameters were varied so as to substantially expand the range of parameters addressed in TM5-853-3, as shown in table 6.2.

<table>
<thead>
<tr>
<th>Scale Parameter</th>
<th>TM5-853-3 m/kg(^{1/3})</th>
<th>Small-Scale Experiments, Rickman (2006), m/kg(^{1/3})</th>
<th>Present Study, Half scale Experiments m/kg(^{1/3})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height of burst</td>
<td>-</td>
<td>0.00</td>
<td>0.20-0.68</td>
</tr>
<tr>
<td>Barrier wall height</td>
<td>0.32-0.52</td>
<td>0.29-1.50</td>
<td>0.41-1.35</td>
</tr>
<tr>
<td>Distance, charge to wall R</td>
<td>0.20-1.20</td>
<td>0.072-6.90</td>
<td>1.25-3.90</td>
</tr>
<tr>
<td>Distance, wall to structure D</td>
<td>0.40-8.00</td>
<td>0.24-11.90</td>
<td>2.16-16.24</td>
</tr>
</tbody>
</table>

6.4.3 Test Setup

The test program included four half-scale plane blast walls with different street elevations. Each wall was situated in front of non-responding round steel pipes mounted on a reinforced concrete foundation (i.e., essentially non-responding columns) to represent the target buildings. Three columns were used and positioned at three different distances from the blast wall. This test setup allowed all of the three non-responding
columns to be loaded simultaneously with the same explosive charge, which increased the accuracy of the comparison between the loadings on the three columns. The height of each column was 6 m and represented a half scale model. The first column was erected at a distance of 2 m; the second, at 6 m and the third, at 9.65 m. Four reflected pressure transducers were embedded in a vertical array on the center line of the target column at heights of 0.75 m, 2.25 m, 3.75 m, and 5.25 m to measure the reflected pressures in the presence and absence of the blast wall in front of the target building.

On the test site, the foundation was buried in a pit, which was excavated in the same position relative to each wall; then the specimen was aligned to its final position using lifting crane after which the excavation was backfilled and compacted as shown in Figure 6.6. Concrete block wing walls were erected around the RC wall on both sides to increase the width of the wall to prevent air blast pressure from wrapping around the wall.

![Image](image_url)

**Figure 6.6: The Process to Position the Blast Wall**

A reinforced concrete slab was constructed in front of the blast wall to represent the street and a replaceable steel plate with a thickness of 20 mm was positioned on the slab, which was laid under the charge to avoid excessive damage to the slab. This setup give a fully reflective surface. The test setup is illustrated in Figure 6.7. Figure 6.8 shows the test site prior one of the detonations.
Figure 6.7: Schematic of Blast Wall and Parameters Test Setup (Elevation View)

P1 = 0.75 m, P2 = 2.25 m, P3 = 3.75 m, P4 = 5.25 m

Figure 6.8: Test Site Prior One of the Detonation (a) One Meter Street Elevation (b) 0.5 meter street elevation

Figure 6.8: Test Site Prior One of the Detonation (a) One Meter Street Elevation (b) 0.5 m Street Elevation
Ammonium Nitrate-fuel oil mixture (ANFO) was used as the explosive material; non-fragmenting cube wood containers of different dimensions were used based on the charge weight. Each charge was positioned 3 m from the test wall along the front elevation and was placed on a wooden table, which had the height adjusted to position the center of the explosive at the mid height of the wall. Since this was a half-scale model, the charge was elevated to 0.5 m above the ground. The initiation point was fixed at the top of the charge and the charge boosted by a suitable quantity of PETN explosive. Figure 6.9 illustrates the preparation of the ANFO charge.

Figure 6.9: Preparing the ANFO Charge

The experimental program was established with the aim of providing as wide a range of geometric conditions as possible. Variations in the distance of the blast wall relative to the target building and the charge weight $W$ were examined, while the blast wall height, the charge to wall distance, the HOB, and the transducer configuration were kept constant.

The shock wave parameters were measured using a high speed data acquisition system which includes National Instruments data recording hardware and LabVIEW software, for the data collection and analysis, and a signal conditioner as well as sensors transducers and cables that are connected to the test specimen. The sampling rate was up
to 100 ks/s/channel. Reflected pressures were measured using a piezoelectric pressure sensor screwed onto the non-responding columns and their orientation was perpendicular to the blast wave. Sensors from PCB Piezotronics models 102B, 102B03, and 102B04 were utilized and the sensors were installed on non-responding columns that represented the target buildings to determine the reflected pressure along the height of the rigid column.

Blast wave characteristic, including incident and reflected pressures were recorded as well as the post-blast damage and mode of failure of the blast walls were observed. Experimental observations were used to evaluate the performance and to determine the capacity and failure limit states of the concrete wall. For the case with no wall, the blast characteristics were compared with those obtained using A.T.BLAST software to check the results obtained from the experiments.

6.5 Results and Discussion

6.5.1 Observations and Comparison with Results of Other Authors

Previous studies focused on the effect of blast wave propagation in city streets on the adjacent structures. And experiments and calculations with cuboid explosions with reflections are relatively rare in literature. Moreover, to the best of the author’s knowledge, the combined influence of street elevation and blast wall in reducing the blast wave resultants on a target building located behind the wall has not been investigated. Nevertheless, the use of this technique would not only reduce the pressure on the target building located behind the blast wall but also it would prevent excessive damage to the blast wall itself.

A safe scaled distance was used as the parameter in some regulations (US DoD, 2004 and ASCE, 1997) in assessing structural safety and ability to resist airblast loads. The values used in the manuals are for conventional structures without any blast hardening. In this study, the safe scaled standoff distance from the charge to the wall was $1.3 \text{ m/kg}^{1/3}$, which is much smaller than those specified in U.S. DoD (2004) and ASCE (1997) respectively,
which are 4.46 m/kg$^{1/3}$ and 4.65 m/kg$^{1/3}$. Moreover, there was no significant observable wall deformation or any sign of local failure in all of tests. It should also be noted that the effects of differences in structural materials or their configurations are not taken into consideration in these guidelines. Moreover, the definition of structural damage here is not clear.

FEMA 428 (2008) shows a generic interaction between the weight of the explosive threat and its distance to an occupied building. However, these generic charts are suitable for conventional construction only. Moreover, these distances are so site-specific that the generic charts provide little more than a general guideline in the absence of more reliable site-specific information. Based on the information provided in the chart, the onset of column failure is associated with stand-off distances in the order of tens of feet. This type of chart can be utilized to demonstrate the blast response of structural member at diverse level of protection. The maximum size of the charge used in the current study was 15.052 kg and the standoff distance from the charge to the blast wall was of 3 m. These values are out of the range of the chart.

### 6.5.2 Reflected Pressure and Impulse Distribution

To verify the experimental results, the A.T.BLAST program was used and the numerical results were compared as shown in Tables 6.3 – 6.10. Two different charge sizes at two different standoff distances were used in the case of where there was no wall for these comparisons. The results showed a good agreement between the measured and the numerical values for the reflected pressure while the program overestimated the reflected impulses.
Table 6.3: Reflected Pressure Comparison of the Computed and the Experiments

<table>
<thead>
<tr>
<th>Reflected Pressure</th>
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<tbody>
<tr>
<td>R, m</td>
</tr>
<tr>
<td>P1</td>
</tr>
<tr>
<td>P2</td>
</tr>
<tr>
<td>P3</td>
</tr>
<tr>
<td>P4</td>
</tr>
</tbody>
</table>

Table 6.4: Reflected Pressure Comparison of the Computed and the Experiments

<table>
<thead>
<tr>
<th>Reflected Pressure</th>
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<tbody>
<tr>
<td>R, m</td>
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<tr>
<td>P1</td>
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<tr>
<td>P2</td>
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<td>P3</td>
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<td>P4</td>
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Table 6.5: Reflected Pressure Comparison of the Computed and the Experiments

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<tbody>
<tr>
<td>R, m</td>
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<tr>
<td>P2</td>
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<tr>
<td>P3</td>
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<td>P4</td>
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Table 6.6: Reflected Pressure Comparison of the Computed and the Experiments

<table>
<thead>
<tr>
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<tbody>
<tr>
<td>R, m</td>
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<tr>
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<tr>
<td>P2</td>
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<tr>
<td>P3</td>
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<tr>
<td>P4</td>
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</table>
Table 6.7: Reflected Impulse Comparison of the Computed and the Experiments

Results

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</thead>
<tbody>
<tr>
<td>R, m</td>
<td>θ</td>
<td>W, Kg</td>
<td>A.T.BLAST, KPa-ms</td>
<td>Experiment, KPa-ms</td>
<td>Difference %</td>
<td></td>
</tr>
<tr>
<td>P1</td>
<td>9.20</td>
<td>1.56</td>
<td>7.248</td>
<td>223.50</td>
<td>58.9</td>
<td>74</td>
</tr>
<tr>
<td>P2</td>
<td>9.37</td>
<td>10.77</td>
<td>7.248</td>
<td>216.86</td>
<td>56.5</td>
<td>74</td>
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<tr>
<td>P3</td>
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<td>P4</td>
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<td>27.31</td>
<td>7.248</td>
<td>184.40</td>
<td>42.5</td>
<td>77</td>
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Table 6.8: Reflected Impulse Comparison of the Computed and the Experiments

Results

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<tbody>
<tr>
<td>R, m</td>
<td>θ</td>
<td>W, Kg</td>
<td>A.T.BLAST, KPa-ms</td>
<td>Experiment, KPa-ms</td>
<td>Difference %</td>
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<tr>
<td>P1</td>
<td>9.20</td>
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<tr>
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<td>27.31</td>
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<td>305.8</td>
<td>110</td>
<td>84</td>
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</tbody>
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Table 6.9: Reflected Impulse Comparison of the Computed and the Experiments

Results

<p>| | | | | | | |</p>
<table>
<thead>
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</tr>
</thead>
<tbody>
<tr>
<td>R, m</td>
<td>θ</td>
<td>W, Kg</td>
<td>A.T.BLAST, KPa-ms</td>
<td>Experiment, KPa-ms</td>
<td>Difference %</td>
<td></td>
</tr>
<tr>
<td>P1</td>
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<td>155.0</td>
<td>18.6</td>
<td>88</td>
</tr>
<tr>
<td>P2</td>
<td>12.97</td>
<td>7.76</td>
<td>7.248</td>
<td>153.0</td>
<td>63.0</td>
<td>59</td>
</tr>
<tr>
<td>P3</td>
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<td>147.3</td>
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<td>140.5</td>
<td>45.4</td>
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Table 6.10: Reflected Impulse Comparison of the Computed and the Experiments

Results

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</thead>
<tbody>
<tr>
<td>R, m</td>
<td>θ</td>
<td>W, Kg</td>
<td>A.T.BLAST, KPa-ms</td>
<td>Experiment, KPa-ms</td>
<td>Difference %</td>
<td></td>
</tr>
<tr>
<td>P1</td>
<td>12.85</td>
<td>1.12</td>
<td>15.05</td>
<td>258.0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>P2</td>
<td>12.97</td>
<td>7.76</td>
<td>15.05</td>
<td>254.2</td>
<td>98.0</td>
<td>62</td>
</tr>
<tr>
<td>P3</td>
<td>13.26</td>
<td>14.19</td>
<td>15.05</td>
<td>245.2</td>
<td>78.6</td>
<td>68</td>
</tr>
<tr>
<td>P4</td>
<td>13.70</td>
<td>20.29</td>
<td>15.05</td>
<td>232.6</td>
<td>78.74</td>
<td>66</td>
</tr>
</tbody>
</table>
The location of the blast wall relative to the building is protecting is very important because if the wall is too close to the building then the reflections between the wall and the building will cause damage to the building. But if the wall is too far away from the building then there will be no benefit of the wall. In this study, the height of the blast wall was one meter and it was placed 3 m away from the charge. The charge weight varied from 0.454 kg – 15.052 kg and charge was elevated to 0.5 m above the ground. The distances from the wall to the structure were 2 m, 6 m, and 9.65 m. The scaled distances from the wall to structure used in this study was extended up to 16.72 m/kg$^{1/3}$ compared to 8 m/kg$^{1/3}$ and 11.90 m/kg$^{1/3}$ as in the TM5-853-3 and Rickman (2006) studies, respectively.

It is worth noting that in the case of where there is no blast wall in front of the structures the maximum values for the peak reflected pressure and impulse always occur at the ground level. In this study, two different charge sizes were placed at two different standoff distances. It was observed in this study that the erection of a blast wall coupled with reduction in the street elevation provided a significant reduction in the reflected pressure at the lower level of the structure especially at the shortest distance used in this study and where the street elevation was lowered by 1 m the reduction was significant. It is also observed that a reduction in the reflected pressure at the longest distance used in this study was when the street lowered by 1 m. Figure 6.10 to Figure 6.13 illustrate the pressure-time histories when the target was elevated to 0.75 m on the target building for two different charge sizes and at two different standoff distances from the charge to the target building. The wall height was 1m and the distance from the charge to the wall was 2 m.
Figure 6.10: Reflected Pressure Time History for Charge = 7.248 kg and Standoff Distance Charge to Building = 9.2 m

Figure 6.11: Reflected Pressure Time History for Charge = 15.052 kg and Standoff Distance Charge to Building = 9.2 m
Figure 6.12: Reflected Pressure Time History for charge = 7.248 kg and Standoff Distance Charge to Building = 12.85 m

Figure 6.13: Reflected Pressure Time History for Charge = 15.052 kg and Standoff Distance Charge to Building = 12.85 m

The efficiency of the blast wall erected around the building to be protected coupled with the lowered street elevation in reducing pressure is demonstrated by utilizing the ratio of the reflected pressure or impulse where there was a blast wall in place to where there was no wall. This ratio (reduction factor) provides a direct measure of the effect of a blast
wall. A comparison of the peak reflected pressure and the associated impulse distribution over the height of the building both with and without a blast wall and with different street elevation is shown in Figure 6.14 to Figure 6.21. The analysis of the experimental data allowed several general trends to be observed. First, the maximum effectiveness of the combination of the blast wall and the lowering of the adjacent street elevation was obtained when the wall to structure distance of 9.2 m was used along with the lowest street elevation. However, the effectiveness of the blast wall decreases as the street elevation increases.

It is observed that the use of a blast wall coupled with decrease in the street elevation reduces the peak blast pressure and the impulse. Thus, it is obvious that the erection of a blast wall would deflect the blast waves and prevent the full impact of the blast wave from reaching the target structure and significantly reduce the reflected pressure and the impulse obtained compared to the erection of a blast wall alone. For the pressure adjustment factors, the reduction over the height of the structure was nearly identical in each case at the smallest standoff distance (wall-to-structure). The impulse reduction factors were increased for all of the scenarios in this particular illustration for the street elevation of 0 m and 1.0 m. However, the reduction factors were quite large where the street elevation was 0.5 m.

The enhancement of the impulse loads (reduction factor > 1) is a significant finding. This situation generally occurs at locations on the structure at heights above the height of the protecting blast wall. The worst configuration for such enhancement scenarios are those with a large wall-to-structure standoff distance with a minimum street elevation.
Figure 6.14: Effect of Street Elevation on Pressure Factors, \( W \) of 7.248 kg and \( D \) of 9.2 m

Figure 6.15: Effect of Street Elevation on Pressure Factors, \( W \) of 7.248 kg and \( D \) of 12.85 m
Figure 6.16: Effect of Street Elevation on Pressure Factors, W of 15.052 kg and D = 9.2 m

Figure 6.17: Effect of Street Elevation on Pressure Factors, W of 15.052 kg and D of 12.85 m
Figure 6.18: Effect of Street Elevation on Impulse Factors, W of 7.248 kg and D of 9.2 m

Figure 6.19: Effect of Street Elevation on Impulse Factors, W of 7.248 kg and D of 12.85 m
Figure 6.20: Effect of Street Elevation on Impulse Factors, W of 15.052 kg and D of 9.2 m

Figure 6.21: Effect of Street Elevation on Impulse Factors, W of 15.052 kg and D of 12.85 m
The best results were obtained when the wall-to-structure distance was at a minimum. And the largest reductions were found toward the bottom of the structure. As the standoffs distance was increased, there was often a kink point slightly above the base of the structure and the location of this point moves up to a higher location on the structure as the standoff distance increases. For the short distance from the wall to the structure, the $AF_p$ was at maximum at the base of the structure. At the interim distance, there was a kink point just slightly above the base. In general, the kink point was more towards the top of the structure at the largest standoff distances.

6.6 Summary and Conclusions

The U.S. Army Security Engineering Manual (TM5-853-3) coupled with UFC 340-02 uses the adjustment factor approach (the ratio of pressure or impulse with the blast wall compared to that of where there is no wall) to calculate the reduction in pressure or impulse behind a plane wall, but the TM5-853-3 manual is restricted to use by only the U.S. military. The formulations used in this manual are based on small scale tests. Moreover, these formulations were never validated with Full-Scale experiments. The formulations in these manuals, too, are for spherical charges initiated at its center with an emphasis on mid-field and far-field overpressures.

It is important to note, too, that since the charge shapes selected by the terrorist organizations are unknown and could be non-spherical (cylindrical, or cuboid) and have different orientations and detonation points within the explosives, they will produce different distributions of the overpressures than would a spherical charge of the same size especially in the immediate vicinity of the detonation. The authors of the manuals also did not consider the reflection due to the vicinity of the structures; the street elevation, or the structure geometry. Hence, these manuals are not useful. Nevertheless, the experiments conducted along with the FE codes such as LS-DYNA can be used to assess the overpressure of the detonation of the non-spherical charges and the size of the reflection due to the geometry of the structure or that the adjacent structures.
An extensive series of half-scale experiments were carried out to study the effects of the erection of blast walls coupled with changes in street elevation in changing blast wave propagation and the distribution of blast pressure on a building façade located behind a wall. Reinforced concrete walls at half-scale were subjected to close range high explosive blast tests at the SSF Explosive Ranges in Saudi Arabia. Generally speaking, it is worth noting that when there was no wall present, the reflected blast resultants were higher than when a wall was present. It was also observed that, if the point on the target was not in a direct line of sight of the charge, there was a reduction in the reflected pressure. It was also observed that when the distance from the wall to the structure was short, the blast wall was more effective in reducing the peak reflected pressure and the impulse even though the duration of the positive phase of the blast pressure was longer. Hence, the erection of a blast wall was found to significantly reduce the peak pressure exerted on the building surface. This effect was more significant for the lower part of the building than for the upper part. For greater standoff distances, this effect was not as significant for the upper part of the building. Further, it was found that a higher level of reduction could be obtained by decreasing the street elevation adjacent to the wall. Hence, a lowering of a street elevation coupled with the erection of a blast wall was found to play an important role in the mitigation of the blast resultants. And during this study a street elevation of 1.0 m was found to be the most effective in reducing the blast pressure along the height of the building located behind the wall.

The perimeter wall was constructed to increase the standoff distance between the explosion and the protected building to reduce the blast resultants on the structure. The cost was very little compared to that of strengthening the building with a protective material. In addition to erecting blast wall around the building, paying careful attention to the street layout can play an important role in mitigating the effect of blast loads on buildings. The street layout, too, can have a significant impact in reducing the blast resultants on buildings. In order to quantify the level of protection provided, the pressures created behind the blast wall along the height of the protected building are compared to the pressures obtained in the absence of a blast wall.
The experimental data indicated that the proposed solution provided a substantial reduction in the reflected pressure. Using a combination of erecting a blast wall, decreasing the street elevation and providing a substantial standoff distance was found to be a preferable solution to induce a significant pressure reduction as well as reduce the number of human causalities and injuries.

6.7 Acknowledgment

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6.8 References


Chapter 7

7 Response of T- Reinforced Concrete Wall Protected with Aluminum Foam Subjected to Close-in Explosion: Full Scale Experiments

7.1 Introduction

The increase in the threat by the terrorist organizations using explosive weapons has resulted in structures and infrastructures in Saudi Arabia and across the globe becoming more vulnerable to extreme impulsive loading. Hence, the need for a reliable performance assessment of structural components such as reinforced concrete panels subjected to blast loading has become an important issue. Such an assessment must include careful consideration of several key factors. For example, the air-blast phenomenon takes place over very short time intervals, measured in milliseconds. And, the magnitude of the blast pressure is proportional to the size of explosive and is related as \( Z = R/W^{1/3} \), where \( R \) is the standoff distance from the center of the charge to the target and \( W \) is the charge weight measured in kg equivalent of TNT (DOD, 2008). The shape of the charge must also be taken into consideration. However, existing analytical and empirical methods to predict the air blast pressure in the near-field regime are based on derivations of bare spherical high explosives. And these methods are not suitable for charges of different shapes if accurate blast wave parameters are to be set (Ritzel 2009). To realistically evaluate the available empirical equations and to set reliable standards for blast properties, a new code is needed.

To date, there is only one code available worldwide. The code was published by the US Army Corps of Engineers in 1990, namely TM5-1300 which entitled “Structure to resist effects of accidental explosion”. This code provides not only information on blast property calculations but also details for the design of protective Reinforced Concrete (RC) structures subjected to high explosive detonations. However, the blast properties, which are currently available, are applicable to a limited range of applications (e.g. the pressures of hemispherical and spherical explosions at certain distances and the pressures on finite and infinite walls located perpendicular to the source of an explosion at set
distances). For complex structures that are subjected to unusual charge geometries, experimental studies, which will provide more reliable information about blast properties, are required.

In terms of the experimental validation of blast wave parameters with regards to their propagation on different structural layouts and configurations, a number of published papers are available for reference. However, most of the experiments were conducted on small-scale models and the results vary widely. This is partly due to the fact that the parameters set for empirical equations proposed by a number of researchers have significant differences. Therefore, close examination of the generally accepted means of calculating blast wave parameters is needed. Another problem concerns the accuracy of the empirical equations proposed that diminishes as the stand-off distance of the explosive grows shorter. Moreover, the aforementioned equations were generated from experimental studies under a limited range of test conditions (i.e. blast took place in a single structure environment in an open flat terrain). Also the majority of the previous studies were of small-scale and numerical approaches because the possibility of conducting full-scale tests was limited due to security restrictions and the large quantity of resources required.

The need for more knowledge of the structural requirements for effective blast resistance in the construction industry has evolved in recent years and the design of blast resistant structures requires knowledge of both blast loading and the behaviour of structures under these loadings. Due to the difficult nature of explosion assessing the effects of this kind of threat on structures is no easy. Hence, full-scale blast tests are required to gain a full understanding of the likely the behaviour of structures under this kind of loading.

The reinforced concrete walls that currently surround buildings in Saudi Arabia and across the globe are not designed to withstand blast loads. Since the perimeter walls are designed to act as a shield for protecting the buildings located behind them, it is very important to design and locate the perimeters walls properly. Also the resistance of these walls can be increased if they are protected by aluminum foam panels, which can be utilized as sacrificial. Sacrificial claddings are frequently utilized in the design of a protective system. Aluminum foams, for example, are lightweight and are one of the
energy absorption materials that are used to mitigate the damage caused by blast loadings. This research explores the potential of using such aluminum foam as a structural retrofitting material for concrete structures being subjected to blast effects. The response of reinforced T-Shaped concrete wall is investigated in this study both with and without different layers of aluminum foam.

The published results for blast wall case revealed that the majority of the experimental studies performed used small-scale models that were subjected to either non uniform or spherical and cylindrical charges. However, the experiments conducted for this thesis involved full-scale models and cuboid shape charges. Moreover, the majority of the blast walls studied by other researchers were solid and only a few of the studies examined reinforced concrete plane walls. In this study, reinforced concrete T-Shaped walls both with and without the protection of aluminum foam were used to examine the effectiveness of this reinforcing technique to improve the resistance of a wall and to reduce the blast resultants along the height of the building located behind a wall.

7.2 Research Significance

The majority of the governmental and residential buildings in Saudi Arabia are surrounded by perimeter walls constructed of either RC or unreinforced masonry. Most of the existing reinforced concrete walls are not designed to resist blast loads. To date based on the information available in the open literature; most research programs have focused on understanding the effectiveness of rigid plane walls to reduce the blast wave resultantson the building to be protected behind them using small-scale models.

Blast loads can be generated and applied to structures by a pressure load using a ConWep computer program or other detonation simulation methods. ConWep is based on a collection of conventional effects calculations provided in TM5-855-1 (U.S. Department of the Army, 1998); Hyde 2005; United States Army Corps of Engineer). ConWep is a standalone program and restricted to be used only by the U.S. Military. But it is also implemented in a LS-DYNA FE program which is available to the researchers. In this method, blast pressure is applied directly to the front surface of the lagrangian structure.
This approach can be used for analysis before the failure of a structural surface. An eroding technique is used in this approach to eliminate the damage elements from the structure to avoid element distortion and the blast load continues to act on the uneroded elements. Therefore, in a simulation using this method the damage to the structure is underestimated because the load is not transmitted to the concrete core after the spalling of the concrete surface. This is one of the disadvantages of this method. Nevertheless, this method is simple and requires for the input only the charge weight in the equivalent of TNT weight and its position relative to structure according to Simoens et al. (2011). As a consequence, the computational cost is less than that in other simulation approaches. Another disadvantage of this method, however, is that it does not account for the blast wave reflections from coming from the ground and the structure. Likewise, the shadowing of the blast wave due to the presence of the blast wall or other adjacent structures cannot be considered in this approach. Also it does not take into consideration the effect of the charge shape, its the ignition point, or the orientation of the charge with respect to the structure.

There is another numerical approach that can be used to model the explosion and the resulting structural damage which involves the modeling of the air and the explosive with a multi-material-arbitrary Lagrangian-Eulrian (MM-ALE) formulation in which a proper equation of state is assigned to the materials and a burn model controls the explosive’s detonation behaviour. In this method, the structure is treated as a Lagrangian and Fluid-Structure-Interaction (FSI) according to Hallquist et al. (2012) and is used for communication between the Lagrangian and the ALE domains. This method is appropriate in case where the charge and the structure are at close-range according to Yi et al. (2013).

There are several advantages that stem from the use of this method over the pressure load method that is used in the ConWep software: (1) the software takes into account the fact that the blast wave load continues to act on the structure after the elements eroded from the structural surface; (2) the reflection and diffraction of the blast wave can be predicted; (3) this method can take into consideration the effect of the charge shape its ignition
points. The main disadvantage of this method is that a large air blast model is usually needed to mitigate the boundary effects and, the increased computation time because of the large domain.

Blast loads in simple geometries can be predicted using empirical or semi-empirical methods, such as those presented in the manuals (UFC 340-01, UFC 340-02). These methods can be used to estimate the blast loading on an isolated structure when there is no barrier. However, there is no direct method that can be used to estimate the blast resultants on a building’s façade if there is a blast wall between the explosive center and the structure. Remenniko (2003) presents a review of the existing methods for predicting blast effects on buildings. However, the existing methodologies for determining the effectiveness of blast walls, such as those presented in (TM5-853-3) are valid over only a very limited range of scaled parameters such as (wall height, charge-to-wall stand-off, and wall-to-structure- distance). Also the empirical design charts it is assumed that there is either a spherical (free-air) or hemispherical (surface) burst and variations in charge shape, charge orientation and the point of detonation of the charge are not considered. In these empirical design charts, too, the ignition point is assumed to be at the center of the spherical charge. The assessment here of the resultants due to air blasts from charges in the free field appears to be sound while the variation in the blast resultants along the building façade protected by a blast wall is not.

It is important to note that a thorough knowledge of blast characteristics is critical to accurately predicting blast effects especially at close range. There are basic elements that have a significant impact on the pressure and impulse values of a blast wave that is propagated in different direction in the air, especially close to an explosion. These elements include wall dimensions, charge shape, ignition point, and type of explosive, structural geometry as well as the orientation of the charge with respect to the structure. The majority of the previous experimental results are related to the spherical shape of the explosive charge and there is a lack of a sufficient number of test results for other charge shapes. Therefore, there are some difficulties in precisely defining the impact of non-spherical charges and their ignition point on structures. In the majority of the previous
studies, too, the charge tests were conducted on unpaved ground, which means that part of the energy associated with the surface burst was lost though the ground and the digging of a crater. In the current study, the charges were placed on rigid ground so that there was a fully-reflected blast wave, which reinforced the blast loading.

The response of concrete structures to blast loading depends on the nature of the blast, the geometric and dynamic structural characteristics involved, and the material’s dynamic response characteristics. The loading regime is characterized by using the Hopkinson’s standoff distance, \( Z \), which is defined as the ratio of the standoff distance to the cube root of the charge weight, and is classified as close-in, near field, or far field, depending on the value of \( Z \) according to Smith et al. (2009). In the far-field design range, blast-generated pressure loads can be considered uniform with respect to the structure and a basic single degree of freedom approximate analysis is often implemented. However, in the close-in design range, blast pressures are non-uniform and the pressure magnitudes can be very high (U.S. Department of Defense [DoD] 2008). A close-in detonation is often considered to have occurred when the scaled distance is less than 1.2 m/kg\(^{1/3}\). Following the detonation of a high explosive at a short scaled distance from a concrete wall, a shock wave is generated. Part of the shock wave that strikes the wall surface is transmitted to the concrete, resulting in a compressive wave. When the transmitted shock wave reaches the back surface, it reflects and results in a tensile wave. If the tensile stress on the back surface is greater than the dynamic concrete’s tensile resistance the concrete will fragment, i.e., spall (DoD, 2008).

A single degree-of-freedom (SDOF) and an advanced (FEM) are available for the designers to design structures that could be subjected to blast loading. However, the challenge lies in using them with an adequate degree of confidence. The SDOF approach, for example, can be used for far range blast loading only (scaled distance \( Z \geq 3 \)) where the blast loading is uniform while this approach cannot be used to determine close-in effects as stated in the ASCE 59-11 standard (2011). The significance of the research presented in this paper is the attempt to provide data that originates from controlled tests for the validation of numerical
models, which are sparingly available in the literature, especially those relating to pressure, strain, and deflection histories.

To address this issue, three specific goals were set for this paper. The first of these was to describe experiments conducted by subjecting regular-strength RC walls, which had been reinforced on both sides with regular strength steel reinforcement, to live blast loading. The second was to measure the acceleration, velocity, displacement and strain experienced by concrete under blast loading. The third was to measure the damage arising from experimentally applied pressure and impulse to the walls and to compare it with ASCE 59-11 (2011) standard. Finally, the fourth goal was to study the blast resultants along the height of the building located behind the blast wall.

From the literature survey, it became evident that while field tests have already been conducted on this subject, there is a lack of controlled experiments and precise data that will serve as useful guidelines in the future. There is also a strong need to characterize the experimental response of RC walls subjected to blast loading to provide accurate response data. Experimental research data that would validate both the advanced finite element and the SDOF models commonly used by designers are also needed to accurately assess their conservativeness. The experimental research data presented in this paper is an attempt to addresses several of these needs.

The aim of the tests was to investigate the behaviour of a reinforced concrete wall subjected to a single blast load after having been retrofitted with aluminum foam. A full-scale blast test was conducted on four T-RC walls reinforced with one or two layers of aluminum foam. The walls were then subjected to a 125 kg ANFO charge (equivalent to 102.5 kg TNT) placed on a wooden table at a height of one meter and at a standoff distance of 4 m. The resulting displacement and the strain response caused by the blast were measured during the test. The blast pressure was also recorded by four pressure gages, which were located along the height of building located behind the wall. Accelerometers were also placed at a point in the middle height of the wall (away from the blast side). Strain gages were also used at different locations along the longitudinal reinforcements on both the front and the back sides of the wall. The acceleration results
were numerically integrated to obtain displacement readings. Most important to this study, a high-speed video camera recorded the blast event.

The level of damages to each of the walls was observed and compared qualitatively to the damage classification in accordance with the ASCE 59-11 (ASCE 2011) standard also using the performance (rotation) limits, and the level of damage to each wall in the current study was in accordance with the code.

The safe scaled distance specified in some regulations such as US DoD, 2004 and ASCE, 1997 was as one parameter in assessing a structure’s ability to resist airblast loads. The safe scaled distance in the current study was compared with those specified by the regulations and those used in previous studies.

7.3 Literature Review

New structures can be designed to resist extreme loads. However, existing structure may not have been designed to resist extreme loads such as those emanating from a terrorist bomb blast. Since the response of such structures when subjected to a blast load is unknown, feasible ways to address this gap in our knowledge must be developed. In the following section, a brief literature review of the behaviour of RC structures subjected to blast loads is presented. A review of established blast propagation concepts is given and the response of RC structures subjected to such a blast loading is discussed.

Scaled distance is used as a parameter in assessing structural safety with respect to resisting airblast loads in the U.S. Dept. of Defense US DoD (2004) guidelines. Whereas the pressure–impulse (P–I) diagrams are used to estimate structural damage against blast loads examined in ASCE (1997). The U.S. DoD (2004) manual specifies a safe scaled distance of $4.46 \frac{m}{kg^{1/3}}$ for unstrengthened buildings to ensure that a building would not be destroyed. The safe scaled distance given in this manual was usually obtained from field blasting tests, which were conducted on either scaled structural models or low-rise residential type structures. One drawback of these guidelines is that the effect of differences in the structural materials and building configurations is not taken into
consideration, although they affect structural performance significantly. Moreover, the definition of structural damage is vague in these guidelines.

ASCE P–I diagrams show that the scaled distance to prevent structural collapse against 1,000 kg TNT blast loads is $4.65 m/ kg^{1/3}$. However, these codes do not explicitly define the structural types or their condition, e.g., reinforced concrete or masonry or steel structures in sound condition or otherwise. They also do not define whether the structure is a low-rise, a medium-rise or high-rise building. Moreover, the explicit descriptions of the various damage scenarios of structures under blast loads at different scaled distances are not given in the codes. Nor this code’s definitions of a structure approaching destruction or “structural collapse” clear. It could be either the total collapse of the structure, or the total collapse of the infilled masonry walls, or the partial collapse of the masonry walls.

Range to effects chart provided by FEMA 428 (2008) shows only a generic interaction between the weight of the explosive threat and its distance from an occupied building. And these generic charts are intended to be for conventional construction only. Moreover, the distances provided in these generic charts provide little more than general guidelines in the absence of more reliable site-specific information. Based on the information provided in the chart, the onset of column failure is associated with stand-off distances in the order of tens of feet.

Zhou and Hao (2008) conducted a numerical analysis of a typical unretrofitted reinforced concrete wall under different blast loads. The objective of their study was to obtain a range-to-effect chart similar to that in (FEMA 428, 2008) for RC walls. According to FEMA 428 (2008), the explosive weight of a bomb used in a terrorist can be divided into four subcategories. These are: luggage bomb, automobiles bomb, vans bomb and truck bomb. According to the numerical results, critical curves and analytical formulae related to bombs in each of these subcategories, different damage conditions were suggested from which safe stand-off distances for different terrorist bombing scenarios were determined. The critical charge weight and – stand-off distance relationships are shown
in Figure 7.1. The three curves are only suitable for a stand-off distance ranging between 2 m and 20 m. FEMA (2008)’s threshold line for failure of concrete columns is also given for comparison. Their results were obtained based on the particular RC wall that was analyzed in their study. Moreover, more analyses are needed on RC walls of different dimensions, with different reinforcement ratios, and under different boundary conditions in order to derive a more useful formula for a quick assessment of the performance of a RC wall subjected to blast loads.

Further research by Wu and Hao (2007a) has been done to fill in this gap for concrete structure. Wu and Hao (2007a) developed an improved approach based on the U.S. DoD Code, which defines various performance levels, including collapse. However, they neglect to consider that besides the charge weight and standoff distance, structural materials and configurations are also two important parameters. However, some tests by Baylot et al. (2005) showed that by increasing the charge weight or decreasing the stand-off distance other types of damage can be observed in addition to building collapse, including cracks, catastrophic breaching, and the scattering of both low and high velocity debris. Therefore, the development of all inclusive and detailed guidelines cover the
major damage levels resulting from blast loads for retrofitted RC and masonry walls is necessary.

Timothy (2011) conducted experimental investigations to evaluate the performance of reinforced concrete walls that were constructed using Normal weight and Fiber Reinforced Concrete with a wide range of construction details under blast loading. Eighteen 1.2 m square wall panels were tested under blast loading of various reinforcement types and thicknesses. ANFO charges were used and placed on a small wooden table at the center of the test site with the height being adjusted to position the explosive in the mid-height of the specimens. The ANFO charges used (weighed 34 lbs.) and they were placed at a standoff distance of 38 inches. They found that the FRC panels with 10 mm diameter steel rebar, which were spaced at 152 mm and that the NWC panels with 10 mm steel rebar, which were spaced at 305 mm and had external GFRP overlays, were the least damaged compared to the other panels used in their study.

Schenker et al. (2008) carried out two full scale field explosion tests on both protected and unprotected concrete slabs to determine the effectiveness of the aluminum foams when they were added as a protective layer to mitigate blast wave loads. The specimens were subjected to a 1000 kg hemispherical TNT charge that was placed at distance of about 20 m. Two strong slabs and pair regular slabs were tested. In each pair, one concrete slab was either fully or partially covered with an aluminum foam layer while the other concrete slab was left bare. The density of the aluminum foam was 0.135 g/cc and the thickness of each layer was 36 mm. The aluminum foam was then covered with a 1.5 mm steel plate.

The study indicated that the unprotected slabs suffered much more damage than did the protected slabs for both the strong and the regular slabs. However, the aluminum foam layers that covered the central part of the concrete slabs were split off the slabs. The study also indicated that the accelerations, velocities, and displacements at the centers of the protected concrete slabs were smaller than those of the unprotected slabs. They found that in terms of the ratio between the accelerations of the protected and the unprotected slabs, the aluminum foam was more effective for the regular slabs. This was because the regular slab was more flexible and, hence was more sensitive to energy dissipation.
Chengqing et al. (2011) performed experimental investigations to study the performance of reinforced concrete slabs both with and without aluminum foam protection. Four reinforced concrete slab protected with different thicknesses of aluminum foam and one unprotected control RC slab were tested. Two specimens were protected with 12.7 mm and 25 mm foam layers with densities of 450 kg/m$^3$ and another pair of specimens was protected with 43.2 mm foam layers with a density of 140 kg/m$^3$. The aluminum foam layers were attached to the top surface of the concrete slabs. A 1.15 mm thick steel cover sheet was attached to the outer face of the aluminum foam layer. The charge shapes used in the tests were cylinders with ratio of diameter to length of 1:1. The specimens were subjected to charges of 8.05 kg and 14.07 kg Composite B at standoff distances of 0.92 m and 1.5 m. The charges were oriented perpendicular to and parallel to the slab surfaces and suspended in the air. The study showed that the 25 mm aluminum foam layer with a density of 140 kg/m$^3$ provided better protection compared to the other specimens in this study where no damage was observed. The maximum accelerometer reading limit was 250 g. Because of this limitation, all of the specimens provided almost the same peak acceleration values. The displacements of the protected slabs were lower than those of the unprotected slabs.

Another study by Chengqing et al. (2011) was carried out on simply supported reinforced slabs protected by aluminum foam layers. The RC slabs were covered in this study by 75 mm foam layer with densities of 420 kg/m$^3$. A 1.15 mm thick steel cover sheet was attached to the outer face of the aluminum foam layer. All of the specimens were subjected to 7.5 kg Composite B charges with spherical shapes and were set at a standoff distance of 1.25 m (scaled distance of 0.64 m/kg$^{1/3}$). The explosive charge was suspended over the slab in each case. The study indicated that the 75 mm aluminum foam was not significantly densified and cracks appeared along the full length of the slab, which were significant in depth indicating that the slab was damaged before the complete densification of the aluminum foam. The reason of this behaviour may be due to the different strengths of the foam plateau stress (5.4 MPa) and the amount of structural yield stress. It is important to note that the aluminum foam needs to match the resistance of the structure to achieve the most effective level of structural protection (Ma and Ye, 2007).

The author designed a cantilever reinforced concrete wall based on UFC-340-02 (2008) manual. The wall was designed to resist a blast load with a capacity of 80 kg of TNT at a standoff distance of 4 m.
A series of half-scale experiments was conducted on RC walls of different shapes and the walls were subjected to charge sizes that varied from 0.45 kg to 14 kg at a constant standoff distance of 2 m. It was found that a wall can resist 12 kg of TNT (equivalent to 96 kg for a full-scale test at a 4 m standoff distance). According to these findings, a T-RC wall was chosen to be investigated in full-scale experiments where the charge size selected was 125 kg ANFO (equivalent to 102.5 kg of TNT). The response of the wall both with and without different layers of aluminum foam was investigated as well as the effects of the reduction in blast wave resultantts along the height of building located behind the wall.

7.4 Experimental Program

Four full-scale reinforced concrete walls of a T-shape were constructed and field tested against a close-in explosion generated by an ANFO charge. The performance of the walls both with and without aluminum foam protection was investigated. Sleeve anchor bolts were used to connect the aluminum foam to the wall. The performance of the non-retrofitted T-RC wall under blast loads was used as a "control" case for comparison purposes. The blast wave resultantts along the height of the building in the absence of the wall were measured and the results were compared with those appearing in the A.T.BLAST program. The design of the reinforced concrete walls and of the non-responding column, which represent the target building, the construction of the specimens, the test matrix and the test setup are presented in this section. All of the instrumentation, construction, and testing took place on the firing range at the SSF test site in Riyadh city, Saudi Arabia.

7.4.1 Test Specimens

A total of five full-scale explosive field tests were conducted in order to discover an effective form of protection for blast mitigating barrier walls both with and without aluminum foam. The T-RC wall represented a full-scale wall which was 4 m wide and 2 m high and was 0.4 m thick. A ready mix concrete was used to construct the test specimens. A normal concrete was used with a compressive strength of 40 MPa and a 10 mm slump. The wall was reinforced on both the front and rear surfaces with 20 mm
diameter steel bars at 200 mm spacing center to center in both ways and the clear concrete cover placed on all sides was 40 mm thick and had 10 mm stirrups; a spacing of 200 mm was used. Standard deformed Grade 60 reinforcing bars with strength of 420 MPa were specified for all reinforcements. It was necessary to place reinforcing steel inside both surfaces of the wall due to the dynamic action affects in which these structures might rebound due to loading reversal. Figure 7.2 illustrates the reinforcement and the dimensions of the T-RC-W.

Figure 7.2: T-RC-W Cross Section.
The foundation used was not the focus of attention in this study as long as it was able impart fixity at the base of the wall. The construction and curing of all the walls and footings were carried out at the SSF main site. The foundation of each wall was cast first. Then the required amount of longitudinal reinforcement was projected from the foundation to ensure the continuity of the wall’s longitudinal reinforcement at the wall’s base, thus ensuring adequate resistance against direct shear failure when acted on by the blast. Figure 7.3 shows the formwork and the reinforcing process.

![Formwork and Reinforcing Process](image)

**Figure 7.3: Construction of RC wall (a) Reinforcement (b) Formwork**

The second stage was to construct the walls as shown in Figure 7.4. The walls were cast with a concrete pump truck and consolidated with a hand vibrator, as shown in Figure 7.4. The walls were cured 28 days before the actual field tests and cubic samples were taken to verify their compressive strength.
A heavy duty crane and a trailer truck were required to lift and transport the specimens to the test site. Hooks had been installed in the specimens to facilitate their transportation. Figure 7.5 shows the transportation, installation and positioning of the specimens.

The aluminum foam that was used in the present study had air-filled closed cells. The energy absorption capability of this foam stems from the energy required to compress the air filling the closed cells and crushing the cell walls. The previous statistics from
previous studies of the aluminum foam’s performance under blast loading [(Su Yu et al. (2008)] showed that the density and the thickness of the aluminum foam played significant roles in determining their energy absorption capability. Increasing the density of the aluminum foam or using thicker foam increases the energy absorbing capability of the aluminum foam plate. Therefore, the properties of the aluminum foam selected in the current study were based on the optimum findings of the previous studies. Details and some properties of the aluminum foams layer that was used in the present experiments are summarized in Table 7.1.

<table>
<thead>
<tr>
<th>Table 7.1: Detail and Properties of the Aluminum Foam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density, kg/m³</td>
</tr>
<tr>
<td>Thickness, mm</td>
</tr>
<tr>
<td>Dimension, m</td>
</tr>
<tr>
<td>Cover steel plate both sides, mm</td>
</tr>
</tbody>
</table>

7.4.2 Test Matrix:

Five full-scale blast tests were conducted in front of a target building using close-in charges (scaled standoff distance, $Z = 0.86 \text{ m/kg}^{1/3}$). The first test was conducted in the absence of the wall to measure the reflected pressure along the height of the building while the other four tests were designed to study the response of T-RC walls both with and without the protection of different layers of aluminum foam. The wall measured 4000 mm wide, 2000 mm high, and had a thickness of 400 mm. The standoff distance from the charge center to the wall was kept constant at 4 m and the distance from the wall to the building was 12 m. The explosive material used in all cases was ANFO, a mixture of ammonium nitrates and fuel oil.

Table 7.2 presents the test matrix of the T-RC wall specimens which were tested under each blast load. The walls were designated as follows: T-RC-W1, T-RC-W2, T-RC-W3, and T-RC-W4. Each wall was subjected to single shot. T-RC-W1 was not retrofitted and its performance under blast load was used as a "control" case for comparison purposes. T-RC-W2 was protected with one layer of aluminum foam on the front surface of the wall. T-RC-W3 was protected with one layer of aluminum foam on both the front and back
surfaces of the wall. T-RC-W4 was protected with two layers of aluminum foam on the front surface and one layer on the back surface of the wall.

**Table 7.2: Test Matrix**

<table>
<thead>
<tr>
<th>Shot #</th>
<th>Specimen</th>
<th>Charge size ANFO (kg)</th>
<th>TNT equivalent, Kg</th>
<th>Standoff distance charge-to-wall, m</th>
<th>Standoff distance charge-to-building, m</th>
<th>Scaled distance, m/kg^{1/3}</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>T-RC-W1</td>
<td>125</td>
<td>102.5</td>
<td>4</td>
<td>16.4</td>
<td>0.86</td>
</tr>
<tr>
<td>2</td>
<td>T-RC-W2</td>
<td>125</td>
<td>102.5</td>
<td>4</td>
<td>16.4</td>
<td>0.86</td>
</tr>
<tr>
<td>3</td>
<td>T-RC-W3</td>
<td>125</td>
<td>102.5</td>
<td>4</td>
<td>16.4</td>
<td>0.86</td>
</tr>
<tr>
<td>4</td>
<td>T-RC-W4</td>
<td>125</td>
<td>102.5</td>
<td>4</td>
<td>16.4</td>
<td>0.86</td>
</tr>
</tbody>
</table>

The parameters studied in this experimental program were the feasibility of using different layers of aluminum foam as a protective layer on the surface of a wall as well as their ability to reduce the blast wave resultants along the height of the building behind the wall. The criterion for the selection of each charge size and standoff distance was based on the results achieved in previous work conducted in Saudi Arabia by the author and the shortest distance from the street to the perimeter. The behaviour and the responses of the protected walls were compared to those of the unretrofitted T-RC-W1 in terms of the level of damage observed as well as the acceleration, velocity, displacement, and strains.

### 7.4.3 Test Setup:

The test program included four full-scale T-RC blast walls. Each wall was situated in front of a non-responding round steel pipe that was mounted on a reinforced concrete foundation (i.e., essentially a non-responding column) to represent the target building. One column was used and positioned at a distance of 12 m from the blast wall. The height of the column was 12 m and it represented a full-scale model. Four reflected pressure transducers were embedded in a vertical array along the center line of the target column at heights of 1.5 m, 4.5 m, 7.5 m, and 10.5 m to measure the reflected pressure both in the presence and the absence of the blast wall located in front of the target building.
On the test site, the foundation was first buried in a pit, which was excavated to secure each wall in the same position, then the specimens were aligned using a lifting crane to their final positions after which the excavations were backfilled and compacted as shown in Figure 7.6. Concrete block wing walls were erected around the RC wall on both sides to increase the width of the wall to prevent air blast pressure from wrapping around the wall.

![Figure 7.6: The Process to Position the Blast Wall](image)

A reinforced concrete slab was constructed in front of the blast walls and a replaceable steel plate with a thickness of 20 mm was positioned on the slab, which was laid under the charge to avoid excessive damage to the slab. This setup provided a fully-reflective surface. The test setup is illustrated in Figure 7.7. Figure 7.8 shows the test site prior to one of the detonations.
Figure 7.7: Schematic of Blast Wall and Parameters Test Setup (Elevation View)

P1 = 1.5 m, P2 = 4.5 m, P3 = 7.5 m, P4 = 10.5 m

Figure 7.8: Test Site Prior One of the Detonations

An ammonium nitrate-fuel oil mixture (ANFO) was used as the explosive material, and non-fragmenting cube wood containers were used of a size that was based on the charge
weight. The charge weight was 125 kg ANFO for each test with an equivalent TNT value of 102.5 kg, and the charges were positioned 4 m from the test walls in the front and were placed on a wooden table at a height adjusted to position the center of the explosive at the mid height of the walls. The charge was elevated to 1.0 m above rigid ground. The charge was arranged by first placing half of the charge weight in the container, followed by adding the booster and blasting cap, which acted as the detonator in the center of the container, and then adding the remaining half of the charge. The ignition point was fixed at the center of the charge. The charge was boosted by a suitable quantity of 0.4 kg pentolite explosive. Figure 7.9 illustrates the preparation of the ANFO charge.

Figure 7.9: Preparing the ANFO Charge

Four pressure gages were used and installed on steel column located behind each wall to measure the blast’s reflected pressure. The tops of these gages were coated with a silicone compound to eliminate the effects of heat radiation during the explosion. To measure the specimen displacements, two accelerometers were used and located on the back of the wall as shown in Figure 7.10. Eight stain gages were used and installed in different locations on the longitudinal reinforcements on both the front and the back of the walls as shown in Figure 7.10. All of the reflected pressures, accelerations and strains data were simultaneously recorded using a high speed data acquisition system and were
based on the transmission cable properties; the data acquisition system was placed 25 m away from the each specimen to provide accurate results and avoid signal distortion.

<table>
<thead>
<tr>
<th>50 cm</th>
<th>SG-1</th>
<th>Acc. (B)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 cm</td>
<td>SG-2</td>
<td>Acc. (A)</td>
</tr>
<tr>
<td>180 cm</td>
<td>SG-3</td>
<td></td>
</tr>
</tbody>
</table>

**Figure 7.10: Locations of the Strain Gages and the Accelerometers**

The measurement of the shock waves, displacements, accelerations and strains is an important element with regards to assessing the performance of or structural response to, blast loading. These parameters were measured using a high speed data acquisition system which includes National Instruments data recording hardware and LabVIEW software for data collection and analysis as well as a signal conditioner, sensor transducers and cables that are connected to the test specimen. The sampling rate here can be up to 100 ks/s/channel. A high speed digital video camera was used to capture the explosion event. A rugged notebook PC was used in conjunction with the digital camera for image acquisition.

Reflected pressures were measured using a piezoelectric pressure sensor, which was screwed onto the non-responding column and their direction was perpendicular to the blast wave. Sensors from PCB Piezotronics models 102B, 102B03, and 102B04 were utilized and the sensors were installed on a non-responding column that represented the target building to determine the reflected pressure along the height of the rigid column.
A uniaxial shock accelerometers model 350A13 Piezoelectric ICP® was selected and used in this study. It had a maximum measurable shock of 10,000g. The sensors were attached by mounting them to the back side of the wall and connected to the data acquisition system module.

Strain gages manufactured by Vishay CEA-06-125UN-350 were used and installed on the vertical reinforcing bars to measure the deformation inside the test specimen after the blast test. These strain gages had a resistance of 350 Ω with a gage factor of 2.11. The rebar were straightened prior to installing the strain gages. In order to protect the steel strain gage from damage due to the concrete pouring and moisture, a coating of an epoxy layer was provided. Shielded vinyl lead wire cables (3 mm diameter 3 cores) were used to connect the strain gages to the data acquisition system. Figure 7.11 shows the installation of the strain gages. Each strain gage was given a designation to indicate its location according to its particular arrangement, i.e. strain gauge 1 at the front surface was designated as SGF-1. Prior to testing and after each installation step, the resistance of each gage was checked for proper resistance using a digital voltmeter as shown in Figure 7.11.

Figure 7.11: Strain Gages Installations

Blast wave characteristic, including reflected pressures were recorded as well as the post-blast damage and the mode of failure of the blast walls were observed. Experimental observations were used to evaluate the wall’s performance and to determine their capacity and failure limits. In the case where there was no wall, the blast characteristics
were compared with those obtained using A.T.BLAST software to validate the results obtained from the experiments.

7.5 Test Results and Discussion

Visual observations as well as the recorded pressures, strains, and accelerations plus a discussion of the results are presented in the following sections. Selected images of the blast explosions are captured by a high speed camera as shown in Figure 7.12. The images show the fireball, shock wave, and the reflection.

Figure 7.12: Selected High Speed Images for Blast Wave Propagation
7.5.1 Post-Blast Observations and Qualitative Wall Damage Classification According to ASCE 59-11

Based on the analysis of the data collected during each test and the close inspection of the post-blast condition of each specimen, the following observations can be made.

7.5.1.1 T-RC-W1 Test

Full-scale blast testing was first conducted on an unprotected T-RC reinforced concrete wall. The wall measured 4 m in wide, and 2 m in high, and had a thickness of 0.4 m. The wall was reinforced with T20 vertical and horizontal bars with 200 mm spacing on both the front and the back sides of the wall, and a 40 mm concrete cover with 10 mm stirrups spaced 200 mm were used. Figure 7.13 and Figure 7.14 show the wall both before and after the test.

The wall was tested at a scaled standoff distance of $Z = 0.86 \text{ m/kg}^{1/3}$. In this case there was no significant observable wall deformation or any sign of local failure in the wall. However, at the connection of the front face of the wall (blast-facing) there was a crack that ran along the full length as shown in Figure 7.14. Cracks were observed also on the back surface of the wall and were located at the mid height of the wall. The crack patterns formed on the back side of the wall are presented in Figures 7.14. There were three major flexural cracks only on the back side of the wall, which stretched horizontally along the full width of the wall.

According to the qualitative damage descriptions given in ASCE/SEI 59-11 and based on the damage observed in this wall, the wall could be classified as Superficial to Moderate damage. Since the purpose of the blast wall was to protect the target building located behind it, this level of damage was considered acceptable.
Figure 7.13: The Front Surface of (T-RC-W1) Before the Test

Figure 7.14: The Front and the Back Surface of (T-RC-W1) After the Test
7.5.1.2 T-RC-W2 Test

This wall (T-RC-W2) was similar to wall T-RC-W1 except that it was protected with one layer of aluminum foam with a thickness of 40 mm on its front surface (blast-facing). Figure 7.15 shows the wall before the test. Similar to the unprotected wall, the crack formation after the test blast indicated only flexural cracks, which stretched horizontally across the full width of the wall. The cracks were observed only on the back surface of the wall. The core of the aluminum foam was entirely intact and was still well bonded to the concrete wall. However, a portion of the steel skin cover spilt off from the aluminum foam core. Because the pressure of the close range blast loading was not uniform, this skin played an important role in determining the behaviour of the foam as illustrated in Figure 7.16. According to the qualitative damage descriptions provided in ASCE/SEI 59-11 and based on the damage observed to this wall, the wall could be classified as Superficial damage.

![Figure 7.15: The Front and the Back Side of the (T-RC-W2) Before the Test.](image)

(a) Front Surface  
(b) Back Surface
7.5.1.3 T-RC-W3 Test

This wall (T-RC-W3) was similar to wall T-RC-W1 except that it was protected with one layer of aluminum foam with a thickness of 40 mm on both the front and back surfaces. Figure 7.17 shows the wall before the test. Similar to the damage suffered by the unprotected wall, the crack formation included only flexural cracks, which stretched horizontally across the full width of the wall. However, these cracks reduced to only minor. The core of the aluminum foam was entirely intact and was still bonded well with the concrete wall. Once again a portion of the steel skin cover spilt off the aluminum foam core. Because the pressure of the close range blast loading was not uniform, the skin played an important role in determining the behaviour of the foam as illustrated in Figure 7.18. According to the qualitative damage descriptions provided in ASCE/SEI 59-11 and based on the damage observed to this wall, the damage of the wall could be classified as being Superficial. Hence, this wall experienced the least damage when compared to that suffered by the other walls in the current study.
7.5.1.4 T-RC-W4 Test

This wall (T-RC-W4) was similar to wall T-RC-W1 except that it was protected with two layers of aluminum foam with thickness of 40 mm on the front surface and one layer on the back surface. Figure 7.19 shows the wall before the test. Similar to the unprotected wall, the crack formation consisted of only flexural cracks, which stretched horizontally.
along the full width of the wall, but were reduced to only minor cracks. Most of the aluminum foam layers were spilt off the front surface of the concrete wall while the aluminum foam was entirely intact and was still well bonded to the back surface of the concrete wall and suffered no damage. These observations point towards the need to perform further evaluations of the possible benefits of further increasing the numbers of aluminum foam layers.

Such evaluations would include an examination of both the overall protection offered by retrofitting an RC wall with several layers of aluminum foam and to how improve the bond of the aluminum foam layers. There was no significant observable wall deformation or sign of local failure in the wall while at the connection of the front face of the wall (blast-facing) there was a crack, which ran along the full width as shown in Figure 7.20. According to the qualitative damage descriptions provided in ASCE/SEI 59-11 and based on the damage observed to this wall, the damage of this wall could be classified as Superficial to Moderate.

Figure 7.19: The Front and the Back Side of the (T-RC-W4) Before the Test.
Figure 7.20: The Front and the Back Side of the (T-RC-W4) after the Test.

7.5.2 Wall Response and Quantitative Wall Damage Classification According to ASCE 59-11

Displacement time histories, which were taken at the middle of all the walls, were computed. Accelerometers were used to measure the acceleration. Then the displacements were computed as the second integration of the acceleration-time histories.
The accelerations that were measured at point (A) are shown in Figure 7.21 to Figure 7.23. It can be clearly seen that the acceleration of the protected wall (T-RC-W3) was less than that for the other walls. This wall was protected with one layer of aluminum foam on both its front and the back surfaces.

Figure 7.21: Measured Acceleration – Time History (T-RC-W2)

Figure 7.22: Measured Acceleration – Time History (T-RC-W3)
The velocity-time histories that were calculated at point (A) by integrating the measured acceleration-time histories are shown in Figure 7.24. It can be clearly seen that the velocity acquired by the protected wall (T-RC-W3) was smaller than that acquired by the other walls examined in this study.

Figure 7.23: Measured Acceleration – Time History (T-RC-W4)

Figure 7.24: Calculated Velocity – Time History
The displacement-time histories at point (A) that were calculated by the double integration of the measured acceleration-time histories are shown in Figure 7.25. It can be clearly seen that the displacement reached by the protected wall (T-RC-W3) was smaller than that reached by the other walls.

![Figure 7.25: Calculated Displacement – Time History](image)

The peak deflections from the wall displacement response histories were converted to an approximate support rotation by taking the arctangent of the ratio of the deflection to the deflected length. By using the ASCE 59-11 performance (rotation) limits, each wall in the current study was assigned a damage level as classified by the code as illustrated in Table 7.3.

<table>
<thead>
<tr>
<th>Wall</th>
<th>Scaled distance, m/kg(^{1/3})</th>
<th>Deflection, mm</th>
<th>Rotation, θ</th>
<th>Code-Quantitative Damage State</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-RC-W-2</td>
<td>0.86</td>
<td>12.31</td>
<td>0.88</td>
<td>Superficial</td>
</tr>
<tr>
<td>T-RC-W-3</td>
<td>0.86</td>
<td>7.31</td>
<td>0.53</td>
<td>Superficial</td>
</tr>
<tr>
<td>T-RC-W-4</td>
<td>0.86</td>
<td>34.77</td>
<td>2.49</td>
<td>Moderate</td>
</tr>
</tbody>
</table>

The time-dependent strains that were measured at different locations in the front and back of the wall indicated that the strain measured on the protected wall (T-RC-W3) was
significantly less than the strain measured on the other walls examined in this study. A comparison of the time-dependent strains at two locations namely, SGF2 for the front wall and SGB4 on the back of the wall, are shown in Figure 7.26 to Figure 7.31.

![Figure 7.26: Measured Strain – Time History (T-RC-W2), Front Side Point 2](image1)

![Figure 7.27: Measured Strain – Time History (T-RC-W2), Back Side Point 4](image2)
Figure 7.28: Measured Strain – Time History (T-RC-W3), Front Side Point 2

Figure 7.29: Measured Strain – Time History (T-RC-W3), Back Side Point 4
Figure 7.30: Measured Strain – Time History (T-RC-W4), Front Side Point 2

Figure 7.31: Measured Strain – Time History (T-RC-W4), Back Side Point 4
7.5.3 Comparison of Safe Scaled Distance with Published Results by Other Authors

The quantity of the explosive and the standoff distance between the explosive and the target building are the main factors contributing to building damage. In this study, the distance of the explosive from the site to the target building was converted to a quantity that was identified as the Hopkinson-Cranz or cube root scaled distance (Z) and it was taken to be equal to the distance from the charge to the target divided by cube root of its weight (equivalent to TNT charge weight).

The safe scaled distance is used as the parameter in some regulations (e.g., US DoD, 2004 and ASCE, 1997) in assessing the structural safety of building to resist airblast loads. The values used in these manuals are for conventional structures only without any blast hardening; these values are 4.46 m/kg$^{1/3}$ and 4.65 m/kg$^{1/3}$, respectively. The scaled distance used in this study was 0.86 m/kg$^{1/3}$ and it was much smaller than that specified in the regulations. Moreover, there was no significant observable wall deformation or any sign of local failure in all of tests. It should be noted that the effects of differing structural materials and building configurations are not taken into consideration in these guidelines. Moreover, the definition of structural damage is not clear.

The safe scaled distance used in this study was also smaller than that suggested by Zho and Hao (2008). It should also be noted that the critical curves and the analytical formulae that they presented were based on numerical results only and were not validated by experimental results. These authors also did not consider the potential impact of walls having different dimensions, or different reinforcement ratio, or being under different boundary conditions.

There are also other studies by Chengqing (2011), which were performed experimentally on RC slabs protected with aluminum foam of different densities and thickness that were subjected to blasts at different scaled distances, and to charges of different shapes. However, the charges were suspended in the air and hence, there was no reflection from the ground. The author of these studies indicated that the aluminum foam provided good
protection to the RC slabs but neglect to include information about the safe scaled distances.

7.5.4 Blast Pressure Distribution behind the Wall

Figure 7.32 shows the typical pressure and impulse time histories captured by one of the reflected pressure gages after the detonation of 125 kg of ANFO at a 16.4 m standoff distance. They show the typical features of a pressure profile induced by the detonation of a conventional explosive, i.e. zero time, exponential decay, positive phase, and negative phase.

![Figure 7.32: The Reflected Pressure and Impulse Time History Profile](image)

A.T.BLAST software was used to compute the reflected shock wave parameters during the positive phase only, Pressure (P), Impulse (I), and positive phase duration (t_d). Table 7.4 shows the reflected pressure results obtained from this program where there is no wall in front of the building to verify the experimental results. The results showed good agreement.
Table 7.4: Reflected Pressure Comparison of the Computed and the Experiments

<table>
<thead>
<tr>
<th></th>
<th>R, m</th>
<th>( \theta )</th>
<th>W, Kg</th>
<th>A.T.BLAST</th>
<th>Experiment</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>16.41</td>
<td>1.750</td>
<td>125</td>
<td>219.05</td>
<td>232.6</td>
<td>-6%</td>
</tr>
<tr>
<td>P2</td>
<td>16.77</td>
<td>12.10</td>
<td>125</td>
<td>207.39</td>
<td>149.6</td>
<td>39%</td>
</tr>
<tr>
<td>P3</td>
<td>17.64</td>
<td>21.62</td>
<td>125</td>
<td>183.54</td>
<td>141.2</td>
<td>30%</td>
</tr>
<tr>
<td>P4</td>
<td>18.95</td>
<td>30.10</td>
<td>125</td>
<td>155.48</td>
<td>158.6</td>
<td>-2%</td>
</tr>
</tbody>
</table>

Another goal of the experiment was to measure the effectiveness of a T- Reinforced Concrete wall both with and without aluminum foam and assess its effectiveness to reduce structural loads transmitted to the rigid face located behind the barrier wall. Four reflective pressure gauges were placed on the structure along the centerline of the structure at heights of 1.5 m (Target 1), 4.5 m (Target 2), 7.5 m (Target 3), and 10.5 m (Target 4). During the experiment, all data were captured. The explosive threat was a 125 kg ANFO charge. The charge-to barrier standoff was 4 m, and the charge-to-structure standoff was 16.4 m.

The vertical axis represented the height of the measurement location on the building, while the pressure and impulse were plotted along the horizontal axis, to produce a profile along the building height. The corresponding impulse values (area under pressure-time history) were computed by the integration of the pressure-time history. The negative phase values were not considered in this study because they generally have small peak pressures and long durations.

A comparison of the measured reflected pressure along the height of the building for all of the walls is shown in Figure 7.33 while Figure 7.34 shows the comparison of the computed impulses. It was observed that the wall (T-RC-W3) was more effective in reducing pressures and the corresponding impulses. In the case where there was no wall; the signals from the lower transducers located at Targets 1 and 2 were interrupted, which prevented the calculations of the reflected impulses.
Figure 7.33: The Effect of Protection T-RC Wall in Reducing the Reflected Pressure along the Height of Building behind the Wall.

Figure 7.34: The Effect of Protection T-RC Wall in Reducing the Impulse along the Height of Building behind the Wall.
7.6 Summary and Conclusions

The aim of the tests conducted here was to investigate the behaviour of a reinforced concrete wall subjected to a single blast after having been retrofitted with aluminum foam. A Full-Scale blast tests were conducted on four T-RC walls protected with one or two layers of aluminum foam. The walls were subjected to a 125 kg ANFO cuboid charge (equivalent to 102.5 kg TNT) that was placed on a wooden table at a height of one meter and at standoff distance of 4 m. The charge was ignited at its center. The resulting displacement and the strain response caused by each blast were measured during the test.

During this time, the blast pressures were recorded by four pressure gages, which were located at different heights along the face of the building behind the wall. Accelerometers were also placed at the middle height of the wall (away from the blast side). Strain gages were used at different locations on the longitudinal reinforcements both on the front and back sides of the wall. The acceleration results were numerically integrated to obtain displacement readings. The response of the wall to each blast was monitored by a high speed data acquisition system, which was developed for this study. More important to this study, a high speed video camera recorded the blast event.

The level of damage to each wall was visually observed and compared qualitatively with the damage classifications in accordance to the ASCE 59-11 (ASCE 2011) standard as well as the performance (rotation) limits. Each wall in the current study was classified by damage level in accordance to the code.

Safe scaled distance is used in some regulations (e.g., US DoD, 2004 and ASCE, 1997) as one parameter in assessing the structural safety of a building to resist airblast loads. The safe scaled distance in the current study was compared with those specified in the regulations and in previous studies. Since the purpose of blast wall is to protect the target building located behind it, then this level of damage can be acceptable for the canopies walls (T-RC walls) because the combination of the energy reflection and energy absorption by the canoped walls can mitigate the effect of blast wave loading developed
behind the wall and the canopy will still remain in place and reduced the magnitude the potential damage of the blast wave diffracting over the wall.

The present experimental results showed that there was a significant reduction in terms of the safe scaled distance as well as the blast wave resultants located behind the wall compared to the previous results obtained by other researcher. This is may be due to the nature of the full-scale experiment in which a real wave propagation problem can be simulated by taking into consideration real conditions. However, the results follow the same general trend with respect to the shape of the blast wave. It can be concluded that a 125 kg of ANFO and below cannot destroy a blast wall. Thus, the building being protected by a wall would be safe from the blast effects.

The existing methodologies for determining the effectiveness of blast walls, such as those presented in (TM5-853-3) are valid within only a very limited range of scaled parameters (wall height, charge-to-wall stand-off, and wall-to-structure- distance) and the methodologies only examined plane rigid walls. The empirical design charts assume either a spherical (free-air) or a hemispherical (surface) burst and do not take into considerations variations in charge shape, charge orientation or its point of detonation. In the empirical design charts, the point of detonation was assumed to be in the middle of the sphere charge. The assessment of damage caused by the of air blasts from charges in the free field is well understood while the variation in blast resultants occurring along a building’s façade protected by a blast wall is not fully understood. Alternatively, the results of these experiments or the FE codes such as LS-DYNA can be used to assess the overpressure of the detonations of the non-spherical charges and the reflection due to the geometry of the target structure or that of adjacent structures.

However, the methodology used in these manuals is similar to that used in this study. The reflected pressures and impulses generated along the height of the building located behind each wall were investigated. It was found that the T-RC-W3 provided a higher reduction in the reflected pressure and impulse compared to the other walls investigated in this study.
It was observed that T-RC-W3 which was protected by one layer of the aluminum foam on both its front and back surfaces, experienced less damage compared to the other walls investigated. There was also a reduction in the acceleration and in the displacement as well as in the strains in the current study. The study also showed greater reduction in the reflected pressures and the impulses generated along the height of the protected building located behind the wall. The system developed here revealed the best means to protect a T-RC wall and reduce the blast wave resultants on a building located behind a wall. Surprisingly, T-RC-W4 did not perform well even though it was protected with two layers of aluminum foam on the front face and one layer on the back of the wall.

It is recommended that further research be conducted for walls of a T-Shape to improve the connections. Moreover, more research is needed on reinforced concrete walls with a T-Shape to study the effect of aluminum foam layers, their densities and the necessary anchorage techniques, which could include epoxy adhesives, combined with mechanical fasteners to strengthen the bond between the aluminum foam and the walls.

7.7 Acknowledgment
The authors gratefully acknowledge the financial support provided by the Saudi Government.
7.8 References


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Chapter 8

8 Performance of Double Reinforced Masonry Wall Retrofitted with Aluminum Foam under Near-Field Blast Loading: Full Scale Experiments

8.1 Introduction

One of the most common methods of construction in buildings is the use of concrete masonry walls. Concrete masonry provides a fast and inexpensive way to construct buildings of various heights. Hollow concrete masonry walls are widely used in building construction in Saudi Arabia. The masonry units come in many different sizes and shapes, and can be made with a variety of materials including concrete, and clay. A concrete masonry unit (CMU) is made from a mixture of Portland cement and aggregate under controlled conditions and must meet the requirements of the American Society for Testing and Materials (ASTM) standards. Masonry walls are site constructed using manufactured masonry units and site mixed mortar. The blocks are categorized based on their weight (lightweight, normal weight and heavyweight).

Lightweight units are used under non-load bearing conditions. Normal weight and heavyweight blocks are used in structural masonry applications. Masonry walls also typically increase the fire resistance of a wall, or of its structural elements. Masonry walls can be single- or multi-wythe; the interior cells of the units comprising the masonry walls may be empty or grouted. “Reinforced masonry” generally refers to cases where steel reinforcing bars have been inserted vertically within the interior cells, and then grouting only those cells (partially grouted) or all cells (fully grouted). Horizontal reinforcing can also be used in the form of rebar being laid horizontally within grouted cells and/or as wire mesh joint reinforcement. Because the mass of a structure in dynamic loading plays an important role in the overall resistance, fully grouted wall sections generally perform much better than ungrouted sections.

Building envelopes usually comprise nonreinforced masonry walls built into beam-column frames. However, due to low flexural capacity and brittle mode failure caused by
weak mortar tensile bond strength, masonry walls do not generally provide any resistance against out-of-plane lateral pressure that may result from blast loads. Under blast loading, the failure of a masonry wall is likely to be sudden and severe, producing wall fragments that may cause injuries (Baylot et al. 2005) exceeding those due to the blast itself as shown in Figure 8.1. Hence, there is a need to strengthen such walls in a quick and effective way. Different approaches may be used to harden buildings. Some of the traditional methods used to upgrade masonry structures are: filling of cracks and voids by grouting, stitching of large cracks and other weak areas with metallic or brick elements, single or double sided jacketing by cast-in-situ concrete, etc. Another option includes the addition of a steel column and a plate where a number of steel columns is secured behind the wall and connected to the building frame. Steel plates connect the flanges of the columns together thereby producing a tension membrane (Ward 2004). Although the techniques mentioned above are effective in increasing the strength and stiffness of unreinforced masonry buildings, they add considerable mass to the structure that can alternately modify the dynamic response characteristics of the structure (Velazquez Dimas and Ehsani 2000).

Figure 8.1: Failure Masonry Wall Due to Blast Loading
8.2 Statement of Problem

The majority of the governmental and residential buildings in Saudi Arabia are surrounded by either unreinforced masonry or reinforced concrete perimeter walls. Most existing masonry walls have little strength to withstand blast loads and the unreinforced masonry (URM) walls have a low resistance against out-of-plane blast loading due to their low flexural capacity and their brittle mode of failure. To date based on the information available in the open literature; most research programs have focused on understanding the effectiveness of the plane rigid walls using small scales to reduce the blast wave resultants on a protected building behind it.

Blast loads can be generated and applied to the structures using a ConWep computer program or detonation simulation methods. ConWep is based on a collection of conventional effects calculations provided in TM5-855-1 (U.S. Department of the Army, 1998); Hyde 2005; United States Army Corps of Engineers). ConWep is a standalone program and is restricted to be used only by the U.S. Military but it is also implemented in LS-DYNA, which is available for researchers. In this method, the blast pressure is applied directly to the front surface of the lagrangian structure. This approach can be used for the analysis before the failure of the structural surface. An eroding technique is used in this approach to eliminate damage elements from the structure to avoid element distortion. The blast load continues to act on the uneroded elements. Therefore, the simulation using this method underestimates the damage to the structure because the load is not transmitted to the concrete core after the spalling of the concrete surface; this is one of the disadvantages of this method.

Nevertheless, this method is simple and it requires for the input only the charge weight as the equivalent of TNT weight and its position relative a structure according to Simoens et al. (2011). As a consequence, the computational cost is less than that the incurred other simulation approaches. Another disadvantage of this method is that it does not account for reflections from the ground and the structure. Likewise, this approach cannot take into account the shadowing of the blast wave due to the presence of the blast wall or
other adjacent structures. Also it does not take into account the effects of the charge shape, the ignition point, or the orientation of the charge and the structure.

Another numerical approach that can be used to model the explosion and structure involves modeling of the air and explosive with multi-material-arbitrary Lagrangian-Eulerian (MM-ALE) formulation in which a proper equation of state is assigned to the materials and a burn model controls the explosive’s detonation behaviour. In this method, the structure is treated as a Lagrangian and Fluid-Structure-Interaction (FSI), which is used for communication between the Lagrangian and the ALE domains, according to Hallquist et al. (2012). This method is appropriate at close-range of where the distance between the charge and the structure is short according to Yi et al. (2013). There are several advantages of this method over the pressure load method that is used in ConWep software: (1) the blast wave load continues to act on the structure after the elements erode from the structural surface; (2) the reflection and diffraction of the blast wave can be predicted, and (3) it can consider the effect of the charge shape and the ignition points. The main disadvantage of this method is the large size of the air blast model that usually needs to be included to mitigate the boundary effects, which increases the computation time because of the large domain.

Blast loads in simple geometries can be predicted using empirical or semi-empirical methods, such as those presented in the UFC 340-01 and UFC 340-02 manuals. These methods can be used to estimate the blast loading on isolated structure when there is no barrier. However, there is no direct method that can be used to estimate the blast resultants on a building façade if there is a blast wall between the explosive center and the structure. Remenniko (2003) presents a review of the existing methods for predicting blast effects on buildings The existing methodologies for determining the effectiveness of blast walls, such as those presented in (TM5-853-3) are valid over only a very limited range of scaled parameters (wall height, charge-to-wall stand-off, wall-to-structure-distance). The empirical design charts assume either a spherical (free-air) or hemispherical (surface) burst and do not show variations in charge shape, charge orientation or the point of detonation of the charge. In the empirical design charts, the
point of detonation is assumed to be in the middle of a spherical charge. The assessment of air blasts from charges in the free field is well understood while the variation in blast resultants along a building’s façade that is protected by a blast wall is not.

The knowledge of blast characteristics is very important in predicting blast effects - especially those at close range. There are basic elements that have a significant impact on the pressure and impulse values of a blast wave that is propagated in a different direction in air, especially close to an explosion. These elements include: dimensions and, shape of charge as well as the ignition point, type of explosive, and structure geometry plus the orientation of the charge and the structure.

The majority of the previous experimental results are related to the spherical shape of an explosive charge. As a result, there is a lack of a sufficient number of test results for other charge shapes. Therefore, there are some difficulties in precisely defining the impact of the blast wave of non-spherical charges and its ignition point on structures.

The aim of this test was to investigate the behaviour of reinforced hollow concrete blocks (CMU) wall subjected to a single blast load after having been retrofitted with aluminum foam. A full-scale blast test was conducted on three masonry walls reinforced with one or two layers of aluminum foam. The walls were then subjected to a 25 kg ANFO charge (equivalent to 20.5 kg of TNT) placed on a wooden table at a height of one meter and at standoff distance of 4 m. The resulting displacement and the strain response caused by blast were measured during the test. For each test, a charge of explosive material with a weight deemed by the SSF to be representative of a terrorist threat was placed at a particular distance from the wall (typically 4 m) and detonated.

During this time, the blast pressure was recorded by four pressure gages, which were located along the height of building behind the wall. Accelerometers are also placed at the top height and middle of the wall (away from the blast side). Gages were used at different locations in the front and at the back of the wall. The acceleration results were numerically integrated to obtain displacement readings. More important to this study, high speed video recorded the behaviour of the walls during the blast load.
8.3 Literature Review

The safe scaled distance given in some regulations (US DoD, 2004 and ASCE, 1997) was used as the parameter in assessing the structure’s ability to resist airblast loads. The U.S. DoD (2004) specifies a safe scaled distance of $4.46 \ m/kg^{1/3}$ for unstrengthened buildings to ensure that the building would not be destroyed. The scaled distance given in such regulations is usually obtained from field blasting tests on scaled structural models or on low-rise residential-type structures. The downside of the guidelines is that the effects of the structural materials and the various configurations are not taken into consideration even though they affect structural performance significantly. Moreover, the definition of structural damage is vague in these guidelines.

Wu and Hao (2007) proposed safe scaled distances for masonry infilled Reinforced Concrete (RC) frame structure subjected to air blast loads using a homogenized model for masonry. In their study, three structural models represented low and medium rise unstrengthened RC frame structures with infilled masonry walls were analyzed. They were a one-storey, a two storey, and an eight-storey RC frame building. The size of the one storey building was 5.0 x 5.0 x 3.3 m, and the reinforced concrete roof was 150 mm thick. The columns and beams of the building had a cross section of 300 x 300 mm and 200 x 300 mm with a 2% longitudinal reinforcement. The hoop reinforcements for the columns and beams were 10 at 200 mm. The height of each storey was 3.3 m. The span length and width of the eight-storey building were 10.0 x 5.0 m, respectively. The height of all the storeys was 3 m. A uniform cross section of 400 x 400 mm for the columns and 200 x 400 mm for the beams was used. A longitudinal reinforcement ratio of 3% was used. Unreinforced solid clay brick masonry with a thickness of 240 mm was used and the size of the brick was 230 x 110 x 76 mm. Based on their numerical studies, the scaled distance necessary to prevent the collapse of one-storey and two-storey masonry infilled RC frame building was about $1.80 \ m/kg^{1/3}$ while for the eight-storey RC building, the scaled distance to prevent its collapse was $1.18 \ m/kg^{1/3}$. The safe standoff distance to prevent excessive damage to the low-rise structures against a blast load was $4.5 \ m/kg^{1/3}$, whereas it was $5.6 \ m/kg^{1/3}$ for medium rise structures.
Ahmad S. et al. (2014) conducted full scale test on a cantilevered masonry wall constructed from solid clay bricks, the wall was subjected to six different amounts of explosives from 4 kg to 14 kg (equivalent to 2.4 kg to 8.4 kg of TNT) in increments of 2 kg; the standoff distances were 3 m and 4 m respectively. The wall’s dimensions were 2 m x 2 m with a thickness of 0.37 m. A nitroglycerin based dynamite explosive was used. The charge was placed on a wooden table 1 m above unpaved ground. The pressure time history, the acceleration time history and the strain were all measured. At a scaled distance of 1.81 m/kg$^{1/3}$, a large longitudinal crack along the front and back sides of the wall was observed. The brick wall collapsed at a scaled distance of 1.72 m/kg$^{1/3}$.

Darren L. Rice et al (2006) performed numerical analyses to study the response of a protective blast wall. The protective blast wall consisted of two brick walls with sand in between and a lower concrete Jersey-barrier placed in front of the wall. The response of the wall (both with and without the protective Jersey barrier) to several charge sizes was investigated. The response to a car-sized charge showed only moderate damage to the top of the front brick layer. In contrast, a much larger truck-sized charge was able to deform the complete movable section of the wall. For this sized charge, the added momentum due to the brick and sand impact on the protected target will significantly increase the total blast loading.

Numerous studies have been performed in an effort to improve the out-of-plane behaviour of URM wall systems, and have investigated various methods of retrofitting structures to mitigate the effects of blast loading. The use of modern materials, such as fiber-reinforced polymers (FRPs) and sprayed on polyurea, to retrofit URM wall systems have proven to be highly effective in improving both the load resistance and the deformability of URM walls subjected to out-of-plane loads (Velazquez-Dimas et al. 2000; Tumialan and Nanni 2001; Carney and Myers 2003). Additionally, the use of highly deformable elastomeric surface coatings to retrofit URM infills has been found to be highly effective in reducing the masonry debris scatter that is typically associated with URM infills subjected to blast loads (Connell 2002).
Davidson, et al (2004) used full-scale explosive tests to determine the effectiveness of using sprayed-on polymers to improve the blast resistance of unreinforced masonry walls. They concluded that the sprayed-on polymer retrofit approach to strengthening masonry walls against blast loads was an effective technique. Davidson, et al (2005) experimentally and numerically studied the damage and failure mechanisms of polymer reinforced concrete masonry walls subjected to blast loadings. They observed that a thin elastomeric coating on the interior face of the wall can be effective in minimizing the fragmentation and potential for collapse of unreinforced concrete masonry walls resulting from a blast. They also found that elongation capacity was more important for damage reduction than having a high level of stiffness, so an effective balance between stiffness and elongation potential was required. Finite element results indicated that a spray-on polymer reinforcement approach can be effective in reducing the vulnerability of unreinforced CMU walls subjected to blast loading.

Browning R.S. et al. (2008) conducted full scale testing on both fully and partially grouted concrete masonry walls with both vertical and horizontal joint reinforcement. The testing involved panels both with and without a clay brick veneer and polystyrene foam insulation. They found that the veneer enhanced the wall’s resistance due to the added mass, but did not significantly increase the section moment of inertia through composite action. They also found that the veneer ties provided sufficient strength and stiffness to transfer the forces from the reflected pressure from the veneer exterior to the structural wythe without significantly loading the insulation. It was found that the ungrouted cells of the partially grouted exterior walls tended to breach and turn into hazardous fragments similar to those of unreinforced masonry; therefore partially grouted walls should not be used when designing structures to resist a significant blast (UFC 3-340-02, 2008).

SU Yu et al (2008) carried out numerical simulations to investigate the performance of aluminum foam-protected URM walls subjected to blast loads. They used numerical calculations of the response and damage to 2500 mm×2500 mm× 110 mm URM wall both with and without retrofitting under airblast loads. The units used were typical 10-
hole cored clay brick, with nominal dimensions of 230 mm×110 mm×76 mm. the effect of airblast loads acting on the surfaces of a URM wall and an aluminum foam protected URM wall at different scaled distances with different charge masses was examined.

The size of the blast load was estimated using the TM5-1300 manual. A triangular pressure function with a peak pressure of $P_{r,max}$ and duration $t_d$ was assumed in the analysis. The pressure was uniformly applied over the incident face of the URM walls and aluminum foam protected URM walls. It was found that, the critical scaled distance to prevent failure of the URM wall was 9.0 m/kg$^{1/3}$. When the URM wall was subjected to smaller blast loads, such as at a scaled distance below 9 m/kg$^{1/3}$, it was usually damaged due to the growth of both shear cracks and the tensile cracks in the mortar joints, which had the appearance of step cracks. But, when the URM wall was subjected to larger blast loads such as those fired at a scaled distance of 4 m/kg$^{1/3}$, the masonry wall was blown out immediately.

The researchers found that when the URM wall was protected with one layer in the front surface, the wall suffered only light damage at the scaled distance of more than 4 m/kg$^{1/3}$ while when the scaled distance reached 3.3 m/kg$^{1/3}$, the aluminum foam sheet began to show signs of damage and debonding between the steel sheets/masonry interface was observed. The collapse of the protected wall occurred as the scaled standoff distance reached 2.3 m/kg$^{1/3}$. It was found that when the URM wall was retrofitted with a layer of 40 mm thick aluminum foam on both surfaces, the debonding failure between the aluminum foam and the steel sheets/URM wall occurred at a scaled standoff distance of 2.3 m/kg$^{1/3}$ and the wall failure occurred when scaled standoff distance reached 1.8 m/kg$^{1/3}$.

They also found that the thickness and density of the aluminum foam sheet played a significant role in mitigating the blast effects on the URM walls. It was also found that the higher the density, the smaller the response, i.e., the more effectively it mitigated the blast effects on the URM wall. It was also found that the greater the thickness, the smaller the response. That is, the more effectively it mitigated the blast effects on the URM wall. They found that a retrofitted URM wall with aluminum foam sheets on both sides could
absorb more than 14 times the blast energy of an URM wall without aluminum foam sheets, indicating that aluminum foam is a very effective means of mitigating blast effects on a URM wall.

Compounded with the uncertainties inherent in blast load and material properties, an accurate modeling of CMU wall behaviour when subjected to blast loads is a difficult problem to solve. These analytical complexities are traditionally avoided by using empirical design rules based on experience and experimental evidence (U.S. DOE 1992). Associated with these empirical rules, however, are significant inaccuracies, particularly when new geometry, material, types of construction, and blast variables are present. To avoid these problems, a number of researchers have analytically replicated CMU wall behaviour, typically with the finite element (FE) method. Some of these investigators have included Bogosian (1997).

The previous studies were limited to the application of brick to reduced scale unreinforced concrete block walls using numerical analysis. The goal of this study is to investigate the response of full-scale double reinforced hollow concrete block masonry walls when under blast loads and to examine the usefulness of aluminum foam of different layers as a protection material. Furthermore, the pressure distribution along the height of a building located behind a wall is investigated.

Recently, aluminum foam has attracted considerable attention in retrofitting concrete members and interest in material has extended to retrofitting masonry structures because aluminum foam panels can be applied to reinforced masonry walls. In this study, the investigation was focused on the aluminum foam material as well as on using polyurethane foam to fill the cells. However, there is still a need to develop a cost-effective RMW wall, which has an easy installation hardening technique to protect it against blast loads. The use of composite materials and the application of novel installation technique can enlarge the available data base and may lead to practical solutions to current problems. Further experiments are therefore required to gather enough data to allow researchers to conduct analyses and to process the results.
Proof-of-concept tests were performed using a blast-loaded double masonry wall and hollow concrete blocks, which were connected to increase the integrity of the wall and retrofitted with the aluminum foam material. In this study four full-scale explosive tests were conducted with the intent of determining the effectiveness of aluminum foam to improve the blast resistance of masonry walls. The tests were conducted to evaluate the aluminum foam application and the failure modes as well as to assess the general effectiveness and level of protection provided by the aluminum foam. This work describes the performance of reinforced masonry walls both with and without aluminum foam subjected to blast loading. Both unstrengthened and strengthened walls were tested and the tests were performed at the SSF Ranges in Saudi Arabia.

The data collected during explosive testing is often very limited due to the large amounts of dust, debris, and vibration that result from an explosion. However, the typical data that can be collected within reasonable levels of risk to the testing equipment includes: deflection histories, reflected pressure histories, and high-speed video recordings. When this data is successfully collected, the analyst can gain an understanding of the response of the system.

8.4 Experimental Program

Four full-scale double reinforced hollow concrete masonry walls were constructed and field tested under blast loads generated by an ANFO charge. All of the walls were built using 200 mm two-cell concrete blocks. The walls being tested were fully grouted, where there were hollow voids in the CMU blocks they were filled with polyurethane foam. In all of the walls, a reinforcing bar was placed in every cell; horizontal reinforcement was also used. Hence, a new construction method of reinforced hollow concrete masonry wall is proposed in this study. Construction method of the wall provided way to cause the blocks to work as one unit.

The dimensions of each wall were 10 blocks wide (4000 mm) by courses 10 blocks high (2000 mm). The wall thickness was two blocks (400 mm). All of the walls were face-shell bedded with 10 mm mortar joints. The walls were double and the arrangement of
the hollow concrete block units integrated each wall. All of the walls were constructed in running bond fashion to simulate the most common type of construction used in Saudi Arabia. After the walls cured, they were retrofitted with different types of aluminum foam layers. Sleeve anchor bolts were used to connect the aluminum foam to the walls. The performance of a non-retrofitted RMW wall under blast loads was used as a "control" case for comparison purposes.

8.4.1 Material Properties

The masonry walls were constructed using hollow concrete block units (CMU). Hollow concrete blocks (400 x 200 x 200 mm) were used. A hollow concrete block unit with two cores was used as shown in Figure 8.2. Vertical bars were placed in the center of each cell and horizontal reinforcements were placed between the courses. The walls were fully grouted and polyurethane foam was used to fill the cells of the hollow concrete units. Polyurethane foams are a lightweight material, which is widely used as an insulation and core material. It is a quick-expanding type of spray foam. The density is 62 kg/m³ when it is installed in the cells under no pressure (i.e. free to expand). The wall construction and its completed state are shown in Figure 8.3.

Figure 8.2: Standard Concrete Masonry Hollow Concrete Units Size Used in Research
Aluminum foams with air-filled closed cells were used in the present study. Its energy absorption capability stems from the energy required to compress the air filling the closed cells and then crushing the cell walls. It was found numerically in the previous studies of the aluminum foam performance under blast loading (Su Yu et al. [2008]) that the density and the thickness of the aluminum foam play significant roles in determining their energy absorption capabilities. By increasing the density of the aluminum foam or by using thicker foam, the energy absorbing capabilities of the aluminum foam plate will be increased. Therefore, the properties of the aluminum foam selected in the current study were based on the optimum findings in the previous studies. Details and some properties of the aluminum foam layers that were used in the present experiments are summarized in Table 8.1.

**Table 8.1: Detail and Properties of the Aluminum Foam**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density, kg/m$^3$</td>
<td>420</td>
</tr>
<tr>
<td>Thickness, mm</td>
<td>40</td>
</tr>
<tr>
<td>Dimension, m</td>
<td>0.5 x 0.5</td>
</tr>
<tr>
<td>Covered with 1.5 mm steel plate</td>
<td>yes</td>
</tr>
</tbody>
</table>
8.4.2 Test Matrix

Table 8.2 presents the test matrix of the RMW wall specimens which were tested under the blast loads. The walls were labeled from RMW1-RMW4 and each wall was subjected to a single shot. Walls (RMW-1-NP) and (RMW-2-NP) were not retrofitted and their performance under the blast load was used as a "control" case for comparison purposes. RMW-3-P1 was protected with one layer of aluminum foam on the front surface of the wall. RMW-4-P2 was protected with two layers of aluminum foam on the front surface and one layer on the back surface of the wall.

<table>
<thead>
<tr>
<th>Shot #</th>
<th>Specimen</th>
<th>Charge size ANFO (kg)</th>
<th>TNT equivalent, Kg</th>
<th>Standoff distance charge-to-wall, m</th>
<th>Standoff distance charge-to-building, m</th>
<th>Scaled distance, m/kg 1/3</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>RMW-1-NP</td>
<td>25</td>
<td>20.5</td>
<td>4</td>
<td>16.4</td>
<td>1.46</td>
</tr>
<tr>
<td>2</td>
<td>RMW-2-P1</td>
<td>25</td>
<td>20.5</td>
<td>4</td>
<td>16.4</td>
<td>1.46</td>
</tr>
<tr>
<td>3</td>
<td>RMW-3-P2</td>
<td>25</td>
<td>20.5</td>
<td>4</td>
<td>16.4</td>
<td>1.46</td>
</tr>
<tr>
<td>4</td>
<td>RMW-4-NP</td>
<td>125</td>
<td>102.5</td>
<td>4</td>
<td>16.4</td>
<td>0.86</td>
</tr>
</tbody>
</table>

Figure 8.4 shows the masonry concrete wall reinforcement and configuration layout. The parameters studied in this experimental program are as follows: the infill material (polyurethane foam), the hollow concrete block unit arrangements, and the feasibility of using the aluminum foam as a protective layer on the surface of a wall. The criterion for the selection of each charge size and standoff distance was based on the results obtained in previous studies and on the shortest distance allowed from the street to the perimeter wall in Saudi Arabia.
Figure 8.4: Masonry Concrete Wall Reinforcement and Configuration Layout
8.4.3 Test Setup and Instrumentation

The walls were precisely positioned in place on the test site for each test. Reinforced concrete wing walls were also erected on each side of the walls to eliminate the clearing effect around the edges of the walls.

Four pressure gages were used and were installed on a steel column located behind each wall to measure the blast’s reflected pressure. The tops of these gages were coated with a silicone compound to eliminate the effects of heat radiation during the explosion. To measure the specimen displacements, two accelerometers were used and located at the upper course and in the middle height of the wall as shown in Figure 8.5. Eight strain gages were used and installed in different locations on the front and on the back of the wall as shown in Figure 8.5. All of the reflected pressures, displacements, and the strains data were simultaneously recorded using a high speed data acquisition system. Based on the transmission cable properties, the data acquisition system was placed 25 m away from each specimen to provide an accurate reading to the results and avoid signal distortion.

![Figure 8.5: Locations of the Strain Gages and the Accelerometers](image)

The measurement of the shock wave, displacement, accelerations and strains is an important element with regards to the performance or structural response to blast loading. These parameters were measured using a high speed data acquisition system, which included National Instruments data recording hardware and LabVIEW software for data...
collection and analysis, a signal conditioner plus sensor transducers and cables that were connected to the test specimen. The sampling rate was up to 100 ks/s/channel.

Reflected pressures were measured using a piezoelectric pressure sensor screwed onto the non-responding columns; their direction was perpendicular to the blast wave. Sensors from PCB Piezotronics models 102B, 102B03, and 102B04 were utilized and the sensors were installed on non-responding column that represented the target buildings to determine the reflected pressure along the height of the rigid column.

A uniaxial shock accelerometers model 350A13 Piezoelectric ICP® was selected and used in this study. It had a maximum measurable shock of 10,000g. Figure 8.5 shows the sensor and how the sensor was attached by mounting it to the back side of the wall and how it was connected to the data acquisition system module.

Strain gages manufactured by Vishay CEA-06-125UN-350 were used and installed on the vertical reinforcing bars to measure the deformation inside the test specimen after the blast test. These train gages had a resistance of 350 Ω with a gage factor of 2.11.

The rebar should be straightened prior installing the strain gages. In order to protect the steel strain gage from damage due to concrete pouring and moisture, a coating with an epoxy layer was provided. Shielded vinyl lead wire cables (3 mm diameter 3 cores) were used to connect the strain gages to the data acquisition system. Figure 8.6 shows the installation of the strain gage. Each strain gage was given a designation to indicate its location according to the particular arrangement, i.e. strain gage 1 was designated as SGF-1. Prior to testing and after each installation step, the gage resistance was checked for proper resistance using a digital voltmeter as shown in Figure 8.6.
The walls were subjected to a 25 kg ANFO which has an equivalent TNT value of 20.5 kg for pressure calculations. In this study, a cuboid charge shape was used. The charge was arranged by first placing half of the charge weight in a container, followed by the placement of the booster and the blasting cap, which acted as the detonator in the center of the container, and then placing the remaining half of the charge in the container. The explosive was placed on a wooden table and elevated to one meter above rigid ground and connected to a detonation device outside the test structure. The detonation device was remotely controlled from a bunker. Accelerometers gages were mounted on each wall both at the top-height and in the middle to record the deflection history of the walls. Four pressure gages (referred to as gage 1 to 4) were mounted on a steel stand to record the reflected pressure. A high-speed digital video camera was used to capture the explosion event. A rugged notebook PC was used in conjunction with the digital camera for image acquisition. Figure 8.7 shows a completed wall just prior to the test.
8.5 Test Results

The visual observations as well as the recorded pressures, strain, and accelerations are presented in the following sections as well as a discussion of the results. Selected high speed images of the blast explosion captured by the high speed camera are shown in Figure 8.8. The images show the fireball, shock wave, and the reflection.
8.5.1 Post-Blast Observations and Qualitative Wall Damage Classification According to ASCE 59-11.

The American Corps of Civil Engineers standard [ASCE/SEI 59-11 (ASCE 2011)] for the design and analysis of structures subjected to blast loading uses qualitative damage indicators to classify the performance of the structural components and the expected level of damage. Four damage levels are classified in this standard: Superficial (visible permanent damage is unlikely), Moderate (permanent damage might be visible but is repairable and component failure is unlikely), Heavy (significant damage, mostly non-repairable and component failure remains unlikely), and Hazardous (component fails).
The tested walls were visually inspected after each blast test and the results are discussed in the following sections.

8.5.1.1 RMW-1-NP Test

Full-scale blast testing was conducted on unprotected fully grouted double reinforced concrete masonry walls. The walls were reinforced with T20 vertical bars in the center of each cell with 200 mm spacing and T12 horizontal joint reinforcement. According to the manual (UFC 3-340-02, 2008), partially grouted walls should not be used when designing them to withstand a significant blast. In the current study, expanded polyurethane foam was used to fill the cells of the hollow concrete; this material was selected due to its ability to reduce the blast wave transmission. A double wall was constructed by adding hollow concrete blocks to make an integration of the wall where the head joints in the back wall were different from those in the front wall to enhance the resistance of the whole wall. This wall was tested under a specific scaled distance \((z = 1.46 \text{ m/kg}^{1/3})\).

Figure 8.9 and Figure 8.10 show the wall before and after the test. The following damage was documented: horizontal cracks where appeared at the bed joints in the front face at 1\(^{st}\), 4\(^{th}\), 5\(^{th}\), 6\(^{th}\), and 10\(^{th}\) courses in the front of the wall (due to loss of bond between the mortar and the block). Minor horizontal bed joint cracks developed in the back surface at the 4\(^{th}\), 5\(^{th}\), and 6\(^{th}\) courses; the upper course, the 10\(^{th}\) exhibited lateral offset cracks indicating an out of plane sliding shear failure as shown in Figure 8.10. No mortar spalling was observed. The back wall suffered less damage due to the construction technique and the use of the polyurethane foam. According to the qualitative damage descriptions provided in ASCE/SEI 59-11 and based on the damage observed to this wall, the degree of damage to the wall could be classified as being Superficial to Moderate. Since the presence of the blast wall was to protect the target building located behind it, this level of damage is considered to be acceptable.
8.5.1.2 RMW-2-NP Test

The unretrofitted reinforced masonry wall was also tested higher scaled standoff distance ($Z = 0.86$ m/kg$^{1/3}$). Due to the large blast size, the wall completely collapsed after the blast event. Moreover, the debris was found approximately 15 m away from the wall. Figure 8.11 shows the wall before the test and Figure 8.12 shows the wall after the test.
According to the qualitative damage descriptions provided in ASCE/SEI 59-11 and bases on the damage observed in this wall, the damage to the wall classified as Hazardous.

The reinforced masonry wall was protected by one layer of aluminum foam on its front side using anchoring devices as shown before the test in Figure 8.13. The following damage was documented: the aluminum foam layer that covered the wall did not split off and only the 10th course on the back of the wall exhibited lateral offset damage and a...
wider crack opened up as shown in Figure 8.14. The protected wall suffered less damage compared to that of the unprotected wall at the same scaled distance (1.46 m/kg$^{1/3}$). According to the qualitative damage descriptions used in ASCE/SEI 59-11 and based on the amount of damage observed, the damage to the wall could be classified as Superficial.

**Figure 8.13:** The Front and Back Side of Protected Wall with One Layer of Aluminum Foam on the Front Side before the Test

**Figure 8.14:** The Front and Back Side of Protected Wall with One Layer of Aluminum Foam on the Front Side after the Test
8.5.1.4 RMW-4-P2 Test

When the wall was protected by two layers of aluminum foam on the front side and one layer of the aluminum foam on the back side as shown before the test in Figure 8.15, a portion of the aluminum foam in the front was split off the wall while the layer on the back side remained in place. Horizontal cracks at the bed joints on the front side of the wall at the 1st, 2nd, 4th, 5th, and 10th courses were observed as shown in Figure 8.16. However, these cracks were smaller than those observed in the unprotected wall. An observation of the wall, from the front, revealed that the unprotected wall suffered considerable more damage than did the protected one. Based on the damage observed to this wall, the wall could be classified as having suffered Superficial to Moderate damage. It was also observed that there was spalling at the bottom of the wall due to the foam splitting off from the wall. When using more than one layer of the aluminum foam to protect the wall, there was a need to have a special anchoring device to prevent the tensile failure of the foam.

![Figure 8.15: The Front and Back Side of the Protected Wall with Two Layer of Aluminum Foam on the Front side and One Layer on Back Side before the Test](image-url)
Figure 8.16: The Front and Back Side of the Protected Wall with Two Layer of Aluminum Foam on the Front Side and One Layer on the Back Side after the Test

8.5.2 Wall Response and Quantitative Wall Damage Classification According to ASCE 59-11

Displacement time histories at the upper and middle course of all the walls were measured. However, only the results observed at the top of the wall are presented here. In tracking these results a positive displacement value indicates an inward wall displacement while a negative value indicates an outward displacement. The accelerometers were used to measure the acceleration; the displacements were then computed as the second integration of the acceleration-time histories. The accelerometer gage 2 failed and did not provide any useful information.

The accelerations that were measured at the top of the wall are shown in Figures 17 to 19. It is clearly seen that the accelerations at the top of the protected wall are significantly smaller than those of the unprotected wall.

The velocity-time histories that were calculated at the top of the wall by integrating the measured acceleration-time histories are shown in Figure 20. It can be clearly seen that the velocities acquired by the protected wall are smaller than those acquired by the unprotected wall.
The displacement-time histories at the top of the wall that were calculated by a double integration of the measured acceleration-time histories are shown in Figure 21. It can be clearly seen that the displacement occurring with the protected wall is smaller than that occurring with the unprotected wall.

The time taken by each wall to reach its maximum displacement was longer than the entire positive phase duration of the blast event. This finding confirmed the assumption that the walls studied in the current study respond in their impulsive loading regime (Baker et al. 1983) under the scaled distance used in this study.

![Figure 8.17: Measured acceleration – time histories at the top of the unprotected wall](image)
Figure 8.18: Measured acceleration – time histories at the top of the wall protected with one layer of aluminum foam

Figure 8.19: Measured acceleration – time histories at the top of the wall protected with two layers in front and one layer in the back of the wall by aluminum foam
The peak deflection from the wall displacement response histories were converted to an approximate support rotation by taking the arctangent of the ratio of the deflection to the deflected length. By using the ASCE 59-11 performance (rotation) limits, each wall in the current study was assigned a damage level and classified by a code as illustrated in Table 8.3.
### Table 8.3: Wall Support Rotations and Corresponding Code Damage States

<table>
<thead>
<tr>
<th>Wall</th>
<th>Scaled distance, m/kg$^{1/3}$</th>
<th>Deflection, mm</th>
<th>Rotation, $\theta$</th>
<th>Code-Quantitative Damage State</th>
</tr>
</thead>
<tbody>
<tr>
<td>RMW1-NP</td>
<td>1.46</td>
<td>28</td>
<td>0.80</td>
<td>Superficial</td>
</tr>
<tr>
<td>RMW2-P1</td>
<td>1.46</td>
<td>23</td>
<td>0.66</td>
<td>Superficial</td>
</tr>
<tr>
<td>RMW3-P2</td>
<td>1.46</td>
<td>18</td>
<td>0.52</td>
<td>Superficial</td>
</tr>
<tr>
<td>RMW4-NP</td>
<td>0.86</td>
<td>Failure</td>
<td>Failure</td>
<td>Blowout</td>
</tr>
</tbody>
</table>

The time-dependent strains that were measured at different locations in the front and on the back of the wall indicated that the strain measured on the protected wall was significantly smaller than the strain measured on the unprotected wall.

A comparison of the time dependent strains at two locations namely, FSG2 for the front of the wall and BSG2 for the back of the wall, for an unprotected RMW-NP1 wall and protected wall RMW-P1 are shown in Figures 22 to 25.

![Figure 8.22: Measured strain time history in the front side of the unprotected RMW wall.](image-url)
Figure 8.23: Measured strain time history in the front side of the protected RMW wall.

Figure 8.24: Measured strain time history in the back side of the unprotected RMW wall.
Figure 8.25: Measured strain time history in the back side of the protected RMW wall.

Hence, a significant enhancement to the out-of-plane blast resistance of retrofitted walls compared to that to non-retrofitted walls from using the proposed system was observed. And the effectiveness of the proposed retrofit technique was evaluated. The wall did not fail in either experiment. The main finding of this study is that utilizing a double RMW wall with an arrangement of the hollow concrete block units will increase the integrity of the wall and infilling the cavities with polyurethane foam will enhance the resistance of the wall.

8.5.3 Comparison of Safe Scaled Distance with Published Results by Other Authors

The quantity of the explosive used and the standoff distance between the explosive and the target building are the main factors contributing to building damage. Here the distance of the explosive from the site to the target building is converted to a quantity called the Hopkinson-Cranz or the cube root scaled distance (Z), and it is taken to be equal to the distance from the charge to the target divided by cube root of weight (equivalent to a TNT charge weight).
The safe scaled distance is used as the parameter in some regulations (US DoD, 2004 and ASCE, 1997) in assessing the structural safety of a structure to resist airblast loads. The values used in the manuals are for conventional structures without any blast hardening and these values are 4.46 m/kg$^{1/3}$ and 4.65 m/kg$^{1/3}$, respectively. Moreover, there is no significantly observable wall deformation or any sign of local failure in all of the tests cited. It should be noted, too, that the effects of structural materials and configurations are not taken into consideration in these guidelines. Moreover, the definition of structural damage is not clear in these guidelines.

Wu and Hao (2007) proposed a safe scaled distance for masonry infilled reinforced concrete frame structure based only on numerical studies. They utilized unreinforced solid clay brick with a thickness of 240 mm. They found that the scaled distance to prevent the collapse of a low rise building was 1.8 m/kg$^{1/3}$ and that for high rise building was 1.18 m/kg$^{1/3}$. The results of another numerical study conducted by Su et al. (2008) indicated that the critical to prevent failure of an unreinforced masonry wall constructed of 10-hole cored clay units with a thickness of 76 mm is 9 m/kg$^{1/3}$ and at scaled distance of 4 m/kg$^{1/3}$ from the charge the URM wall will be blown out. They also found that when the wall was protected by one layer aluminum foam on the front surface it suffered only light damage when the scaled distance reached 4 m/kg$^{1/3}$. And, as the scaled distance reached 3.3 m/kg$^{1/3}$, the aluminum foam sheets were seriously damaged although the URM wall suffered only light damage. They also found that the wall collapsed when the scaled distance reached 2.3 m/kg$^{1/3}$ while if the wall were to be retrofitted with a layer of 40 mm thick aluminum foam on both surfaces, the debonding failure between the aluminum foam and the steel sheets/URM wall occurred at a scaled distance of 2.3 m/kg$^{1/3}$. Wall failure occurred at a scaled distance 1.8 m/kg$^{1/3}$.

Ahmad et al. (2014) performed an experiment on only one unprotected cantilevered masonry wall constructed from solid clay bricks with a thickness of 370 mm. They found that at a scaled distance of 1.81 m/kg$^{1/3}$ a large longitudinal crack appeared along both the front and the back sides of the wall and that the wall collapsed at a scaled distance of 1.72 m/kg$^{1/3}$.
The previous studies were limited to the use of brick on reduced scale unreinforced concrete block walls using numerical analyses or experimental tests. In the current study, double reinforced masonry walls both with and without the protection of aluminum foam were investigated under the scaled distance of 1.46 m/kg$^{1/3}$. It was found that the unretrofitted reinforced masonry wall suffered only light to moderate damage while the RMW retrofitted wall with one layer of aluminum foam on the front surface suffered only light damage where the aluminum foam was attached to the wall and only a portion of the upper course in the back of the wall exhibited lateral offset damage. When the RMW was retrofitted with two layers of aluminum foam on the front surface and one layer on the back surface, only a portion of the aluminum foam on the front surface was split off the wall while the aluminum foam on the back of the wall was still intact. As a result, the scaled distance considered in the current study compared to the safe scaled distances found in the reviewed previous studies was much smaller especially compared to 4.46 m/kg$^{1/3}$ and 4.65 m/kg$^{1/3}$ as specified in U.S. DoD (2004) and ASCE (1997), respectively.

**8.5.4 Blast Pressure Distribution behind the Wall**

The reflected pressure was measured and recorded for every shot using four pressure transducers installed along the height of the building located behind the blast wall. Figure 8.26 illustrate the pressure-time history as well as the impulse-time history profile of the blast wave detected at one of the pressure transducers. Here, the pressure signal resembles the classical shape of blast wave pressure history. The corresponding impulse values were computed by the integration of the pressure-time history. The negative phase values were not considered in this study because they generally produce small peak pressure of long durations.
It is worth noting that where there was no blast wall in front of the structures the maximum values for the peak reflected pressure and impulse always occurred at the ground level. It was also observed in this study that the protective blast wall provided a significant reduction in the reflected pressure and the impulse at the lower level of the structure at the scaled distance of, $Z=1.46 \text{ m/kg}^{1/3}$. Figures 27 to 29 show the reflected pressure at the lower level of the building behind the blast wall while the reflected impulses (area under the reflected pressure-time histories) are shown in Figures 30 to 32.
Figure 8.28: Reflected pressure time history measured at the lower part of the building.

Figure 8.29: Reflected pressure time history measured at the lower part of the building.
Figure 8.30: Reflected impulse time history measured at the lower part of the building.

Figure 8.31: Reflected impulse time history measured at the lower part of the building.
Figure 8.32: Reflected impulse time history measured at the lower part of the building.

A comparison of the reflected pressure and impulse produced along the height of the building for all of the walls is shown in Figure 8.33 and Figure 8.34. It should be noted that the reduction in the pressure is about 24% for wall W3 while the reduction in impulse is 20% compared to the non-retrofitted wall. It is observed that the wall W3 was more effective in reducing pressure. The vertical axis represents the height of the measurement location on the building, while the reflected pressure is plotted along the horizontal axis, to produce a profile along the height of the building.
Figure 8.33: The effect of protection RMW wall in reducing the reflected pressure along the height of building behind the wall.

Figure 8.34: The effect of protection RMW wall in reducing the impulse along the height of building behind the wall.
8.6 Summary and Conclusions

Four full-scale tests were conducted at the SSF Ranges, Riyadh city, Saudi Arabia on fully-grouted double reinforced masonry walls both with and without aluminum foam layers and with polyurethane foam filling in the cells of the hollow concrete units. Each wall was subjected to single shot, using a charge of 25 kg ANFO, which had an equivalent TNT value of 20.5 kg at a standoff distance of 4 m, one of the walls was subjected to a 125 kg charge (equivalent to 102.5 kg TNT) at the same distance. For each test, the walls were placed at the same standoff distance and orientation with respect to the blast source. The shape of the charges was cuboid and they were ignited at the top.

Since the majority of the previous studies were limited to brick on reduced scale unreinforced concrete block walls using numerical analysis with charges being detonated either on or above unpaved ground, some energy would be absorbed by the ground. The contrast, the blast load in the current study was above rigid ground and it was magnified by the reflection off the rigid ground. The goal of this study was to investigate the performance of a full-scale double reinforced hollow concrete block masonry walls under blast loads and to examine the use of aluminum foam with different protective layers. In addition, the pressure distributions along the height of building behind the wall were investigated.

The response of the walls to blasts was monitored by high-speed data acquisition systems. The accelerations, velocities, displacements, and strains experienced by the protected walls were significantly smaller than those of the unprotected wall. The time taken by each wall to reach its maximum displacement was longer than the entire positive phase duration of the blast event. This finding confirmed the assumption that the walls examined in the current study respond in their impulsive loading regime (Baker et al. 1983) under the scaled distance in this study.

The level of damage to each wall was visually observed and compared qualitatively to the damage classification in accordance to ASCE 59-11 (ASCE 2011) standards as well as by using the performance (rotation) limits; each wall in the current study was classified
according to its damage level as specified in the standards. The results confirmed the
effectiveness of the aluminum foam retrofit technique for blast protection. This
retrofitting prevented damage to the whole panel. It was observed that the protected wall
RMW-P1 (one layer aluminum foam on the front surface of the wall) suffered less
damage than did the other walls investigated in this study. Because RMW-P2 was
protected by two layers of aluminum foam on the front surface of the wall and one layer
on the back surface of the wall, there was a reduction in the acceleration, velocity,
displacement, and strain. However, it suffered damage at a greater height than when the
wall was protected by only one layer and a portion of the aluminum foam spilt off the
front wall. It was observed that when multi layers of aluminum foam were used there was
a need to use special anchoring devices to prevent the tensile failure of the foam.

The safe scaled distance provided in some regulations such as (US DoD, 2004 and
ASCE, 1997) was used as the parameter for assessing the wall’s ability to resist airblast
loads. The safe scaled distance used in the current study was compared with those
specified in the regulations and in the previous studies. It was found that the safe scaled
distance in the current study was much smaller.

Hence, the existing methodologies for determining the effectiveness of blast walls, such
as those presented in (TM5-853-3), are considered to be valid over only a very limited
range of scaled parameters (wall height, charge-to-wall stand-off distance, wall-to-
structure standoff distance) and is based on plane rigid walls. In the empirical design
charts it is assumed that there is either a spherical (free-air) or a hemispherical (surface)
burst. Nor are the variations in charge shape, charge orientation or the point of detonation
of the charge taken into consideration. In the empirical design charts, the point of
detonation is assumed to be in the middle of a sphere charge. Moreover, the assessment
of air blasts from charges in the free field is well understood while the variation in blast
resultants along a building’s façade, which is protected by a blast wall, is not fully
understood. Alternatively, new experiments or the FE codes such as LS-DYNA can be
used to assess the overpressure resulting from the detonation of non-spherical charges
and the reflection due to the geometry of the structure or that of the adjacent structures.
However, methodology similar to those used in these manuals was used in this study. Here the resulting reflected pressures and impulses along the height of the building located behind each wall were investigated. It was found that the RMW-P1 provides a greater reduction in the reflected pressure and impulse than did the other walls investigated in this study.

8.7 Acknowledgment
The authors gratefully acknowledge the financial support provided by the Saudi Government.

8.8 References


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Chapter 9

9 Conclusions and Future work

9.1 Conclusions

A high speed data acquisition and analysis system for real experiments were developed in this study along with in-house LabVIEW programs. The techniques for the proper mounting of the piezoelectric sensors were also worked out. This system was used for field blast tests to study the response of the reinforced concrete and reinforced masonry walls under blast loading as well as to measure the blast wave parameters.

The current study aimed to conduct a half-scale experiments to investigate the effectiveness of reinforced concrete walls with different shapes as well as the effectiveness of utilizing reinforced concrete walls coupled with decreasing the street elevation in reducing the blast resultants along the height of the protected building behind the perimeter wall.

Moreover, full-scale experiments were also conducted to investigate the effectiveness of reinforced canopied walls as well as the effectiveness of double fully grouted reinforced masonry walls infilled with polyurethane foam. The walls were tested both with and without aluminum foam retrofitting and were subjected to both close-in and nearby explosions. In addition, the reduction level of the blast wave resultants along the height of a protected building located behind these walls was investigated.

The main conclusions drawn from this study are made as follows:

- The efficiency of blast walls in reducing reflected pressure was clearly demonstrated by comparing the ratio of the measured reflected pressure or impulse with a blast wall to no blast wall present.
- The numerical results on rigid walls indicated that:
- The concept of changing the shape of the blast wall is effective in reducing the peak reflected pressure and the impulse.
- The maximum reduction pressure at the lower level of the building is 75% for the plane wall and 80% for the canopy wall.
- The experimental results indicated that the blast wall with Y-shape with opening is the most effective shape in mitigating the blast resultants behind it.
- Using a combination of erecting a blast wall, decreasing the street elevation and providing a substantial standoff distance found to:
  - Provide significant pressure reduction
  - Reducing the number of human causalities and injuries.
- The level of damage to each wall:
  - visually observed and classified qualitatively to the damage classification in accordance to ASCE 59-11 (ASCE 2011) standards
  - Using the (rotation) limits each wall also classified quantitatively to the damage classification in accordance to ASCE 59-11 (ASCE 2011) standards.
- The safe scaled distances used in the current study are much smaller compared with those specified in the regulations and in previous studies.
  - $0.86 \text{ m/kg}^{1/3}$ for the reinforced concrete plane wall,
  - $0.99 \text{ m/kg}^{1/3}$ for the reinforced canopy wall and
  - $1.46 \text{ m/kg}^{1/3}$ for the reinforced masonry wall.
- The results confirmed the effectiveness of the aluminum foam retrofit technique for blast protection and it was observed that walls protected by aluminum foam on both front and back surfaces, experienced less damage compared to other walls investigated.
- The construction method of the reinforced masonry wall significantly enhanced its performance sustaining blast loads, even without protection by the aluminum foam.
o The current data indicates that the blast wall is most effective when explosive charge is placed within two wall height (2H) from the blast wall and within thirteen wall height (13H) from the protected building.

o The high speed data acquisition system and the installation techniques as well as the operations procedures which were developed in this study is a useful tools to predict the blast wave characteristics as well as to assess the behaviour of the structures subjected to blast loading.

o More research is needed on to study the effect of using several layers of aluminum foam layers and the necessary anchorage techniques to strengthen the bond between the aluminum foam and the walls.

9.2 Recommendations for future research

The assigned objectives for this study were achieved. However, further experimental and numerical work is needed and the following is a list of a recommendations based on the outcome of the current study.

- Further experiments are needed to investigate the influence of charge shape, orientation and the point of detonation as well as the blast wall height on Air-Blast blasting.

- Further experiments are needed to investigate anchorage techniques, which could include epoxy adhesives, combined with mechanical fasteners to strengthen the bond between the aluminum foam and the walls.

- Further Computational Fluid Dynamics (CFD) simulations are needed to predict the blast pressure behind the blast wall for various blast environments.

Also there is an important need to conduct experiments to investigate:

- The performance of the precast panels under blast loading
- The effectiveness of using different underground wave barriers to reduce the blast-induced ground shock.
- Identifying the quantity and the distance of the explosions using ground vibration or sound levels criterion.
- The blast effects on human and to develop a personal protective material to resist the effects of blast loading.
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