

Master Thesis

NUMERICAL SIMULATION OF STRENGTHENED UNREINFORCED
MASONRY (URM) WALLS BY NEW RETROFITTING TECHNOLOGIES FOR
BLAST LOADING

BY

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DECLARATION

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ABSTRACT

Terrorism has become a serious threat in the world, with bomb attacks carried out both inside and outside buildings. There are already many unreinforced masonry buildings in existence, and some of them are historical buildings. However, they do not perform well under blast loading. Aiming on protecting masonry buildings, retrofitting techniques were developed. Some experimental work on studying the effect of retrofitted URM walls has been done in recent years; however, these tests usually cost a significant amount of time and funds. Because of this, numerical simulation has become a good alternative, and can be used to study the behaviour of masonry structures, and predict the outcomes of experimental tests.

This project was carried out to find efficient retrofitting technique under blast loading by developing numerical material models. It was based on experimental research of strengthening URM walls by using retrofitting technologies under out-of-plane loading at the University of Adelaide. The numerical models can be applied to study large-scaled structures under static loading, and the research work is then extended to the field of blast loading. Aiming on deriving efficient material models, homogenization technology was introduced to this research. Fifty cases of numerical analysis on masonry basic cell were conducted to derive equivalent orthotropic material properties. To study the increasing capability in strength and ductility of retrofitted URM walls, pull-tests were simulated using interface element model to investigate the bond-slip relationship of FRP plates bonded to masonry blocks. The interface element model was then used to simulate performance of retrofitted URM walls under static loads. The accuracy of the numerical results was verified by comparing with the experimental results from previous tests at the University of Adelaide by Griffith et al. (2007) on unreinforced masonry walls and by Yang (2007) on FRP retrofitted masonry walls. To study the debonding behaviours of retrofits

bonded to masonry, and find appropriate solution to protect certain masonry walls against blast loading, various retrofitting technologies were examined. The simulation covers explosive impacts of a wide range of impulses. Based on this work, pressure-impulse diagrams for different types of retrofitted URM walls were developed as a design guideline for estimating the blast effect on retrofitted masonry walls.

The outcomes of this research will contribute to the development of numerical simulation on modelling retrofitted URM walls, improving the technique for explosion-resistant of masonry buildings, and providing a type of guideline for blast-resistant design.

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1. INTRODUCTION

1.1. BACKGOURND

The protection of structures against blast loads is a government research priority for “Safeguarding Australia” against terrorism. Unreinforced masonry (URM) construction, which is widely used in public buildings, is extremely vulnerable to blast loads. An effective solution to mitigate blast effects on URM construction is to strengthen the masonry using retrofit technologies. Hence, developing retrofit technologies for URM construction is necessary and imperative.

Retrofit URM constructions are currently in their infancy around the world (Buchan and Chen 2007; Davidson et al. 2005; Davidson et al. 2004b; Hamoush et al. 2001; Romani et al. 2005; Tan and Patoary 2004; Urgessa et al. 2005; Ward 2004; Yang and Wu 2007). Categories of available masonry retrofit include: conventional installation of exterior steel cladding or exterior concrete wall, and new technologies such as external bonded (EB) FRP retrofit technologies, catch systems, sprayed-on polymer and/or a combination of these technologies (Davidson et al. 2005; Davidson et al. 2004b). However, most of the current research focuses on studying the behaviours of retrofitted masonry walls under static, cyclic or seismic loading {Hamoush, 2001 #175; Malvar, 2007 #578; Silva, 2001 #512; Yang, 2007 #407}. Recently, blast tests have been conducted to investigate retrofitting techniques to strengthen unreinforced masonry (URM) walls against blast loading (Baylot et al. 2005; Carney and Myers 2005; Myers et al. 2004; Romani et al. 2005). Therefore, it is urgent to study the behaviours of retrofitted URM walls under blast loading, and develop an efficient retrofitting solution to enhance blast resistance of masonry structures.

The analyses of retrofitted masonry member against static, cyclic or seismic loading have received considerable attention in recent years (Baratta and Corbi 2007; Bastianini et al. 2005; El-Dakhakhni et al. 2004; ElGawady et al. 2006b; ElGawady et al. 2007; Hamoush et al. 2002; Hamoush et al. 2001; Korany and Drysdale 2006; Shrive 2006; Silva et al. 2001; Willis et al. 2006). Empirical, analytical and numerical methods have been developed to estimate the response of retrofitted masonry under quasi-static loads (Cecchi et al. 2004; Ceechi et al. 2005; ElGawady et al. 2006a; Hamed and Rabinovitch 2007; Korany and Drysdale 2007a; Korany and Drysdale 2007b; Wu and Hao 2007a; Wu et al. 2005). 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 test data available. Although analytical methods can perform quick and reliable analysis, it is sometimes not possible to obtain analytical solutions due to the complexity of the problems. The finite element method, which is widely used in practical engineering, provides explicit and direct results.

The analysis of masonry members with retrofits subjected to blast loads is currently still in its initial stages. For example, conventional design guidelines (American Society of Civil Engineers (ASCE) 1997; American Society of Civil Engineers (ASCE) 2007; Department of Defence (DoD) 1990) reference using a “Single Degree of Freedom” (SDOF) model in the blast analysis and design of retrofitted masonry member (Biggs 1964). Although the SDOF method is easy to implement and is numerically efficient, it has a number of drawbacks. For example, it cannot capture a variation in mechanical properties of a cross-section along the member, cannot simultaneously accommodate shear and flexural deformations, and cannot allow varying distribution of blast loading spatially and temporally. All of this is in contrast to finite element analysis, where these accommodations are possible. Thus there is a need to develop a finite element model to analyse the dynamic response of retrofitted masonry members against blast loads.

1.2. SCOPE AND OBJECTIVES

The primary aim of this project is to establish numerical models to investigate the behaviours of retrofitted URM walls under blast loading. To achieve this goal, there were four milestones during the project:

- 1) **Simulation of URM walls using homogenization technique.** This consists of: (a) building masonry basic cell (MBC), (b) identifying material models for brick and mortar, and (c) deriving equivalent material properties of masonry basic cell. The basic material properties of brick and mortar were gained from material tests (Griffith et al. 2007). By simulating the behaviours of MBC under various load statements, the equivalent material properties were derived from the simulated stress-strain curves of MBC. Based on the equivalent material properties, a three-dimensional (3D) homogenized model was derived. This homogenized model was validated in simulating full-scaled URM walls.
- 2) **Developing bond-slip model by simulating pull-tests.** The interface bond/slip characteristics between FRP and masonry govern the performance of retrofits. Aiming on gaining reliable results, the bond behaviours should be simulated accurately. In this thesis, interface and contact models were used in simulating pull-test including externally bonded (EB) and near surface mounted (NSM), meaning accurate results were obtained. The validated homogenized masonry models together with reliable interface models between masonry and FRP were applied in the simulation of full-scaled retrofitted URM walls under quasi-static loads.
- 3) **Studying the behaviours of retrofitted URM walls subjected to blast loading.** The validated numerical models are extended to simulate

retrofitted masonry wall subjected to blast loading. Several types of retrofitting techniques were tested. Parametric studies were conducted to simulate masonry walls with different retrofitting techniques subjected to blast loading and effective retrofits are found. A comparison of the effectiveness of various retrofitted masonry walls was plotted.

- 4) **Developing pressure-impulse (P-I) diagrams as design guideline.** Based on simulation results, two critical damage levels were identified for the retrofitted masonry walls. As a type of design guideline, P-I diagrams were developed, in which both the effect of pressure and impulse were well considered.

1.3. THESIS OUTLINE

In Chapter 1, background, scope and objects of this project are introduced. The brief summary of this thesis will be presented in the following content in this chapter.

Chapter 2 presents relevant literature on URM walls and retrofitted URM walls subjected to blast loading. The commonly used retrofitting techniques on concrete and masonry structures are summarized. The brief overview of methods on estimating blast loading is described. Proposed methods, which were used to analyse behaviours of masonry walls, are also introduced.

Chapter 3 presents homogenization approach. The equivalent material properties of URM were derived from the behaviour of the constitutive materials (brick and mortar) in a basic cell. The derived homogenized properties of the masonry basic cell were used to simulate the performance of masonry under static loading. Results of the simulation under static loading were validated by experiments. Both the distinct

model in which bricks and mortar modelled separately and the derived homogenized models were used to simulate response of masonry walls under static loading and blast loading. Numerical results of URM walls by using finite element method are presented.

Chapter 4 presents the simulations of pull-tests. Based on test results, two types of bond-slip model for FRP plates bonded to masonry were employed to simulate interface between FRP plates and masonry. Validation of the numerical model for retrofitted URM walls under static loading is described. The simulation results were verified by full scale of retrofitted masonry wall tests.

In Chapter 5, numerical modelling of retrofitted URM wall under blast loading is presented. The validated numerical models, including masonry, bond-slip and retrofitting material models, were used to predict the out-of-plane capacity of retrofitted URM walls subjected to blast loading. Various retrofitting technologies were simulated, parametric studies were carried out, and their effectiveness under blast loading was compared. As final goal, P-I diagrams for different types of retrofitted masonry wall were developed.

Conclusions and recommendations are given in Chapter 6. The outcomes from this investigation are summarized.

Key input decks of masonry basic cell, URM walls and pull-tests are reported in Appendix A, B and C, respectively. Those for retrofitted URM walls under static loading and blast loading are included in Appendix D and E, respectively.

2. LITERATURE REVIEW

2.1. INTRODUCTION

Masonry walls are widely used in Australia, but are not commonly designed with blast resistance in mind. In recent years, several retrofitting reinforced technologies have been developed to strengthen reinforced concrete structures, which have been extended to apply to unreinforced masonry (URM) structures. However, few investigations have focused on strengthening URM walls to resist blast loads (Ward 2004).

This literature review summaries the damage to unreinforced masonry walls subjected to blast loading, and examines the current available retrofitting technologies for strengthening masonry structures. Examples of such technologies are near-surface mounted FRP, external bonded FRP, sprayed-on polyurea and aluminium foam, all of which are considered appropriate for strengthening URM walls. Since this project focuses on studying the behaviours of URM walls under blast loading, methods of estimating blast loading are presented. In addition, a review of primary techniques in estimating the response of masonry walls under blast loading, especially the finite element method, is provided. A review of some current design guidelines for blast loading is also included in the following literature review.

2.2. BACKGROUND OF URM STRUCTURES

Unreinforced masonry (URM) construction is widely used in Australia, as it provides a combination of structural and architectural elements. This method is attractive and

durable, and provides effective thermal and sound insulation and excellent fire resistance (Page 1996). However, it is found that URM construction is extremely vulnerable to terrorist bomb attacks since the powerful pressure wave at the airblast front strikes buildings unevenly and may even travel through passageways, resulting in flying debris that is responsible for most fatalities and injuries. In order to protect URM construction from airblast loads, an effective solution is to strengthen the masonry using retrofitting technologies.

Old masonry construction is usually designed without considering the effects of blast-resistance. In general design, masonry is considered to have little tensile strength. For this reason, negative factors affecting the stability of masonry structures, such as the crack and breathing phenomenon observed in blast events, have not been studied widely. In Australia, a large number of buildings were constructed using masonry without additional protection to resist blast events, as bomb attacks or explosive accidents seldom happen in Australia. However, in recent years, with the rising threat of terrorism, protection of many existing buildings, structures and facilities against airblast loading is receiving more and more attention.

Some research on masonry structures against blast loading has been carried in recent years. Baylot et al. (2005) studied the blast response of lightly attached concrete masonry cell walls. Unretrofitted concrete masonry cell (CMU) walls and several different types of retrofits were tested under blast loading, with results showing that URM walls failed on light impulse and produced high velocity debris under high impulse. The researchers also found that debris from failing masonry wall and collapse are two main types of damage to URM wall subjected to blast loads. Because of the different properties of the cells and mortar, URM walls have weak planes due to the low tensile strength at each cell-mortar interface. The failures of masonry walls under blast loads are likely to be localized. They produce damage from wall fragments, which would injure the people behind the wall or destroy other structure, and debris with high velocity will damage other nearby structures. Muszynski and

Purcell (2003) tested four unretrofitted URM walls with different standoff distances. All mortar joints failed, some masonry blocks spalled and breaching occurred under high explosive detonations. Experiments (Davidson et al. 2005; Muszynski and Purcell 2003) showed that cracking usually occurred on the inter surface of masonry walls under light explosions, and appeared around breaching under high explosive blasts. Catastrophic breaching or even collapse happened when explosion came to high enough or the stand-off distances were small enough and wall failed in that case.

In summary, due to the shortcomings of masonry construction subjected to airblast loading, it is necessary to find efficient retrofitting technologies, study the behaviours of retrofitted URM walls under airblast loading, and develop an efficient mitigating solution to enhance blast resistance of URM construction.

2.3. CONVENTIONAL METHODS FOR URM STRENGTHENING

An effective solution to mitigate blast effects on URM construction is to strengthen the masonry using retrofit technologies. However, retrofit URM constructions are currently in their infancy around the world (Buchan and Chen 2007; Davidson et al. 2005; Davidson et al. 2004b; Ward 2004). Categories of available masonry retrofit include: conventional installation of exterior steel cladding or exterior concrete wall, and new technologies such as external bonded (EB) FRP plating, metallic foam cladding, sprayed-on polymer and/or a combination of these technologies (Davidson et al. 2005; Davidson et al. 2004b; Schenker et al. 2008; Schenker et al. 2005).

2.3.1. Fibre Reinforced Polymers

Fibre reinforced polymers (FRP) have a variety of advantages over other materials, such as lower density, high stiffness and strength, adjustable mechanical properties,

resistance to corrosion, solvents and chemicals, flexible manufacturing and fast application (Bastianini et al. 2005). They have been widely used in structural repairing and seismic resistance, and in recent years some studies for explosion resistance using FRP have been conducted. A variety of retrofitting technologies have been used to strengthen reinforced concrete (RC) structures (i.e. beams and columns) (Oehlers and Seracino 2004). Some of them have already been used to retrofit masonry walls, for example, near surface mounted (NSM) FRP plates and externally bonded (EB) FRP plates (Figure 2.1), which have high satisfactory performance and wide usage for enhancing RC structures. These technologies have proven to be an innovative and cost effective strengthening technique under out-of-plane static loading for strengthening masonry walls.

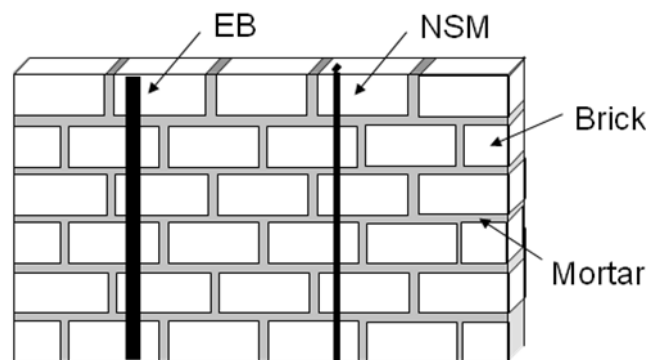


Figure 2.1 Samples of EB & NSM FRP plates

Near-surface mounted (NSM) FRP plates, which have been successfully used for strengthening concrete members, have been extended to retrofit masonry structures. Some recent tests under cyclic loading (Liu et al. 2006; Mohamed Ali et al. 2006) showed that the NSM plates can be used to strengthen RC structures with little loss of ductility, and increase the overall shear capacity substantially. Two experiments (Galati et al. 2006; Turco et al. 2006) showed that the NSM plates increased the flexural capacity (from 2 to 14 times), strength, and ductility of URM walls significantly. However, few studies on the behaviour of URM structures under blast loading have been conducted.

The key factor in increasing ductility and preventing the intrusion of wall fragments into occupant areas is the ability to absorb strain energy (Davidson et al. 2004b). Some recent experiments (Davidson et al. 2004b; Muszynski and Purcell 2003) on EB retrofitting techniques indicated that the high stiff FRP materials, such as steel plate and carbon fibre reinforced polymer (CFRP) used to retrofit masonry walls appeared less effective than low stiff materials under blast loads. An experimental work (Muszynski and Purcell 2003) tested air-entrained concrete (AEC) masonry walls retrofitted with carbon fibre reinforced polymer (CFRP) and Kevlar/glass (K/G) hybrid that is less stiff than CFRP. The residual displacements of CFRP structure were higher than the K/G Hybrid structure, which indicated the low stiff material would provide more ductility and absorb more strain energy, with bonding being another critical factor. Externally bonded techniques could be applied to strengthen masonry walls, when retrofitting materials that balance stiffness, strength, and elongation capacity become available. Therefore, GFRP appears a good option, as it is cost-effective and easier to apply, compared with the rigid material such as CFRP and steel plates.

Since the performance of FRP-strengthened URM walls is often controlled by the behaviour of the interface between the FRP and masonry, it is very important to study the bond-slip relationship in detail. Debonding could occur between the inter-surfaces of high stiff FRP materials and masonry when structures are subjected to out-of-plane loads. Stress concentration is also a problem if FRP is bolted on masonry walls. Screws can be used to fix the FRP materials, but may become a significant hazard, like debris, when subjected to blast loading. Therefore, it may not be a suitable for application on masonry walls.

Strengthening techniques such as near-surface mounted (NSM) FRP plates and externally bonded (EB) FRP plates have been used to increase the flexural strength of masonry structures (Yang 2007). The behaviour at the interface between FRP and masonry is an important consideration in the analysis and design of masonry

retrofitted with EB and NSM plates. Pull tests, in which an FRP strip or plate is bonded to a masonry prism and loaded in tension, are often used to study the bond-slip relationship of FRP-to-masonry. In the last decade, considerable research, including experimental, analytical and numerical approaches, has been conducted to investigate the FRP-to-concrete bond behaviour (Al-Mahaidi and Hii 2007; Lu et al. 2006; Lu et al. 2007; Mosallam and Mosalam 2003; Neale et al. 2005; Oehlers and Seracino 2004; Teng et al. 2006; Willis et al. 2004). Recently similar experimental and analytical studies have been carried out in investigation of the FRP-to-masonry bond behaviour (Yang 2007). However, little research has focused on numerical simulation of the bond behaviour of masonry retrofitted by using EB glass FRP (GFRP) strips and NSM carbon FRP (CFRP) plates.

2.3.2. Spray-on Polyurea

Spray-on polyurea is new technique using urea-based or polyurea-based coating sprayed on the surface of masonry walls. It produces a tensile membrane, which prevents spalls significantly. The material is cheap, but needs careful surface preparation before application (Ward 2004). Polyurea has low stiffness, and Davidson et al.'s study (2004b) demonstrated that it could enhance the flexural ability of URM wall and reduce debris effectively. Coated and non-coated wall panels were tested to establish the effectiveness of spray-on polyurea, with results showing that compared with stiffer materials, polyurea can absorb strain energy and keep fragments within a safe area. Further research (Davidson et al. 2005) found that stiff composite materials, such as woven aramid fabrics or CFRP, can also reduce fragments effectively. However, compared with polyurea, they are more expensive, which limits their applicability. Baylot et al. (2005) studied debris hazard from masonry walls against blast loads. Three types of retrofits (FRP, polyurea, steel) with different amount of grout and reinforcement were tested to find the most effective retrofitting technology for decreasing the degree of hazard under blast loads. The panels retrofitted by

sprayed-on polyurea performed well and succeeded in reducing the hazard level inside. The previous tests indicated that spray-on polyurea can be an effective technique for increasing the ductility of masonry walls.

2.3.3. Aluminium Foam

Aluminium foams are new, lightweight materials with excellent plastic energy absorbing characteristics that can mitigate the effects of an explosive charge on a structural system by absorbing high blast energy. The material behaves closely to that of a perfect-plastic material in compression, which makes aluminium foam an attractive choice for use in sacrificial layers for blast protection. Airblast tests on aluminium foam protected RC structural members have been conducted recently and it was found that aluminium foam was highly effective in absorbing airblast energy and thus successfully protected RC structural members against airblast loads (Schenker et al. 2008; Schenker et al. 2005). Due to its properties, it is believed that aluminium foam would also be very effective in protecting of URM constructions against airblast loads, although no tests have been performed. Since field airblast tests are very expensive and sometimes not even possible due to safety and environmental constraints, numerical simulations with a validated model provide an alternative method for an extensive investigating the effects of aluminium foam in mitigating airblast loads on the URM construction.

2.4. ESTIMATING RESPONSE OF MASONRY WALLS UNDER BLAST LOADING

2.4.1. Estimation of Blast Loading

(1) Empirical methods

The explosion considered here is a surface explosion with the charge placed about one metre above the ground. Considering that a bomb attack is often carried out in a

vehicle, which isolated from the ground, the ground shock can be diminished (Luccioni et al. 2004). Henrych (1979) developed empirical formulae for estimating the blast pressure history. In 2005, (Alia and Souli 2006; Remennikov and Rose 2005; Wu and Hao 2005, Shi, 2007 #484) improved Henrych's theory by enabling calculation of the full pressure time history. The U.S. Army developed a blast-resistant design manual TM-5-1300, which provides some empirical curves to predict blast loads. However, the load time history is simplified as a triangle shape, and the load rise period is ignored.

The typical simulated pressure shock wave time histories in the air are shown as Figure 2.2, where T_a is the shock wave front arrive time, T_r is the rising time from arrival time to peak value, P_{so} is the peak pressure, and T_d is the decreasing time from peak to ambient pressure. The shock wave rises to the peak value suddenly (this history is often ignored, as the rising time is very short), and then decreases back to ambient value before entering a negative phase.

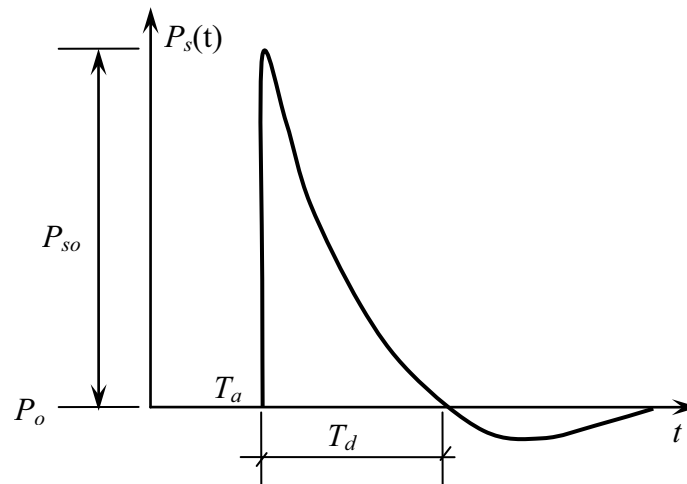


Figure 2.2 Typical free-air pressure time history

With a ground explosion in a free-air burst, a shock wave, having a hemispherical front (Figure 2.3) is produced. The formulae for an explosion in a free-air can be used for contact explosion, except that the charge weight must be substituted for half of the

value in free-air burst.

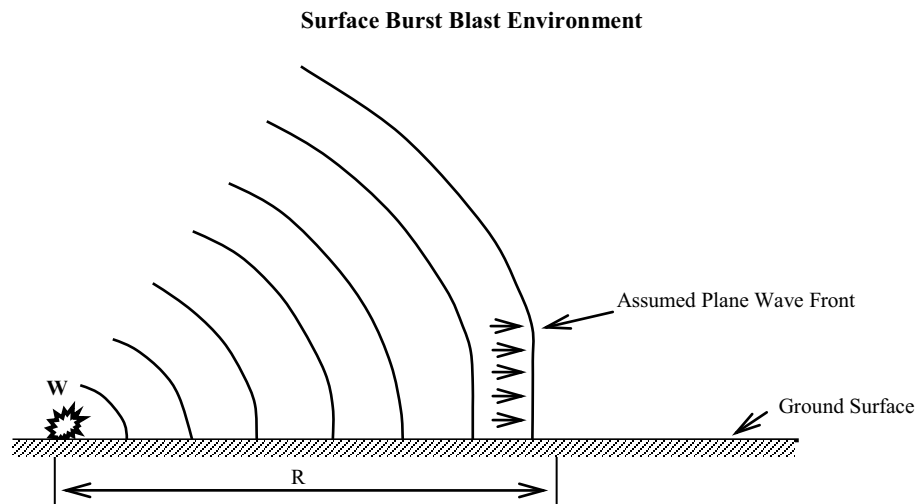


Figure 2.3 Surface burst blast environment

(2) Code solution

For design purposes, a number of codes were developed to estimate blast loading and the response of structures. The codes for military and munitions use cover more comprehensive statements including:

- "Structures to Resist the Effect of Accidental Explosions." *TM-5-1300*, US Department of the Army, Navy and Air Force Technical Manual, Washington, D.C.
- "Design of Structures to Resist Nuclear Weapons Effects", *ASCE Manual 42*, American Society of Civil Engineers
- "Fundamentals of protective design for conventional weapons." *TM-5-855-1*, Headquarters. Washington, DC.

Given scaled-distance, a simplified airblast load curve can be derived. Usually, the positive phase is considered and the rise time is ignored. To provide a more detailed and approximate pressure history, a computer program ConWep (Hyde 1993) was

developed as an application of the code TM-5-855-1 (Headquarters 1986), and has been incorporated into finite element programs AUTODYN and LS-DYNA (Randers-Pehrson and Bannister 1997). Given charge weight and stand-off distance, the blast history can be calculated automatically and applied to the surface of specimens.

(3) Numerical simulation of explosion

For explosions in complex environments, in which shock waves travel through complex routes or wave fronts impact on uneven surfaces, the previous methods do not give reliable results. Therefore, numerical simulations were developed to cover this shortcoming. In this method, the charge was simulated as a type of explosive material. Air is modelled as fluid and could be coupled with the charge to get a more accurate pressure history and numerical results. The whole process of explosion can be presented, and complicated phenomena can be observed. Recently, some studies (Alia and Souli 2006; Remennikov and Rose 2005; Wu and Hao 2005, Shi, 2007 #484) were carried out using this method; however, there are some disadvantages which should be noted. Firstly, the simulation involves a high number of calculations. Therefore, blast at far stand-off distances becomes time-consuming. Secondly, the application is complex, with some issues like the dimensions of the element closed to the charge and near the concerned area, such as the contact surface between air and specimens, requiring careful consideration. To have the negative phase of the pressure history, the fast reduction of air pressure due to the leakage of gas may also be a computing problem. Thirdly, equation of the gas should be modified to consider the behaviour of the air under high temperature and high pressure, especially for a close explosion.

2.4.2. Finite Element Method

Numerical simulation is a cost-effective method for investigating the behaviour of masonry structures. Compared with experiments, it gives better understanding of the detailed process of events. The numerical simulation has become a widely used method for investigating behaviours of structures under static or dynamic loading, with a significant amount of research showing that it could produce considerable coincidental results with experimental data. This section overviews the estimation of blast loading, material properties for simulation, and some major numerical methods.

(1) Continuum model and discrete model

The continuum model considers the masonry material as a continuum medium, and is applicable to analysing a large-scale masonry wall in some early investigations (Anthoine 1995; De Buhan and De Felice 1997; Pegon and Anthoine 1997). Research showed that after varying the bond pattern, neglecting the head joints, or assuming plane stress states, reasonable estimates of the global elastic behaviour of masonry were obtained. However, as Anthoine (1995) indicated, a careful examination of the elastic stresses that develop in the different constitutive materials shows that the situation might be quite different in the non-linear range (damage or plasticity). To obtain reliable equivalent material properties of masonry material, homogenization is critical in numerical analysis.

The discrete model has been developed to perform linear and nonlinear analyses of masonry structures. It is computationally intensive, making it a time-consuming method, and is therefore generally only suitable for simulating the fracture behaviours of small specimens (Ma et al. 2001). In this study, the specimens are full-scaled masonry walls made of cored brick and mortar joint. Therefore, to avoid the calculating problem, the homogenized model is preferable, which is discussed in the

following section.

(2) Homogenized model

The homogenization technique has been used in the past to derive the equivalent material properties and failure characteristics for solid brick masonry. Considerable research has been conducted in the last decade to investigate the complex mechanical behaviour of solid brick masonry structures using various theoretical and numerical homogenization techniques (Anthoine 1995; Luciano and Sacco 1997; Ma et al. 2001; Milani et al. 2006a; Milani et al. 2006b; Wu and Ha 2006; Zucchini and Lourenco 2004). It has been shown that using homogenized material properties can give a reliable estimate of masonry response under both static and dynamic loading. However, substantially less computational time is required to perform the analysis of masonry structures as compared with distinct model in which bricks and mortar joints are separately discretized. Recently, the homogenization technique has been used to derive equivalent material properties of hollow concrete block masonry (Wu and Hao 2007b), in spite of this, no study has been conducted to analyse the response of masonry structure constituted by cored brick units jointed with mortar using the homogenization technique. Due to the complex geometric properties of the cored brick unit, it is very complicated and time consuming to use the distinct model to perform the analysis on this kind of masonry structure. Therefore, it is of importance if the equivalent material properties of this masonry structure can be derived.

As masonry is a composite structure constituted by bricks and mortar, using the discrete method to compute large scale of masonry walls often requires a significant amount of time. The homogenized technique, which is used to derive the behaviour of the composite from geometry and behaviour of the basic cell, has been developed to simplify the computation. Some homogenization models of URM structures subjected

to blast loading has been investigated by researchers (Anthoine 1995; Cecchi and Di Marco 2002; ElGawady et al. 2006a; Luccioni et al. 2004; Milani et al. 2006a; Wu and Ha 2006; Zucchini and Lourenco 2004) in recent years.

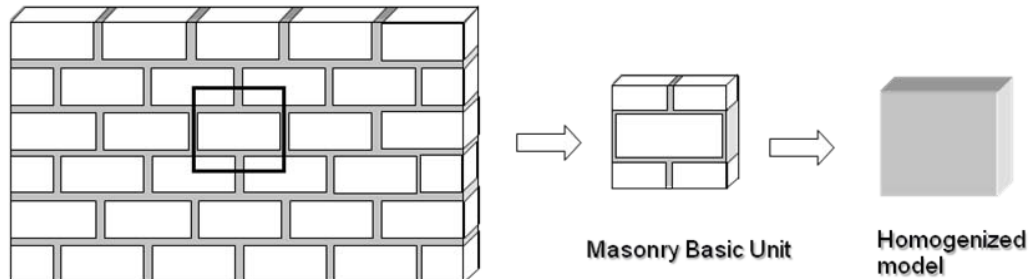


Figure 2.4 Homogenization of Masonry Material (Wu and Ha 2006)

The homogenization approach is shown above in Figure 2.4. Determining the basic cell is the first stage of homogenization. The basic cell contains all the geometric and constitutive information of the masonry, and is modelled to calculate the equivalent elastic constants and failure modes of masonry structures. Its volume depends on the bonding formats and retrofitting modes. Header bond shown in Figure 2.4 is commonly used for homogenization. More complex bond types require cells with greater dimensions, which are divided into small elements to calculate the constants.

Some recent research (Cecchi et al. 2004; Ceechi et al. 2005) began to focus on homogenizing CFRP retrofitted masonry structures. Firstly, the reinforcement and masonry were homogenized separately, then the homogenization of reinforced masonry was obtained by integrating the constitutive function of masonry and reinforcement along the thickness of the wall (Ceechi et al. 2005). Moreover, the authors developed a numerical finite element single-step homogenization procedure, which can be used as an example for modelling retrofitted masonry walls.

2.4.3. Design Guideline

According to previous studies, URM walls are weak, brittle, and have low ductility under blast loading. In order to develop effective retrofitting technologies, major damage levels should be studied, due to their significant hazard to occupants and surrounding constructions. Some experimental tests have been done to investigate the behaviour of URM walls under blast loads showing the major damage. Some countries, such as the U.S. through its Department of Defence, developed a blast evaluation guideline. Scaled distance is defined as $R/W^{1/3}$, where R is the stand-off distance and W is the TNT charge weight, which is used as a parameter by U.S. DoD Code (1999) to evaluate the structural safety under blast loads. The safe scaled distance is specified as $4.46 \text{ m/kg}^{1/3}$ for unstrengthened buildings to ensure the buildings are not destroyed. However, the description of damage level from U.S. DoD Code is vague, and further research (Wu and Hao 2007a) has been 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 the charge weight and stand-off distance, structural materials and configurations are also two important parameters. However, some tests (Baylot et al. 2005) showed by increasing the charge weight, or decreasing the stand-off distance other types of damage can be observed in addition to collapse, including cracks, catastrophic breaching, and low and high velocity debris. 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.

2.5. SUMMARY

This literature review has considered the behaviours of URM walls under blast loading, and was suggested that the retrofitting technologies can be applied to

strengthen masonry constructions. Still, a suitable solution is required to provide the better protection against all blast loads. According to published studies, existing retrofitting technologies are efficient in providing protection to concrete and masonry structures. Commonly used and newly developed retrofitting technologies on masonry structures have been reviewed, including externally bonded FRP, near-surface mounted FRP, spray-on polyurea and aluminium foam. It is found that previous research primarily focused on studying behaviours of URM walls under static or blast loading, or studied the FRP retrofitted URM walls under static loading or quasi-static loading. Hence, more research on the retrofitted URM walls against dynamic loading, such as blast loading, is needed.

To investigate the effectiveness of various retrofitting methods, the major damage modes were identified. It is crucial to qualify the damage levels for developing the design guideline, and it is expected that the previous damage levels and tests data could be used to check the effectiveness of different retrofits. Finite element analysis with blast loading calculated from a design code can be used to study the dynamic behaviours of retrofitted masonry walls under blast loads. At present, there is no industry guideline available for blast-resistant design of masonry structures, and it is therefore expected that, this project will contribute to its development.

3. NUMERICAL SIMULATION OF URM WALLS USING THE HOMOGENIZATION TECHNIQUE

3.1. INTRODUCTION

Homogenization techniques have been used to derive the equivalent material properties of masonry for many years. However, no research has been conducted to derive the homogenized model of the standard ten-core brick masonry wall, commonly used in Australia. In this chapter, the homogenization technique was used to model a three-dimensional masonry basic cell, which contains all the geometric and constitutive information of the masonry wall, in a finite element program to derive its equivalent mechanical properties. The detailed material properties of mortar and brick were modelled using a numerical analysis. By applying different loading conditions on the surfaces of a basic cell, stress-strain curves of the basic cell under various stress states were simulated. Using the simulated stress-strain relationships, the homogenized material properties and failure characteristics of the masonry unit were derived. The homogenized 3D model was then utilized to analyse the response of a masonry wall with and without pre-compression under out-of-plane loads (Griffith et al. 2007). The same masonry wall was also analysed with distinct material modelling, and the efficiency and accuracy of the derived homogenized model were demonstrated.

3.2. HOMOGENIZATION PROCESS

Homogenization techniques can be used to derive the equivalent material properties of a composite from the geometry and behaviour of the representative volume element.

Masonry is a composite structure constituted by bricks and mortar. Thus, the homogenization technique can be used to derive the equivalent material properties of masonry unit.

In this section, a highly detailed finite element model was used to model a three-dimensional basic cell to derive the equivalent material properties for a homogenous masonry unit. Various load cases were applied to the basic cell surfaces to derive average stress-strain relationships of the homogenous masonry unit under different stress states. The average elastic properties and failure characteristics of the homogenous masonry unit are obtained from the simulated results. The numerical results are verified from comparison to experimental results from previous tests undertaken at the University of Adelaide, along with numerical results from simulation using a distinct finite element model. The derived equivalent material properties can be utilized to simulate large-scale masonry structures and predict their failure modes under out-of-plane loading.

3.2.1. Homogenization Technique

Traditionally, laboratory tests are performed to obtain average stress and strain relationships of a specimen, required to find the homogenized properties of composite materials such as concrete with aggregates and cement. However, for masonry structures, it is often too difficult to conduct the laboratory test. To overcome this difficulty, the numerical homogenization method was used in this study to derive its equivalent material properties. Figure 3.1 shows the homogenization process for a basic cell, which contains all the geometric and constitutive information of the masonry wall. The basic cell was modelled, separately, with individual components of mortar and brick. Constitutive relations of the basic cell can be set up in terms of average stresses and strains from the geometry and constitutive relationships of the

individual components. The average stress and strain $\bar{\sigma}_{ij}$ and $\bar{\varepsilon}_{ij}$ are defined by the integral over the basic cell as

$$\bar{\sigma}_{ij} = \frac{1}{V} \int_V \sigma_{ij} dV \quad \text{Eq. 3-1}$$

$$\bar{\varepsilon}_{ij} = \frac{1}{V} \int_V \varepsilon_{ij} dV \quad \text{Eq. 3-2}$$

where V is the volume of the basic cell, σ_{ij} and ε_{ij} are stress and strain components in an element. By applying various displacement boundary conditions on the surfaces of the basic cell, the equivalent stress-strain relationships of the basic cell were established. In addition, the equivalent material properties of the basic cell were derived from the simulated stress-strain curves. However, to simulate the performance of the basic cell under different loading conditions in a finite element program, the material properties of mortar and brick should be determined.

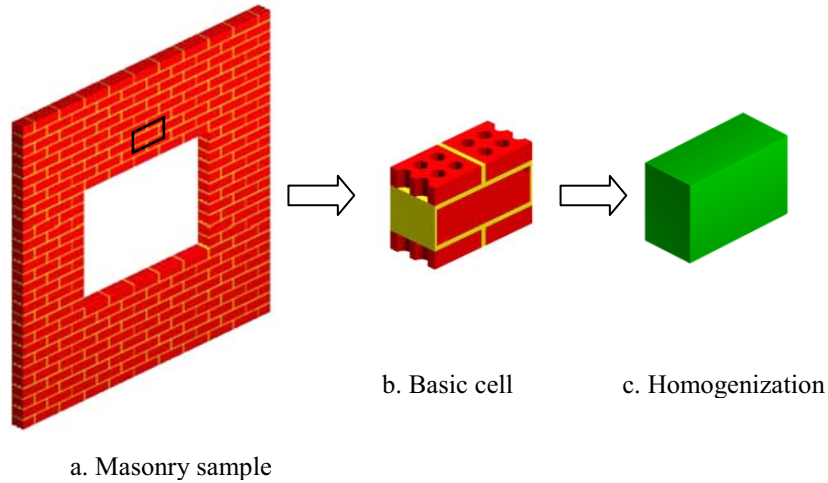


Figure 3.1 Homogenization of masonry material

3.2.2. Material Models for Brick and Mortar

In order to derive the equivalent inelastic material properties of the basic cell, reliable

material models for brick and mortar are important. The yield criterion for quasi-brittle materials such as brick and mortar is usually based on Drucker-Prager strength theory as shown in Figure 3.2.

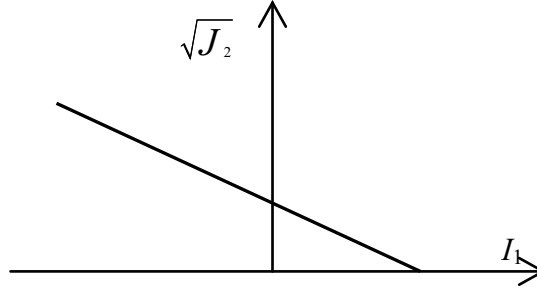


Figure 3.2 Drucker-Prager yield surface

The yield criterion on Drucker-Prager yield condition is given by:

$$\alpha I_1 + \sqrt{J_2} - k = 0 \quad \text{Eq. 3-3}$$

where J_2 is the second invariant of the deviatoric stress tensor S_{ij} , and I_1 is the first invariant of the stress tensor, given by

$$J_2 = \frac{1}{2} S_{ij} S_{ij} \quad \text{Eq. 3-4}$$

$$I_1 = (\sigma_1 + \sigma_2 + \sigma_3) \quad \text{Eq. 3-5}$$

α is the pressure sensitivity coefficient and k is a material constant. Let σ_t and σ_c be the yield stresses in uniaxial tension and compression respectively. On the yield surface, $J_2 = \frac{1}{3} \sigma_1^2$, k is obtained from yield condition as:

$$k = \frac{2\sigma_t\sigma_c}{\sqrt{3}(\sigma_t + \sigma_c)} \quad \text{Eq. 3-6}$$

If,

$$m = \frac{\sigma_c}{\sigma_t}, \quad \text{Eq. 3-7}$$

then,

$$\alpha = \frac{m-1}{\sqrt{3}(m+1)} \quad \text{and} \quad k = \frac{2\sigma_c}{\sqrt{3}(m+1)} \quad \text{Eq. 3-8}$$

The constants α and k can be determined from the yield stresses in uniaxial tension and compression.

Typical 10-core clay brick unit manufactured by Hallet Brick Ptd Ltd with nominal dimensions of $230 \times 110 \times 76 \text{ mm}^3$, as shown in Figure 3.3, was used in this study. The detailed dimensions and locations of ten cores are also shown in Figure 3.3. The mortar consisted of cement, lime and sand mixed in the proportions of 1:2:9, and the 10-core clay brick unit and a 10 mm thick mortar joint were used in this study. The same material properties for bed and head joints were assumed.

A set of material tests were performed to gain the primary parameters for subsequent simulations by Griffith (2007). The tests included bond wrench tests to gain flexural tensile strength of the masonry, masonry unit beam tests to gain lateral modulus of rupture of the brick units, and compression tests of a 5-layer-brick model to gain compressive strength of the masonry and Young's modulus. Table 3.1 lists material properties for mortar and brick. Details about the masonry properties are presented elsewhere (Griffith et al. 2007).

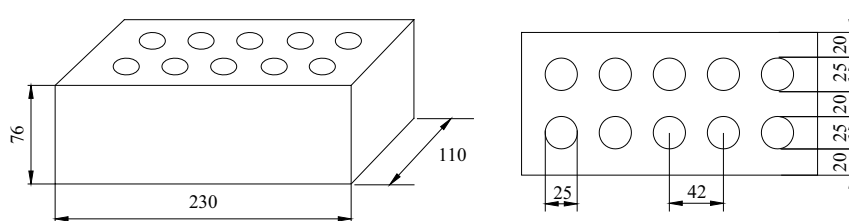


Figure 3.3 Nominal dimensions of brick unit (mm)

Using material properties, the material constants α and k in the above Drucker-Prager

strength model were derived, with their values listed in Table 3.1, and material properties for brick and mortar were coded into a finite element program LS-DYNA (LSTC 2007). The key parameters for using in simulations of masonry basic cell are listed in Table 3.1.

Table 3.1 Material properties for brick and mortar

c	E_c, E_t (GPa)	G_c (GPa)	ν	σ_t (MPa)	σ_c (MPa)	α	k (MPa)
brick	5.27	2.2	0.2	3.55	35.5	0.47	3.73
mortar	0.44	0.18	0.3	0.6	6.14	0.47	0.65

A general-purpose finite element program LS-DYNA was used in this study to calculate the stress-strain relationships of the basic cell as shown in Figure 3.1b. LS-DYNA provides a variety of material models for analysing masonry structures. According to a previous research (Davidson et al. 2004a), four material models perform well in simulating bricks under blast loading. The possible candidates are “Soil and Foam”, “Brittle Damage”, “Pseudo Tensor”, and “Winfrith Concrete”. The material Soil and Foam is a cost-effective model with fewer inputs, and still gives reliable results. The yield criterion of the material model “Soil and Foam” is based on Drucker-Prager strength theory as follows

$$\phi = J_2 - [a_0 + a_1 p + a_2 p^2] \quad \text{Eq. 3-9}$$

where p is hydro pressure, which is equal to $I_1/3$. On yield surface, it has

$$J_2 - [a_0 + a_1 p + a_2 p^2] = 0 \quad \text{Eq. 3-10}$$

Then, constants a_0 , a_1 and a_2 in Soil and Foam model are given by:

$$\begin{aligned}
 a_0 &= k^2 \\
 a_1 &= -6\alpha k \\
 a_2 &= 9\alpha^2
 \end{aligned}
 \tag{Eq. 3-11}$$

Considering the limited material properties and the efficiency of simulation, the “Soil and Foam” model in LS-DYNA was selected to model both brick and mortar in this study, as the model is efficient and requires fewer inputs. The model simulates crushing through the volumetric deformations, and a pressure-dependent flow rule governs the deviatoric behaviour with three user-specified constants. Volumetric yielding is determined by a tabulated curve of pressure versus volumetric strain as shown in Figure 3.4 (LSTC 2007). The actual input constitutive relationships are shown in Figure 3.5, and elastic unloading from this curve is assumed to be a tensile cut-off. One history variable, the maximum volumetric strain in compression, is given. If the new compressive volumetric strain exceeds the given value, loading is indicated. When the yield condition is violated, the updated trial stresses are scaled back. If the hydrostatic tension exceeds the cut-off value, the pressure and the deviatoric tensor would be zeroed (Davidson et al. 2005; LSTC 2007).

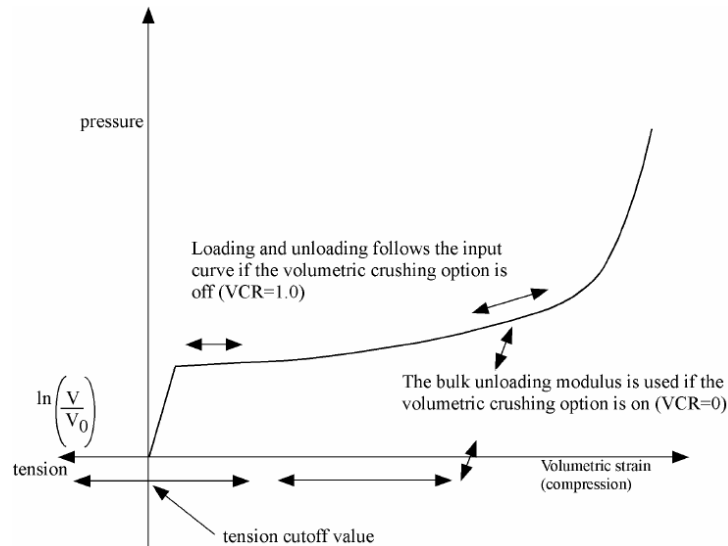


Figure 3.4 Volumetric strain versus pressure curve for soil and crushable foam model (LSTC 2007)

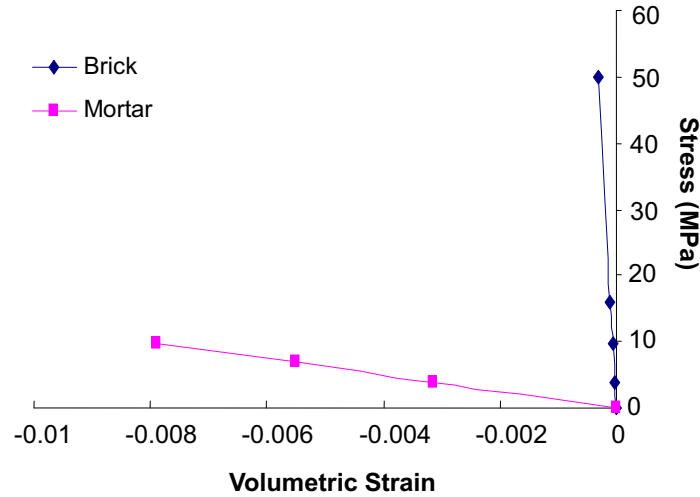


Figure 3.5 The input elastic constitutive relationships of brick and mortar

The model simulates flow rule through the volumetric deformations. The yield surface is a surface of revolution about the hydrostat in the principal stress space. A planar end cap is assumed for the yield surface so that the yield surface is closed. Given the right input, the model turns to the Drucker-Prager model, in which pressure is taken to be positive. Volumetric strain is given by the natural log of the relative volume and is negative in compression (LSTC 2007).

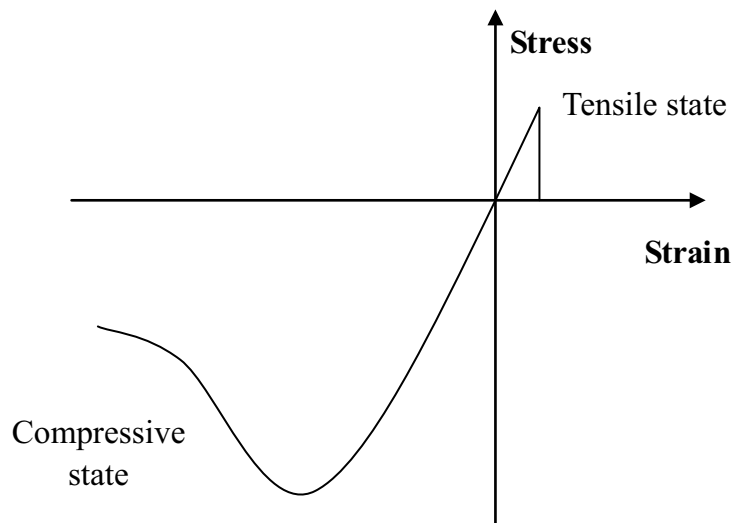


Figure 3.6 Theoretical stress-strain relationship

The theoretical stress-strain relationships are presented in Figure 3.6. In actual

simulations of complex models, the equivalent tensile curve shows some ductility. This is because individual elements did not fail at the same time under tension. Therefore, there were always some elements that could carry loads until the specimen was cut-through.

(1) Identification of inputs for numerical model

Although the key parameters have been already obtained from material tests, there are still some parameters that have not yet been determined. For example, parameters such as the bulk modulus were derived by numerical simulations, while key parameters such as the shearing modulus and cut-off tensile strength were estimated from the test results directly.

For common bricks and mortar, m (Eq. 3-7) equalled 10. Thus, for brick, a_0 , a_1 and a_2 equalled 2.82×10^{12} , 4.76×10^{16} and 2.008. For mortar, a_0 , a_1 and a_2 equal 4.16×10^{11} , 1.83×10^6 and 2.008. The material card used in the analysis for “Soil and Foam” is listed in Table 3.2 with corresponding tabulated values. Values for the bulk unloading modulus, volumetric strain values, and their corresponding pressures were estimated from the results of Griffith’s tests (Griffith and Vaculik 2005) firstly, and then were verified by simulating the compression of 5-layer-brick model.

Description of the input parameters is listed in Table 3.2. The shear modulus G was calculated from Young’s modulus by using formula $G = \frac{E}{2(1+\gamma)}$, and a_0, a_1, a_2 were calculated from Eq.3-10. The unloading bulk modulus can be gained from test, and must be greater than Young’s modulus. However, in this study, trial simulations were carried out to estimate the value of BULK, and it was found to be approximately 2.5 times greater than Young’s modulus (1.8×10^{11} Pa). The experimental tensile strengths were reported, ignoring the presence of the cores. Hence, for the detailed finite element model, the test values were adjusted to account for the holes in the bricks.

Considering stress concentration, actual values should be greater than the calculated ones. Trial simulations were carried out to obtain the final values. The tensile strength for brick and mortar are 7.1MPa and 0.8MPa (Table 3.3), respectively.

Table 3.2 Description of the inputs

Input	Description
G	Elastic shear modulus
BULK	Unloading bulk modulus
A0, A1, A2	Shear failure surface parameters
PC	Pressure cut-off for tensile fracture
VCR	Volume crushing option = 0 means on
REF	User reference geometry to initialize the pressure =0 means off
EPS1-10	Logarithmic volume strains
P1-10	Corresponding pressures

Table 3.3 Summary of input values

Brick

MID	RO	G	BULK	A0	A1	A2	PC
1	2.40E+03	2.20E+10	1.80E+11	5.5548E+13	2.1124E+07	2.008	-7.10E+06
VCR	REF						
0	0						
ESP1	ESP2	ESP3	ESP4	ESP5			
0	-1.18E-04	-2.44E-04	-4.00E-04	-1.10E-03			
P1	P2	P3	P4	P5			
0	3.95E+06	9.88E+06	1.60E+07	5.00E+07			

Mortar

MID	RO	G	BULK	A0	A1	A2	PC
2	2.40E+03	1.84E+08	1.33E+09	4.1540E+11	1.83E+06	2.008	-8.00E+05
VCR	REF						
0	0						
ESP1	ESP2	ESP3	ESP4				
0	-8.97E-03	-1.40E-02	-2.26E-02				
P1	P2	P3	P4				
0	3.95E+06	6.14E+06	9.88E+06				

(2) *Experimental validation*

In order to check the inputs for “Soil and Foam” model, verification was carried out by simulating compression tests. Figure 3.7 shows the configuration of the compression test. A uniform compressive load was applied on the top of the loading face of the specimen from zero at 25 kN increments up to a maximum load of 150 kN. Then, the specimen was subjected to an increasing compressive load until failure.

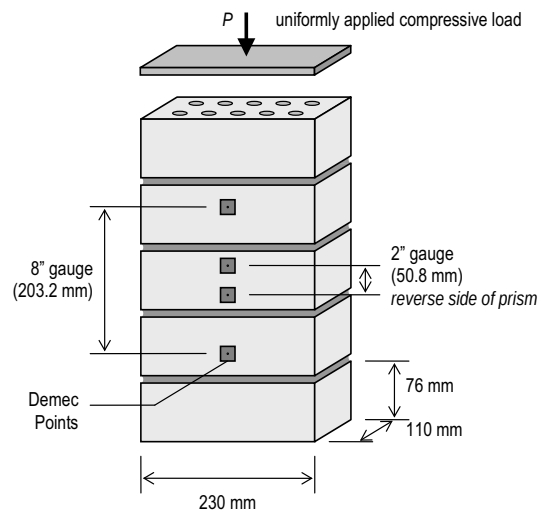


Figure 3.7 Configuration of the compression tests

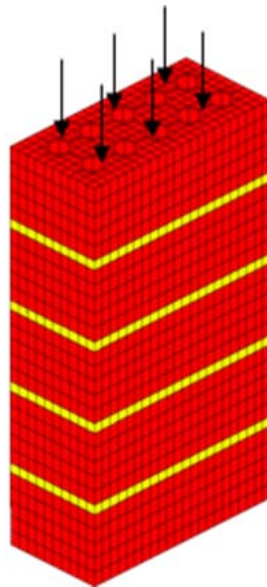


Figure 3.8 5-layer-brick model

Aiming to simulate the compression test, a 5-layer-brick finite element model was built as shown in Figure 3.8. The boundary conditions were set the same as the test, and the results of stress and strain were obtained from the elements with the same location of the gauges in the compression test.

The comparison of the test result and simulation result are presented in Figure 3.9. Due to lack of test data in the plastic phase, the results were only compared in elastic phase. From Figure 3.9, it can be found that the trend line of the simulation result matches well with that of the test result, verifying the input material properties in Soil and Foam model.

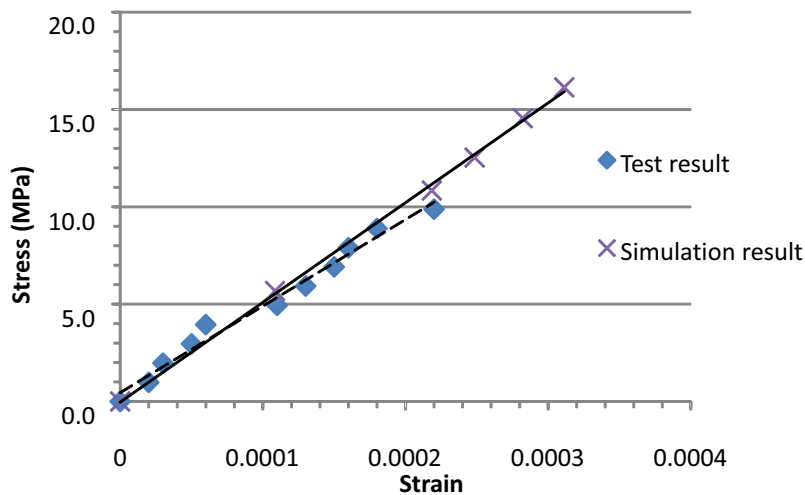


Figure 3.9 Stress-strain curves of the simulation and tests

3.2.3. Masonry Basic Cell and Convergence Tests

The first step of the homogenization process is to pick up masonry basic cell (Figure 3-1) with the common constitutive material properties form target masonry walls. The basic cell should contain all the participant materials, constitute the entire structure by periodic and continuous distribution, and also satisfy the requirement of minimum

size, that is, the cell should be small enough to make the variation of stresses and strains inside it negligible. Based on the above requirements, the basic cell shown in Figure 3.10 is used as a representative volume element.

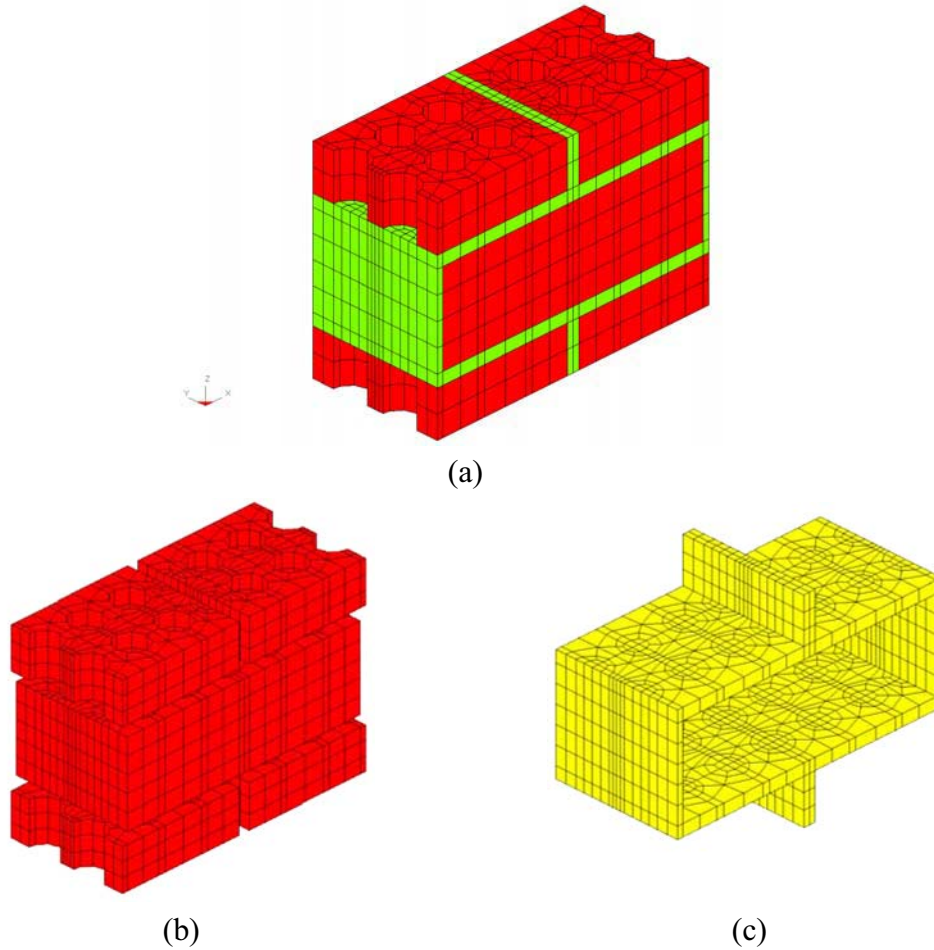


Figure 3.10 Masonry Basic Cell Finite Element Model (3560 elements)

(a) Masonry basic cell, (b) brick part, (c) mortar part

The basic cell was numerically modelled with distinct consideration of individual components of mortar and ten-core brick units. As mentioned previously, the Drucker-Prager strength theory was used for both mortar and brick. The average stress-strain relationships of the basic cell under different stress states were derived by applying various displacement boundaries on the basic cell surfaces. The equivalent material properties and failure characteristics of masonry material were derived from the simulation results.

The masonry basic cell is a finely meshed, 8-node solid element, with 24 degrees of freedom was used to represent the cell. Because the full integration of the element may produce element locking problem, which makes the elements hard to deform, the one-point integration element was used to get correct results. In this case, hourglass energy was monitored to guarantee of the reliable results. Usually, the hourglass energy is limited to 5% of total internal energy.

Convergence tests were conducted to determine the minimum number of elements needed to achieve a reliable estimation. Theoretically, masonry basic cells with more elements give more reliable results, but the calculation time for such a test is significantly greater. Therefore, convergence tests were performed to choose an efficient model. The finite element mesh used in the numerical model of the basic cell is shown in Figure 3.10. As shown, the 10-core clay brick unit and mortar in the basic cell were discretized into a number of solid elements. A convergence test on the influence of element size on computational accuracy was carried out by halving the size of the element for both brick and mortar while keeping loads on the basic cell unchanged. This test was continued until the difference between the results obtained with two consecutive element sizes was less than 5%. The test was performed by applying simple elastic properties to the basic cells, and setting them under compressive state. The boundary condition was set as vertical uniaxial compression, the bottom was all fixed, and displacement through the Z axial was applied as loading on the top.

Five models with different numbers of elements were tested, with the results summarized in Table 3.4. The model with the largest number of elements (23750) was considered to provide the most reliable result, and, as such, the results of the other four models were compared with it. In this simulation, the average stress, strain and Young's modulus of central elements were compared. From the results presented in Table 3.4, it is concluded that all the models gave reliable results. Because of this, the most effective model with 3560 elements for masonry basic cell was chosen.

Table 3.4 Average stress and strain of central elements

Model	Stress (MPa)	Strain (1×10^{-4})	Young's Modulus (MPa)	Difference
3560	-2.03	-5.30	3826	0.15%
5760	-2.03	-5.30	3829	0.25%
6144	-1.97	-5.16	3825	0.13%
10208	-2.01	-5.27	3823	0.07%
23750	-2.02	-5.30	3820	

Because of the complex internal structure of the cored brick, it would be difficult to build a model with less than 3000 elements. Moreover, the dimensions of elements should be kept similar to ensure the reliability of results. Considering the influence of this factor, models with fewer elements were not tested. Thus, 3560 eight-node solid elements were used in the numerical model of the basic cell to achieve the reliable estimation. The final numerical model used in the simulation is shown in Figure 3.10(a). Figure 3.10 (b) and (c) show two parts – bricks and mortar joint, and containing 3560 elements totally.

3.2.4. Simulated Stress-Strain Relationships of the Masonry Basic Cell

The masonry basic cell was simulated under varieties of loading states to plot stress-strain curves and derive the equivalent material properties. The loading states include compression-compression, compression-tension, shearing, and compression-tension-shearing. For compressive or tensile stress state, uniform displacements were applied as compressive loading or tensile loading on the surfaces of masonry basic cell.

To gain the equivalent material properties and yield surface, the response of the basic cell under compressive-compressive, compressive-tensile, tensile-tensile,

compressive-shear and tensile-shear stress states were simulated. Over 50 cases were simulated, and the calculated results are presented in Figure 3.11, Figure 3.12, and Figure 3.13.

Figure 3.11 shows the typical stress-strain curves of the basic cell under uniaxial compressive-compressive stress states. As shown in Figure 3.11a, the uniaxial compressive strength in the Z direction is 15.7 MPa, which is quite close to the experimental result of ultimate masonry compressive strength 16.0 MPa, carried out by Griffith and Vaculik (2007). It was shown that the uniaxial compressive strengths of the basic cell in the X and Y directions were 7.88 MPa and 7.39 MPa from the simulation results of uniaxial compressive-compressive states in X and Y directions, respectively. This indicated that the geometry of hollow bricks with ten cores reduced the compressive strength of the basic cell in both X and Y directions significantly.

As the basic cell is under biaxial or triaxial compressive states, its strength enhancement in the Z direction is not observed, although there are significant strength enhancements in both X and Y directions. When the basic cell is under biaxial compressive loads in the X and Z directions, as shown in Figure 3.11d, its maximum compressive strength in the Z direction is 15.0 MPa, slightly smaller than its uniaxial strength. The maximum strength in the Y direction is 24.8 MPa, which is much higher than its uniaxial compressive strength. It is also shown in Figure 3.11f that the maximum compressive strengths of the basic cell under triaxial compressive states in X, Y and Z directions are 8.73, 17.4 and 13.8 MPa, respectively. In addition, the compressive strength in the Z direction is slightly smaller than its uniaxial compressive strength. It should be noted that due to different dimensions of the basic cell in X, Y and Z directions, the ratios of the displacement must be set appropriately. In the X and Z directions, as shown in Figure 3.11d, and in the X, Y and Z directions, as shown in Figure 3.11f, the ratios are set to be 4:3 (u:w) and 4:2:3 (u:v:w) according to the dimension of the representative element. This ensures that the strain ratios in Y and Z directions and in X, Y and Z directions are about 1:1 and 1:1:1.

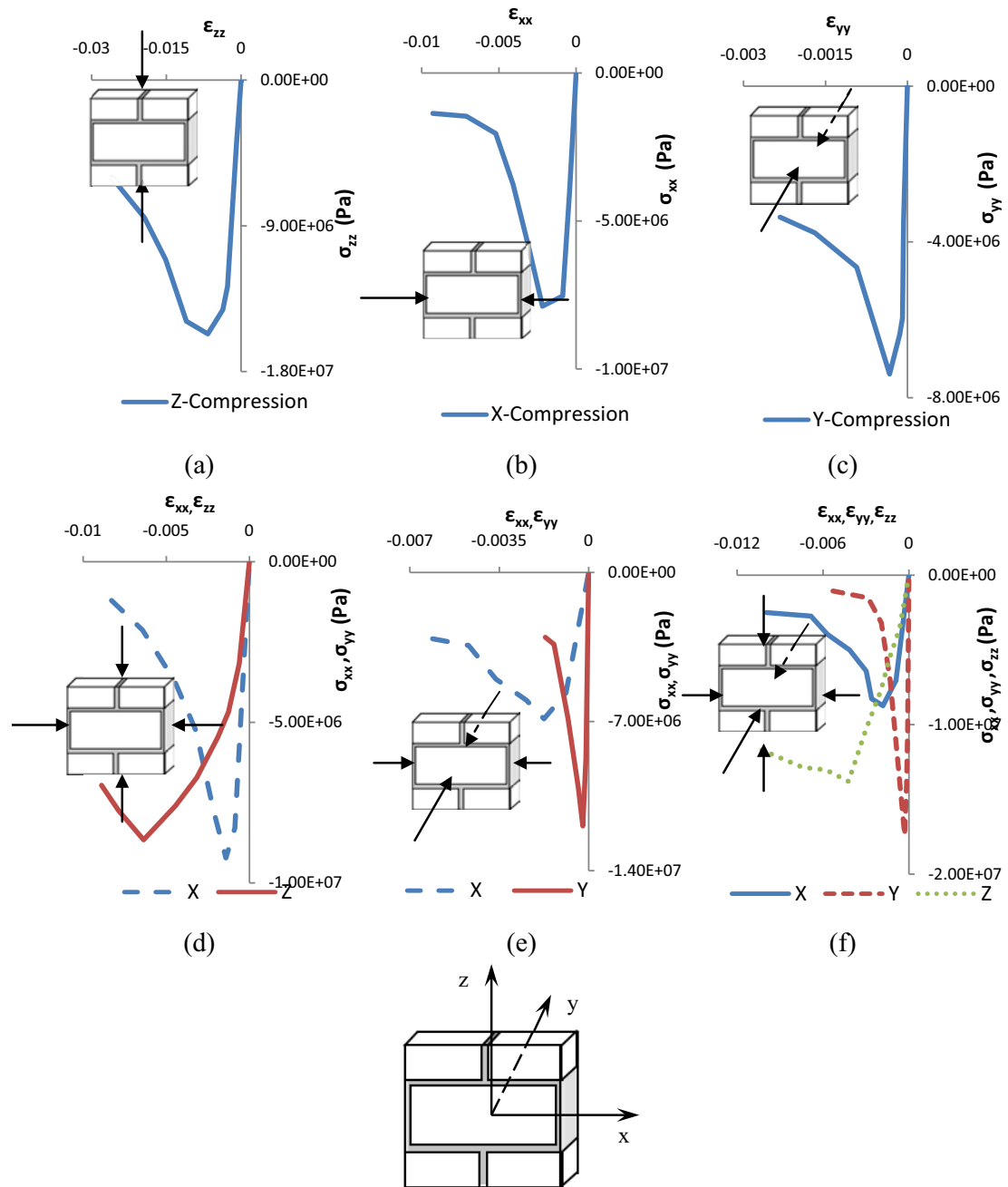


Figure 3.11 Typical stress-strain relationships of the basic cell in compressive-compressive stress states.

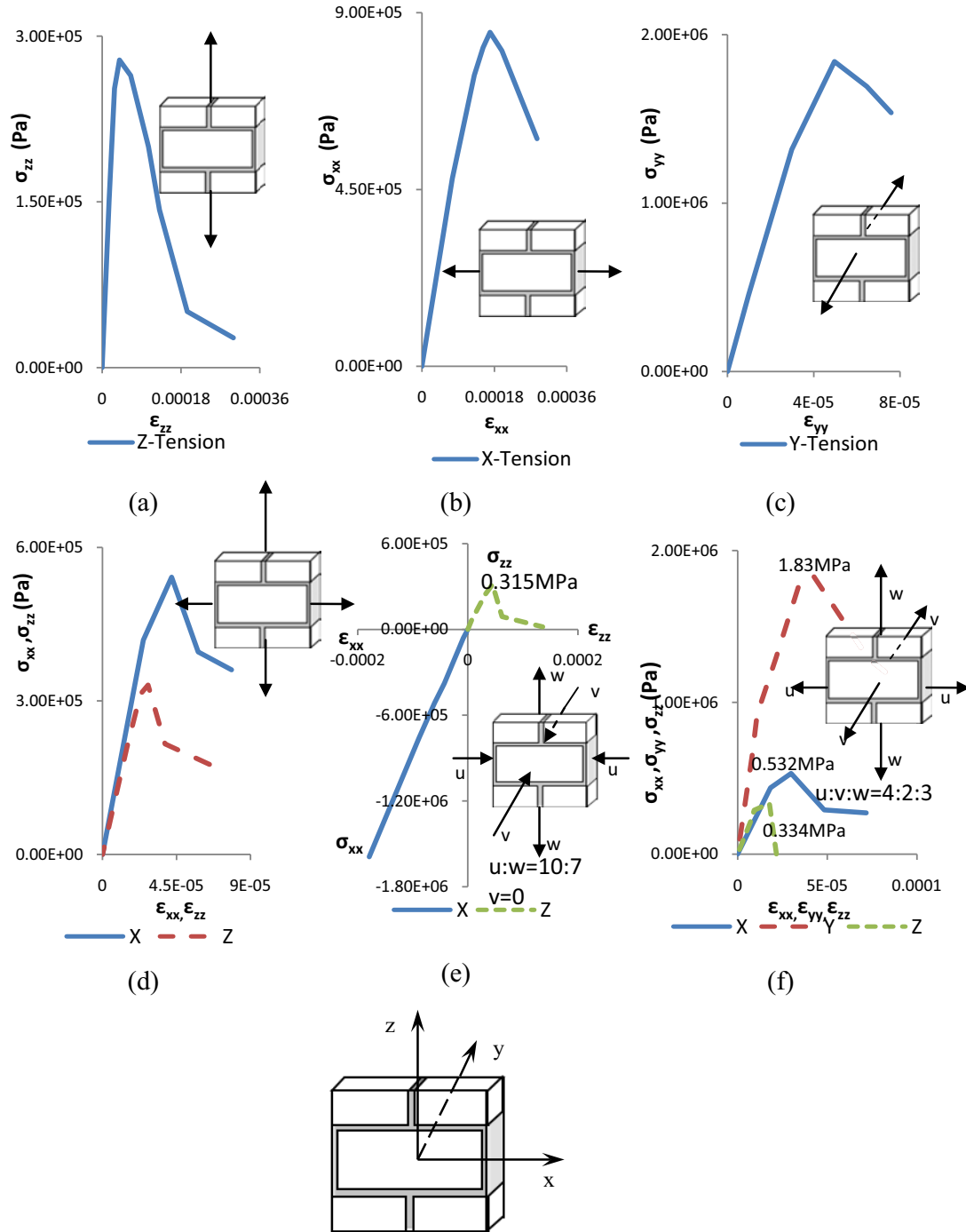


Figure 3.12 Typical stress-strain relationships of the basic cell in compression-tension and tension-tension stress states.

Figure 3.12 shows the typical stress-strain curves in compressive-tensile and tensile-tensile stress states. The uniaxial tensile strengths in the X, Y and Z directions are 0.85 MPa, 1.84 MPa and 0.28 MPa, respectively. It can be seen that the tensile

strength of the basic cell in the Z direction is much smaller than tensile strength of mortar (0.6 MPa) as the volume of the cores is counted as part of the total volume of the basic cell, as well as geometric size influence. The simulated results also indicate that there is not a significant tensile strength enhancement in the Z direction when the basic cell is under biaxial or triaxial tensile stress. In a tensile-compressive stress state, the ultimate tensile strength slight increases and it can be observed from Figure 3.12e that the basic cell fails owing to tensile strain before the compressive strength reaches the maximum value. When the basic cell is in triaxial tensile states (see Figure 3.12f), its tensile strengths in the X and Y directions are reduced, although there is a slight increase in its tensile strength in the Z direction.

The representative stress-strain curves of the basic cell under the compressive-shear and tensile-shear stress state are shown in Figure 3.13. The ultimate shear strengths τ_{zx} , τ_{zy} and τ_{yx} under pure shear condition are 0.78 MPa, 1.58 MPa and 1.28 MPa, respectively. It is also shown in Figure 3.13b that under compressive-shear stress state, the basic cell fails due to shear strain before the compressive strength reaches the maximum value.

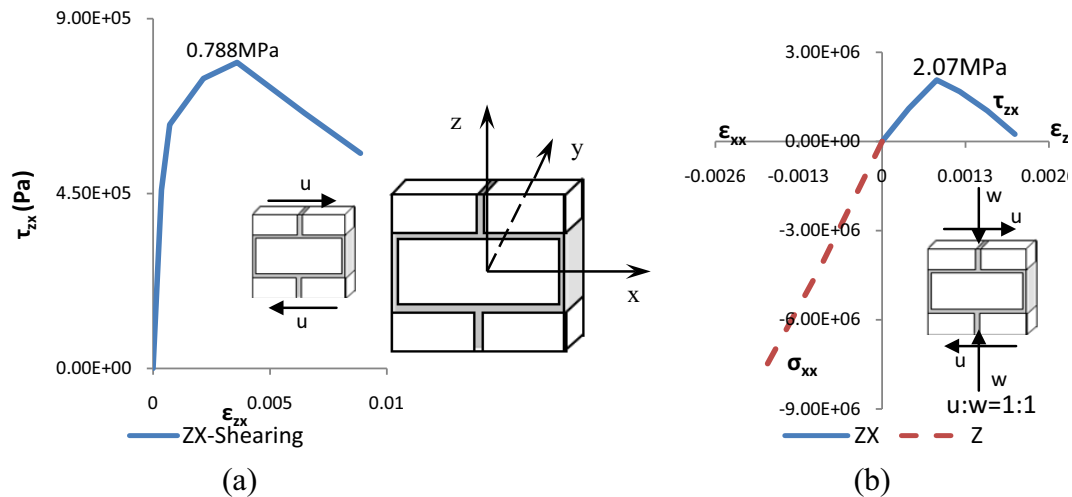


Figure 3.13 Stress-strain relation of the masonry basic cell in a shear stress state

3.2.5. Equivalent Material Properties

The properties of masonry can be obtained from the simulated stress-strain relations shown in Figure 3.11 to Figure 3.13. The equivalent elastic moduli and Poisson's ratio of the masonry material can be calculated using the stress-strain relations corresponding to the uniaxial compressive conditions in the three directions presented in Eq. 3-12. Based on the simulated stress-strain curves, the equivalent material properties of the basic cell such as Young's moduli, shear moduli and Poisson's ratio were derived as listed in Table 3.5.

$$\begin{aligned}
 \bar{E}_{xx} &= \frac{\bar{\sigma}_{xx}}{\bar{\epsilon}_{xx}}, \bar{\nu}_{xy} = \frac{\bar{\epsilon}_{yy}}{\bar{\epsilon}_{xx}}, \bar{\nu}_{xz} = \frac{\bar{\epsilon}_{zz}}{\bar{\epsilon}_{xx}} \\
 \bar{E}_{yy} &= \frac{\bar{\sigma}_{yy}}{\bar{\epsilon}_{yy}}, \bar{\nu}_{yx} = \frac{\bar{\epsilon}_{xx}}{\bar{\epsilon}_{yy}}, \bar{\nu}_{yz} = \frac{\bar{\epsilon}_{zz}}{\bar{\epsilon}_{yy}} \\
 \bar{E}_{zz} &= \frac{\bar{\sigma}_{zz}}{\bar{\epsilon}_{zz}}, \bar{\nu}_{zx} = \frac{\bar{\epsilon}_{xx}}{\bar{\epsilon}_{zz}}, \bar{\nu}_{zy} = \frac{\bar{\epsilon}_{yy}}{\bar{\epsilon}_{zz}} \\
 \bar{G}_{xy} &= \frac{\bar{\tau}_{xy}}{\bar{\epsilon}_{xy}}, \bar{G}_{yz} = \frac{\bar{\tau}_{yz}}{\bar{\epsilon}_{yz}}, \bar{G}_{zx} = \frac{\bar{\tau}_{zx}}{\bar{\epsilon}_{zx}}
 \end{aligned}
 \tag{Eq. 3-12}$$

Table 3.5 Equivalent material properties

Young's Modulus / Pa			Shear Modulus / Pa			Poisson's Ratio		
E _{xx}	E _{yy}	E _{zz}	G _{xy}	G _{yz}	G _{zx}	xy	yz	zx
7.49E+9	4.82E+10	6.82E+9	7.35E+9	3.24E+8	1.28E+9	0.250	0.269	0.205

Compressive strength / Pa			Tensile strength / Pa			Shearing strength / Pa		
X	Y	Z	X	Y	Z	XY	YZ	ZX
-7.88E+6	-7.39E+6	-1.57E+7	8.50E+5	1.84E+6	2.78E+5	1.28E+6	1.56E+6	0.788E+6

3.2.6. Development of Failure Criterion of Masonry Basic Cell

The equivalent strength envelope for ten-core brick masonry can be derived using the

ultimate strength from the simulated stress-strain curves. Since the masonry is an orthotropic material, conventional strength criteria such as the Drucker-Prager or Mohr-Coulomb strength criterion, are not suitable for representing the strength envelope of the orthotropic basic cell. The observations demonstrate that there is little strength enhancement in the Z direction and therefore the failure criteria in this direction follows maximum normal stress criteria for both tensile and compressive strength. Orthotropic failure criteria were derived from Hashin's work (1980) on composite material. In the XY plane its failure surface of the basic cell is shown in Figure 3.14. Therefore, the failure criteria are represented by tensile and compressive failure in the Z direction, as

$$\sigma_{zz} = X_C \quad \text{Eq. 3-13}$$

$$\sigma_{zz} = X_T \quad \text{Eq. 3-14}$$

Tensile and compressive failure in the XY plane, are given as

$$\frac{1}{Y_T}(\sigma_{xx} + \sigma_{yy})^2 + \frac{\tau_{xy}^2 - \sigma_{xx}\sigma_{yy}}{S_C^2} = 1 \quad \text{Eq. 3-15}$$

$$\frac{1}{Y_C} \left[\left(\frac{Y_C}{2S_C} \right)^2 - 1 \right] (\sigma_{xx} + \sigma_{yy}) + \frac{1}{4S_C^2} (\sigma_{xx} + \sigma_{yy})^2 - \frac{(\tau_{xy}^2 - \sigma_{xx}\sigma_{yy})}{S_C^2} = 1 \quad \text{Eq. 3-16}$$

where X_T and X_C are the tensile strength and compressive strength in Z direction; Y_T and Y_C are average tensile and compressive strength in X and Y directions; S_C is shear strength in the XY plane. Using the simulated data, it is found that $X_T = 0.28$ MPa, $X_C = 15.7$ MPa, $Y_T = 1.35$ MPa, $Y_C = 7.65$ MPa. To simplify the problem, the failure criteria in XY plane is expressed in terms of the principle stresses in the XY plane, σ_1 and σ_2 , which are derived from σ_X , σ_Y and τ_{XY} in plane stress system.

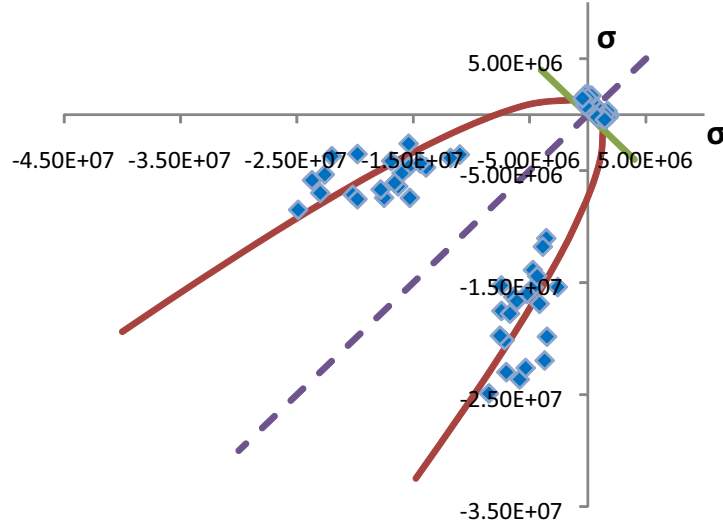


Figure 3.14 Failure curve in XY plane

3.3. SMEARED CRACK MODEL

Smeared crack models have been used to simulate the non-linear macro-level behaviour of URM structures for many years, as they are computationally inexpensive and often provide reasonably accurate results (Burnett et al. 2007; Lee and Fenves 1998; Lotfi and Shing 1991). In this model, many parallel cracks of infinitely small opening are assumed to be continuously distributed over the element where the crack develops. Due to this, the material stiffness and strength are reduced in the direction normal to the cracks after the peak strength of the masonry is reached. Therefore, the model considers an equivalent material in which the influence of the opening cracks is incorporated into the constitutive law which is non-linear and exhibits strain-softening behaviour. This enables the evolution of the cracking process from crack initiation down to full fracture to be modelled. This constitutive law (LSTC 2007) is given by

$$\sigma_i = \begin{cases} E\varepsilon_i & 0 \leq \varepsilon_i \leq \varepsilon_i^{ini} \\ \bar{\sigma} \left(1 - \frac{\varepsilon_i - \varepsilon_i^{ini}}{\varepsilon_i^{ult} - \varepsilon_i^{ini}} \right) & \varepsilon_i^{ini} < \varepsilon_i \leq \varepsilon_i^{ult} \\ 0 & \varepsilon_i > \varepsilon_i^{ult} \end{cases} \quad \text{Eq. 3-17}$$

with $\bar{\sigma}$ the ultimate stress, ε_i^{ini} the threshold strain and equal to $\bar{\sigma}/E$, E is masonry stiffness, and ε_i^{ult} the ultimate strain which is obtained by relating the crack growth energy and the dissipated energy,

$$\varepsilon_i^{ult} = \frac{2G_c A}{V\bar{\sigma}} \quad \text{Eq. 3-18}$$

where G_c is the fracture energy release rate, V is the element volume and A is the area perpendicular to the principal strain direction. The tensile fracture energy released rate G_I , taken from experimental work by Rots (1991), was 10N/m for brick-mortar interface and, from a study by van Zijl (2004), it was 12N/m for clay brick masonry. According to the experimental study by Pluijm (1997) on masonry under tensile strength ranging from 0.22 MPa to 0.36 MPa, the values of G_I could be range from 1.7 N/m to 6.8 N/m. In this study, the tensile fracture energy release rate G_I was estimated to be 12.6 N/m, using the simulated tensile stress-strain curves of the representative basic cell of masonry as shown in Figure 3.10. The shear fracture energy release rate G_{II} was found to be 59 N/m for brick-mortar interface by Burnetta et al. (2007). According to the study by Moyneaux et al. (2002), a linear relationship between shear fracture energy G_{II} and the normal compressive stress σ was identified as $G_{II} = 130\sigma + 30$. In this study, the shear fraction energy release rate was calculated as the area between the shear stress-displacement curve and residual shear stress lever, shown in Figure 3.15. Based on the simulated shear stress-strain curves from the representative basic cell of masonry, G_{II} was found to be 45.7 N/m, quite similar to the value estimated by Burnetta et al. (2007). The above parameter values for the smeared crack models were used in this study for the analysis of the out-of-plane test.

Orthotropic elastic material properties of the homogenized model were used in the smeared crack model, and failure was assumed to be perpendicular to the direction of principle strain. Damage described using linear softening stress strain relationship can occur in interlaminar normal and interlaminar shear direction. An initial crack length

a and crack opening displacement u are assumed existing inside masonry elements. When the crack opening displacement increases by Δa , crack length grows from a to $a + \Delta a$, and the resistant load will decrease by ΔF . The process is illustrated in Figure 3.16.

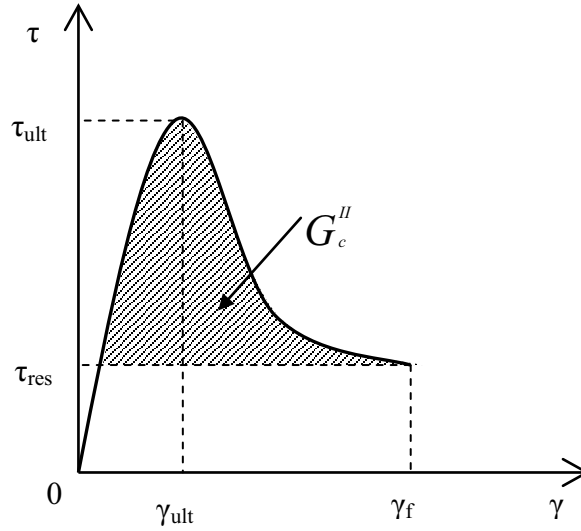


Figure 3.15 Shear softening modes for the Mode II

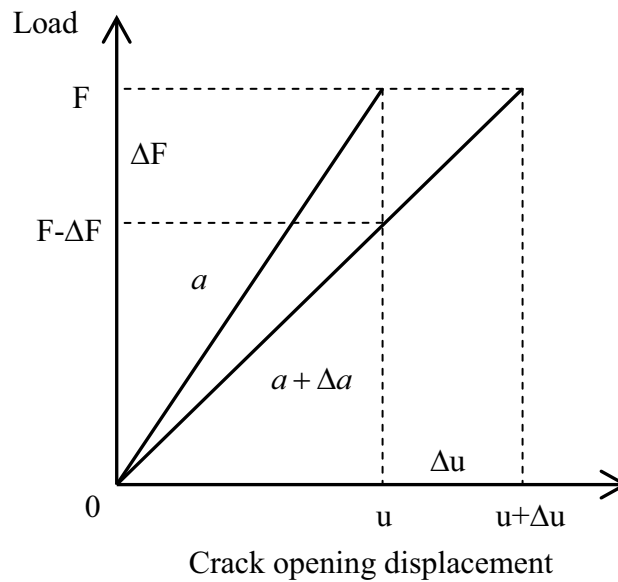


Figure 3.16 Load-displacement relations during crack propagation

Failure is described using linear softening stress strain curves for interlaminar normal and interlaminar shear direction, which must firstly be specified. Damage can occur in

interlaminar normal direction and a single interlaminar shear direction. For the normal component, failure can only occur under tensile loading and for the shear component, the behaviour is symmetric around zero. There are two ways of applying a force to enable a crack to propagate are identified in this model, being “Mode I crack”, opening mode (Figure 3.17I, a tensile stress normal to the plane of the crack) and “Mode II crack”, sliding mode (Figure 3.17II, a shear stress acting parallel to the plane of the crack and perpendicular to the crack front).

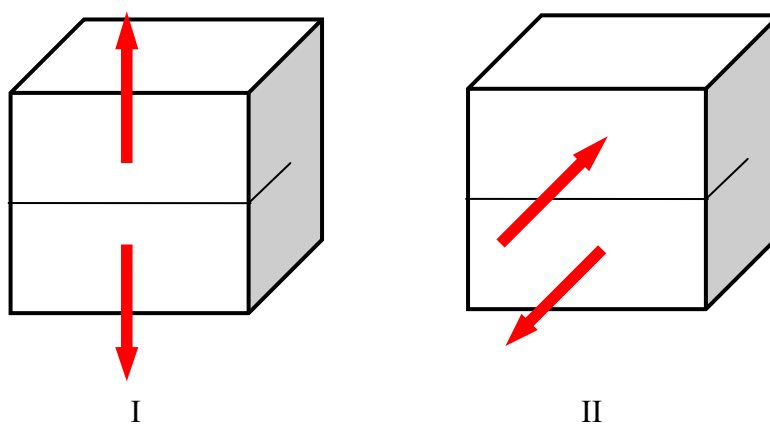


Figure 3.17 Smeared crack model under mode I and II

Two principle failure directions were specified for this model. Z axial was defined as the normal direction, and an ultimate normal tensile stress was given as 0.85 MPa. Due to torsion shear failure in bed joint, stepped failure was observed in the tests of URM walls by Griffith et al. (2007). Therefore, XY was defined as the shear direction, and a derived ultimate shear stress was given as 1.28MPa.

3.4. VALIDATION OF HOMOGENIZED MODEL

3.4.1. Experiments of Masonry Walls

Two short masonry walls were tested under uniform static loading by Griffith et al.

(2007). The experimental results were used to validate the numerical results. And the configuration of this experiment is presented in Table 3.6. Bottom edges were mortar bonded to the floor, and laterally supported by steel members, meaning, the edge connection was considered as fixed. Steel angles were used to provide lateral restraint on the top edges for both the wall with pre-compression and the wall without pre-compression. Restrain of the vertical edges was carefully considered, due to its significant effect on the results of two-way bending test. As shown in Figure 3.18, return walls were used to support the main walls, and were restrained from rotation. A uniform vertical pre-compression 0.1 MPa of stress was applied to the top of a short wall.

Table 3.6 Wall geometry and boundary conditions (Griffith et al. 2007)

Wall Geometry and Support Conditions	Pre-compression (σ_v)
	0.1 MPa
	0 MPa

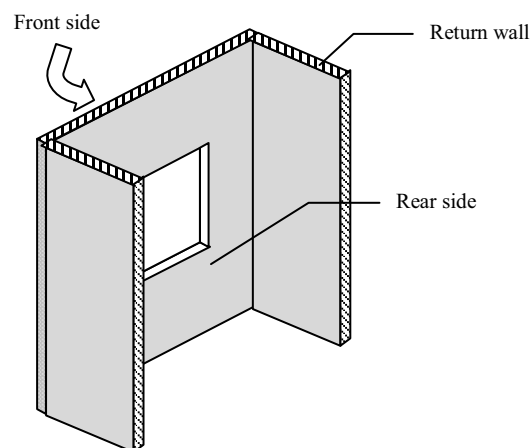


Figure 3.18 Short return walls used to stabilize walls

A uniform out-of-plane pressure was applied on the outside surface of the main wall. Airbags were used to provide the static loading, and distribute the pressure uniformly. Only the solid portions of the walls were acted on by airbags, meaning the opening part did not carry any loads. The arrangement of the airbags is shown in Figure 3.19. The load applied on the wall from the airbags was measured using load cells positioned between the airbag backing board and the reaction frame. In addition, the pressure acting on the wall surface was calculated by dividing the total load by the area of the wall. Linear variable differential transformers (LVDT) were used to measure displacements at different targets. The out-of-plane pressure applied to both of the short walls reached 8.5KPa. Details about the experimental study can be found in (Griffith et al. 2007).

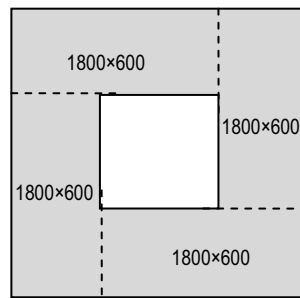


Figure 3.19 Airbag arrangement

3.4.2. Simulation of Masonry Walls

The developed homogenized material model was used to simulate the response of an unreinforced masonry (URM) wall under out-of-plane static loading, with and without pre-compression 0.1 MPa in the vertical direction as shown in Figure 3.20. The wall was $2.5\text{m} \times 2.5\text{m}$ in dimension and had a concentrically positioned opening of $1.2\text{m} \times 1.0\text{m}$. The same masonry wall was also analysed with a distinct model in which brick and mortar materials were discretized. The distinct model was built based on the masonry basic cell, consisting of about 50 thousand elements. As this model

has the same structure as the the actual walls, it was expected to present reliable results for numerical validation of the much simple homogenized model. Figure 3.21 shows the distinct and homogenized models of the masonry walls, which included 319,854 and 3,988 elements respectively. The material models of mortar and brick as well as the homogenized material model for masonry, including the equivalent elastic properties and failure criteria, were inputted into the computer program LS-DYNA in an orthotropic composite damage model for analysis.

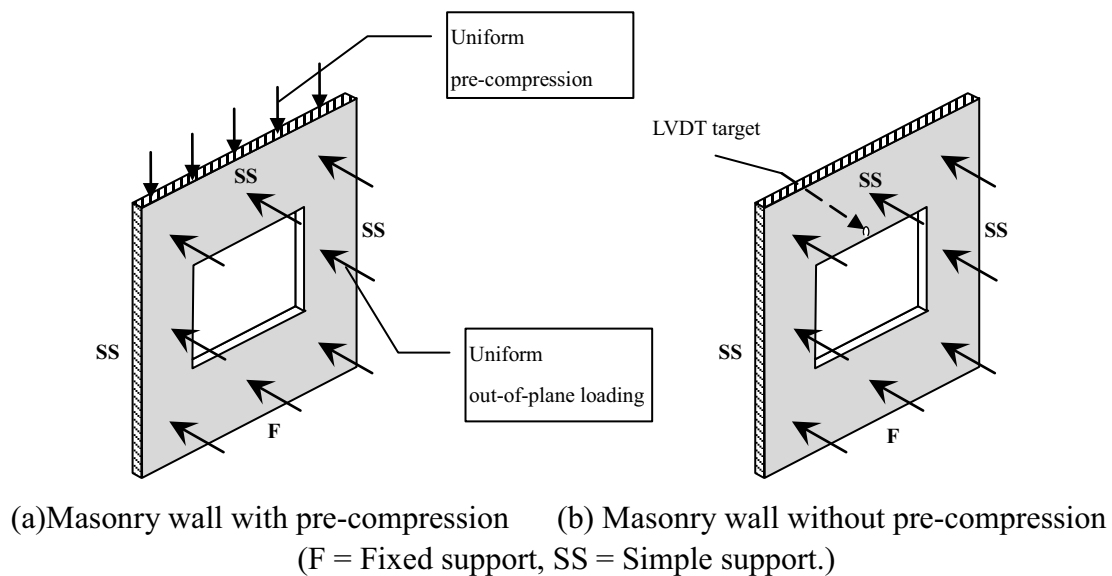


Figure 3.20 Configuration of URM wall with opening

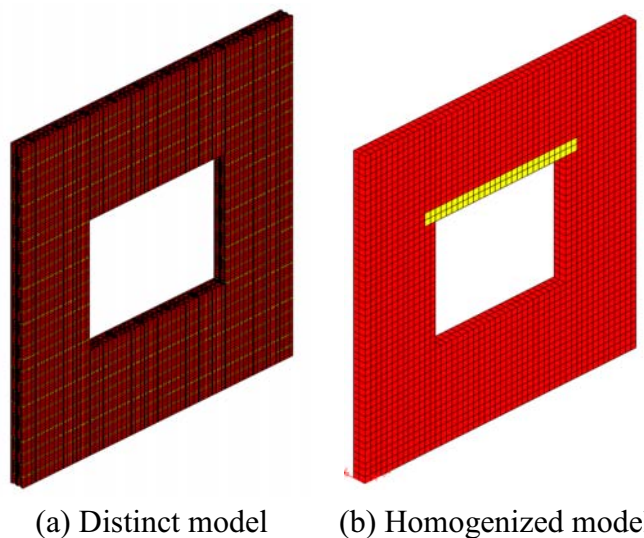


Figure 3.21 Distinct model and homogenized model of the URM walls wall opening

3.4.3. Experimental and Numerical Validation

The test data was used to verify the experimental results. The numerical verification was achieved by comparing the simulation results of the distinct and homogenized models with test data. Results of the pressure-displacement relationship and crack patterns were compared with results from simulations of distinct models as experimental validation.

Figure 3.22 shows the pressure-displacement relationship derived from tests and a numerical simulation of the wall with and without pre-compression 0.1 MPa at a target. As shown in Figure 3.22a, both the homogenized model and distinct model give a good prediction of the URM wall response without pre-compression, as compared with those obtained by experimental tests. However, with the same computer system, the time required to obtain a solution using the distinct model was 20 hours, while only 4 minutes were needed for the simple homogenized model. Again, similar responses were observed from the both models in comparison with the test results with pre-compression 0.1 MPa, as shown in Figure 3.22b. The results of the simulation with the smeared-crack model are also plotted in Figure 3.22b, and it can be seen that the curves of the simulation and test match well. However, crack patterns affect the section of curve where step cracks appear in the test. In the simulation using the smeared-crack model, the crack pattern (Figure 3.24) was not as accurate as in the distinct model. Therefore, from comparison of the pressure-displacement curves, more stiffness was observed from the smeared-crack model. With the same computer system, the calculation time for the smeared-crack model was 15 minutes.

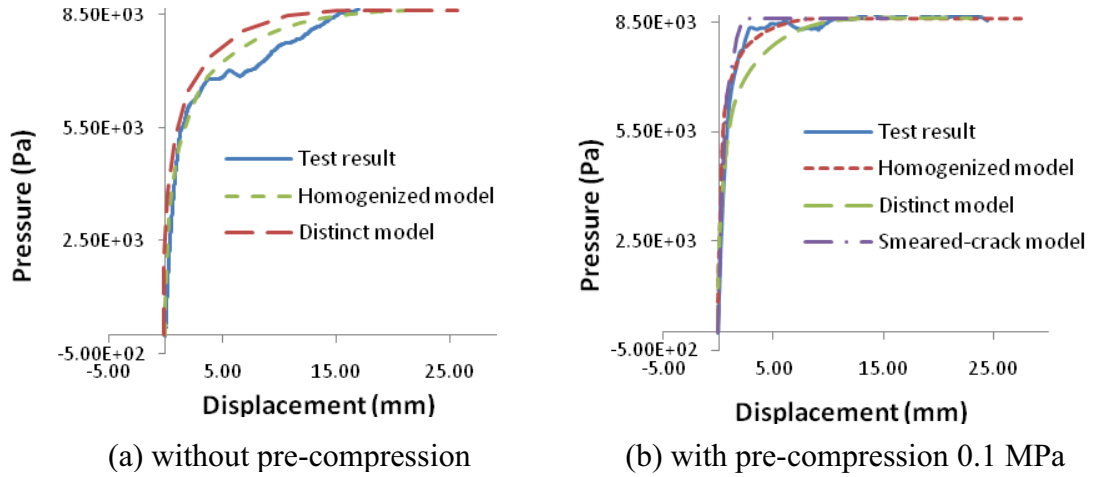


Figure 3.22 Comparison of results from the short wall with and without pre-compression test and simulation

By defining an ultimate strain for materials, elements can be removed during simulation. In this way, cracks were simulated using a distinct model on URM walls shown in Figure 3.23. Compared with test results, crack patterns match quite well in these two cases. The cut-through cracks were not observed, indicating that the failure of bricks was not accurately modelled in the numerical simulation.

It should be noted that although the homogenized model gives a reliable estimation of the global response of URM wall to static loads in far less time than the distinct model, it may yield inferior predictions of crack patterns of the URM wall compared with the distinct model. This is because the weak mortar joints may significantly affect the crack patterns. Figure 3.23 shows cracking patterns from tests with pre-compression 0.1 MPa in comparison with simulation of distinct model. The shading indicates the displacement distribution normal to the plane of the wall. As shown, the distinct model gives an accurate prediction of the crack patterns, whereas, the homogenized model does not simulate crack patterns well. Therefore, for simulating local damage of URM walls, the distinct model is a useful tool, although it is computational intensive.

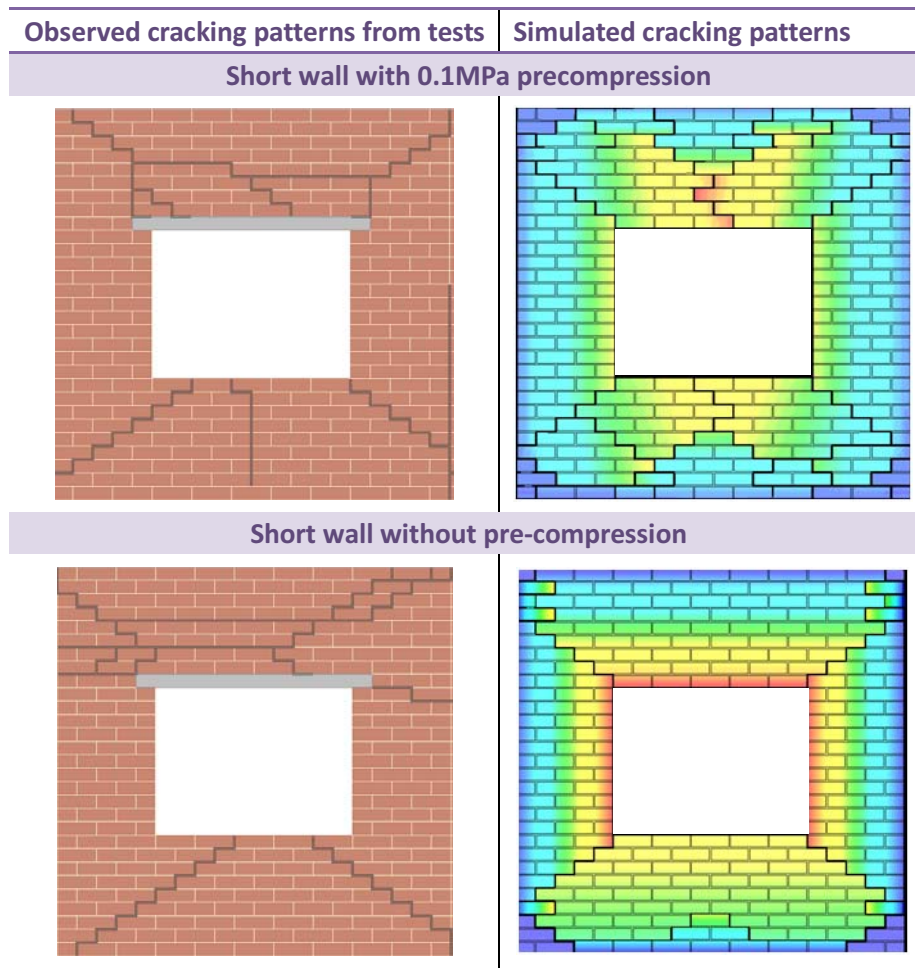


Figure 3.23 Crack patterns of tests and simulations

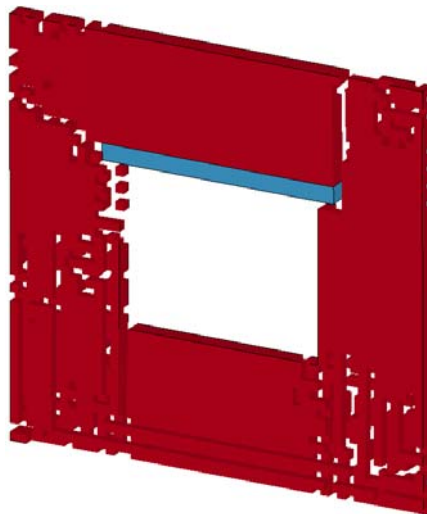


Figure 3.24 Crack patterns of smeared-crack model

3.5. CONCLUSIONS

This chapter presented numerical investigation of the ten-core brick URM wall using the homogenization technique. The equivalent material properties of the masonry unit such as the elastic moduli and failure characteristics were derived by numerical simulation of a basic cell under various boundary conditions. The developed homogenized model is then used to simulate the response of a URM wall with an opening under static loading. It was found that the simulated results using the homogenized model agree well with those obtained from the distinct model and test results. However, far less time is required for a solution using the homogenized model in comparison with distinct model. The developed homogenized model can be used to simulate large-scale masonry structures under various loads. It is worth noting that although the homogenized model has demonstrated its computational efficiency to predict the global response of the URM wall, it may not give a good simulation of local damage such crack patterns of the URM wall in comparison with the distinct model.

4. SIMULATION OF FRP REPAIRED URM WALL UNDER OUT-OF-PLANE LOADING

4.1. INTRODUCTION

The retrofitting of masonry structures with near-surface mounted (NSM) FRP plates and externally bonded (EB) FRP plates has proven to be an innovative and cost effective strengthening technique. The behaviour of such FRP-strengthened URM walls is often controlled by the behaviour of the interface between the FRP and masonry, which is investigated using a pull-test commonly. In modelling the performance of the FRP retrofitted URM wall properly, the key step is to simulate the interface behaviour between masonry and FRP retrofits.

Numerical methods have been used to simulate the interfacial behaviour of FRP-to-concrete (Al-Mahaidi and Hii 2007; Lu et al. 2006; Lu et al. 2007). Usually, there are two approaches to model debonding behaviour in FRP strengthened RC members. One approach is to employ a layer of interface elements with zero-thickness between the FRP and concrete element to simulate debonding failure. Although the bond slip behaviour can be specified in the interface elements, it is not a truly predictive model due to the zero thickness assumption for the interface elements. The second approach is to use a thin layer of concrete elements adjacent to the adhesive to simulate cracking and debonding failure. However, some research has shown that it is difficult to use appropriate concrete models to simulate debonding behaviour. Although the interfacial behaviour of FRP-to-concrete bond has been studied in pull tests recently, few studies have been conducted to investigate the bond-slip and load-displacement behaviour of the FRP-to-masonry interface in pull tests.

In this Chapter, a numerical model will be used to simulate the response of the FRP repaired URM wall under out-of-plane loads. The FRP-to-masonry interface is modelled by a layer of interface elements or contact surface of zero thickness. The interface element model and contact surface model were validated by simulating the bond-slip behaviour of pull tests of both EB and NSM CFRP plate bonded to a five-brick high masonry prism. The masonry prism in pull tests was modelled either by the derived homogenized model or by the commonly used smeared crack model. A distinct model was also employed to model masonry prism behaviour for a comparison. The efficiency and accuracy of the homogenized model was verified from simulation of the pull tests in comparison with the distinct model and the smeared crack model. The homogenized model, together with the interface element model, was then employed to simulate a severely damaged URM full-scale wall, previously tested under reversed-cyclic loading, repaired with NSM CFRP plates under out-of-loads. The smeared crack model was also used to model the response of the FRP repaired URM wall. It was found that the simulated results predicted using the homogenized model fitted well with test data.

4.2. MATERIAL MODELS IN SIMULATION

4.2.1. Masonry

The distinct model, homogenized model and smeared crack model validated in Chapter 3 were used to model the performance of the 10-core clay brick masonry in both pull tests and full scale wall under out-of-plane loading tests.

4.2.2. FRP Models

FRP composites, which are adhesively bonded to the masonry, can be modelled using an elastic-brittle material model. Both CFRP and GFRP plates were used in pull-tests. The reinforcing strips used in NSM pull-test were carbon fibre strip CFRP. The width of the carbon FRP strip was 20mm, and the thickness was 1.2mm. The material properties of CFRP were tested by Yang (2007) and the manufacture with results shown in Table 4.1. The average values appear to be comparable with the manufacturer's data.

Table 4.1 Carbon FRP material properties (Yang 2007)

NOTE:
This table is included on page 55 of the print copy of
the thesis held in the University of Adelaide Library.

Table 4.2 GFRP material properties (Yang 2007)

NOTE:
This table is included on page 55 of the print copy of
the thesis held in the University of Adelaide Library.

The glass FRP (GFRP) material properties were determined based on the tensile test performed by Yang (2007) and are summarised in Table 4.2. The average of rupture

strain was found to be approximately 11500 microstrain. The experimental values for Young's modulus and strength of the GFPR strip are 19.3 MPa and 223 MPa, respectively.

4.2.3. Bond-Slip Models

Adhesive material is used in practice to produce a continuous bond between the FRP and masonry. It can help FRP strips to develop full performance by transferring shear stress inside the layer of interface between FRP and masonry. Therefore, the interface is the key component of FRP-to-masonry bond. The behaviour of interface between the masonry and FRP is based on the strength properties of the epoxy adhesive. The adhesive had tensile strength of 13.9 MPa and Young's modulus of 6.7 GPa. The tensile strength of the adhesive material is much greater than that of masonry, hence, a failure surface was found in the masonry, but not in the adhesive layer in experiments. Therefore, to achieve the goal of simulating the pull test and studying the debonding behaviours, the interface consisting of the adhesive layer and a thin masonry layer must be simulated accurately. The interface was modelled using two methods in this study: a thin layer of interface element model and a contact surface model.

Figure 4.1 illustrates the interface element model and contact surface model. As shown in Figure 4.1a, the interface elements with a thickness of 1mm are adjacent to FRP plates and masonry while the FRP plate and masonry are contacted directly in a contact model as shown in Figure 4.1b. Since there is no thin layer of interface elements in the contact surface model, the number of elements used model will be reduced. Therefore, the contact model can be solved much more quickly.

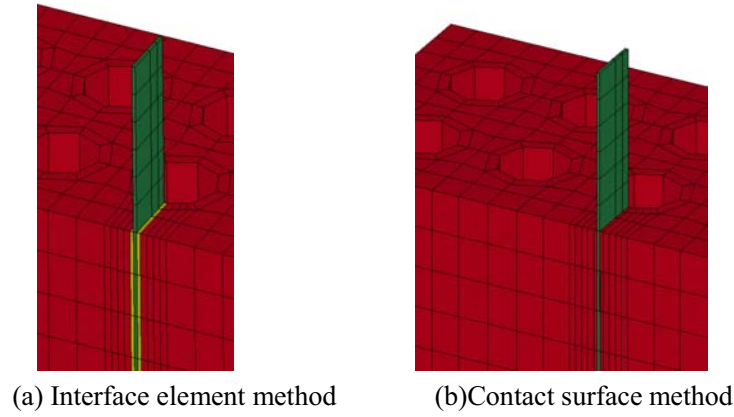


Figure 4.1 Interface elements model and contact model

For the interface element model, the interface was modelled as a thin layer of elements with thickness of 1 mm. The interface element behaved like an isotropic elastic material. The strength criterion of the interface material was dominated by debonding failure, i.e., shear failure. The post-failure process of the interface elements was controlled by fracture energy, which can be determined from the shear-slip curve. Figure 4.2a shows the experimental local bond-slip curves from pull tests, which can be idealised as a bi-linear bond-slip model as shown in Figure 4.2b (Yang 2007). Both shear debonding failure and tensile failure dominate the strength criterion of the thin layer interface material. The post-failure process of the interface material is controlled by shear fracture energy and tensile fracture energy, which equals to the area under the curves as shown in Figure 4.3a, and can be estimated by the local bond-slip models in pull tests. The relationship between shear stress and local slip can be identified by defining the ultimate stress τ_f , the corresponding slip at peak shear stress, δ_I , and slip at zero shear stress, δ_f . The shear fracture energy was estimated according to the average value of the areas under experimental bond-slip curves in a previous study (Yang 2007). σ_{ft} is assumed to be the tensile strength of brick units and tensile fracture energy rate $G_{ft} = 13.2 \text{ J/m}^2$, $\sigma_{ft} = 3.55 \text{ MPa}$ (Seracino et al. 2007). The inputs of τ_f and G_f will vary with different retrofitting techniques. It was found that for the NSM model, the maximum shear strength was 14.5 MPa, and shear fracture energy was 5000 N/m. For EB model, the maximum shear strength and shear fracture energy were found to be 5.87 MPa and

1700N/m, respectively. The above parameter values were introduced in the material model MAT_ARUP_ADHESIVE in LS-DYNA to simulate the interface between FRP strips and masonry. Inputs are summarized in Table 4.1.

NOTE:
This figure is included on page 58 of the print copy of
the thesis held in the University of Adelaide Library.

(a) Experimental results from pull-test

(b) Idealised experimental results

Figure 4.2 Behaviour of bond-slip relationship (Yang 2007)

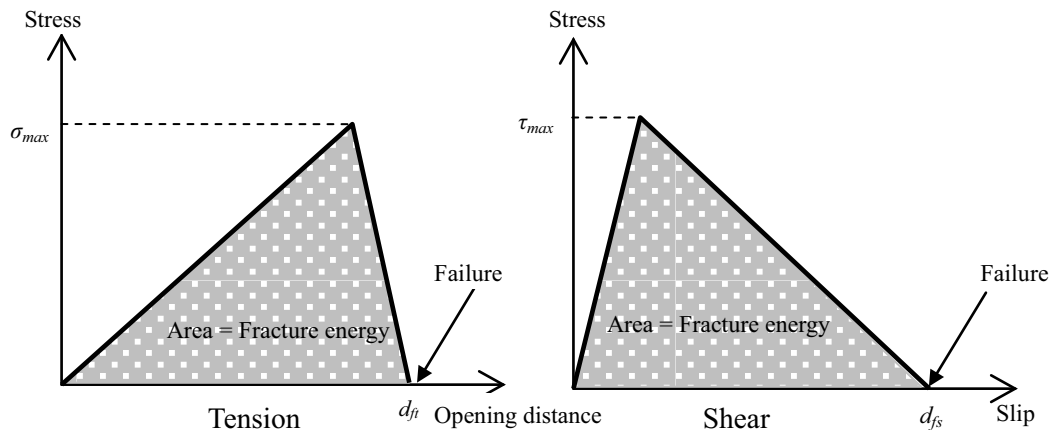


Figure 4.3 Stress-displacement curves of interface element model

Table 4.3 Inputs of interface model for various retrofits

Retrofits	τ_f (MPa)	G_f (J/m ²)
NSM CFRP	14.5	5000
EB CFRP	7.87	600.8
EB GFRP	5.87	1700
Steel plate	7.87	600.8

Before yielding, the material model behaves like an isotropic elastic material. The yield and failure surfaces of the interface elements are treated as a power-law combination of direct normal stress and shear stress across the bond in Eq. 4-1. The yield criterion considered both tension and shearing for the interface element as defined and shown in Figure 4.4.

$$\left(\frac{\sigma_t}{\sigma_{ft}}\right)^2 + \left(\frac{\tau}{\tau_f}\right)^2 = 1 \quad \text{Eq. 4-1}$$

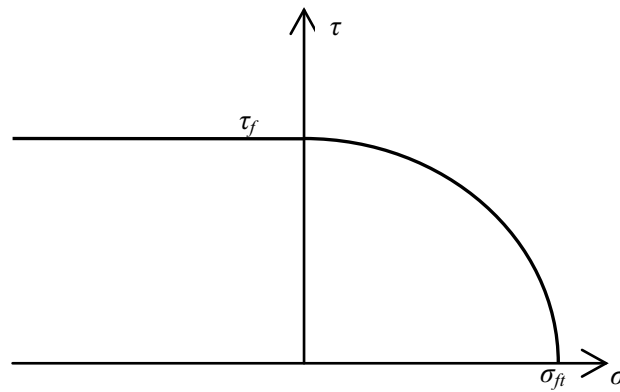


Figure 4.4 Yield criterion for interface material model

As an alternative method, the FRP-to-masonry interface was also modelled using contact between FRP plates and masonry directly. The bond-slip behaviour in the contact surface models was also defined as a bi-linear shear-slip curve (Figure 4.3a). The failure shear stress τ_f , failure tensile stress σ_f and failure slip δ_f were input as control parameters. In the contact surface model, FRP plates and masonry were considered as initially tied with contact nodes. Tiebreak will not occur until the failure stress criterion is satisfied. A separation of contact parts at specific shear or normal stresses is provided in the failure criterion, which is identified by

$$\left(\frac{|\sigma_n|}{\sigma_f}\right)^2 + \left(\frac{|\tau_s|}{\tau_f}\right)^2 \geq 1 \quad \text{Eq. 4-2}$$

where σ_n and τ_s denote the normal and shear stresses, and σ_f and τ_f denote the ultimate normal and shear stresses, respectively. The damage is defined as a linear

function of deformation between nodes initially in contact. After the stress reaches the ultimate stresses, damage initiates and stress is scaled by the linear damage function. When the deformation is increased to the critical failure slip δ_f , the damage is completed, and the contact fails. After the failure, only friction was considered between the initially contacted parts. Assuming there is no load reversals, the energy released rate due to the failure of the interface is approximately $0.5 \times S \times \delta_f$, where

$$S = \sqrt{\max(\sigma_n, 0)^2 + |\sigma_s|^2} \quad \text{Eq. 4-3}$$

at initiation of damage.

In LS-DYNA, contact model “CONTACT AUTOMATIC SURFACE TO SURFACE TIEBREAK” was selected to perform the contact analysis. For the NSM model and EB model, the ultimate shear stress τ_f is equal to 14.5MPa and 5.9MPa, respectively. Because the failure surface was observed to be in experimental masonry tests, the ultimate normal stress for the FRP-masonry interface element was defined as 0.6MPa and 1.8MPa for the NSM and EB models, respectively. Based on experimental results, the failure slip was set as 1.25mm and 0.95mm for the NSM model and EB model, respectively.

4.3. VALIDATION OF THE NUMERICAL MODEL USING PULL TESTS

4.3.1. Pull Test Program

Pull tests are usually used to investigate the bond behaviours of EB GFRP and NSM CFRP to masonry (Griffith et al. 2007). Figure 4.5 shows the experimental specimens. As shown, a five-brick high masonry model was used for each pull-test and bricks were bonded by using 10 mm thick common mortar. Glass FRP strips were used in the externally bonded specimen in the tests, while carbon FRP strips

were used in the near surface mounted specimen. In the testing, the bottoms of the specimens were fixed, and a tensile load was applied to the top of FRP strips until debonding occurred. The load and strains along FRP strips were recorded in these pull tests as shown in Figure 4.5. The local bond-slip curves and global load-displacement curves were estimated from the recorded data. Figure 4.6 shows debonding failed along the FRP strips within masonry, while the adhesive material was undamaged. Therefore, the interface between masonry and FRP strip was the key component. Coding the material models for FRP, masonry and the interface into a finite element program LS-DYNA, the interface element model and contact surface model were validated by simulating the bond behaviours of EB GFRP and NSM CFRP plates to masonry in the pull tests.

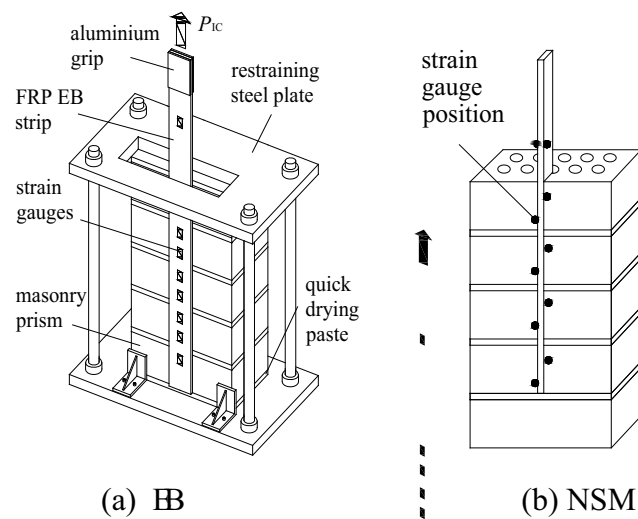


Figure 4.5 Pull-test specimens



Figure 4.6 GFRP fully debonding failure

4.3.2. Distinct Models

A distinct model for masonry introduced in Chapter 3 was used in the simulation of the pull tests. Figure 4.7 shows distinct numerical models of NSM and EB pull-tests. The top surface of the masonry block was fixed in the vertical direction to model the restraint plate, and the bottom of the model was fixed in all degrees of freedom. The tensile load in the numerical model was applied on the top of FRP strips by the displacement control method until debonding occurred. Both CFRP and GFRP were modelled using an elastic-brittle material model. Rupture of FRP plates was controlled using principle strain values in this study. Both the interface element model and contact surface were used to model the interface between FRP and masonry prism in the simulation.

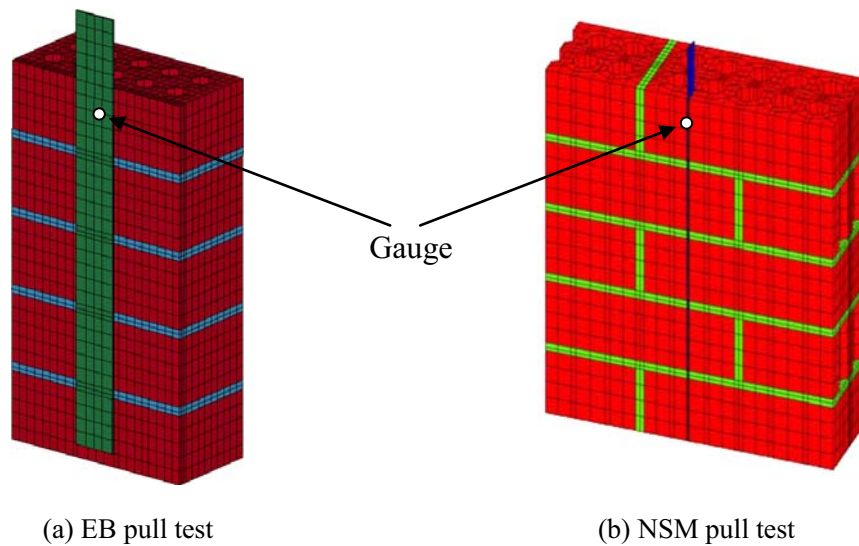


Figure 4.7 Distinct numerical models of NSM and EB pull-tests

Figure 4.8 shows the local bond-slip relationships from experiments and numerical simulation of the pull tests using interface element method. As shown in Figure 4.8a, the interface element model gave good predictions of the local bond-slip relationship

for the EB CFRP strip at 56 mm below the top surface as compared with those obtained from pull tests. Similar results were observed from the simulated local bond-slip relationships of NSM CFRP plate at 20.5 mm below the top surface from pull tests in comparison with the test results as shown in Figure 4.8b. Figure 4.9 shows the corresponding global load-displacement curves from the numerical simulation and test data, where it can be seen that the numerical simulation agreed reasonably well with test data.

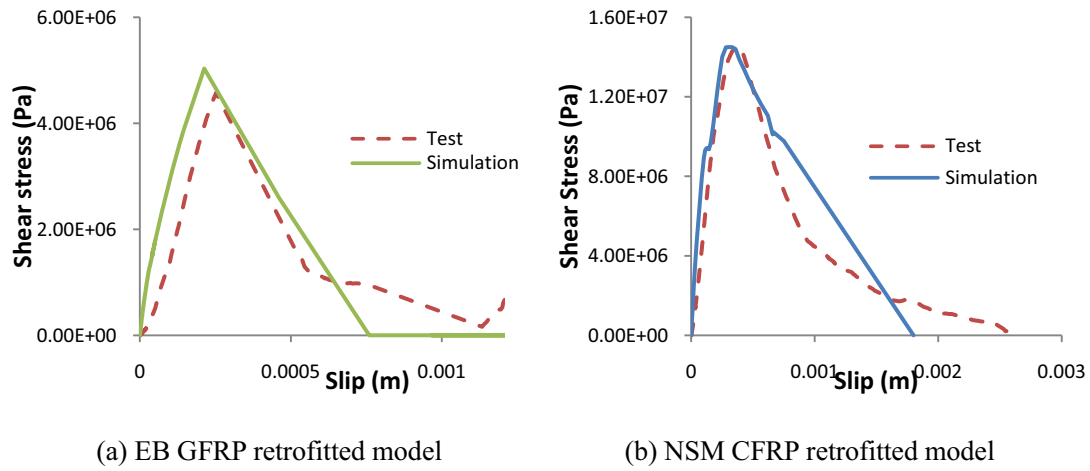


Figure 4.8 Comparison of results of local bond-slip relationships in a pull test using interface element model

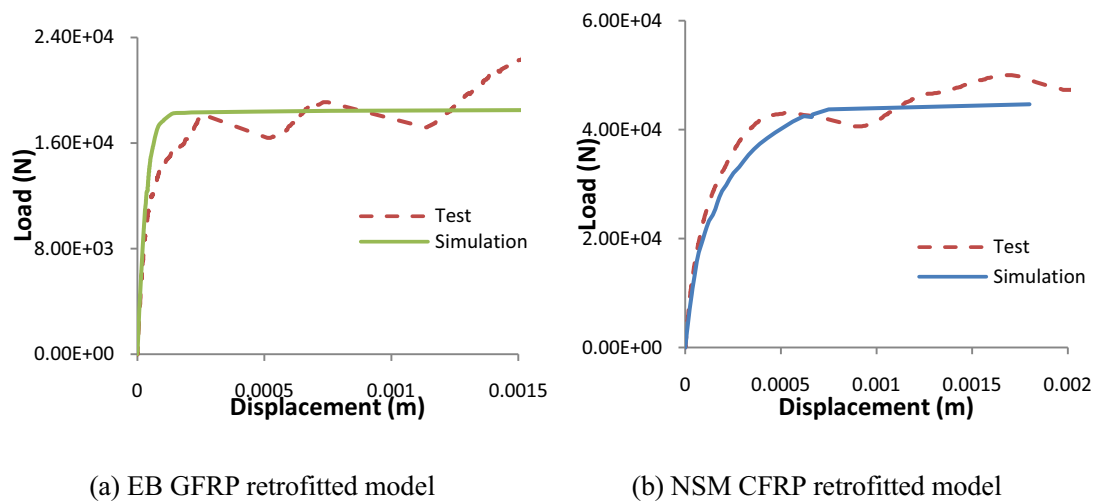


Figure 4.9 Comparison of results of load-deflection curves in a pull test using interface element model

The simulation of local bond-slip relationships in pull tests using contact surface model were also conducted and compared with test data as shown in Figure 4.10. It can be observed that the contact model also predicted the local bond-slip relationships from both NSM and EB FRP plates to masonry very well. Figure 4.11 shows a comparison of results of global load-displacement curves in a pull test using the contact model. As shown, reasonable predictions were obtained for the NSM and EB FRP retrofitted models.

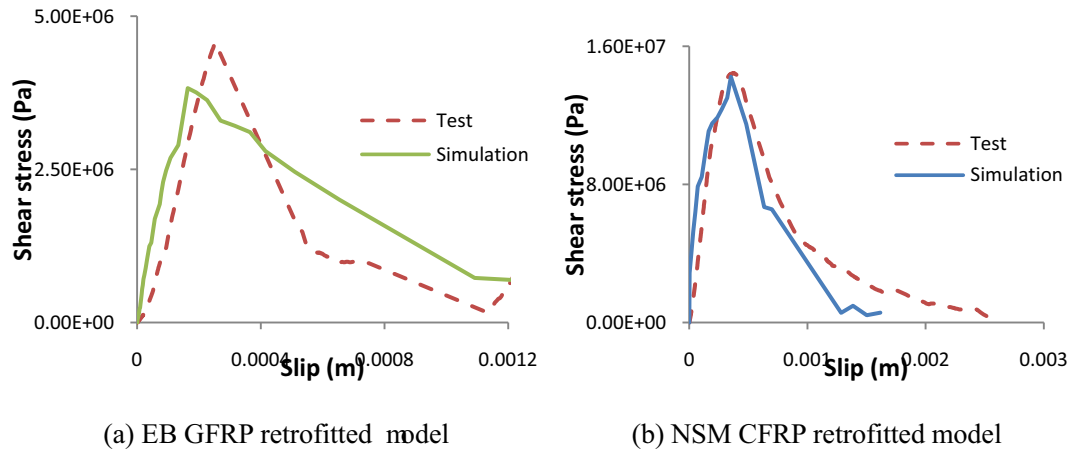


Figure 4.10 Comparison of results of local bond-slip relationships in a pull test using contact model

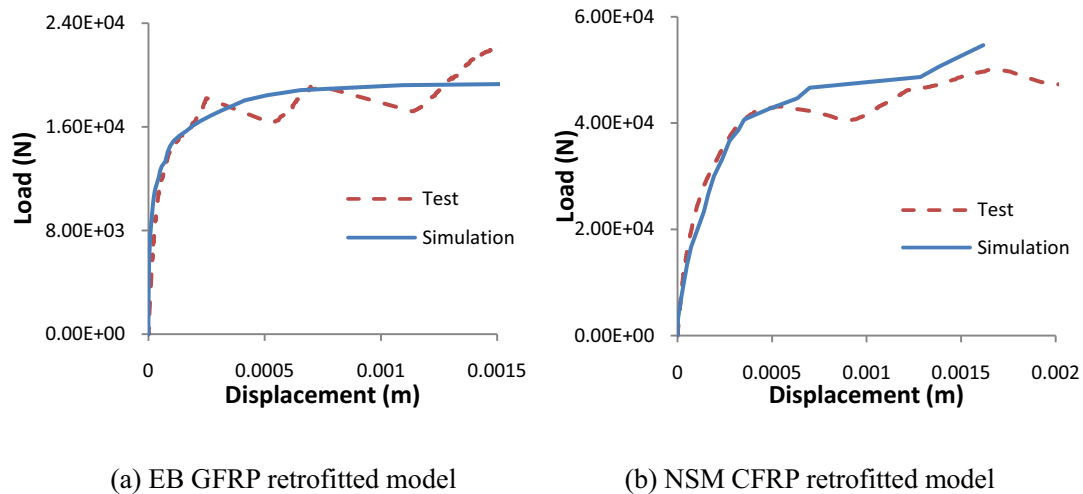


Figure 4.11 Comparison of results of load-deflection curves in a pull test using contact model

In the numerical simulation using an interface element model, failure progress was observed by tracking erosion of interface elements in details (Figure 4.12). Debonding occurred at the top of the bond area while the shear stress reached the ultimate value, after which, failure extended to the bottom quickly. Figure 4.13 shows crack patterns of the simulation, which matches well with test results.

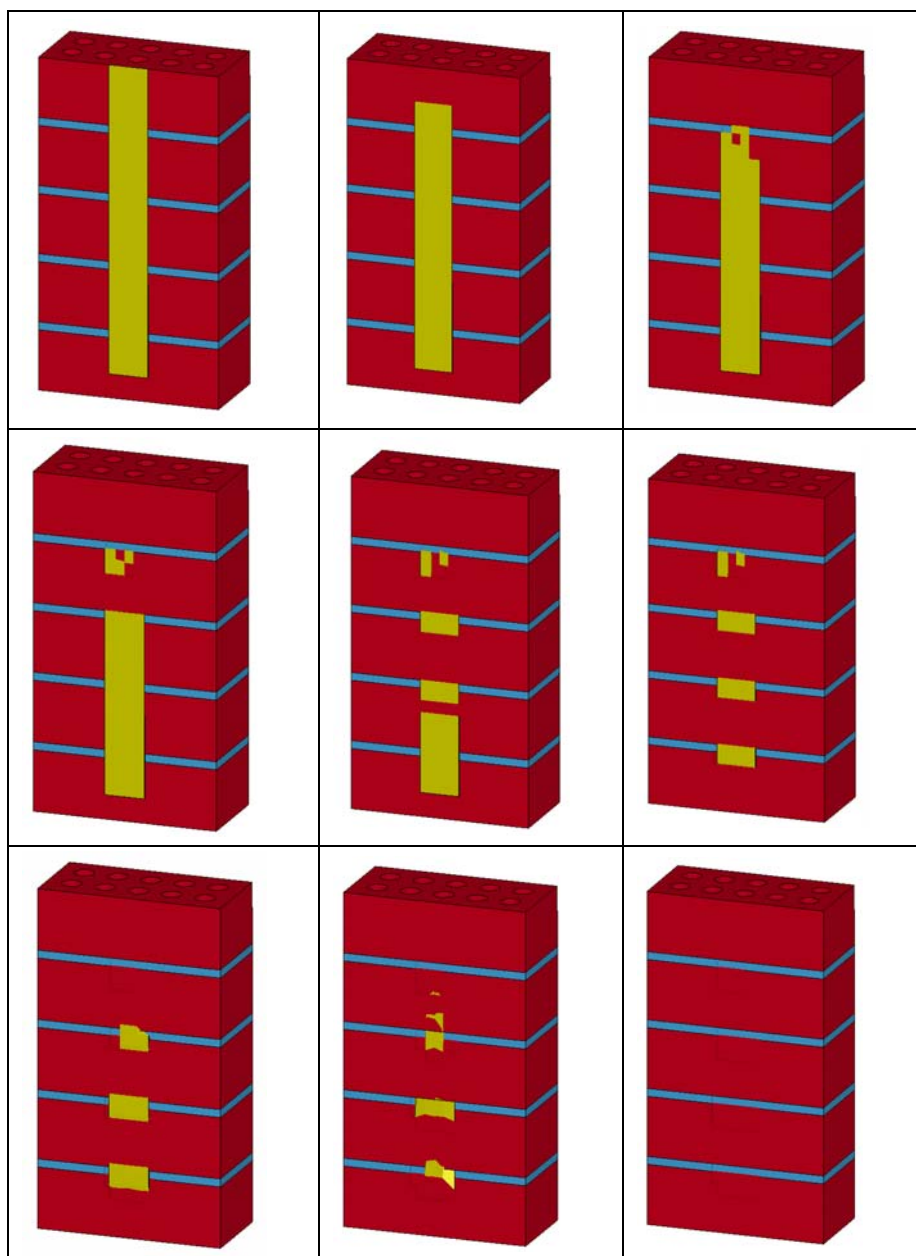
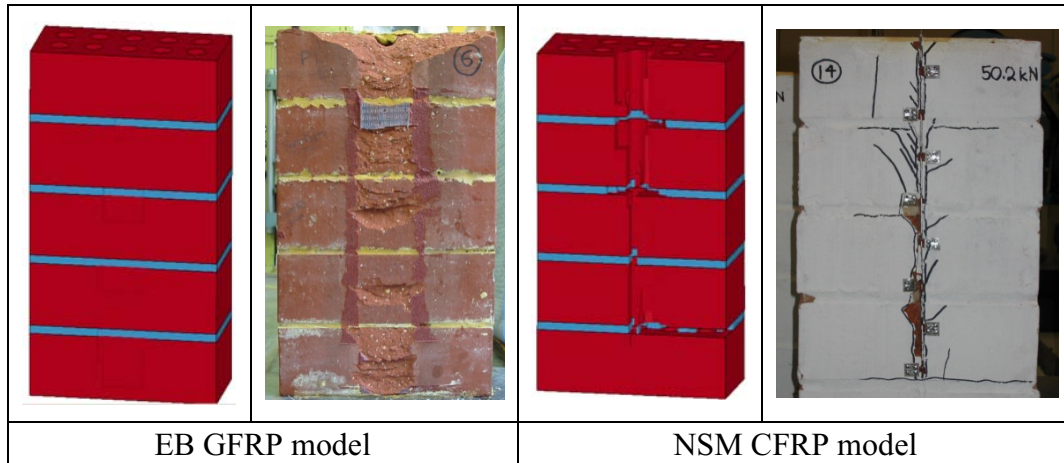


Figure 4.12 Debonding failure progress of EB GFRP model

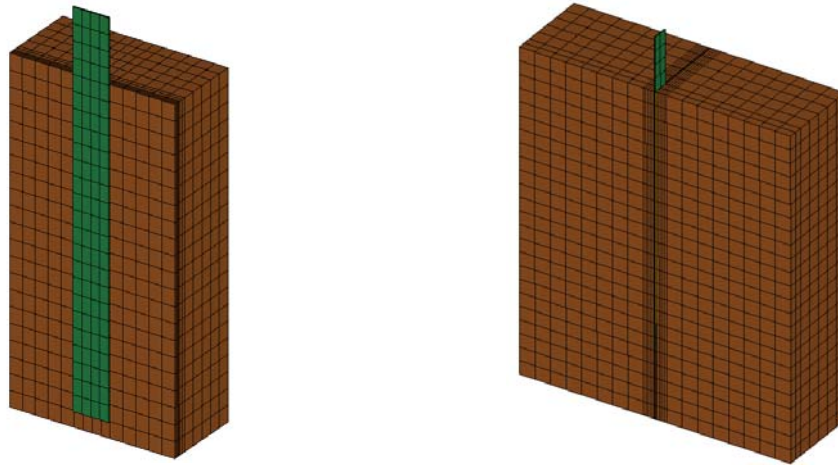
**Figure 4.13 Crack patterns**

It should be noted that although both the interface element method and contact surface method gave reliable estimations of local bond-slip relationships and global load-displacement curves for NSM and EB FRP retrofitted models in pull tests, the time spent in contact model is less than that in interface element model, due to its simple stress transference process. In the models with same number of elements, the contact model saved approximately 50% to 80% calculation time, indicating this model is more efficient than NSM and EB retrofitted members. Moreover, compared with the interface element model, there is less limitation in meshing geometric models, and thus numerical models can be further simplified to save more calculation time. However, the contact surface model may not yield reasonable predictions of debonding failure mechanism of the pull tests as good as the interface element model due to the zero thickness of the interface.

4.3.3. Homogenized Model and Smeared Crack Model

The homogenized model derived in Chapter 3 for masonry together with the elastic material model for FRP and interface element model were coded into the finite element program LS-DYNA to simulate the bond behaviours of EB GFRP and NSM CFRP plates to masonry in pull tests. Figure 4.14a and Figure 4.14b show the

homogenized models of pull-tests of NSM CFRP plates and EB GFRP strips bonded to two five-brick high masonry prisms. In order to check the reliability and computational efficiency of the homogenized model in the numerical simulation, the same pull tests were also analysed with the distinct model and the smear crack model.



(a) Homogenized model of EB pull test (b) Homogenized model of NSM pull test

Figure 4.14 Homogenized models of pull tests

Figure 4.15 shows the local bond-slip relationships from experiments and numerical simulation of the pull tests using the homogenized model and the distinct model. It can be observed in Figure 4.15a that both the homogenized model and the distinct model gave good predictions of the local bond-slip relationship for the EB GFRP strip at 56 mm below the top surface as compared with those obtained from pull tests. More accurate results were observed from the simulated local bond-slip relationships of NSM CFRP plate at 20.5 mm below the top surface from pull tests in comparison with the test results as shown in Figure 4.15b. Figure 4.16 shows the corresponding global load-displacement curves from the numerical simulation and test data, where it can be seen that numerical results from the homogenized model and distinct model agreed reasonably well with test data. It should be noted that although the layout of the five-brick high masonry prism in Figure 4.14a was different from that of basic cell shown in Figure 4.14b, the simulation demonstrated

that both models gave good results, indicating that the homogenized model derived from basic cell of masonry in Chapter 3 can also be used to simulate EB GFRP and NSM CFRP plates to five-brick high masonry prism.

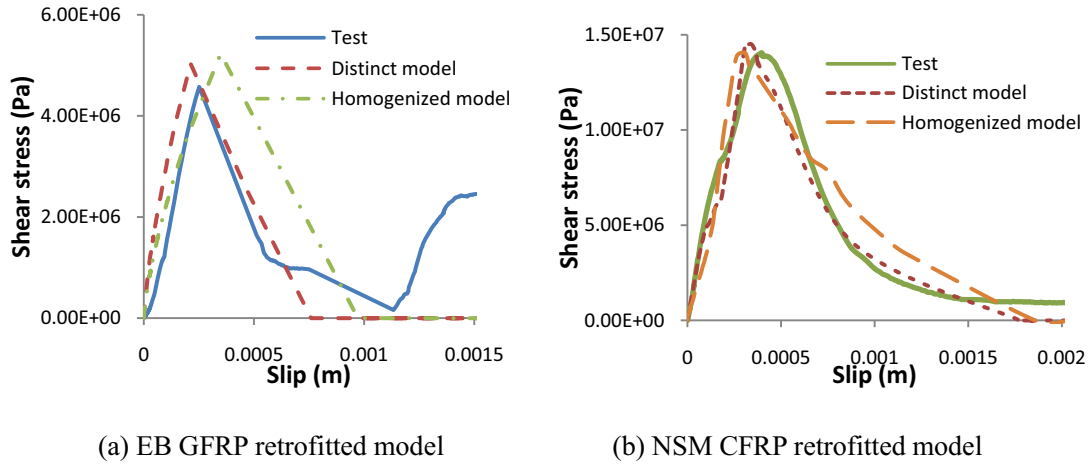


Figure 4.15 Comparison of results of local bond-slip relationships in pull tests

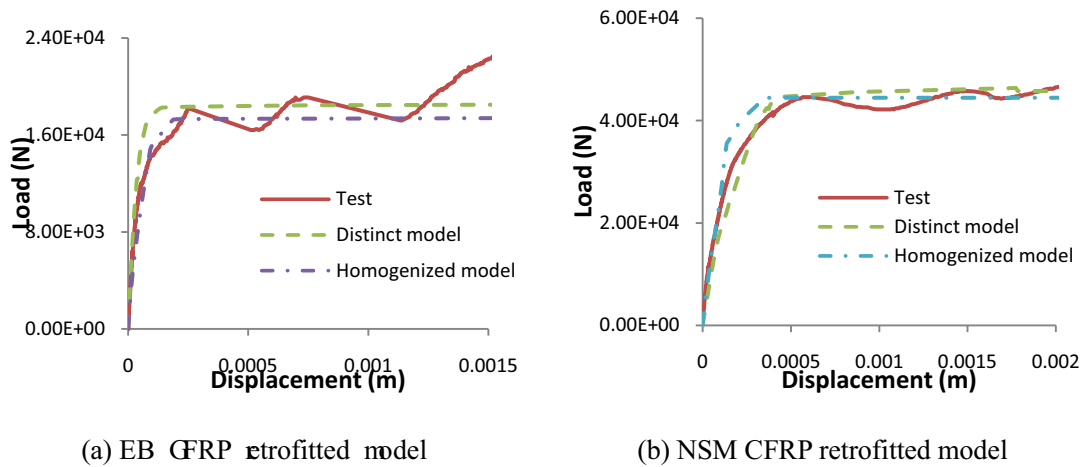


Figure 4.16 Comparison of results of load-deflection curves in pull tests

The same local bond-slip relationships in the above pull tests were also simulated using the smear crack model. Figure 4.17 shows a comparison of the simulated results using the smeared crack model and the distinct model with the test data. It can be observed that the smear crack model also predicted the local bond-slip relationships for both NSM and EB FRP plates bonded to masonry prisms very well. Figure 4.18 shows a comparison of global load-displacement curves in a pull test

using the smear crack model and distinct model. As shown, reasonable predictions were obtained for both FRP strips or plates bonded to masonry prisms in pull tests.

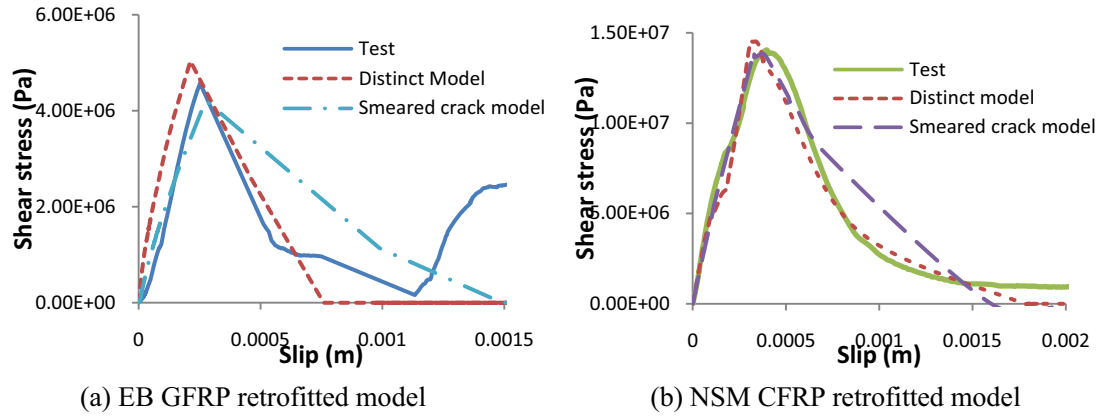


Figure 4.17 Comparison of results of local bond-slip relationships in pull tests

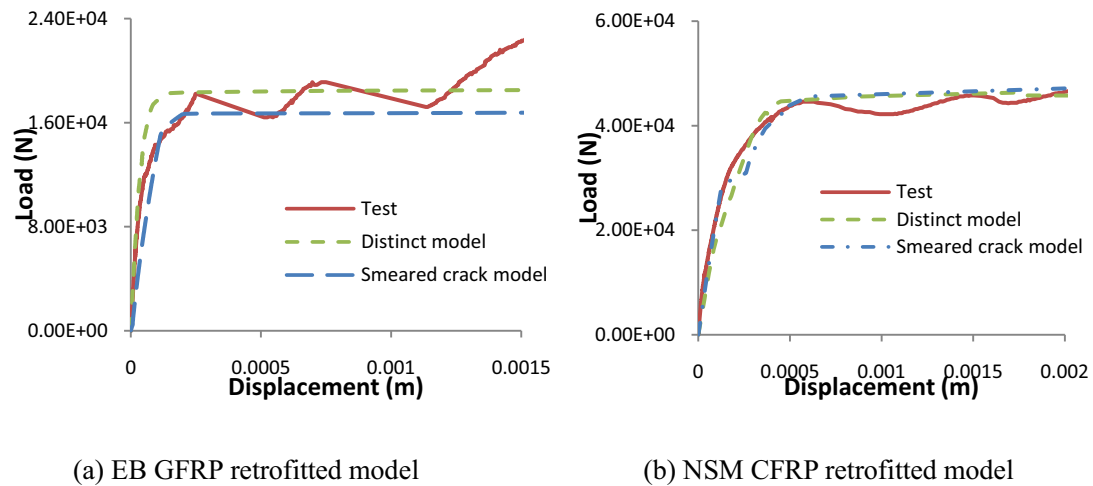


Figure 4.18 Comparison of results of load-deflection curves in pull tests

It should be noted that while distinct, smeared crack and homogenized models all gave reliable estimates of local bond-slip and global load-displacement for pull tests, the solution time varied significantly. In the same pull test simulation, the homogenized model could save about 75% and 90% calculation time, in comparison with the smear crack model and the distinct model. This is shown in Figure 4.19, and indicates that the homogenized model is the most efficient to model NSM and EB plates bonded to masonry prisms in pull tests. It should also be noted that although both the homogenized model and smear crack model gave accurate

prediction of results of pull tests with far less time compared with the distinct model, it may not yield reasonable prediction of debonding failure mechanism of the pull tests as good as the distinct model because the weak mortar joints may significantly affect the debonding process.

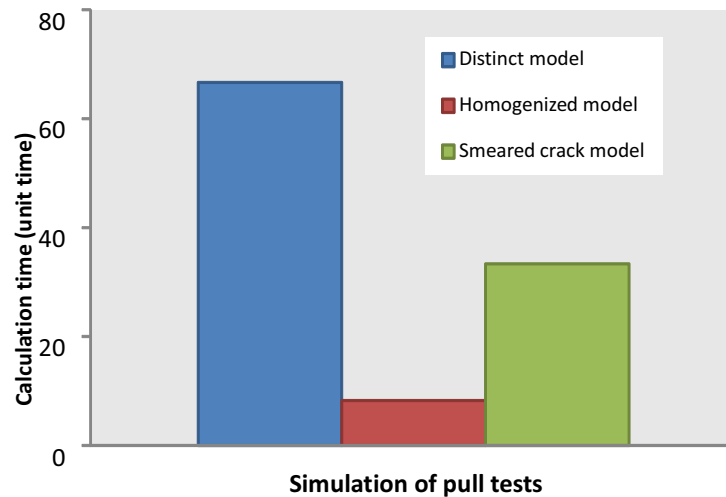


Figure 4.19 Comparison of computing time with different models in pull tests

4.4. APPLICATION OF THE NUMERICAL MODELS FOR FRP REPAIRED URM WALLS UNDER OUT-OF-PLANE LOADING

The above validated numerical models were coded into the finite element program LS-DYNA to simulate the response of two FRP repaired URM walls (with window openings), under reversed-cyclic loading. The two walls were repaired, respectively, with NSM CFRP plates and EB GFRP strips and tested under two-way monotonic out-of-plane bending with pre-compression 0.1 MPa in the vertical direction. The same tests were also analyzed with the smear crack model for a comparison. Figure 4.20 shows the damaged URM wall with opening repaired with two NSM CFRP strips with 20 mm wide x 1.4 mm thick symmetric fixed in vertical direction. The wall configurations and existing crack patterns in the experimental study were also illustrated in Figure 4.20. Figure 4.21 shows the damaged URM wall repaired with

five EB 77 mm wide x 2.0 mm thick prefabricated GFRP strips spaced at 500 mm, with two strips also placed adjacent to the window opening. The details of existing crack patterns are depicted in Figure 4.21 and the experimental setup of the two FRP repaired damaged URM walls were shown in Figure 4.22. In these experimental tests, airbags were used to apply lateral pressure onto the FRP strengthened URM wall specimens to simulate out-of-plane load induced by earthquakes. The load applied on the wall using the airbags was measured using load cells positioned between the airbag backing board and the reaction frame and the pressure acting on the wall surface was calculated by dividing the total load by the area of the wall. Linear variable differential transformers (LVDT) were used to measure displacements at different targets. Strain gauges were placed on the FRP plates at different points to record stress-strain curves. Details about the experimental study can be found in (Yang 2007).

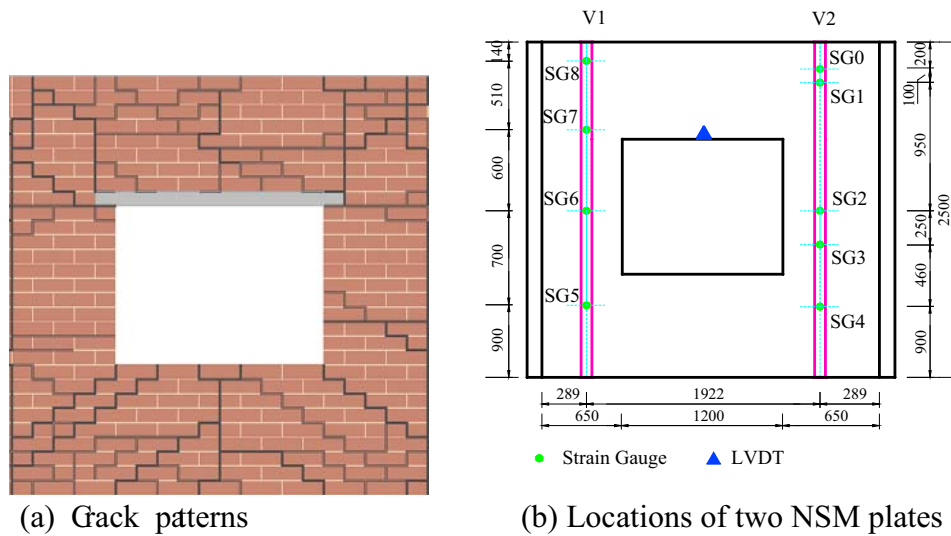


Figure 4.20 Configuration of the damaged URM wall repaired with two NSM plates

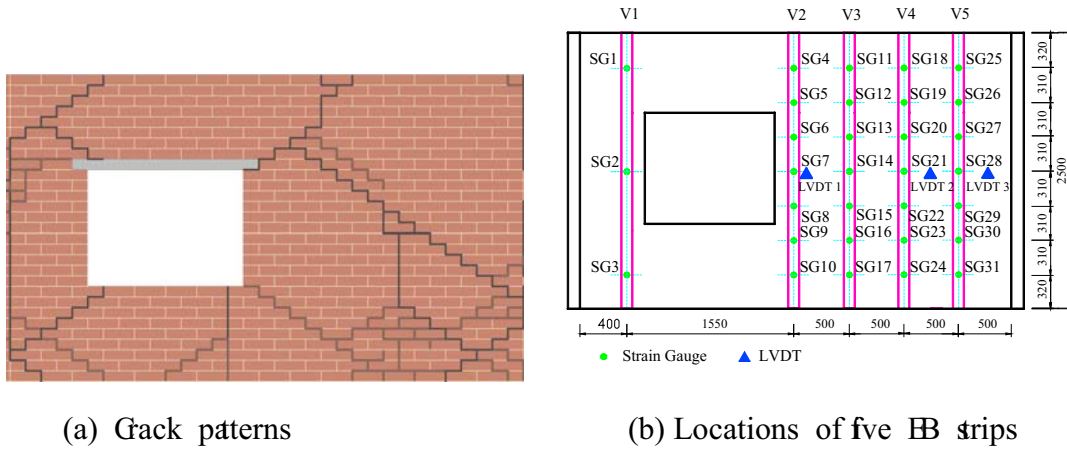


Figure 4.21 Configuration of the damaged URM wall repaired with five EB strips

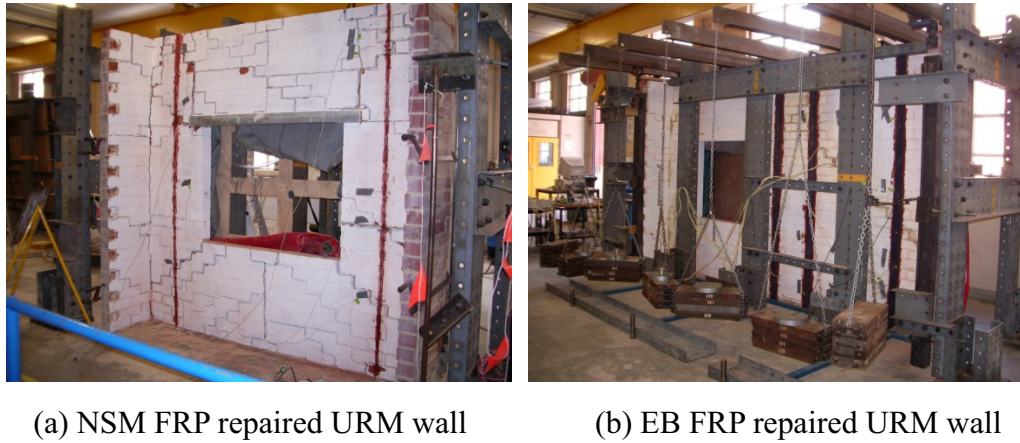


Figure 4.22 Experimental setup for the FRP repaired damaged URM wall

Figure 4.23 shows the numerical models for the two FRP repaired URM walls. Both the homogenized model and smear crack model were used to model the behaviour of masonry. The validated interface element models in the above section were used to model the behaviour of the bond-slip of FRP-to-masonry interface for NSM and EB retrofitting. In the numerical model, existing crack patterns of the two specimens tested under reversed-cyclic loading, shown in Figure 4.20a and Figure 4.21a, were modelled as contact surfaces between different parts of masonry as shown in Figure 4.23. Friction ratio of cracks on the contact surfaces can range from 0.7 to 2.5 (Willis et al. 2004). Since the post-static test cracking patterns on the damaged specimens were generated by the reversed-cyclic loading, the friction coefficient of

these existing cracks on the URM test specimen was determined by simulating the last part of load-displacement curve of the specimen under the repeated cyclic load as shown in Figure 4.24. Trail analysis was carried out by varying friction coefficients between 0.7 and 1.3. Figure 4.25 shows the simulation of the last part of load-displacement curve with various friction coefficients. It was found that a friction coefficient of 0.9 fitted well with test results. Thus the friction coefficient 0.9 was used in the contact model in LS-DYNA program to perform the analysis.

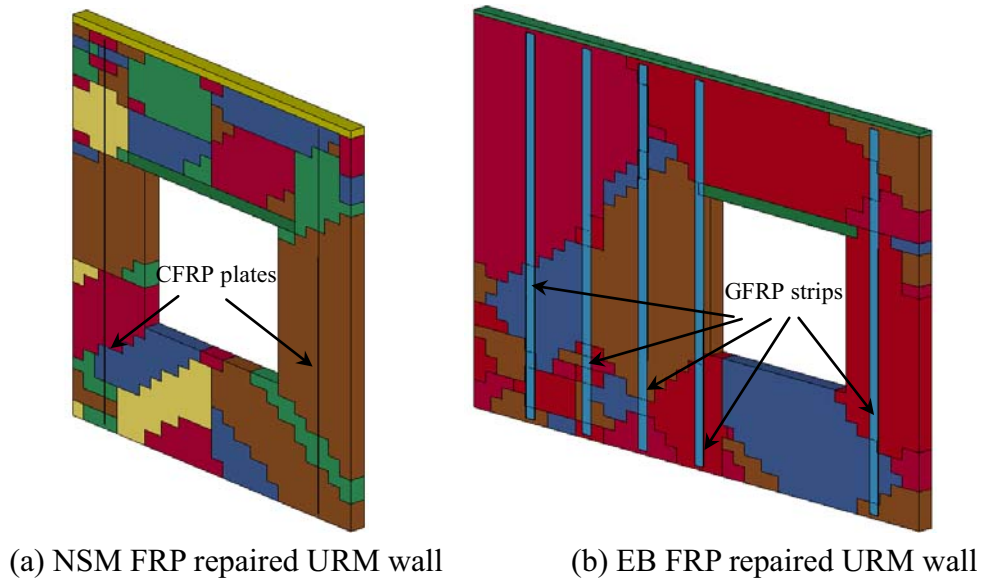


Figure 4.23 Numerical models of the FRP repaired damaged URM walls

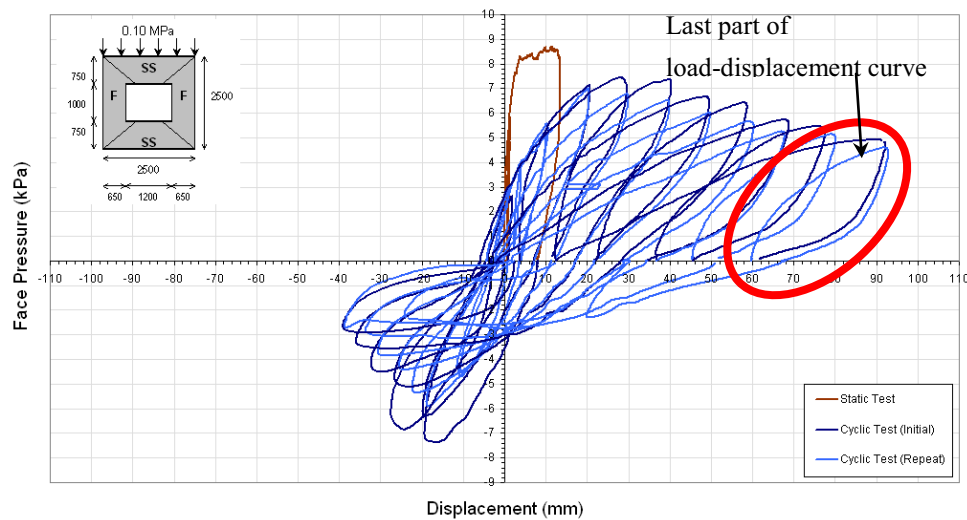


Figure 4.24 Load-displacement curves of the URM walls under the reversed-cyclic loading

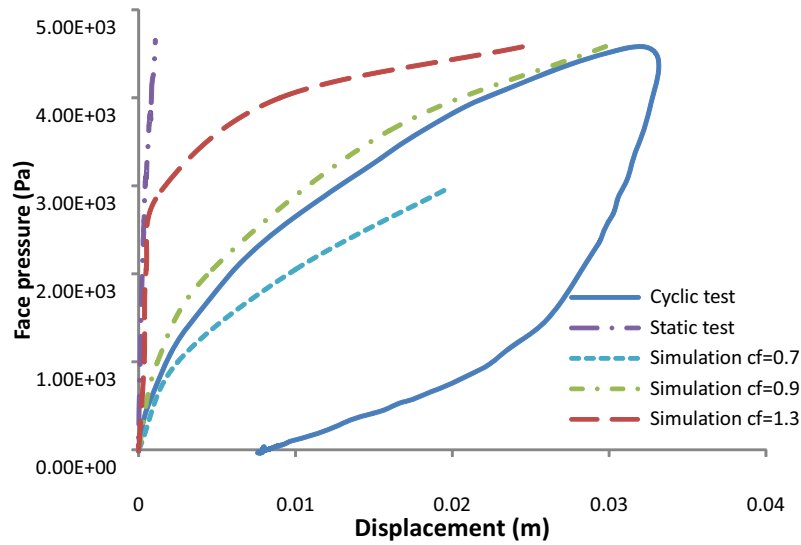


Figure 4.25 Simulation of the last part of load-displacement curve with various coefficients of friction

Figure 4.26 shows load-displacement curves from tests and numerical simulations at the target using the homogenized model and the smear crack model. As shown in Figure 4.27, both the homogenized model and smear crack model gave good predictions of the NEM CFRP repaired URM wall response as compared with those obtained by experimental tests. The distribution of maximum strains along the two EB GFRP plates obtained from numerical simulation using the homogenized model and smear crack model was in comparison with test data as shown in Figure 4.28. As shown, the homogenized model gave a more accurate prediction than the smear crack model. Similar responses were observed from the both models in comparison with the test results of EB GFRP plates repaired URM long wall as shown in Figure 4.29. It should be noted that with the same computer system the time spent for the smeared crack model to solve the problem was much more than for the simple homogenized model.

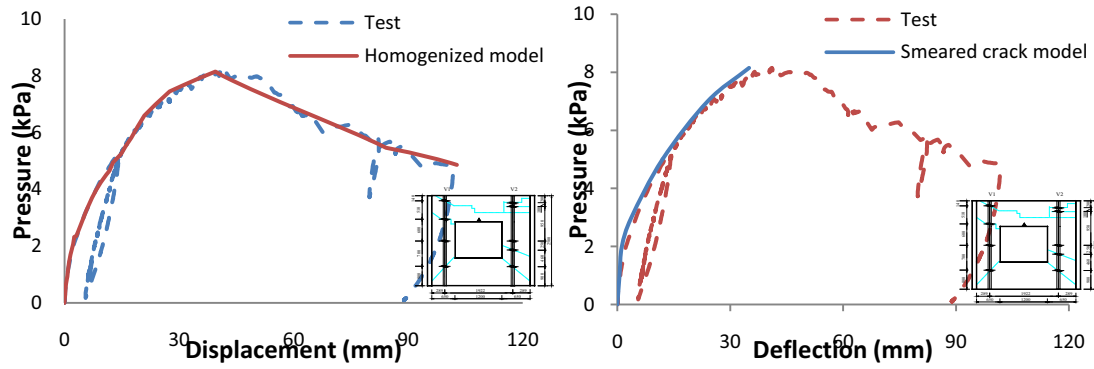


Figure 4.26 Simulation of NSM repaired URM wall using the homogenized model and smear crack model

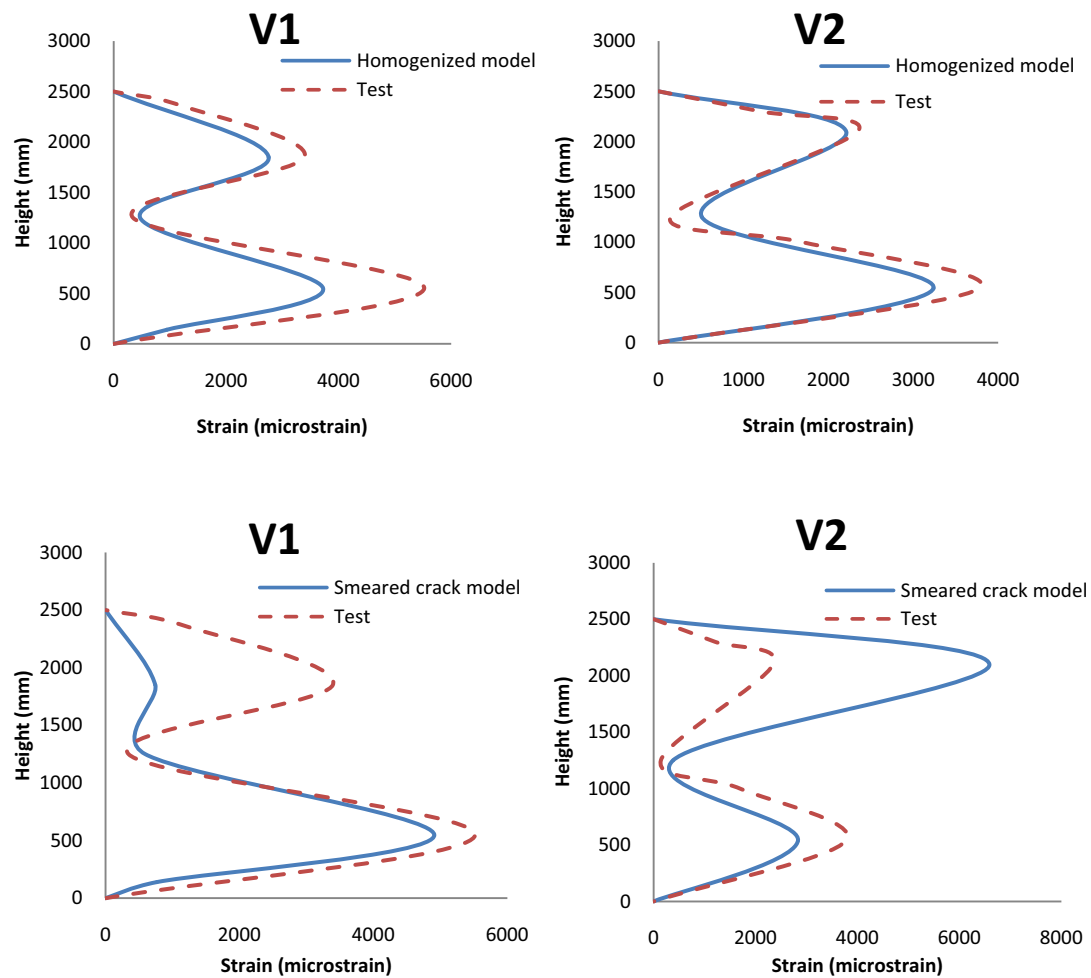


Figure 4.27 Comparison of maximum strain distribution along two NSM CFRP plates

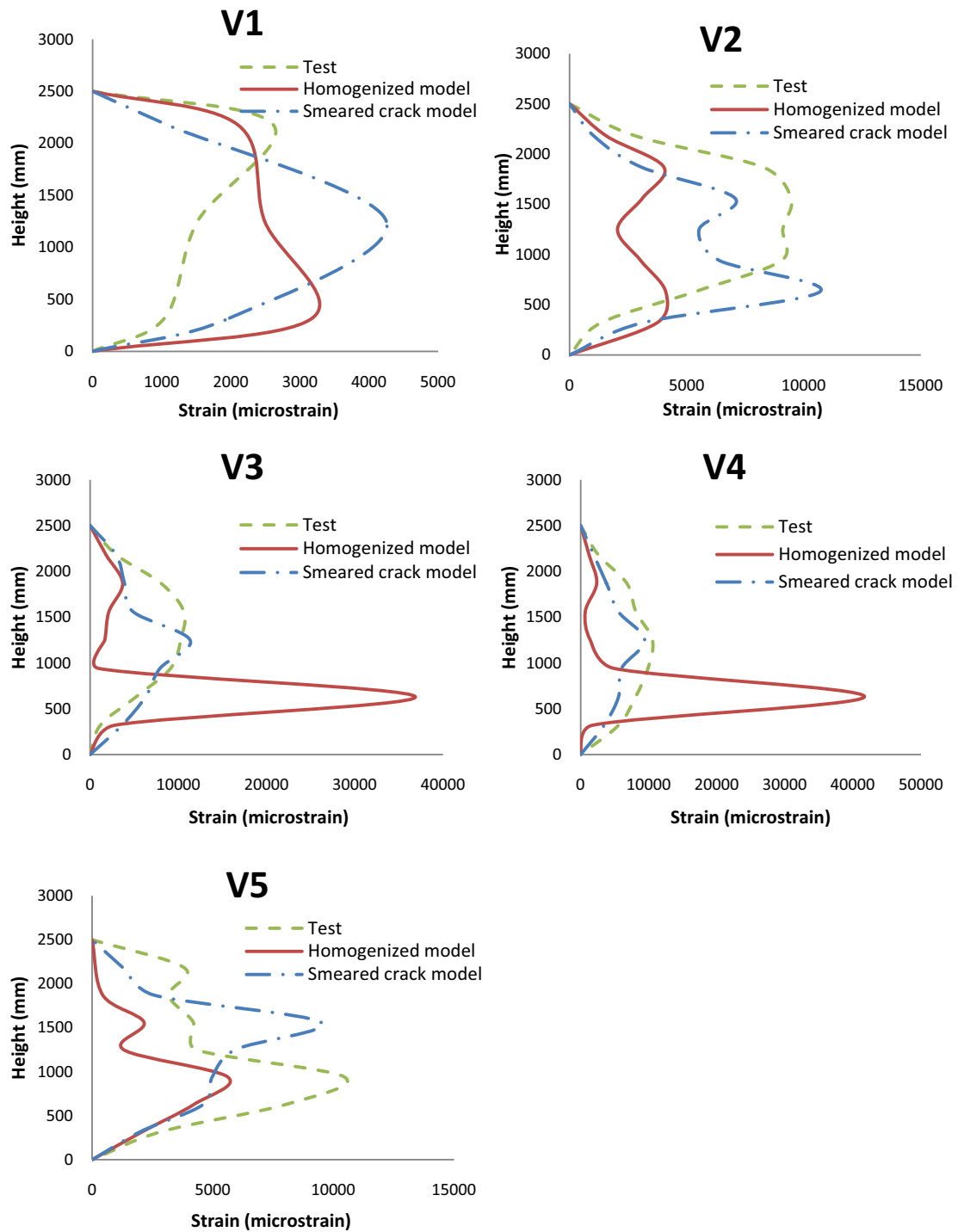


Figure 4.28 Comparison of maximum strain distribution along two EB GFRP plates

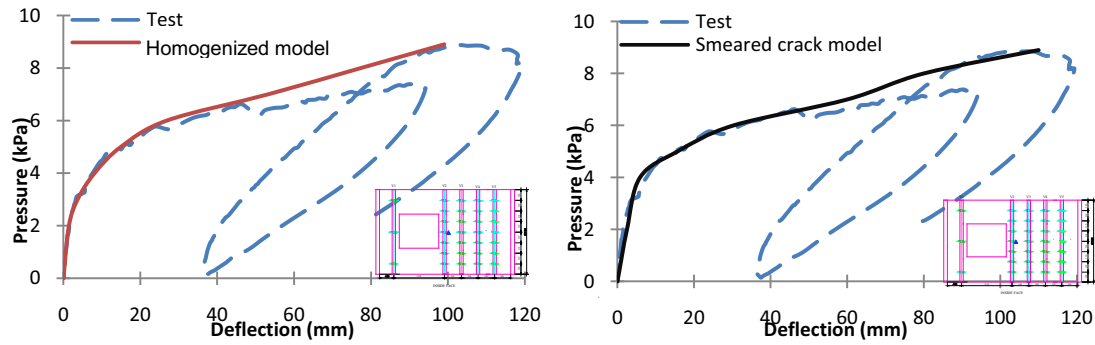


Figure 4.29 Simulation of EB repaired URM wall using the homogenized model and smear crack model

4.5. CONCLUSIONS

Pull tests have been simulated using a contact model and interface element model in the finite element program LS-DYNA. It was found that both the contact model and interface element model gave a reasonable prediction of local bond-slip relationships and global load-deflection curves for both NSM and EB FRP plates to masonry in pull tests. However, less time was required to obtain a solution using the contact model in comparison with interface element model. The contact surface model may not simulate debonding failure mechanism of the pull tests as well as the interface element model due to its zero thickness.

The homogenized model, smear crack model and distinct model have been used to analyse the response of FRP plated masonry prisms in pull tests. It was found that far less time was spent using the homogenized model in comparison with distinct model and smear crack model. The homogenized model and smear crack model together with the interface element model were used to simulate two seriously damaged URM walls retrofitted with NSM and EB plates under out-of-plane loads. The homogenized model has again demonstrated its computational efficiency to predict global response of the two FRP repaired URM walls.

5. MITIGATION OF BLAST EFFECTS ON RETROFITTED URM WALLS

5.1. INTRODUCTION

Unreinforced masonry (URM) construction is extremely vulnerable to terrorist bomb attacks since the powerful pressure wave at the airblast front strikes buildings unevenly and may even travel through passageways, resulting in flying debris that is responsible for most fatalities and injuries. One way to protect URM construction from airblast loads is to strengthen the masonry or to enhance its ductility. Categories of available masonry retrofit include conventional installation of exterior steel cladding or exterior concrete walls, externally bonded FRP plating, metallic foam cladding, spray-on polymer and/or a combination of these technologies (Davidson et al. 2005; Davidson et al. 2004b). However, limited research has been conducted to investigate retrofitting techniques to strengthen unreinforced masonry (URM) walls against airblast loading (Baylot et al. 2005; Carney and Myers 2005; Eamon et al. 2004; Myers et al. 2004; Ward 2004). Therefore, it is necessary to study the behaviours of retrofitted URM walls under airblast loading, and develop efficient retrofit solutions to enhance blast resistance of URM construction.

This chapter presents the results of numerical studies that were conducted to investigate the effectiveness of structural retrofit of URM walls by external bonded (EB) FRP plating, aluminium foam cladding, spray-on polymer and/or a combination of these technologies. A distinct model was used to model the performance of masonry, and the Drucker-Prager strength model verified in Chapter 3 was used to simulate the behaviour of mortar and bricks for masonry structures. An elastic-brittle material model was employed to model the FRP material. The interface element

model described and validated in Chapter 4 was used to model the “partial-interaction” behaviours between the URM wall and the various retrofit materials. The aluminium foam was modelled by a nonlinear elastoplastic material model which was validated by test data from the manufacturer (CYMAT 2003). The spray-on polyurea and steel skin for aluminium foam was simulated using elastoplastic model. The material model “MAT_MODIFIED_HONEYCOMB” in LS-DYNA (Whirley and Englemann 1991) program was used to simulate the performance of aluminium foam protected URM walls subjected to airblast loads. Parametric studies were carried out to investigate the respective efficiency of different retrofitting technologies. Pressure-impulse (P-I) diagrams were used to assess damage levels of the retrofitted URM walls under airblast loads.

5.2. MATERIAL MODELS IN THE SIMULATION

Distinct model for masonry derived in section §3.2.2, and FRP models introduced in section §4.2.2 were used to build models of retrofitted URM walls. With regard to debonding failure due to tension at the interface between the masonry and the bonded retrofit material, tensile failure was employed into the interface element model varied in Chapter 4. Thus, material models for spray-on polyurea, and aluminium foam were introduced in this section.

5.2.1. Material Model for Spray-on Polyurea

Spray-on polyurea is a type of low-stiffness polymer without any fiber reinforcement. Davidson et al. (Davidson et al. 2005; Willis et al. 2004) who tested spray-on polyurea retrofitted concrete masonry walls, reported that the polyurea provided a high level effectiveness of migration against blast by absorbing strain energy and

preventing fragmentation. Compared with stiffer material such as CFRP, it provides a cost-effective solution, and is easy to apply. The material model MAT_PLASTIC_KINEMATIC developed for plastic material in LS-DYNA was used to simulate the spray-on polyurea. It was modelled as an elastoplastic material with material properties obtained from Davidson's tests as summarized in Table 5.1. The failure strain for eroding elements was set as 89% (Davidson et al. 2005).

Table 5.1 Material properties of spray-on polyurea (Davidson et al. 2005)

NOTE:
This table is included on page 80 of the print copy of
the thesis held in the University of Adelaide Library.

5.2.2. Material Model for Aluminium Foam

Aluminium foams are new, lightweight materials with excellent plastic energy absorbing characteristics that can mitigate the effects of an explosive charge on a structural system by absorbing high blast energy. The typical behaviour of aluminium foam in uniaxial compression is illustrated in Figure 5.1 (CYMAT 2003). As shown, the material closely resembles to that of a perfect-plastic material in compression that makes aluminium foam attractive for use in sacrificial layers for blast protection. Airblast tests on aluminium foam protected RC structural members have been conducted recently what it was found that aluminium foam was very effective to absorb airblast energy (Schenker et al. 2008; Schenker et al. 2005). Due to these results, it was believed that aluminium foam would also be very effective for protection of URM construction against airblast loads although no tests have been performed. Since field airblast tests are very expensive and sometimes not even possible to conduct due to safety and environmental constraints, numerical

simulations with a validated numerical model was used here to provide an alternative method for investigating the effectiveness of aluminium foam to mitigate airblast loads on URM construction.

NOTE:
This figure is included on page 81 of the print copy of
the thesis held in the University of Adelaide Library.

Figure 5.1 Schematic stress- strain curve of aluminium foam (CYMAT 2003)

Aluminium foam sheets have a natural directionality, and the numbering convention of material directions is shown in Figure 5.2. As noted above, it has the ability to dissipate energy as a cellular solid due to very early onset of plastic yielding and large plastic deformation capability as shown in Figure 5.1. To model the real anisotropic behaviour of the aluminium foam, a nonlinear elastoplastic material model (LSTC 2007) was used separately for all normal and shear stresses. For the uncompacted material, the trial stress components in the local coordinate system are updated according to

$$\sigma_{ij}^{n+1^{trial}} = \sigma_{ij}^n + E_{ij} \Delta \varepsilon_{ij} \quad \text{Eq. 5-1}$$

where E_{ij} is elastic moduli varying from their initial values to the fully compacted values at V_f , linearly with the relative volume V (defined as the ratio of the current volume to the initial volume):

$$E_{ij} = E_{ij}^u + \beta(E - E_{ij}^u) \quad \text{Eq. 5-2}$$

in which E_{ij}^u is elastic/shear modulus in uncompressed configuration,

$$\beta = \max \left[\min \left(\frac{1-V}{1-V_f}, 1 \right), 0 \right] \quad \text{Eq. 5-3}$$

Each component of the updated stresses is independently checked to ensure that it does not exceed the permissible value determined by the load curve; if

$$\left| \sigma_{ij}^{n+1trial} \right| > \lambda \sigma_{ij}(V) \quad \text{Eq. 5-4}$$

then

$$\sigma_{ij}^{n+1} = \sigma_{ij}(V) \frac{\lambda \sigma_{ij}^{ntrial}}{\left| \sigma_{ij}^{ntrial} \right|} \quad \text{Eq. 5-5}$$

where $\sigma_{ij}(V)$ is defined as the stress component by the stress versus volumetric strain curves; and λ is defined as a function of strain rate as the Euclidean norm of the deviatoric strain-rate tensor. For fully compacted material, it was assumed that the material behaviour is elastic-perfectly plastic and the stress components updated based on

$$s_{ij}^{n+1trial} = s_{ij}^n + 2G\Delta\varepsilon_{ij}^{devn+1/2} \quad \text{Eq. 5-6}$$

where the deviatoric strain increment is defined as

$$\varepsilon_{ij}^{dev} = \Delta\varepsilon_{ij} - \Delta\varepsilon_{kk}\delta_{ij}/3 \quad \text{Eq. 5-7}$$

If the effective trial stress exceeds the yield stress for the fully compacted material,

$$s_{eff}^{trial} = \left(3s_{ij}^{trial}s_{ij}^{trial}/2 \right)^{1/2} > \sigma_Y \quad \text{Eq. 5-8}$$

the stress components are simply scaled back to the yield surface

$$s_{ij}^{n+1} = \frac{\sigma_y}{s_{eff}^{trial}} s_{ij}^{trial} \quad \text{Eq. 5-9}$$

Then the hydro pressure is updated as

$$p^{n+1} = p^n - K \Delta \varepsilon_{kk}^{n+1/2} \quad \text{Eq. 5-10}$$

$$K = \frac{E}{3(1-2\nu)} \quad \text{Eq. 5-11}$$

from which, the Cauchy stress is now obtained by

$$\sigma_{ij}^{n+1} = s_{ij}^n - p^{N=1} \delta_{ij} \quad \text{Eq. 5-12}$$

After completing the stress update, they are transformed back to the global coordinate system.

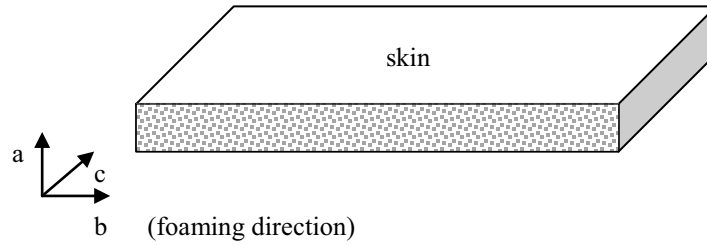


Figure 5.2 Numbering convention of foam material directions

Compressive tests on an aluminium foam sheet (A356SiC040) with length of 400 mm, width of 400 mm and thickness of 40 mm were simulated with the nonlinear elastoplastic material model using LS-DYNA program. Material properties of such aluminium foam are listed in Table 5.2. Figure 5.3 shows a comparison of the simulated stress-strain curve with test data in direction a. It was found that the simulated results agreed well with the experimental data in the manufacturer manual (CYMAT 2003), indicating that performance of the aluminium foam sheet can be effectively simulated with the nonlinear elastoplastic material model. Orthotropic

properties were inputted to get reliable material behaviours according to the manufacturer manual (CYMAT 2003). The simulated stress-strain relationship is plotted in Figure 5.4.

Table 5.2 Material properties of A356SiC040 aluminium foam

Density (kg/m^3)	400	Elastic modulus in a direction (GPa)	0.5
Young's modulus of al (GPa)	71.0	Elastic modulus in b direction (GPa)	0.7
Poisson's ratio	0.33	Elastic modulus in c direction (GPa)	0.85
Yield stress of al (GPa)	0.322	Shear modulus (GPa)	0.92
Compressive strength (MPa)	4.0	Densification Strain (%)	68

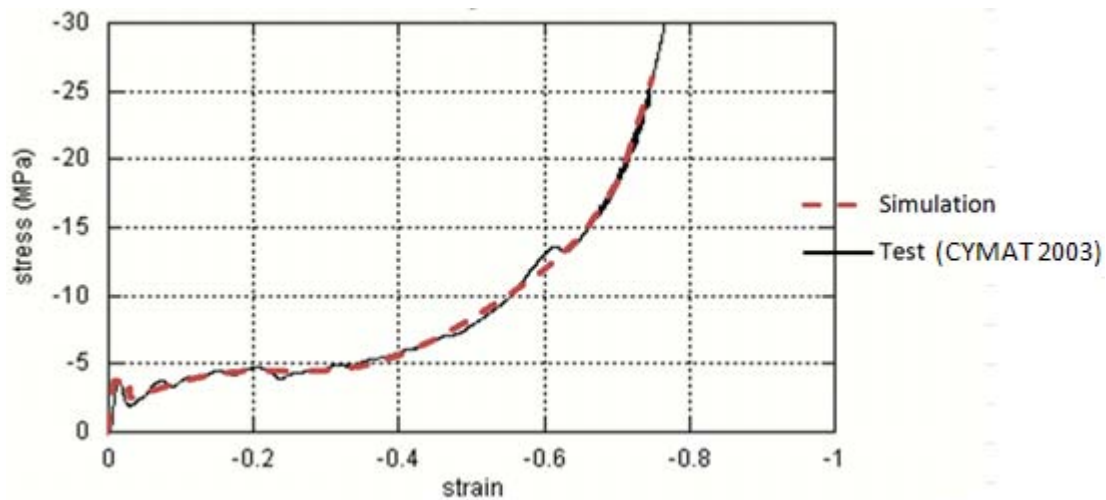


Figure 5.3 Comparison of compressive stress-strain curves between simulation and test in direction a

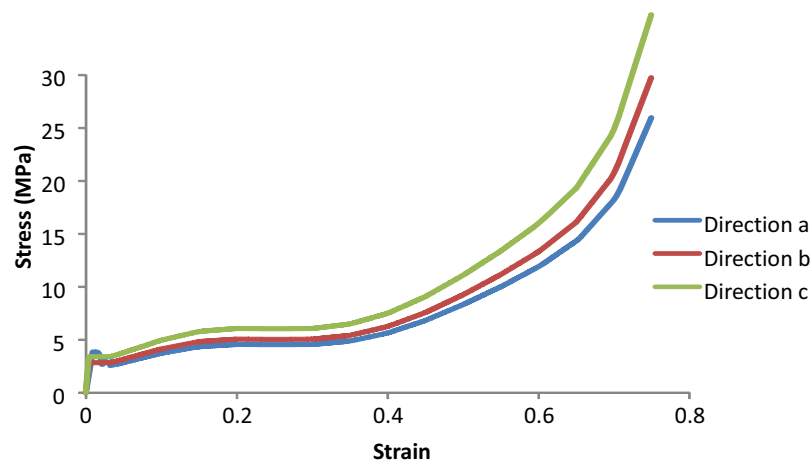


Figure 5.4 Characteristics of compressive stress-strain curves of aluminium foam

in orthotropic directions

5.3. SIMULATION OF RETROFITTED URM WALLS AGAINST BLAST LOADING

The material models for masonry, FRP, polyurea and aluminium foam as well as interface element model were coded into the finite element program LS-DYNA to numerically calculate the response and damage of 2.5m long (b) $\times$ 2.5m high (h) $\times$ 110mm thick URM wall with and without retrofitting under airblast loads.

5.3.1. Blast Loading

The blast environment was set as surface burst blast shown in Figure 5.5. By changing the charge weight (W) and stand-off distance (R) between masonry walls and charge centre, the correlation between the scaled distances ($Z=R/W^{1/3}$) and impulse was identified. The charge weight discussed in section §5.3 is 1000kg TNT. Specimens were located at different stand-off distances to vary the scaled distance. When dimensions of the masonry wall are small compared to the stand-off distance R, the blast loads can be considered as uniform pressure. As shown in Figure 5.6, usually, if the angle θ is less than 5° , the blast wave can be considered to apply a uniform pressure to the wall. In this study, the value of angle θ ranged between 0.6° and 4.2° so that uniform pressure was used and applied on the front surface of URM walls. The blast pressure time history was charted from the U.S DoD code – TM-5-1300 (Department of Defence (DoD) 1990).

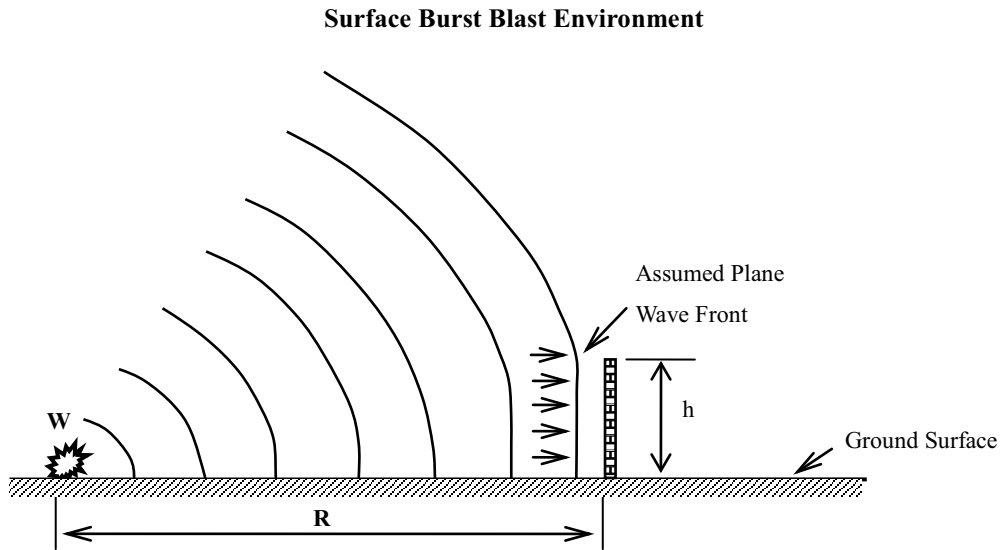


Figure 5.5 Surface burst blast

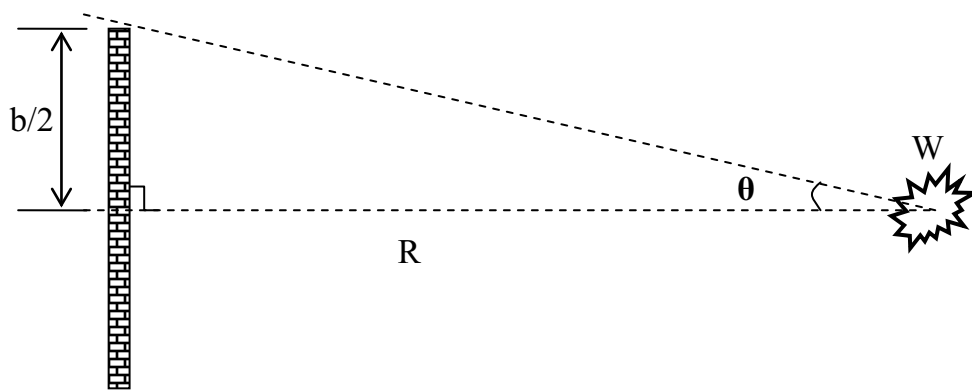


Figure 5.6 Surface burst blast – vertical section

The blast loads were estimated using the idealized pressure-time variation shown in Figure 5.7. The blast wave is characterized by an initial positive phase which consisting of an abrupt rise from ambient (P_o) to peak pressure (P_{so}) followed a decreasing period back to ambient pressure, and the a negative phase in which the pressure drops below the ambient pressure (Department of Defence (DoD) 1990). The pressure-time relationship was approximated in this study by an equivalent triangle curve (idealized positive phase) that is indicated in Figure 5.7.

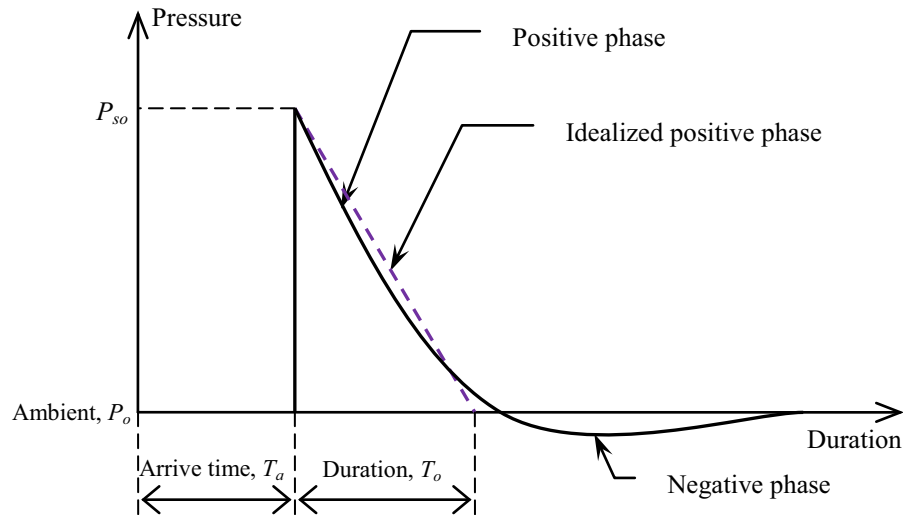


Figure 5.7 Idealized pressure-time variation

The time history was calculated by from empirical methods (Henrych 1979; Wu and Hao 2005) and code solution (Department of Defence (DoD) 1990) which were discussed in Chapter 2.4.1. The peak pressure can be seen in Figure 5.8 for the three methods. Compared with the peak pressure derived from the chart solution, the values calculated by using the empirical theory are much smaller than the DoD value at scaled-distance one. However, the values for all three methods match well at scaled-distances between two and six. Considering that the data for the chart solution using TM-5-1300 were based on tests, which should be more reliable, they were selected for use in the following sections.

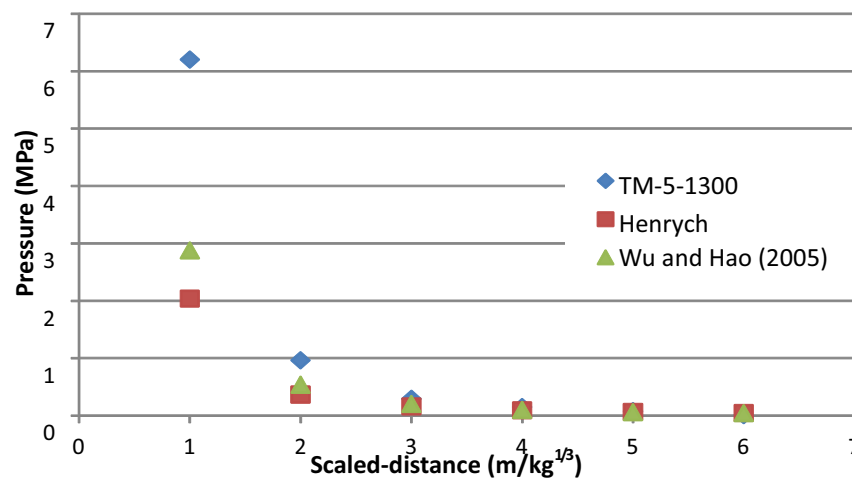


Figure 5.8 Comparison of peak pressure P_{so}

5.3.2. URM Walls

Parametric studies were carried out to estimate the response of the URM walls against airblast loads with a scaled distance increment of $0.01 \text{ m/kg}^{1/3}$. It was found that the critical scaled distance to prevent the URM wall from collapse is $9.0 \text{ m/kg}^{1/3}$. For URM walls under smaller blast loading (i.e. $Z \geq 9 \text{ m/kg}^{1/3}$), damage was due to a combination of growing shear cracks and tensile cracks in mortar joints, demonstrating like step-like cracks as shown in Figure 5.9a. However, URM walls were observed to collapse immediately as shown in Figure 5.9b when subjected to larger blast loading (e.g. $Z = 4 \text{ m/kg}^{1/3}$), and shear failure was found near supports. The performance of non-retrofitted URM walls under blast loads was used as a “control” case for comparison purposes.

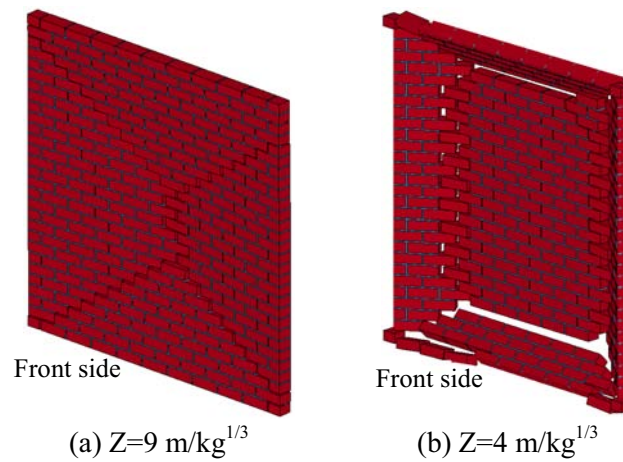


Figure 5.9 Performance of URM wall under different blast loads

5.3.3. NSM CFRP Retrofitted URM Walls

The NSM CFRP technique for the retrofitted URM walls against blast loading was considered first. CFRP plates were applied vertically or horizontally (Figure 5.10) on

the URM wall which was simply supported at its four edges. Blast loading at different scaled distances was applied on the front surface of the wall. Simulation results are shown in Figure 5.11. It was found that maximum blast loads for the vertical or horizontal NSM CFRP retrofitted walls to resist are at scaled distances of $9 \text{ m/kg}^{1/3}$.

The failure models were similar as that of the URM wall. Under light impulse, the tensile and shear failure models were observed in mortar. Step-like cracks were seen and due to the FRP strips, more cracks were found in the central part of the rear side of the wall due to the tensile failure of the mortar. For the horizontal NSM CFRP retrofitted wall, mortar closed to the CFRP strips was damaged due to tensile failure, and horizontal cracks in the mortar were observed near the CFRP strips that reduced the integrity by separating the wall into several pieces. Debonding failure happened near the edges of the vertical NSM CFRP retrofitted wall, and the wall lost the enhancement from NSM CFRP strips in early stage. Compared with the behaviour of URM wall under same blast loading, the vertical or horizontal NSM CFRP retrofits do not increase the load capacity. Therefore, the NSM CFRP retrofitted technique is not considered as a suitable method to retrofit URM walls against blast loading, even if the wall is subjected to light impulse.

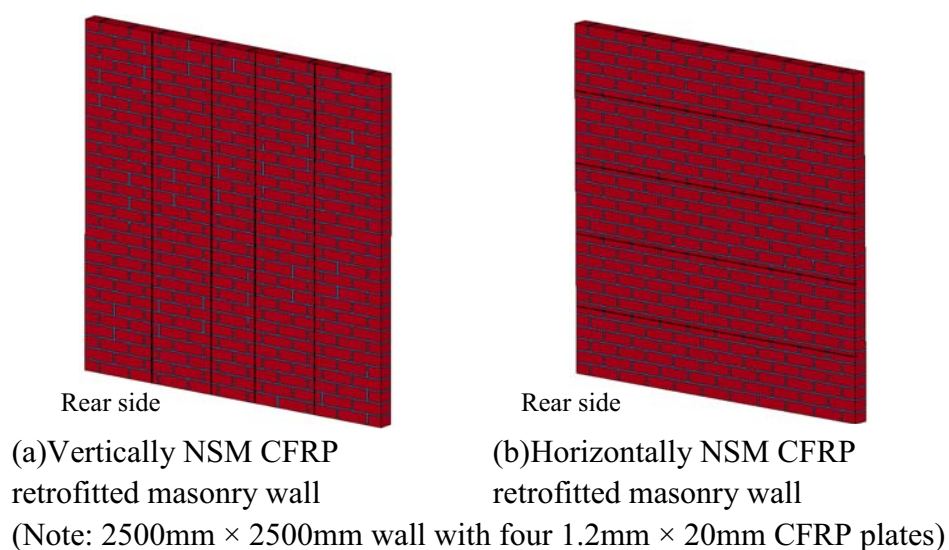


Figure 5.10 NSM CFRP retrofitted URM walls

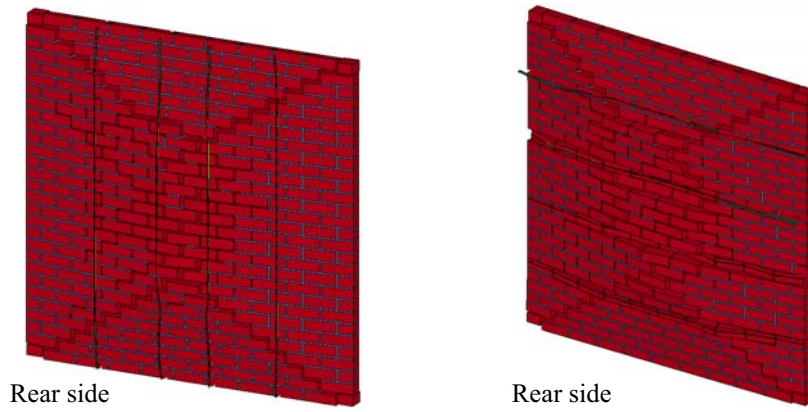


Figure 5.11 Debonding failure of NSM CFRP retrofitted URM walls

5.3.4. EB CFRP or GFRP Retrofitted URM Walls

The EB FRP retrofitting technique was selected next. Figure 5.12a shows four 100mm×2mm GFRP plates applied on the rear surface of URM wall. Numerical simulation results are illustrated in Figure 5.13. As shown, when scaled-distance $Z \geq 5.0 \text{ m/kg}^{1/3}$, step-like cracks were distributed on the most portions of rear surface of the wall, and the debonding of FRP plates was found around the cracks. The GFRP plates still carried loads, and the retrofitted URM wall was kept under light damage level, on which little debonding was observed (Figure 5.13). Some local failure of masonry was seen in the centre of the wall with the debonding failure level at $Z = 5.0 \text{ m/kg}^{1/3}$, and wall failure level was observed at $Z = 4.7 \text{ m/kg}^{1/3}$. Local failure of the masonry was found at the portion of wall without being covered by GFRP plates. It was observed that once the debonding area exceeds 10% of the whole bonded area, the retrofitted walls begin to lose the protection from the FRP retrofits. Thus, the relevant scaled-distance and impulse were defined as critical values of the debonding failure level. The debonding patterns are shown Figure 5.13. The combined effect of horizontal plus vertical GFRP plates was then investigated by applying four vertical and four horizontal GFRP plates with dimension of 100mm×2mm on the rear surface of the URM wall as shown in Figure 5.12b. The scaled-distance of wall failure level is at $4.3 \text{ m/kg}^{1/3}$ (see Figure 5.14), therefore, the additional GFRP plates on the rear

surface slightly enhance the resistance capability. The URM wall with EB GFRP retrofitted on the entire rear surface (Figure 5.12c) was then investigated. Simulation results are summarized in Figure 5.15, indicating the EB GFRP plates installed on the entire surfaces can significantly enhance the blast resistance of the URM wall.

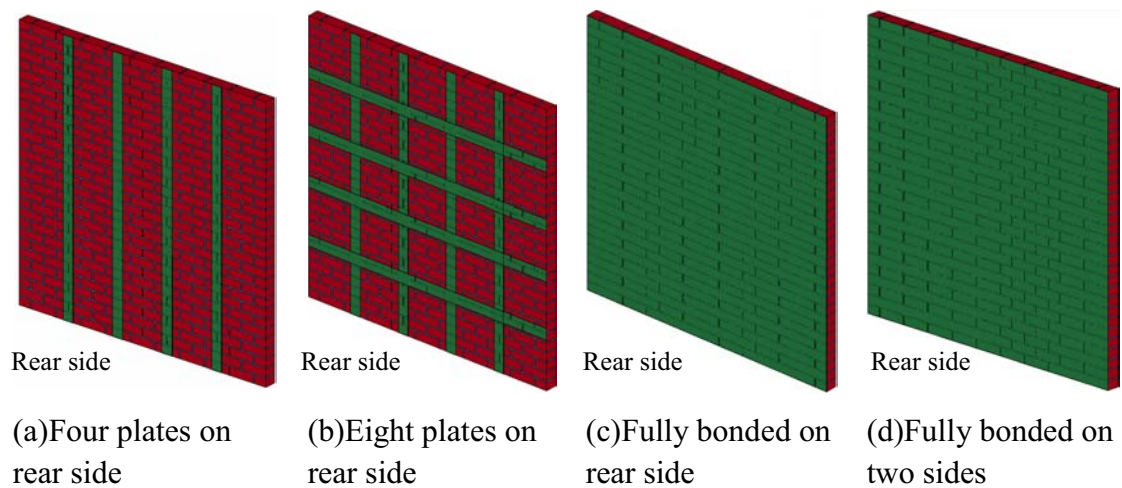
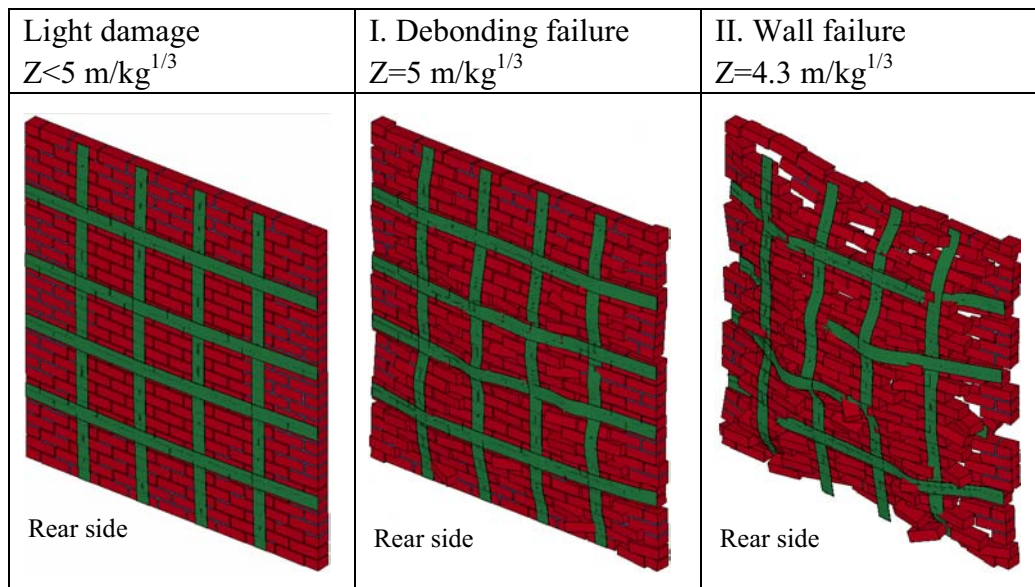
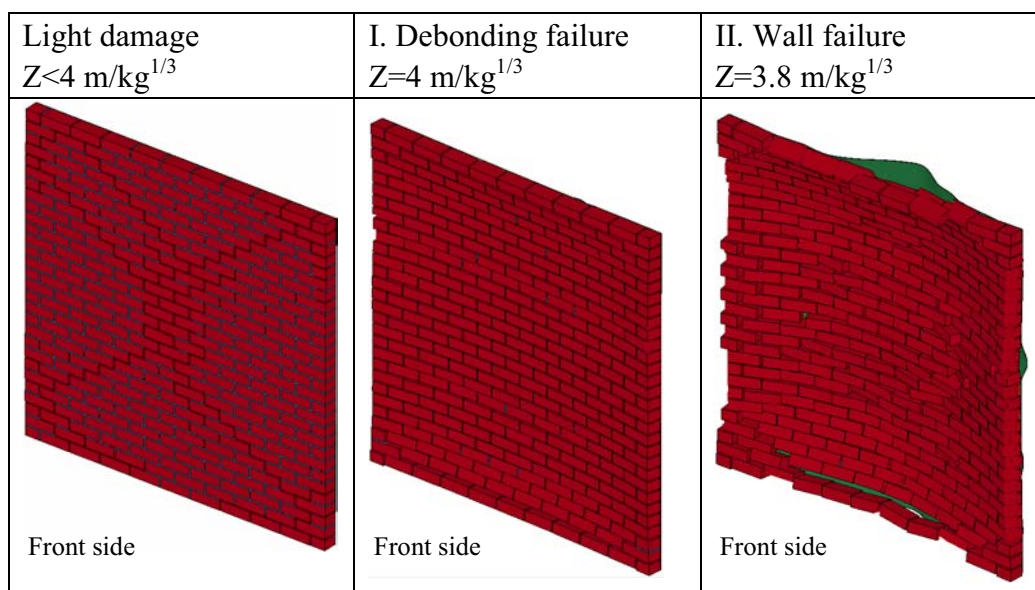


Figure 5.12 EB GFRP retrofitted URM wall

	Light damage $Z < 5 \text{ m/kg}^{1/3}$	I. Debonding failure $Z = 5 \text{ m/kg}^{1/3}$	II. Wall failure $Z = 4.7 \text{ m/kg}^{1/3}$
Crack patterns	 Rear side	 Rear side	 Rear side
Debonding patterns	 Rear side	 Rear side	 Rear side

Figure 5.13 Damaged EB GFRP retrofitted URM wall (4 plates on rear side)**Figure 5.14 Damaged EB GFRP retrofitted URM wall (8 plates on rear side)****Figure 5.15 URM wall fully retrofitted with EB GFRP sheet on rear surface**

The URM walls with EB GFRP retrofitted on the rear surface or on both rear and front surfaces were also investigated by applying GFRP plates on the front surface of the URM wall to prevent the masonry from continuous being damaged during rebounding process. Firstly, four 100mm×2mm GFRP plates were applied on both sides of the wall. The effectiveness of the protection was defined using the

comparison of the maximum impulse capability of the retrofitted walls. Simulation results were compared with other configuration (Figure 5.17) indicating that the additional retrofits on front surface provided little enhancement in comparison with rear surface retrofits only. The URM walls with EB GFRP retrofitted on the entire both rear and front surfaces (Figure 5.12d) were then investigated, and simulation results were summarized in Figure 5.16. As shown, additional GFRP sheets retrofitted on the entire front side improved the enhancement of the URM wall further.

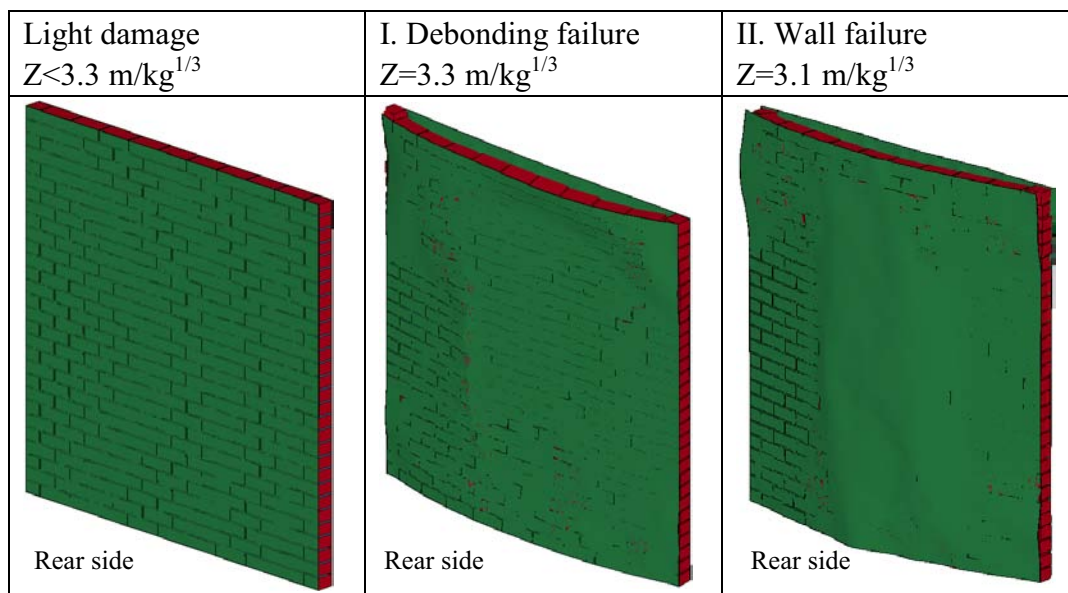


Figure 5.16 URM wall fully retrofitted with EB GFRP sheets on both surfaces

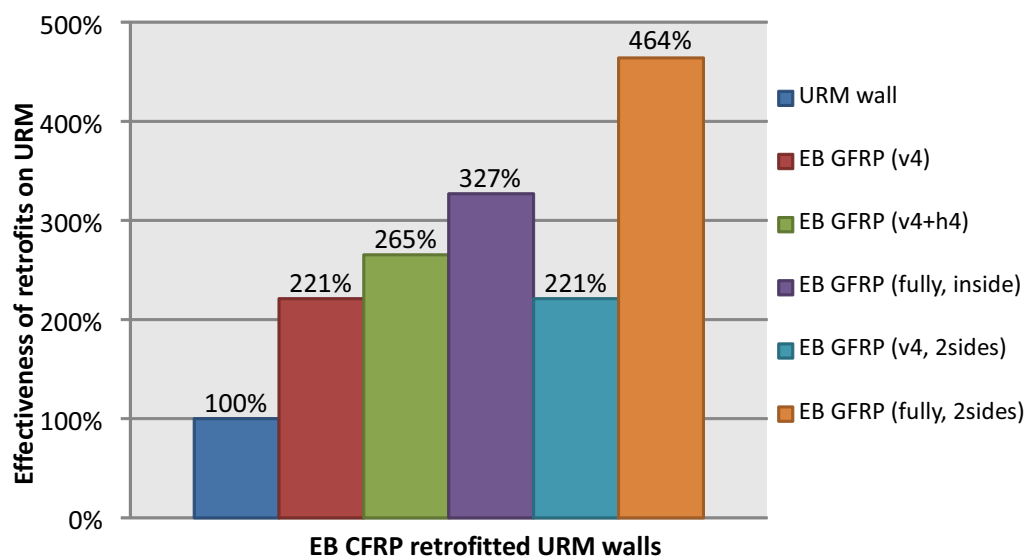


Figure 5.17 Comparison of EB GFRP retrofitted URM walls

A comparison of effectiveness of EB GFRP retrofitted URM walls against blast loading is shown in Figure 5.17. It is observed that GFRP applied on both surfaces provides the best protection by increasing the capability of blast-resistance to 464% compared with unretrofitted URM wall. However, it may not be cost-effective due to increase of cost for the additional layer of FRP sheets.

CFRP retrofitting on URM wall was also investigated. Figure 5.18 shows the URM wall retrofitted by four CFRP plates with dimension of 50mm×1.2mm on the rear surface subjected to blast loading. The simulation results shows that debonding occurred at a scaled distance of $9 \text{ m/kg}^{1/3}$ and wall failure occurs at the scaled distance of $6 \text{ m/kg}^{1/3}$. Thus, the CFRP retrofitting does not increase substantially the blast resistance capability of URM wall.

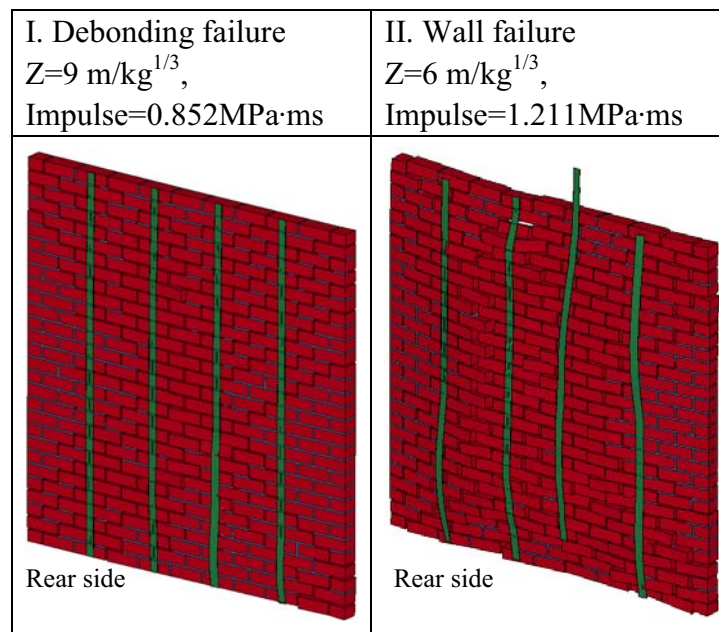
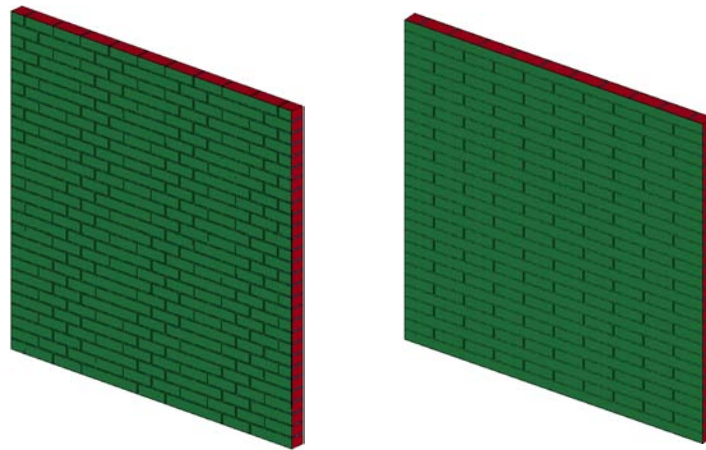


Figure 5.18 EB CFRP retrofitted URM walls (4 plates)

For the walls with CFRP plates bonded on the entire rear surface (Figure 5.19a), wall failure occurred at a scaled distance of $3.5 \text{ m/kg}^{1/3}$ (see Figure 5.20), indicating that entire surface CFRP retrofitting is similarly effective compared with the four vertical

EB CFRP plate retrofitted wall. However, when a layer of CFRP was added to the entire front surface (Figure 5.19b), the wall failed at a scaled distance of $3.3 \text{ m/kg}^{1/3}$, and debonded at scaled distance of $3.7 \text{ m/kg}^{1/3}$, as shown in Figure 5.21. Protection effectiveness of the various EB CFRP retrofits was compared in Figure 5.22, which shows that the effectiveness of blast resistance increases with more CFRP plates. The CFRP installed on both entire sides of the walls provides the best protection to the wall, however, compared with the wall retrofitted only on the entire rear side, the effectiveness was not improved double. Therefore, CFRP retrofitted on front side is not a cost-effective protection.



(a) Fully applied on rear side (b) Fully applied on two sides

Figure 5.19 EB CFRP retrofitted URM walls on entire surface

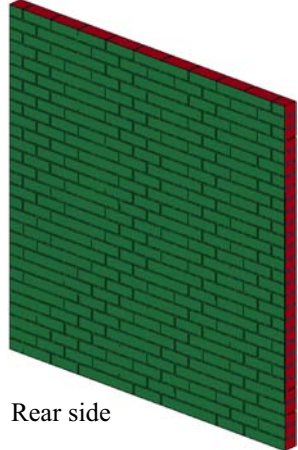
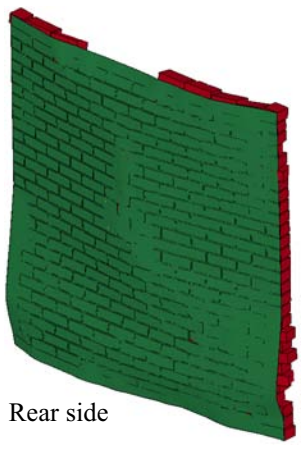
Light damage $Z < 4 \text{ m/kg}^{1/3}$	I. Debonding failure $Z = 4 \text{ m/kg}^{1/3}$	II. Wall failure $Z = 3.5 \text{ m/kg}^{1/3}$
 Rear side	 Rear side	 Rear side

Figure 5.20 Fully EB CFRP retrofitted URM walls on back surface

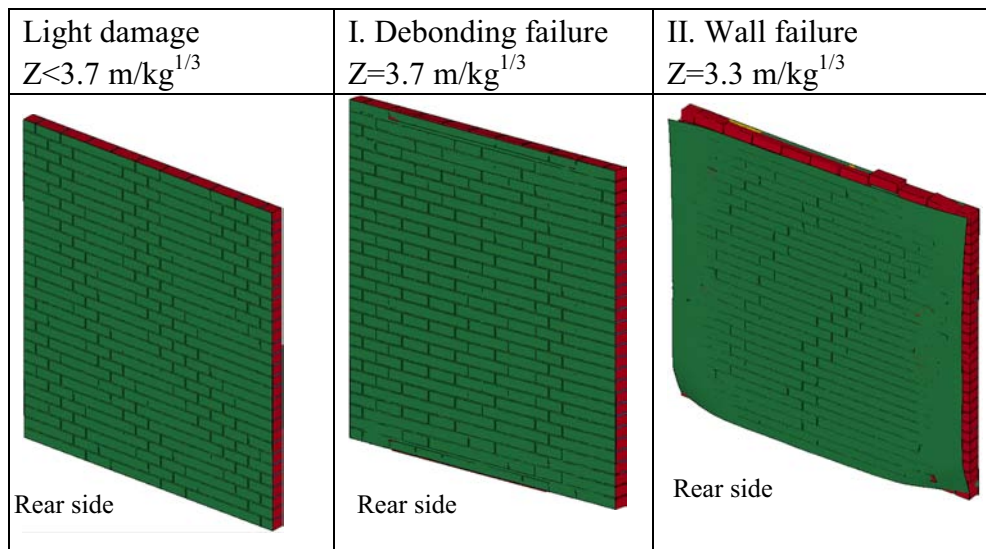


Figure 5.21 Two sides fully EB CFRP retrofitted URM walls

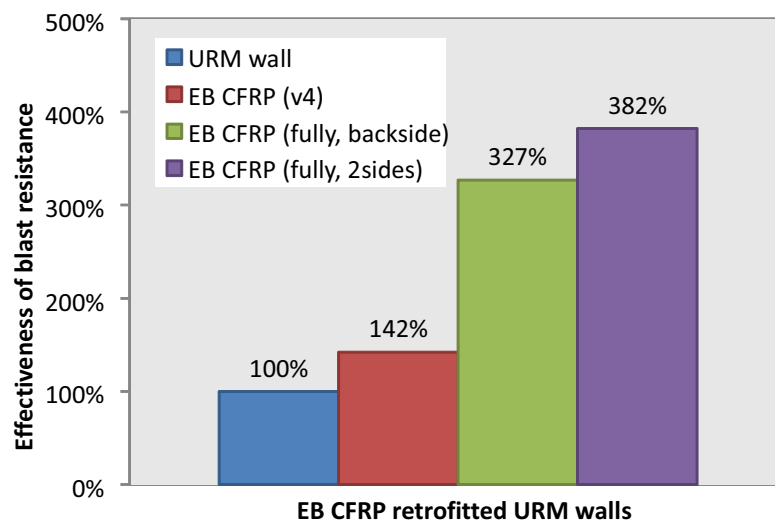


Figure 5.22 Comparison of EB CFRP retrofitted URM walls

A comparison of energy absorption for all of the carbon and glass EB FRP retrofits is shown in Figure 5.23. As shown, the EB GFRP retrofits on both surfaces can increase the unretrofitted URM wall blast-resistance by up to 464%. In general, the trend in results suggest, that full-surface treatments rather than strips will be more efficient. However, due to the expensiveness of the material and other practical issue of application, FRP full-surface retrofitting is a cost effective method.

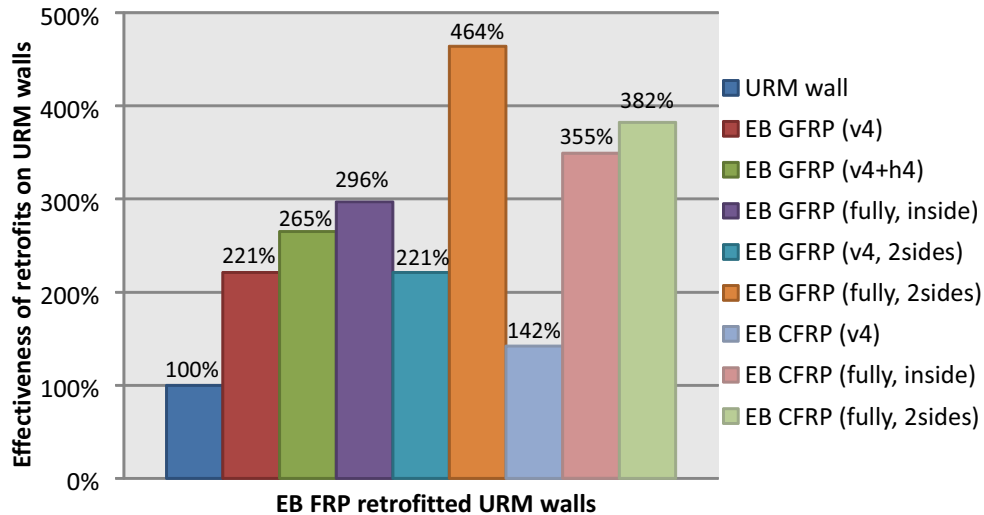


Figure 5.23 Comparison of EB FRP retrofitted URM walls

5.3.5. Spray-on Polyurea Retrofitted URM Walls

A parametric study was carried out to investigate the effectiveness of spray-on polyurea as obviators. The spray-on polyurea retrofitted URM wall was used to study the relationship between the thickness of spray-on polyurea and deflection of the wall at scaled-distance $3 \text{ m/kg}^{1/3}$ and $4 \text{ m/kg}^{1/3}$. The polyurea was applied on both surfaces of the wall and the results are plotted in Figure 5.24. It was found that the thickness influences the effectiveness of the retrofit, with thicker spray-on polyurea giving better protection.

The blast mitigation effectiveness of a layer of 15mm spray-on polyurea was applied to the rear surface of the URM wall is shown in Figure 5.25. In the simulation, the debonding failure was identified by the eroded bricks on the rear surface of the masonry wall. Once the debonding area of eroded surface exceeds about 10% of the entire bonding surface, the mitigation effect begins to decrease seriously. Figure 5.25 shows two failure modes for the retrofits observed in the simulations. Under great pressure, the polyurea would be mutilated closed to supports. Shown in Figure 5.26, local failure and debonding failure were observed. Debonding failure started from the

centre of rear surface and four corners of the masonry wall. Some shear failure of masonry was also observed around the centre and corners of the walls.

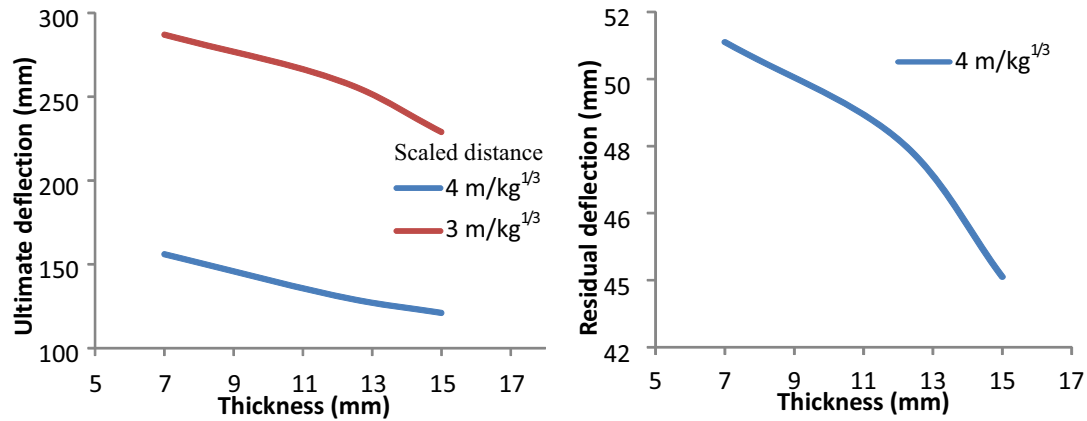


Figure 5.24 Thickness of spray-on polyurea on blast mitigation effects

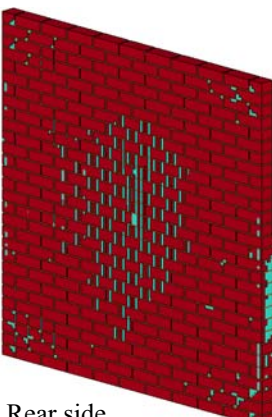
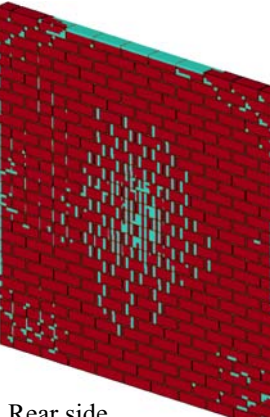
	I. Debonding failure $Z=4 \text{ m/kg}^{1/3}$, Impulse=2.468MPa·ms	II. Wall failure $Z=3.7 \text{ m/kg}^{1/3}$, Impulse=2.786MPa·ms
Defomation	 Rear side	 Rear side
Interface debonding patterns	 Rear side	 Rear side

Figure 5.25 Spray-on polyurea retrofitted URM walls under blast loads

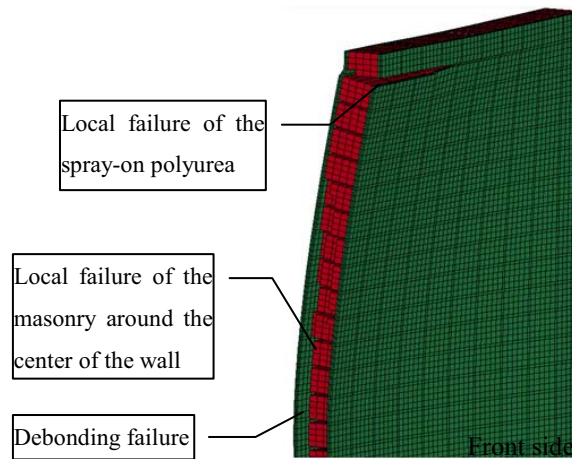


Figure 5.26 Local failure of the spray-on polyurea and masonry (vertical section)

The results for polyurea sprayed on the both surfaces is shown in Figure 5.27. It was observed from the simulation results, that the polyurea on the front surface can enhance the wall by absorbing more strain energy. It was found that the key factor influencing the effectiveness of the retrofits is energy absorbing capability. A comparison of the effectiveness of spray-on polyurea is shown in Figure 5.28. The wall retrofitted by a layer of 15mm spray-on polyurea on its rear surface absorbed three times more impulse energy than the unretrofitted URM wall. The increase of impulse ratio was 859% for the wall retrofitted by spray-on polyurea on both surfaces, indicating that by increasing the ductility, the masonry wall can survive much higher blast impulses.

I. Debonding failure $Z=3.3 \text{ m/kg}^{1/3}$, Impulse=3.257MPa·ms	II. Wall failure $Z=2.3 \text{ m/kg}^{1/3}$, Impulse=7.322MPa·ms
 Rear side	 Rear side

Figure 5.27 Two sides 15mm spray-on polyurea retrofitted URM walls

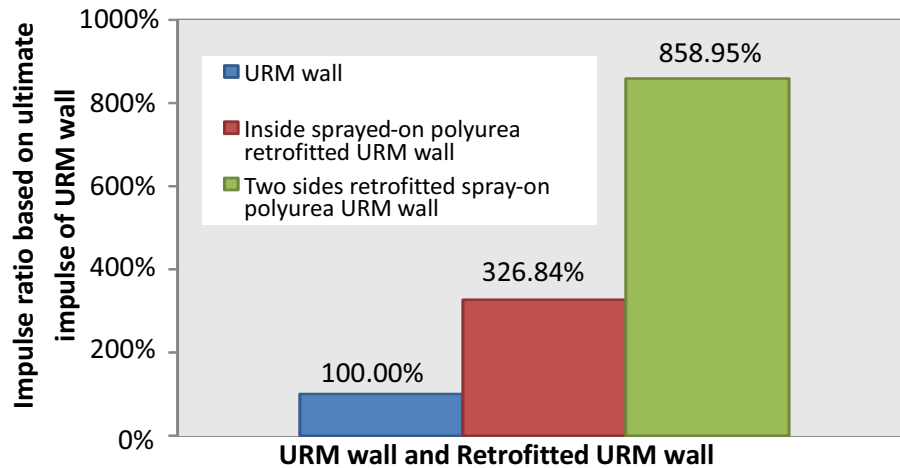


Figure 5.28 Comparison of energy absorption of the spray-on polyurea retrofitted walls

5.3.6. Aluminium Foam Protected URM Walls

Parametric studies were also conducted to study the response of URM walls retrofitted with a layer of aluminium foam sheet (thickness of 40 mm) covered by two 1.5mm steel sheets on the front surface (Figure 5.29). For a scaled distance of more than $4 \text{ m/kg}^{1/3}$ as shown in Figure 5.30a, the protected URM wall suffered only light damage. Once the scaled distance reached $3.3 \text{ m/kg}^{1/3}$, the aluminium foam sheet began to be damaged, and debonding between the steel sheets/masonry interface was found as shown in Figure 5.30b, which demonstrates that the aluminium foam sheet absorbs the airblast energy and mitigates blast effects on the URM wall, even though the URM wall is still kept under light damage condition. The aluminium foam protected URM wall collapsed as shown in Figure 5.30c as the scaled distance reaches $2.3 \text{ m/kg}^{1/3}$. Once the URM wall retrofitted with a layer of a layer of 40mm thick aluminium foam on the both surfaces in Figure 5.31a, debonding failure between the aluminium foam and steel sheets/URM wall did not occur until the scaled distance reached $2.3 \text{ m/kg}^{1/3}$ as shown in Figure 5.31b. URM wall failure only occurred when

the scaled distance reached $1.8 \text{ m/kg}^{1/3}$ as shown in Figure 5.31c. Figure 5.32 shows a comparison of the energy absorption for the aluminium foam protected URM walls. As shown, URM walls retrofitted with aluminium foam sheets on both surfaces can absorb 14 times more blast energy than the unstrengthened URM wall. Even compared with the walls retrofitted with either spray-on polyurea or EB GFRP, the aluminium foam increases the blast-resistance of URM wall about two and four times, respectively, as shown in Figure 5.33. The aluminium foam sheets are very promising for mitigation of blast effects on URM walls.

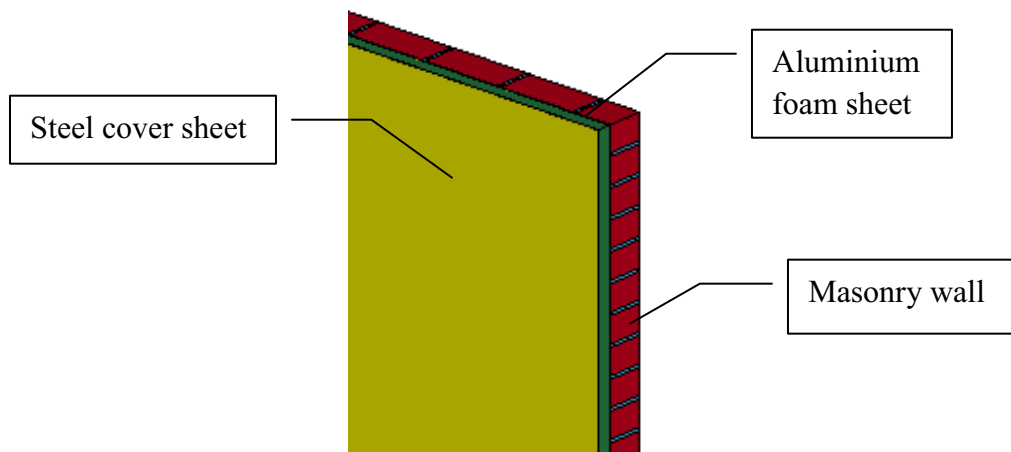


Figure 5.29 Numerical model of aluminium foam protected URM wall

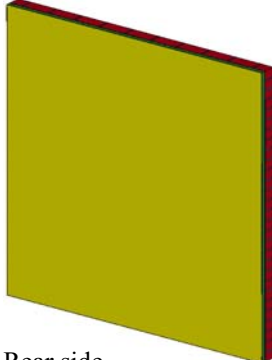
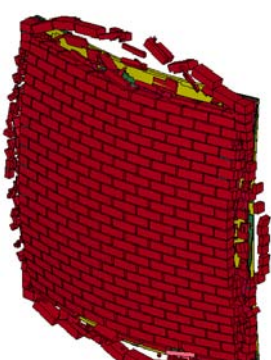
Scaled distance	(a) Undamaged $Z=4 \text{ m/kg}^{1/3}$	(b) Debonding damage $Z=3.3 \text{ m/kg}^{1/3}$	(c) Wall failure $Z=2.3 \text{ m/kg}^{1/3}$
Thickness 40 mm	 Rear side	 Rear side	 Rear side

Figure 5.30 Performance of aluminium foam protected URM walls under blast loads

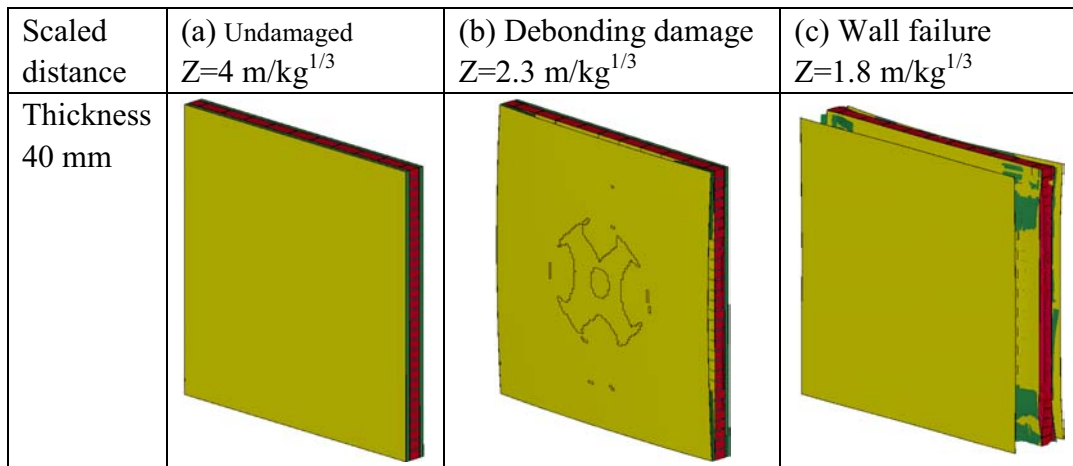


Figure 5.31 Performance of aluminium foam protected two surfaces of URM walls under blast loads

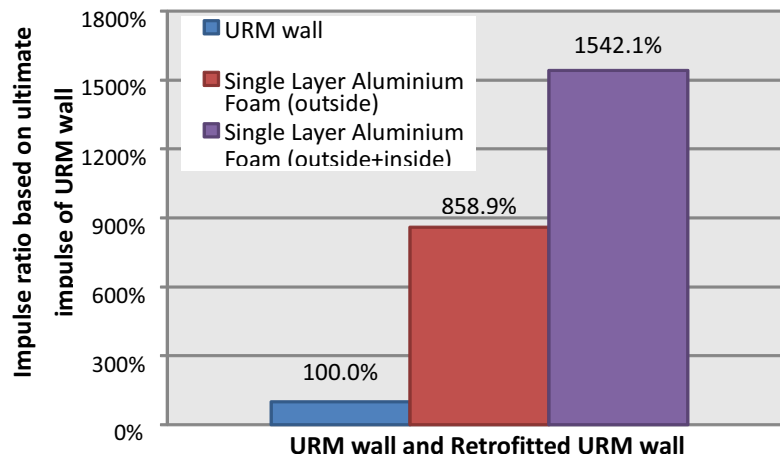


Figure 5.32 Comparison of energy absorption of aluminium foam protected URM walls

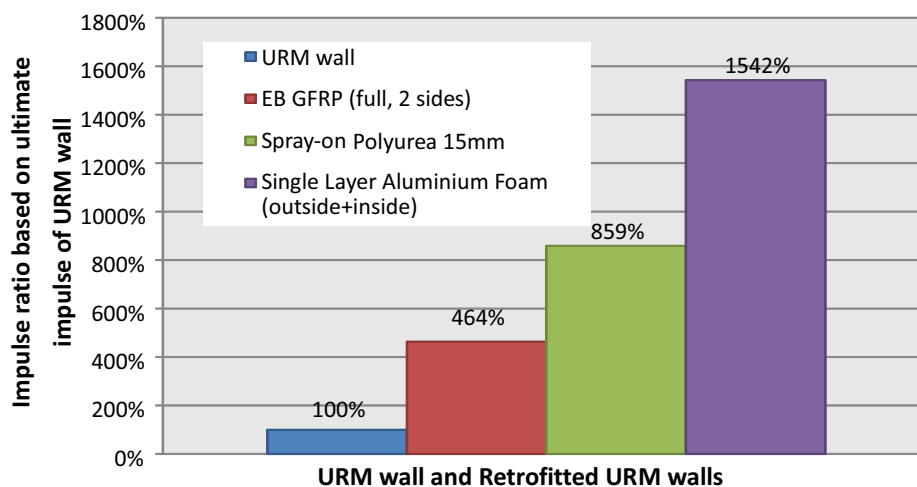


Figure 5.33 Comparison of retrofitted URM walls

Furthermore, different types of aluminium foam sheets can have great influence on its blast energy absorption capacity. Tables 5.3 and Table 5.4 list the material properties for A356SiC030 and A356SiC020 aluminium foams. Parametric studies were conducted to investigate how the material properties of aluminium foam sheets (keeping all the other material properties constant) affect the blast energy absorption capacity on URM walls. Figure 5.34 shows different densities of aluminium foam sheets on the mitigation of blast effects of URM walls. The corresponding response of the aluminium foam protected wall is compared in Figure 5.36. As shown, the higher the density, the smaller the response, that is, the more effective it mitigates blast effects on URM wall. Figure 5.35 shows how thickness of aluminium foam sheets influence mitigation of blast effects on the URM wall and corresponding response of the aluminium foam protected URM walls are compared in Figure 5.37, where it can be seen that the larger the thickness, the smaller the response. Figure 5.38 plots the energy absorption of the aluminium foam retrofitted front wall with different density and thickness. As before, the higher density and thicker foam layers absorb more energy.

Table 5.3 Properties of A356SiC030 aluminium foam

Density (kg/m^3)	300	Elastic modulus in a direction (GPa)	0.300
Young's modulus of al (GPa)	71.0	Elastic modulus in b direction (GPa)	0.460
Poisson's ratio	0.33	Elastic modulus in c direction (GPa)	0.575
Yield stress of al (GPa)	0.322	Shear modulus (GPa)	1.0
Compressive strength (MPa)	2.4	Densification Strain (%)	72

Table 5.4 Properties of A356SiC020 aluminium foam

Density (kg/m^3)	200	Elastic modulus in a direction (GPa)	0.185
Young's modulus of al (GPa)	71.0	Elastic modulus in b direction (GPa)	0.200
Poisson's ratio	0.33	Elastic modulus in c direction (GPa)	0.270
Yield stress of al (GPa)	0.322	Shear modulus (GPa)	0.2
Compressive strength (MPa)	1.2	Densification Strain (%)	80

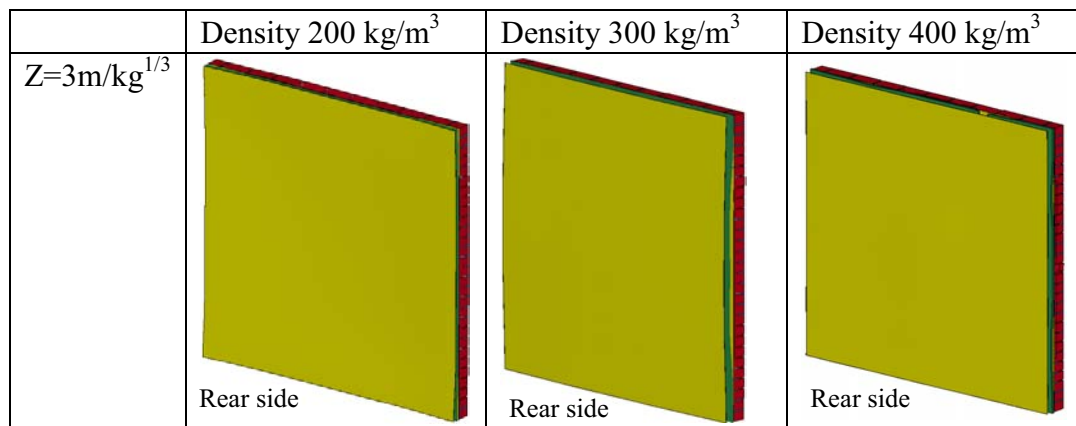


Figure 5.34 Different densities of aluminium foam on mitigation of blast effects on URM wall

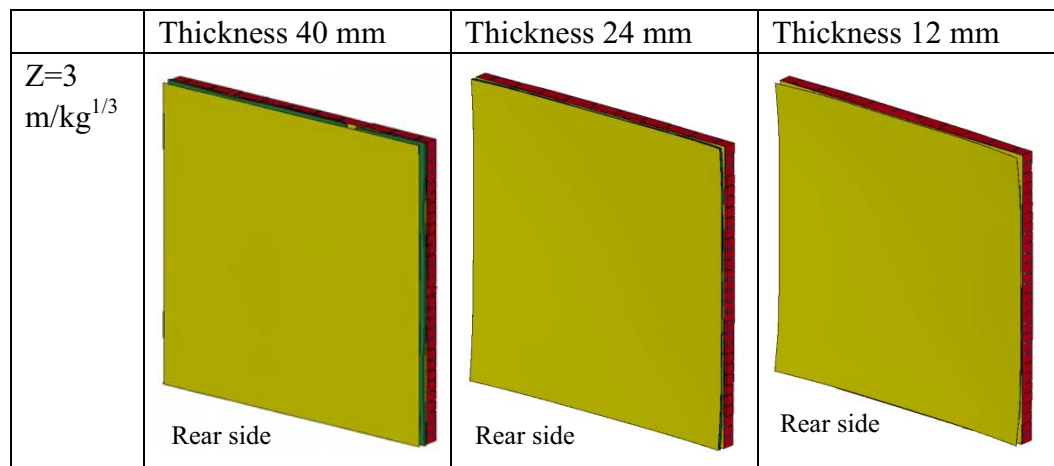


Figure 5.35 Different layer thickness of aluminium foam on mitigation of blast effects on URM wall

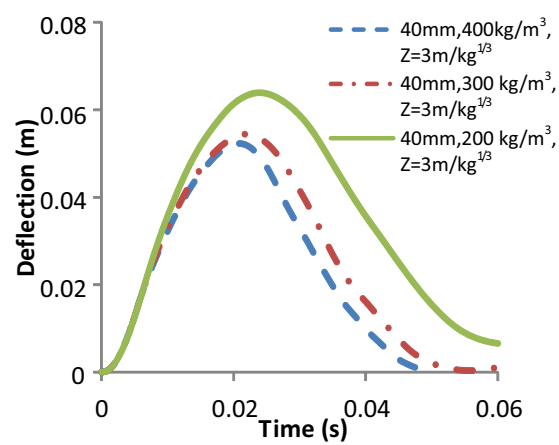


Figure 5.36 Aluminium foam with different densities

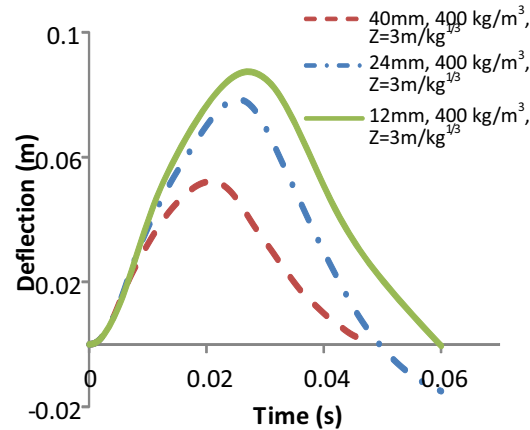


Figure 5.37 Aluminium foam sheet with different thickness

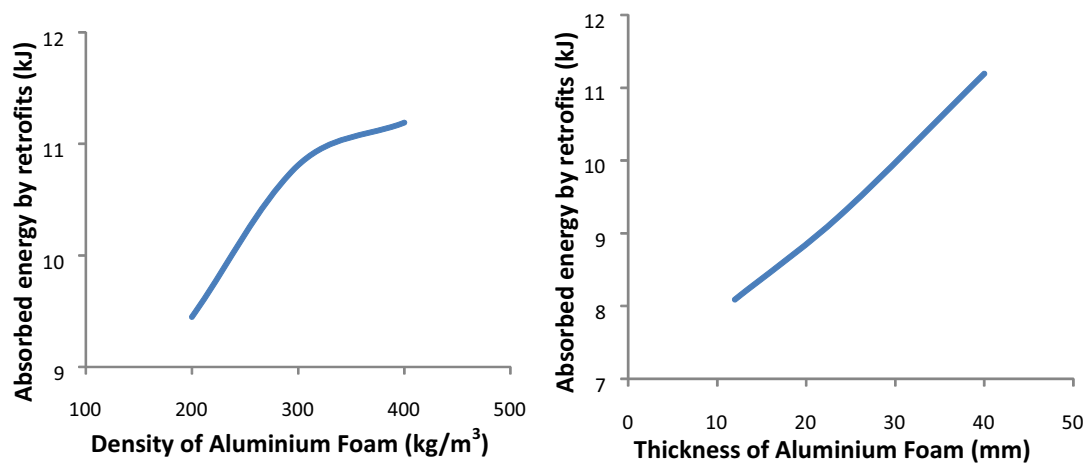


Figure 5.38 Masonry wall protected by aluminium foam with different densities or with different thickness

5.3.7. Combination of Aluminium Foam with Other Retrofits

Finally, retrofits using combinations of aluminium foam with other materials such as 15mm spray-on polyurea, 5mm steel plates or 1.2 mm CFRP plates were applied on URM walls to check the mitigation of blast effect. These results are presented in Figure 5.39, Figure 5.40 and Figure 5.41, respectively. As shown, the spray-on polyurea and CFRP plates increased the flexibility of the masonry wall, but they did not work well under high impulse. That was because the masonry wall is just weak regardless of the impulse, and the aluminium foam can help with reducing the impulse

transferred to the wall by absorbing more of the blast energy. However, the remaining impulse acted on the masonry wall was still too great for the soft retrofits. Therefore, a strong rear support was expected to work best with the aluminium foam. Thus, a layer of 5mm thick steel sheet was applied on the rear surface of the wall. The steel sheet on the rear surface provided better support, allowing the aluminium foam to absorb more energy. A comparison of effectiveness for the URM walls protected by aluminium foam and the combined retrofits is shown in Figure 5.42. The combination of aluminium foam with steel plate performed better than all other combinations, except the double-sided aluminium foam sheet retrofit.

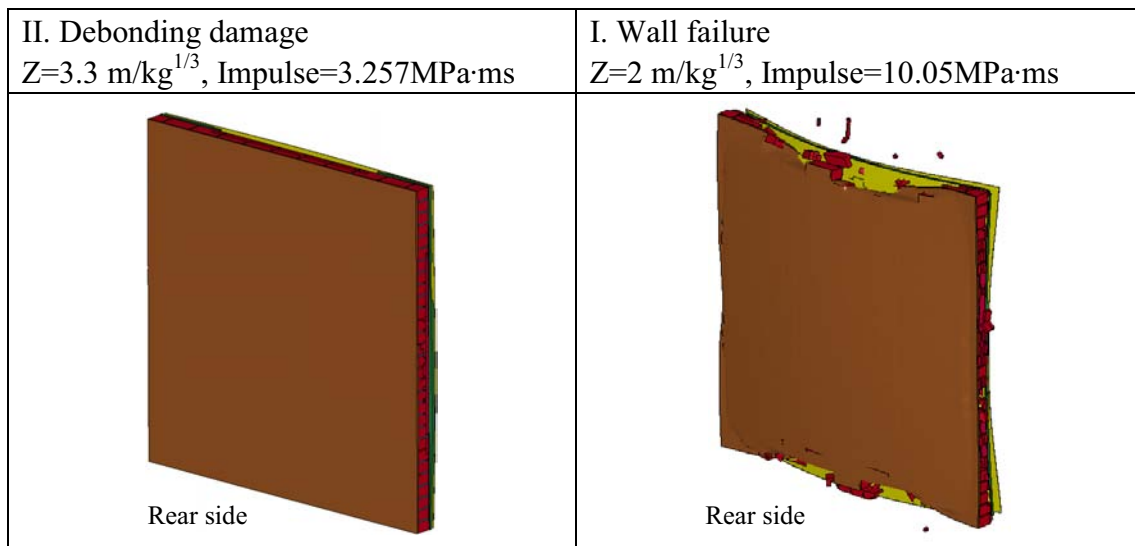


Figure 5.39 Combination of aluminium foam with spray-on polyurea

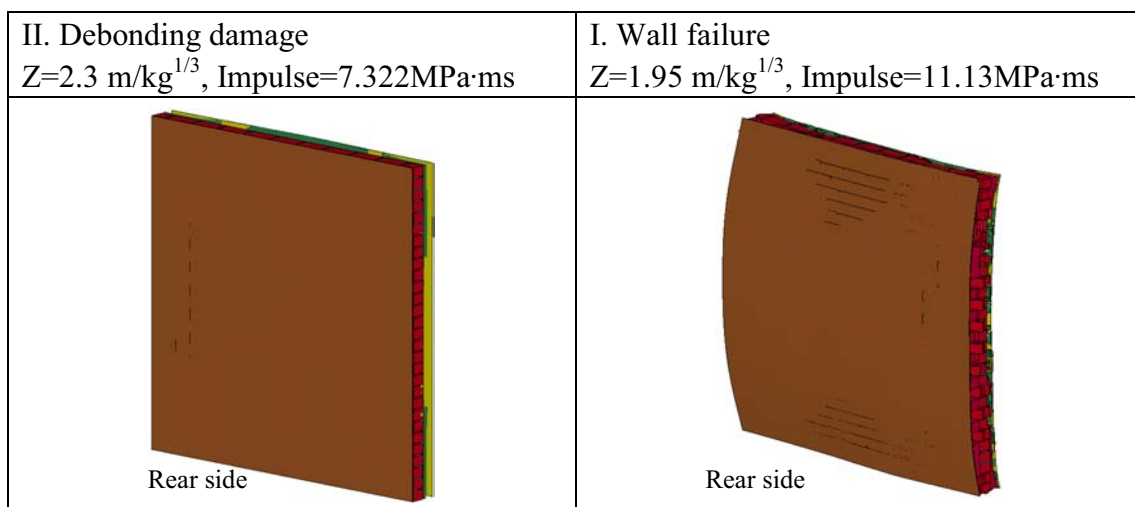


Figure 5.40 Combination of aluminium foam and steel plates

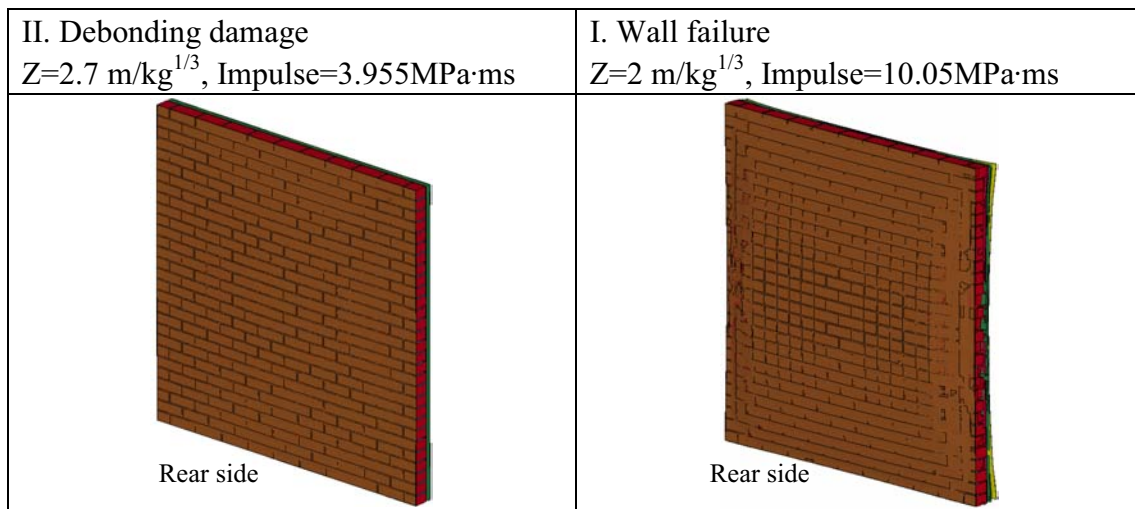


Figure 5.41 Combination of aluminium foam and CFRP plates

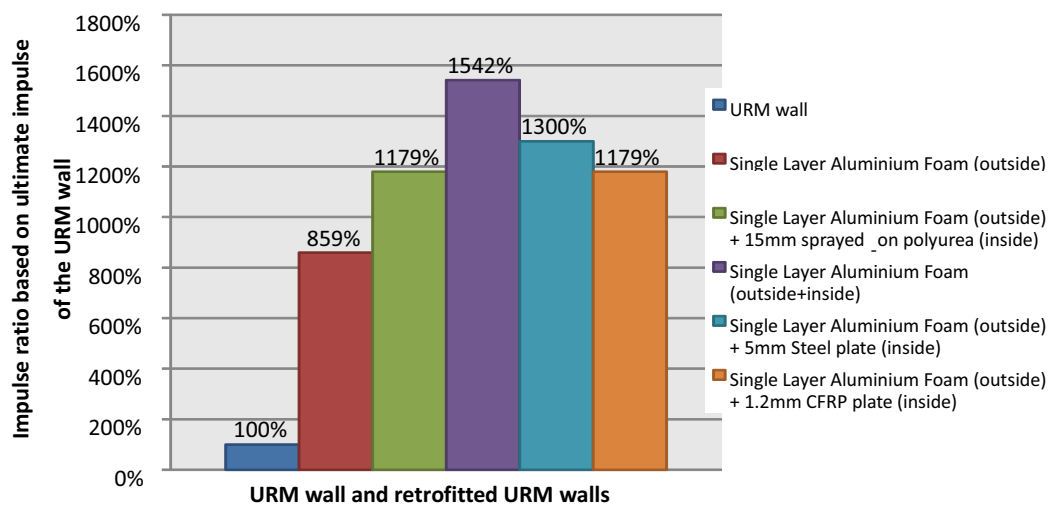


Figure 5.42 Comparison of effectiveness on aluminium foam protected wall

5.4. PRESSURE-IMPULSE (P-I) DIAGRAM FOR RETROFITTED URM WALLS

Current design permit specify the use of pressure-impulse (P-I) diagrams to assess damage levels of structures against airblast loads. Using the numerical model, parametric studies were conducted to derive P-I diagrams for damage assessment of aluminium foam (A356SiC040) protected URM walls. Before deriving the P-I

diagram, damage levels for aluminium foam protected URM walls should be defined.

For URM wall, the ultimate deflection at instability δ_u is predicted by using a one-way vertical bending theory derived by Willis (Willis et al. 2004),

$$\delta_u = t \left(1 - \frac{\sigma_v + 0.25 \rho g h}{f_{mc}} \right) \quad \text{Eq. 5-13}$$

where t is the thickness of the URM wall, σ_v is the pre-compressive stress, ρ is the density of the URM, g is the acceleration due to gravity, h is the height of wall, and f_{mc} is the ultimate compressive stress of mortar. The relationship of f_{mc} and f_{mt} is expressed as follows (MacGregor 1988),

$$f_{mt} = 0.53 \sqrt{f_{mc}} \quad \text{Eq. 5-14}$$

where f_{mt} is the ultimate tensile stress of mortar. The material properties used in this study are presented in Table 5.5, which gives an ultimate deflection of the 2500mm × 2500mm × 110mm URM wall was estimated to be 108mm based on Eq. 5-13. The ultimate deflection of 108mm was used as the failure criterion for the URM wall, and was also used to decide the failure mode of the foam protected URM walls. Figure 5.43 shows P-I diagram for the URM wall based on the above failure criterion.

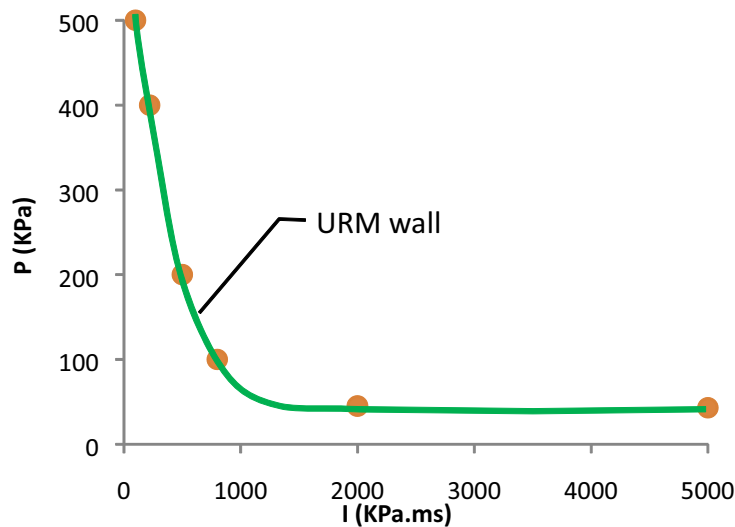
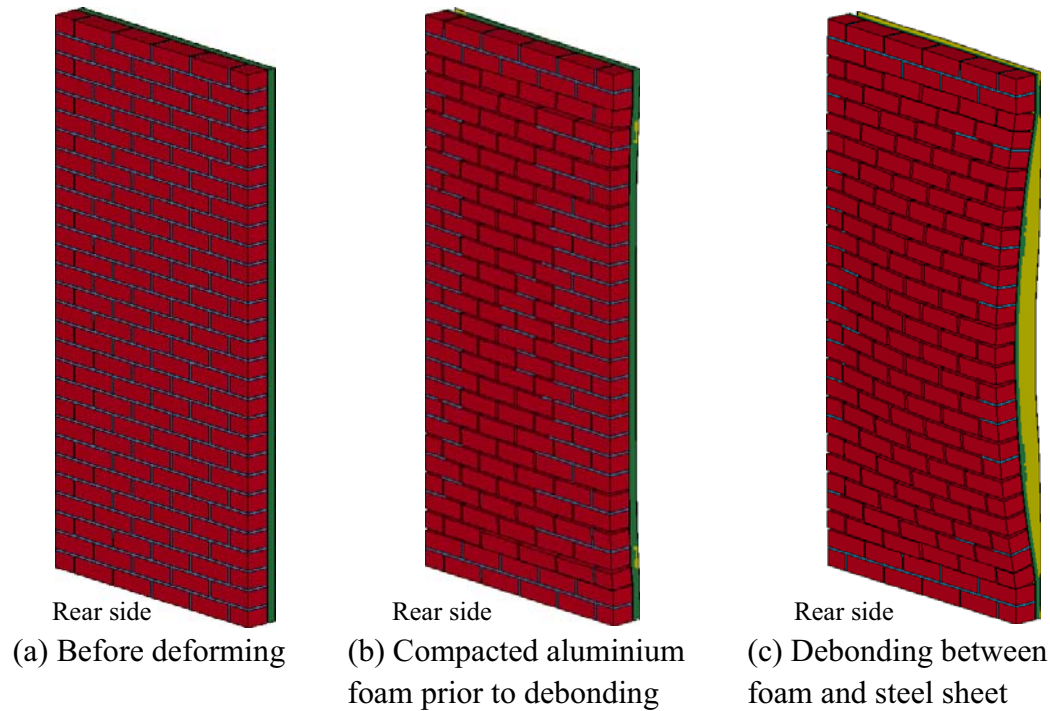


Figure 5.43 P-I diagram for URM walls against airblast loads

Table 5.5 Material properties of URM wall

ρ (kg/m ³)	g (m/s ²)	f_{mt} (Mpa)	t (mm)	σ (MPa)	h (mm)
1800	9.8	0.614	110	0	2500



**Figure 5.44 Deformation process of aluminium foam protected URM wall
(vertical section)**

For aluminium foam protected URM walls, two damage levels are defined: Level 1 foam debonding failure, and Level 2, as an URM wall failure. Debonding between foam and steel sheets/masonry walls will occur when the ultimate deflection of an URM wall exceeds the debonding deflection. Since the elastic modulus of steel sheet is much greater than masonry, debonding begins to occur between the foam and steel sheets rather than between the foam and the masonry. When the debonding area exceeds 10% of the bonding area between foam and steel sheets, the aluminium foam began to damage. Thus, it affects the retrofit effectiveness greatly and characterized as debonding failure, that is, the damage Level 1. Figure 5.44 shows the debonding

failure process of aluminium foam protected URM wall. When a foam protected URM wall is subjected to airblast loads, the foam and the steel sheet will initially deform together with the URM wall (see Figure 5.44b). However, as the deformation of the URM wall increases, debonding occurs between the foam and steel sheets as shown in Figure 5.44c. When the ultimate deflection of the foam protected URM wall reaches 108 mm, it reaches the Damage Level 2, that is, URM wall failure. Table 5.6 characterizes damage levels for aluminium foam protected URM walls under airblast loads.

Table 5.6 Damage levels for aluminium foam protected URM wall

Damage level	Description	Performance
I. Debonding failure	The debonding area exceeds 10% of the bonding area between foam and steel sheets, aluminium foam begin to disintegrate.	Failure of foam happens. Steps cracks can be observed in mortar joints.
II. Wall failure	Protected URM wall reaches its maximum blast resistant capability. Ultimate deflection of foam protected URM wall exceeds the critical deflection 108mm.	Foam definitely fails, and wall collapses. Almost all the mortar joints are damaged.

In this study, damage levels for foam protected URM walls are identified using energy absorption ratio method. The total input energy from a blast impulse is converted into kinetic energy, with the elastic strain energy primarily stored by steel cover sheets, and inelastic deformation strain energy stored by crushing and plastic deformation of masonry and aluminium foam. At the end of the blast event, the retrofitted walls get steady, with most of the input energy being converted to deformation energy stored as internal energy mainly by wall and aluminium foam. Under small impulses, the ratio of energy absorbed by the foam and URM wall (as shown in Figure 5.45) is roughly constant since the foam and the steel sheet deform together with the URM wall. Under greater impulses, the aluminium foam is compacted, and the steel sheets may

also start to debond from the foam. The starting debonding point was defined as Damage Level 1 as shown in Figure 5.45. Further increasing the impulse cause more and more energy to be absorbed by the foam due to more foam cells rupturing until the wall reaches Damage Level 2, as shown in Figure 5.45. At Damage Level 2, the ratio of the energy absorbed by foam reaches a maximum so that it is easily identified in the curves in Figure 5.45 and Figure 5.46. Further impulse increases cause the aluminium foam to be destroyed and the URM wall to collapse. Similar phenomena were observed in the EB FRP plates (Figure 5.47) and spray-on polyurea (Figure 5.48) retrofitted URM walls.

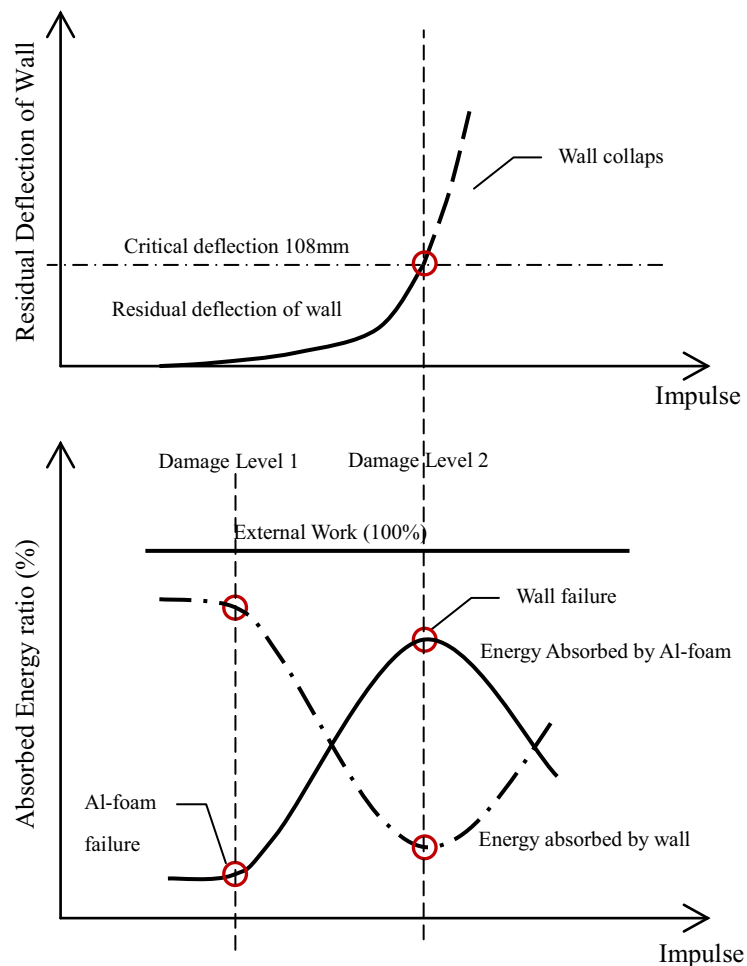


Figure 5.45 Determination of Damage Levels based on energy absorption ratio

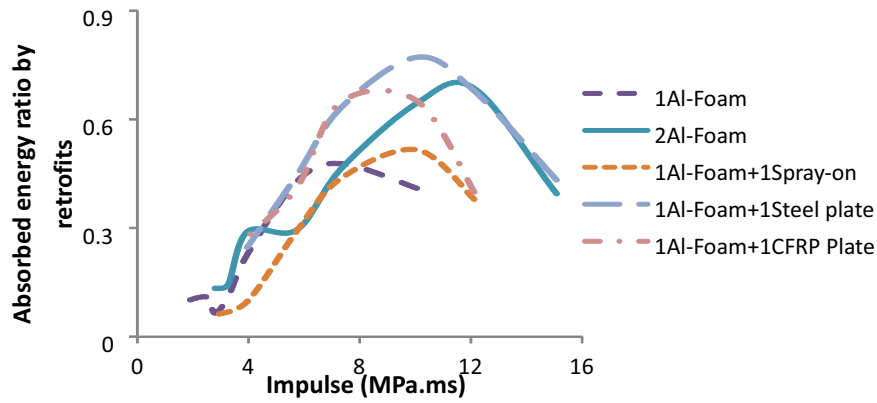


Figure 5.46 Absorbed energy ratio of aluminium foam protected URM walls

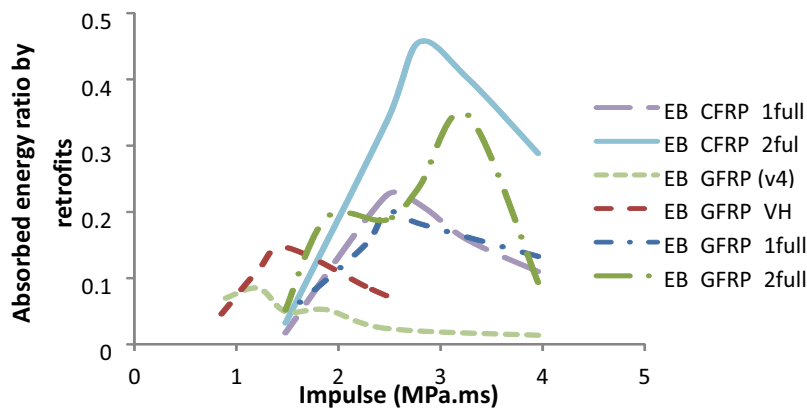


Figure 5.47 Absorbed energy ratio of EB CFRP or GFRP retrofitted URM walls

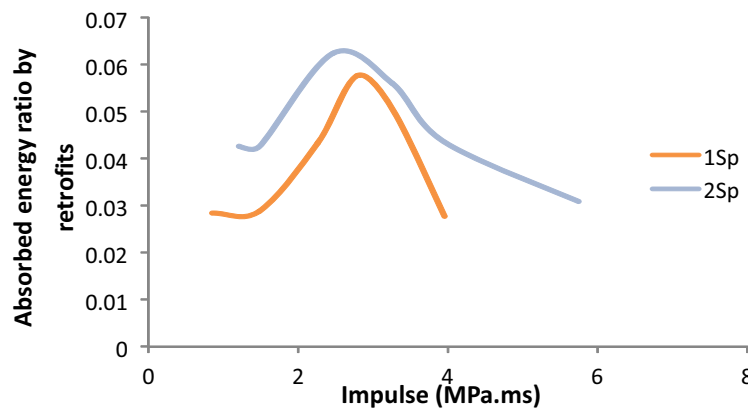


Figure 5.48 Absorbed energy ratio of spray-on polyurea retrofitted URM walls

Based on the damage levels defined in Table 5.6, parametric studies were carried out to derive P-I diagrams for foam protected URM walls. Figure 5.49 and Figure 5.50 show the P-I diagrams for URM walls protected by aluminium foam on the front

surface and the both surfaces at two damage levels. A comparison of the P-I diagrams for URM walls and foam protected URM walls is also shown in Figure 5.51. As shown in Figure 5.50, the foam protected URM walls greatly increase the blast resistant capacity of the URM walls. Using this method, P-I diagrams for URM walls retrofitted by EB CFRP, EB GFRP and spray-on polyurea on rear surface at two damage levels are shown in Figure 5.52, Figure 5.53 and Figure 5.54, respectively.

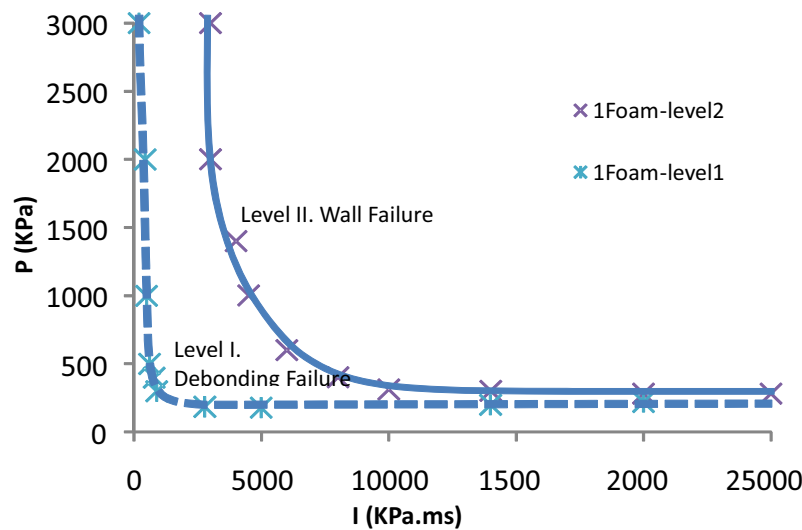


Figure 5.49 P-I diagrams for aluminium foam protected URM walls (front side)

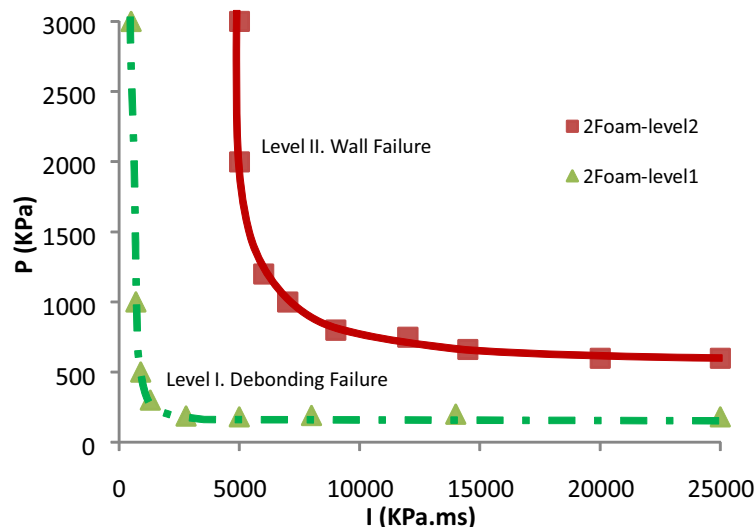


Figure 5.50 P-I diagram of aluminium foam protected URM walls (both sides)

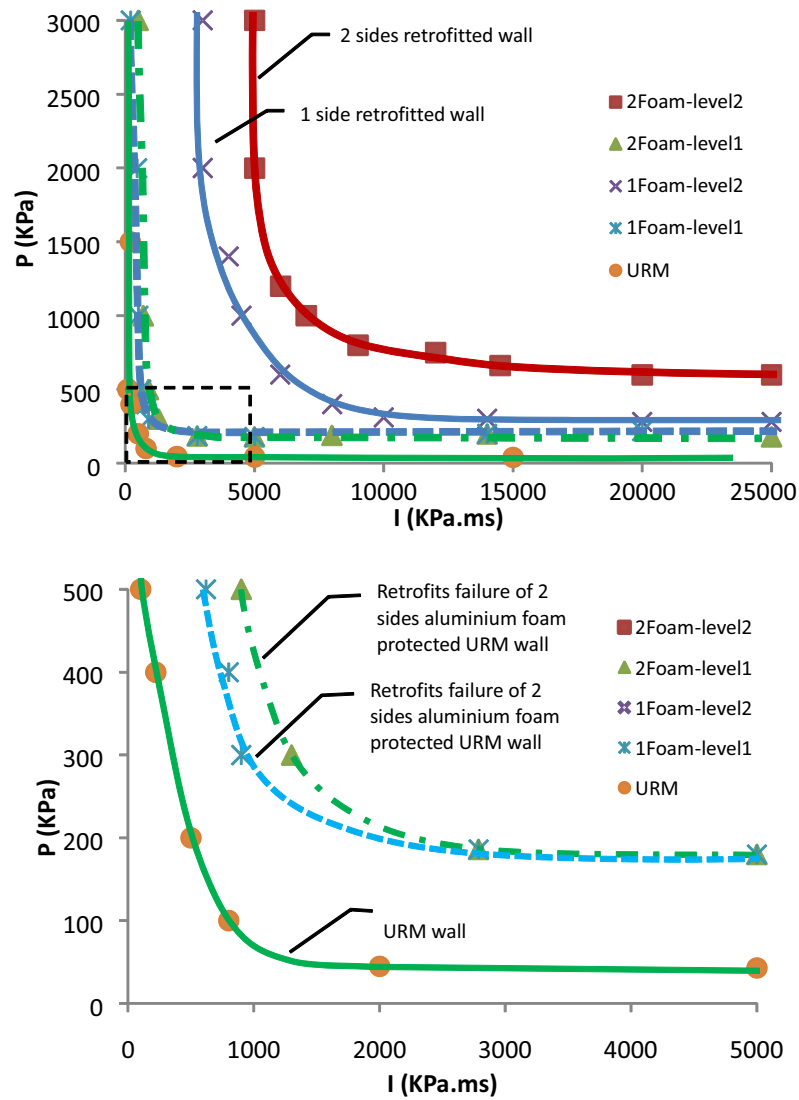


Figure 5.51 P-I diagrams for URM walls and foam protected URM walls

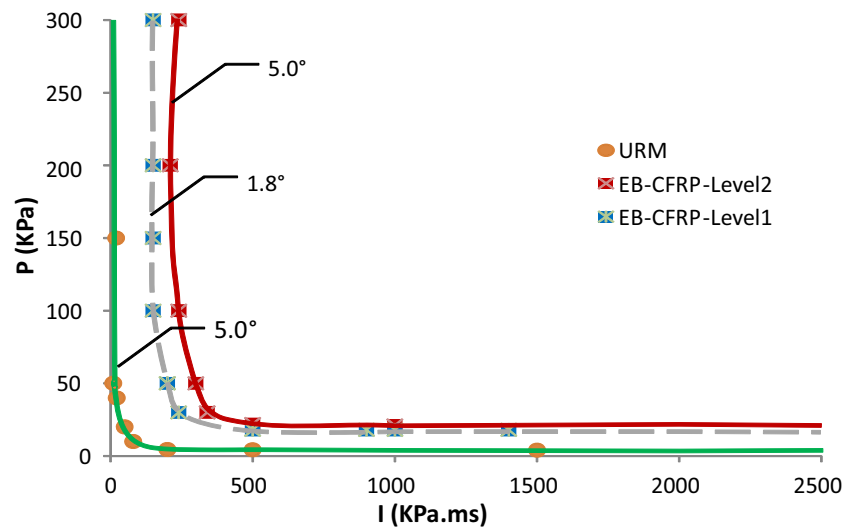


Figure 5.52 P-I diagram for EB CFRP retrofitted URM walls

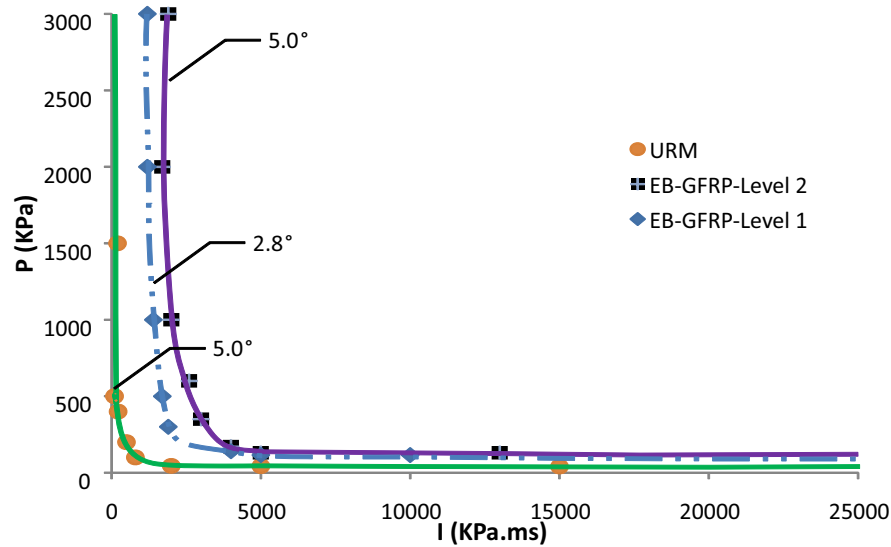


Figure 5.53 P-I diagram for EB GFRP retrofitted URM walls

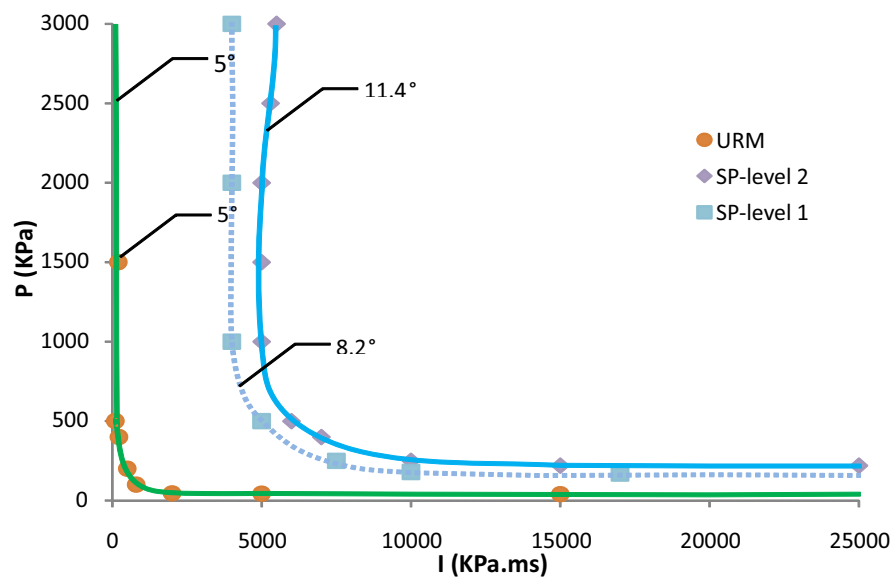


Figure 5.54 P-I diagram for Spray-on polyurea protected URM walls

For the same charge weight, pressure and impulse is plotted in Figure 5.55 as a linear relationship. If using impulse to identify the blast-resistant capability of different retrofitting methods, the effect will vary with charge weight. For different charge weights such as 125kg, 512kg and 1000kg having the same impulse, the mitigation effect is more significant for large charge weight compared with small charge weight.

The relationships acting on the URM wall retrofitted by EB FRP or spray-on polyurea are plotted in Figure 5.56 and Figure 5.57.

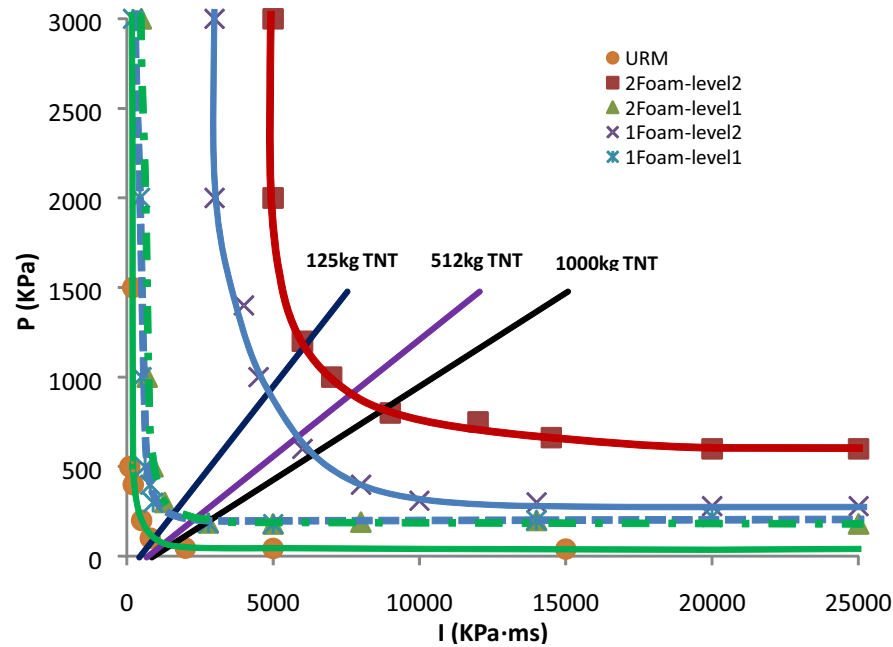


Figure 5.55 Influence of charge weight on effect of protection in P-I diagrams of aluminium foam protected URM walls

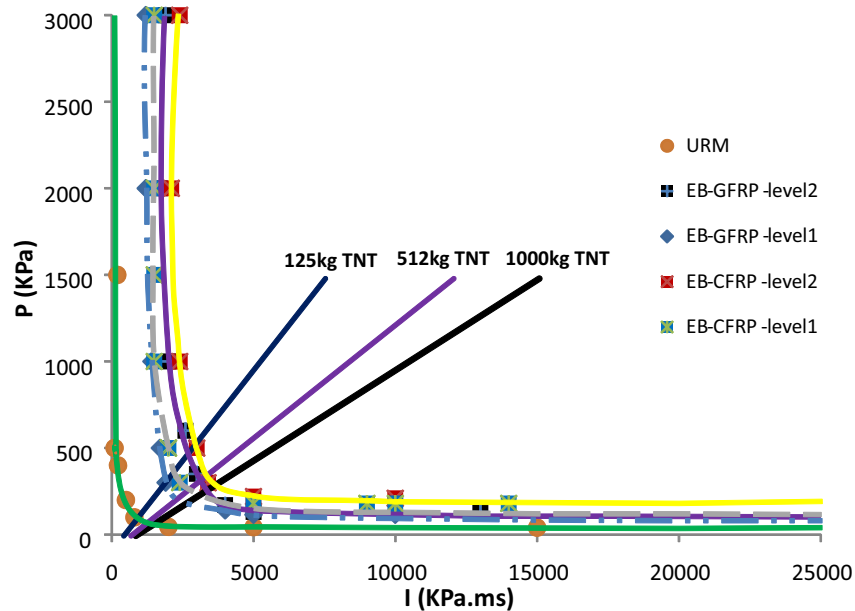


Figure 5.56 Influence of charge weight on effect of protection in P-I diagrams of EB FRP retrofitted URM walls

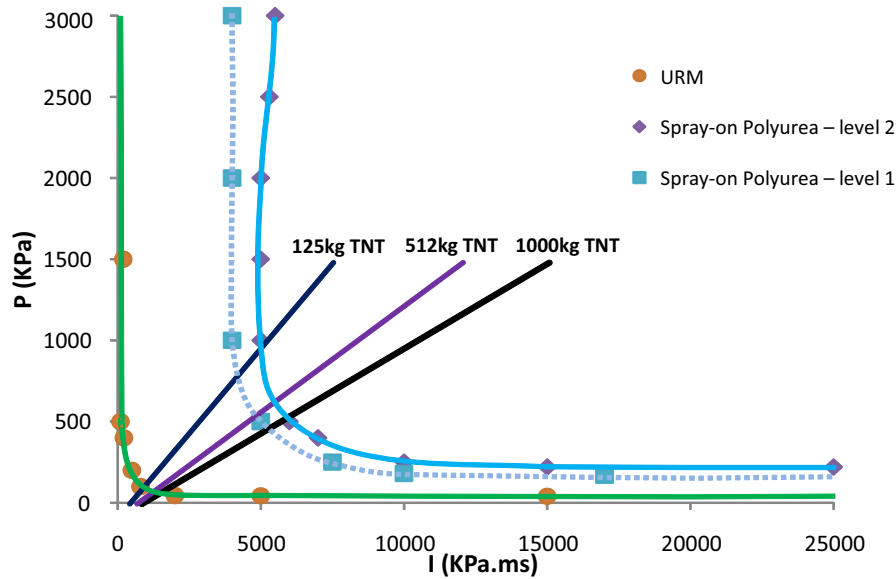


Figure 5.57 Influence of charge weight on effect of protection in P-I diagrams of spray-on polyurea retrofitted URM walls

Comparisons of the P-I diagrams for the URM wall and retrofitted URM walls at damage level II and damage level I are shown in Figure 5.58 and Figure 5.59, respectively. P-I curves for various retrofitting techniques are usually parallel to each other, except the curve for the wall retrofitted by spray-on polyurea. Under small charge weight, the mitigation effect of spray-on polyurea is better than other solutions.

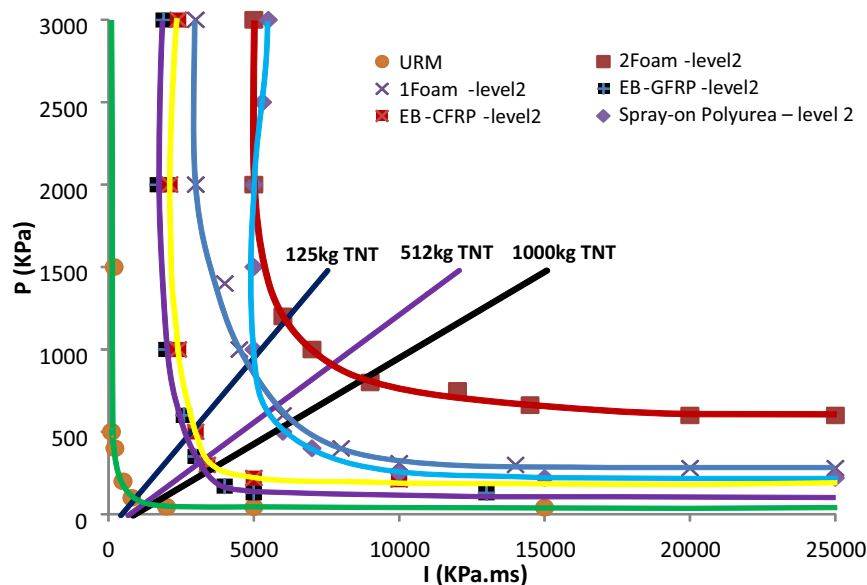


Figure 5.58 P-I diagrams for retrofitted URM walls at damage level II

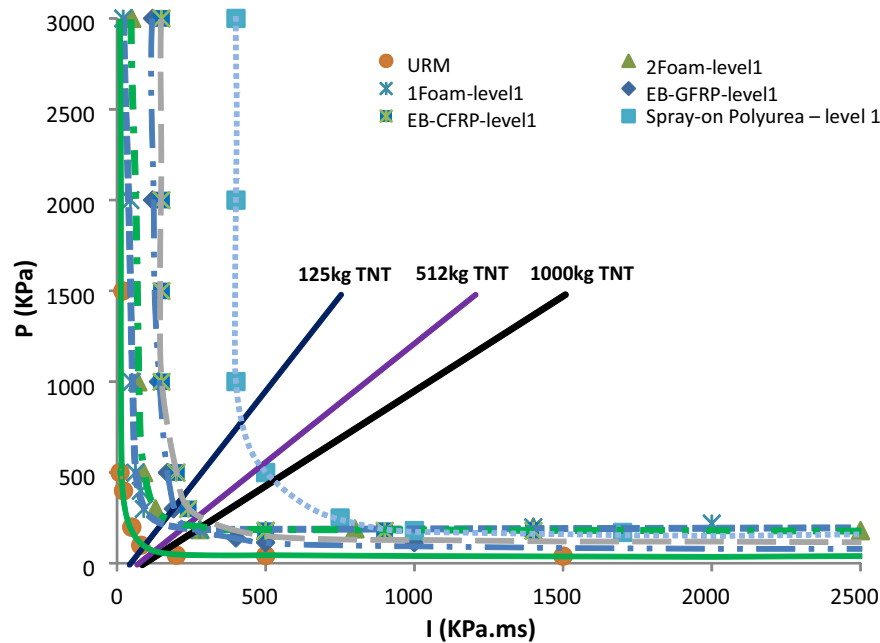


Figure 5.59 P-I diagrams for retrofitted URM walls at damage level I

5.5. CONCLUSIONS

The performance of URM walls protected by various types of retrofitting technologies was simulated numerically in this study. The numerical results indicate that the aluminium foam is the most effective technique for mitigation of blast effects on URM walls. This is because the foam absorbs more blast energy compared with the other retrofitting techniques considered in this study. It was also found that both thickness and density of aluminium foam sheets greatly influences mitigation effectiveness against blast loads on URM walls. Damage levels were defined based on a collapse failure mechanism and energy absorption method. P-I diagrams for EB FRP, spray-on polyurea and aluminium foam protected URM walls based on the simulated results.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1. SUMMARY AND CONCLUSIONS

Masonry buildings exhibit the vulnerability of poor blast-resistant capacity with little ductility. Aiming to find effective strengthening solutions to enhance masonry walls against explosion, this project focused on studying the performance of retrofitting techniques, such as EB FRP and NSM FRP, which have been widely use to strengthen concrete structures, because of its light weight, high strength and durability. However, the performance of the EB and NSM strips retrofits on masonry walls against blast loading was poor. This research showed that, such retrofits failed in shear or bending between strips. Hence, several other new materials, such as spray-on polyurea and aluminium foam, were also studied for mitigation of blast effect. These retrofitting systems were much more efficient.

To study the bonding behaviours between masonry and retrofits, bond-slip models coded in LS-DYNA were used, and compared with pull tests for validation. Stress-slip curves and load-displacement relationship were compared, from which it was found the bond-slip model worked well. A homogenized model which performs efficiently was derived for simulating full scaled retrofitted masonry walls under out-of-plane loading. The models based on test data were verified with test results, and load-displacement curves and strain distribution along the height were compared. Results from the homogenized model matched well with experimental results. It was found that the homogenized model could represent the elastic and plastic behaviours of masonry walls. However, it did not give accurate results for post-failure zone.

The numerical models developed in this study were applied to simulate the behaviours

of retrofitted masonry wall under blast loading. To increase ductility of the wall, a new technique known as spray-on polyurea was employed in this study. It was found that the capability of absorbing strain energy was the key factor that influenced performance. A new energy absorbing material, aluminium foam, was applied to the masonry walls. To investigate the effectiveness of different types of retrofitting materials, two critical damage levels were defined. Based on simulation results, debonding failure level and wall failure level were identified and then extended to greater range of pressure and impulse relationship. Thus, pressure-impulse diagrams for various retrofitting techniques were developed.

It should be noted that the numerical models and developed P-I diagrams were based on one layer of brick masonry wall with thickness of 110 mm, and panel dimensions of 2500mm $\times$ 2500mm. The performance of the retrofits will vary if the thickness or dimensions are changed, especially for the aluminium foam protected masonry walls. If applying the aluminium foam material on stronger masonry wall, the retrofits would likely perform better by enhancing its capability of absorbing energy. The study provides a general approach for simulating the retrofitted masonry walls. However, further research on derived dimensionless P-I diagrams are recommended, which can be applied to wide range of masonry structures.

In summary, it can be concluded that FRP material on masonry used against earthquake loads may not have the same performance in blast environments. The ability to absorb strain energy is important for protecting masonry walls against blast impulses. Further studies should be conducted that focus on the new materials.

6.2. RECOMMENDATIONS FOR FURTHER RESEARCH

Based on the studies described herein, some related aspects requiring further research

have become apparent, namely,

1. Material models for bricks and mortar could be improved to consider microscopic material failures and the effect of strain rate. This would mean more accurate results could be obtained, the relationship between retrofits and masonry would be more reliable, and accurate local failure could be observed in simulation.
2. The bond-slip model in current research is efficient, but could be improved by extending to transfer 3-D stress and strain between masonry and retrofits to simulating the physical behaviours accurately. The reasons behind different types of debonding failures could be further studied in simulation.
3. Experiments on masonry and retrofitted masonry walls under blast loading are required to verify the numerical models. Some phenomena such as local failure at different locations which influence the debonding failure should be checked using test results. Moreover, the P-I diagrams should be validated using experimental data.
4. Dimensionless P-I diagrams are required for design purposes. More data would be required to qualify the damage levels, and other failure modes would also be observed which should be considered in guidelines.
5. Investigation into retrofitted masonry walls under close bursts or explosions at small stand-off distances is deemed to be worthwhile and results could be included in P-I diagrams to improve design guidelines.

NOTATIONS

A	= area perpendicular to the principal strain direction
a_{0-2}	= shear failure surface constants in Drucker-Prager model
E	= modulus of elasticity
E_c	= compressive modulus of elasticity
E_t	= tensile modulus of elasticity
E_{ij}	= elastic moduli of aluminium foam
E_{ij}^u	= elastic/shear modulus in uncompressed configuration
$\bar{E}$	= equivalent moduli of elasticity
f_{mc}	= ultimate compressive stress of mortar
f_{mt}	= ultimate tensile stress of mortar
G	= Elastic shear modulus
G_c	= fracture energy release rate in smeared crack model
G_f	= shearing fracture energy release rate in bond-slip model
G_{ft}	= tensile fracture energy release rate in bond-slip model
G_I	= fracture energy release rate of mode I in smeared crack model
G_{II}	= fracture energy release rate of mode II in smeared crack model
g	= acceleration due to gravity
h	= height of the masonry wall
I	= impulse of blast loading
I_1	= first invariant of the stress tensor
J_2	= second invariant of the deviatoric stress tensor S_{ij}
k	= material constant in Drucker-Prager model
P	= airblast over pressure
P_o	= ambient over pressure
P_r	= reflected pressure
P_{so}	= peak value of incident pressure

p^{n+1}	= hydro pressure
R	= stand-off distance
S_C	= shear strength in XY plane in the homogenized model
S_{ij}	= deviatoric stress tensor
s_{ij}^{n+1}	= stress components of aluminium foam
$s_{ij}^{n+1trial}$	= updated stress components
T_a	= shock wave front arrive time
T_d	= decreasing time from peak to ambient pressure
T_r	= rising time from arrival time to peak value
t	= thickness of the URM wall
V	= volume
V_f	= fully compacted volume of aluminium foam
W	= charge weight
X_T	= tensile strength in Z direction of the homogenized model
X_C	= compressive strength in Z direction of the homogenized model
Y_T	= average tensile strength in X and Y directions
Y_C	= average compressive strength in X and Y directions
Z	= scaled distance
α	= pressure sensitivity coefficient in Drucker-Prager model
Δa	= crack opening displacement increment
δ_f	= slip at zero shear stress
δ_l	= slip at peak shear stress
ε_{ij}	= strain components in an element
$\bar{\varepsilon}_{ij}$	= average strain defined by integral over the basic cell
ε_i^{ini}	= threshold strain
ε_i^{ult}	= ultimate strain
ε_{ij}^{dev}	= deviatoric strain increment
λ	= function of strain rate of the deviatoric strain-rate tensor

ν	= Poisson's ratio
ρ	= density of the masonry
σ_c	= yield stress in uniaxial compression
σ_t	= yield stress in uniaxial tension
σ_f	= ultimate normal stress
σ_n	= normal stress
σ_v	= pre-compressive stress
σ_{ij}	= stress in an element
σ_{ft}	= tensile strength of brick units
$\sigma_{ij}(V)$	= stress component by the stress versus volumetric strain curves
$\bar{\sigma}_{ij}$	= average stress defined by integral over the basic cell
σ_{ij}^{n+1}	= Cauchy stress
τ_f	= ultimate shear stress
τ_s	= shear stress

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APPENDIX A: NUMERICAL MODEL OF THE MASONRY BASIC CELL

```

$-----1-----2-----3-----4-----5-----6-----7-----8
$ LS-DYNA(970) DECK WRITTEN BY : eta/FEMB-PC version 28.0
$ TEMPLATE #: 20040810
$  ENGINEER :
$   PROJECT :
$    UNITS : M, KG, SEC, N
$    TIME : 12:58:56 PM
$    DATE : Tuesday, November 14, 2006
$-----1-----2-----3-----4-----5-----6-----7-----8
*KEYWORD
$-----1-----2-----3-----4-----5-----6-----7-----8
*TITLE
LS-DYNA USER INPUT
$-----1-----2-----3-----4-----5-----6-----7-----8
$
$                                CONTROL CARD
$
$-----1-----2-----3-----4-----5-----6-----7-----8
*CONTROL_TERMINATION
$  ENDTIM   ENDCYC   DTMIN   ENDENG   ENDMAS
      0.15      0      0.0      0.0      0.0
$-----1-----2-----3-----4-----5-----6-----7-----8
$
$                                DATABASE CONTROL FOR BINARY
$
$-----1-----2-----3-----4-----5-----6-----7-----8
*DATABASE_BINARY_D3PLOT
$  DT/CYCL   LCDT   BEAM   NPLTC
      0.010      0      0      0
$  IOOPT
      1
$-----1-----2-----3-----4-----5-----6-----7-----8
$
$                                DATABASE EXTENT CARDS
$
$-----1-----2-----3-----4-----5-----6-----7-----8
*DATABASE_EXTENT_BINARY
$^EXTENT_1
$  NEIPH   NEIPS   MAXINT   STRFLG   SIGFLG   EPSFLG   RLTF LG   ENGFLG
      0      0      3      1      1      1      1      1

```

```

$  CMPFLG  IEVERP  BEAMIP  DCOMP  SHGE  STSSZ  N3THDT
      0      0      0      0      0      0      2
$  NINTSLD
      1
$--+1--+2--+3--+4--+5--+6--+7--+8
$
$
$          PART CARDS
$
$--+1--+2--+3--+4--+5--+6--+7--+8
*PART
BRICK
$  PID      SECID      MID      EOSID      HGID      GRAV      ADPOPT      TMID
      1      1      3      0      0      0      0      0
*PART
MORTAR
$  PID      SECID      MID      EOSID      HGID      GRAV      ADPOPT      TMID
      2      1      4      0      0      0      0      0
$--+1--+2--+3--+4--+5--+6--+7--+8
$
$
$          SECTION CARDS
$
$--+1--+2--+3--+4--+5--+6--+7--+8
*SECTION_SOLID_TITLE
P-1
$  SECID  ELFORM      AET
      1      1      0
$--+1--+2--+3--+4--+5--+6--+7--+8
$
$
$          MATERIAL CARDS
$
$--+1--+2--+3--+4--+5--+6--+7--+8
*MAT_SOIL_AND_FOAM_TITLE
Brick
$  MID      RO      G      BULK      A0      A1      A2      PC
      1  2400.02.2000E+101.8000E+111.3887E+131.0562E+07  2.0083-3550000.0
$  VCR      REF
      0.0      0.0
$  EPS1      EPS2      EPS3      EPS4      EPS5      EPS6      EPS7      EPS8
      0.0-7.503E-05-0.0001876-0.0003037-0.0009494
$  EPS9      EPS10
$
$  P1      P2      P3      P4      P5      P6      P7      P8
      0.0 3950000.0 9880000.01.6000E+075.0000E+07
$  P9      P10

```

*MAT_SOIL_AND_FOAM_TITLE

Mortar

\$	MID	RO	G	BULK	A0	A1	A2	PC
	2	2400.01.8438E+081.3300E+094.1540E+11	1830000.0				2.0083 -614000.0	
\$	VCR	REF						
	0.0	0.0						
\$	EPS1	EPS2	EPS3	EPS4	EPS5	EPS6	EPS7	EPS8
	0.0-0.0094003-0.0165071-0.0236648							
\$	EPS9	EPS10						
\$	P1	P2	P3	P4	P5	P6	P7	P8
	0.0 3950000.0 6140000.0 9880000.0							
\$	P9	P10						

*MAT_SOIL_AND_FOAM_FAILURE_TITLE

Brick-2

\$	MID	RO	G	BULK	A0	A1	A2	PC
	3	2400.02.2000E+101.8000E+111.3887E+131.0562E+07					2.0083-3550000.0	
\$	VCR	REF						
	0.0	0.0						
\$	EPS1	EPS2	EPS3	EPS4	EPS5	EPS6	EPS7	EPS8
	0.0 -0.000075 -0.000188 -0.000304 -0.000949							
\$	EPS9	EPS10						
\$	P1	P2	P3	P4	P5	P6	P7	P8
	0.0 3950000.0 9880000.01.6000E+075.0000E+07							
\$	P9	P10						

*MAT_SOIL_AND_FOAM_FAILURE_TITLE

Mortar-2

\$	MID	RO	G	BULK	A0	A1	A2	PC
	4	2400.01.8400E+081.3300E+094.1540E+11	1830000.0				2.0083 -800000.0	
\$	VCR	REF						
	0.0	0.0						
\$	EPS1	EPS2	EPS3	EPS4	EPS5	EPS6	EPS7	EPS8
	0.0 -0.0094 -0.0165 -0.0237							
\$	EPS9	EPS10						
\$	P1	P2	P3	P4	P5	P6	P7	P8
	0.0 3950000.0 6140000.0 9880000.0							
\$	P9	P10						

\$-----1-----2-----3-----4-----5-----6-----7-----8

```

$
$
$
$
$-----1-----2-----3-----4-----5-----6-----7-----8
*DAMPING_GLOBAL
$^DAMP_1
$    LCID    VALDMP    STX    STY    STZ    SRX    SRY    SRZ
      0      30.0      0.0      0.0      0.0      0.0      0.0
$-----1-----2-----3-----4-----5-----6-----7-----8
$
$
$
$
$-----1-----2-----3-----4-----5-----6-----7-----8
*SET_NODE_LIST_TITLE

$    SID      DA1      DA2      DA3      DA4
      1        0.0      0.0      0.0      0.0
$    NID1      NID2      NID3      NID4      NID5      NID6      NID7      NID8
      1         2         3         4        10        11        18        19
.
.
.
1899
*SET_NODE_LIST_TITLE

$    SID      DA1      DA2      DA3      DA4
      2        0.0      0.0      0.0      0.0
$    NID1      NID2      NID3      NID4      NID5      NID6      NID7      NID8
    25905    25906    25907    25908    25913    25914    25917    25918
.
.
.
26545
$-----1-----2-----3-----4-----5-----6-----7-----8
$
$
$
$
$-----1-----2-----3-----4-----5-----6-----7-----8
*BOUNDARY_PRESCRIBED_MOTION_SET_ID
$^PRESCRIBED MOTION CARD 1
$    ID
      1
$    NSID      DOF      VAD      LCID      SF      VID      DEATH      BIRTH
      2         3         2         2    0.0040    01.0000E+28    0.0

```

```

$-----1-----2-----3-----4-----5-----6-----7-----8
$
$
$                                BOUNDARY SPC CARDS
$
$-----1-----2-----3-----4-----5-----6-----7-----8
*BOUNDARY_SPC_SET_ID
$      ID
      1
$      NSID      CID      DOFX      DOFY      DOFZ      DOFRX      DOFRY      DOFRZ
      1          0          0          0          1          1          1          1
$-----1-----2-----3-----4-----5-----6-----7-----8
$
$                                LOAD CURVE CARDS
$
$-----1-----2-----3-----4-----5-----6-----7-----8
*DEFINE_CURVE_TITLE
LCur_1
$      LCID      SIDR      SFA      SFO      OFFA      OFFO      DATTYP
      1          0      1.0      1.0      0.0      0.0          0
$
      A1          O1
      0.0          0.0
      0.0020      0.000098
      0.0040      0.00019
      0.0060      0.000277
      0.0080      0.00036
      0.010      0.000438
      0.012      0.00051
      0.014      0.000577
      0.016      0.00064
      0.018      0.000698
      0.020      0.00075
      0.022      0.000798
      0.024      0.00084
      0.026      0.000877
      0.028      0.00091
      0.030      0.000937
      0.032      0.00096
      0.034      0.000977
      0.036      0.00099
      0.038      0.000998
      0.040      0.0010
      1.0          0.0010
*DEFINE_CURVE_TITLE
LCur_2

```

\$	LCID	SIDR	SFA	SFO	OFFA	OFFO	DATTYP
	2	0	1.0	1.0	0.0	0.0	0

\$		A1		O1			
		0.0		0.0			
		0.0050		0.000098			
		0.010		0.00019			
		0.015		0.000277			
		0.020		0.00036			
		0.025		0.000438			
		0.030		0.00051			
		0.035		0.000577			
		0.040		0.00064			
		0.045		0.000698			
		0.050		0.00075			
		0.055		0.000798			
		0.060		0.00084			
		0.065		0.000877			
		0.070		0.00091			
		0.075		0.000937			
		0.080		0.00096			
		0.085		0.000977			
		0.090		0.00099			
		0.095		0.000998			
		0.10		0.0010			
		1.0		0.0010			

\$-----1-----2-----3-----4-----5-----6-----7-----8

\$

\$

NODE INFORMATION

\$

\$-----1-----2-----3-----4-----5-----6-----7-----8

*NODE

\$	NID	X	Y	Z	TC	RC
	1	0.0	0.0	0.0	0.0	0.0

.

.

.

27757	0.150132	0.0711325	0.134	0.0	0.0
-------	----------	-----------	-------	-----	-----

\$-----1-----2-----3-----4-----5-----6-----7-----8

\$

\$

SOLID ELEMENTS

\$

\$-----1-----2-----3-----4-----5-----6-----7-----8

*ELEMENT_SOLID

\$	EID	PID	NID1	NID2	NID3	NID4	NID5	NID6	NID7	NID8
----	-----	-----	------	------	------	------	------	------	------	------

1	1	1	2	3	4	5	6	7	8
.									
.									
.									
3560	2	27037	27184	27116	26956	24167	25760	24935	23343

\$-----1-----2-----3-----4-----5-----6-----7-----8

*END

APPENDIX B: NUMERICAL MODELS OF THE PULL-TESTS

1. EB interface element model

```

$-----1-----2-----3-----4-----5-----6-----7-----8
$ LS-DYNA(970) DECK WAS WRITTEN BY: eta/VPG VERSION 3.2
$
$ ENGINEER:
$ PROJECT:
$ UNITS: M, KG, SEC, N
$ DATE: Jul 10, 2007 at 9:43:39
$-----1-----2-----3-----4-----5-----6-----7-----8
*KEYWORD
$-----1-----2-----3-----4-----5-----6-----7-----8
*TITLE

$-----1-----2-----3-----4-----5-----6-----7-----8
*CONTROL_BULK_VISCOSITY
$      q1      q2      type
      1.500  0.06000      1
*CONTROL_CONTACT
$  slsfac  rwpnal  islchk  shlthk  penopt  thkchg  orien  enmass
      0.10000      0.0      1      0      1      0      1      0
$  usrstr  usrfrc  nsbcs  interm  xpene  ssthk  ecdt  tiedprj
      0      0      10      0  4.000      0      0      0
$  sfrc  dfrc  edc  vfc  th  th_sf  pen_sf
      0.0  0.0  0.0  0.0  0.0  0.0  0.0
$  ignore  frceng  skiprwg  outseg  spotstp  spotdel
      0      0      0      0      0      0
*CONTROL_ENERGY
$  hgen  rwen  slnten  rylen
      2      2      1      1
*CONTROL_HOURLASS
$  ihq  qh
      6  0.05000
*CONTROL_OUTPUT
$  npopt  neecho  nrefup  iaccop  opifs  ipnint  ikedit  iflush
      1      3      0      0  0.0      0      100  5000
$  iprtf
      0
*CONTROL_TERMINATION
$  endtim  endcyc  dtmin  endeng  endmas

```

```

0.04600      0      0.0      0.0      0.0
$*CONTROL_TIMESTEP
$$  dtinit    tssfacc    isdo    tslimt    dt2ms    lctm    erode    ms1st
$    0.0    0.90000      0      0.0 -4.000E-6      0      0      0
$-----1-----2-----3-----4-----5-----6-----7-----8
*DATABASE_GLSTAT
$    dt    binary
    .1000E-2      1
*DATABASE_MATSUM
$    dt    binary
    .1000E-2      0
$-----1-----2-----3-----4-----5-----6-----7-----8
*DATABASE_BINARY_D3PLOT
$  dt/cycl    lcdt    beam    npltc
    .100E-3      0      0      0
$  ioopt
    1
*DATABASE_BINARY_D3DUMP
$  dt/cycl
    100000.00
$-----1-----2-----3-----4-----5-----6-----7-----8
*DATABASE_EXTENT_BINARY
$CardName:EXTENT_1
$  neiph    neips    maxint    strflg    sigflg    epsflg    rltflg    engflg
      2      0      3      1      1      1      1      1
$  cmpflg    ieverp    beamip    dcomp    shge    stssz    n3thdt
      0      0      0      0      0      0      2
$  nintsld
    1
$-----1-----2-----3-----4-----5-----6-----7-----8
*PART
BRICK
$  pid    secid    mid    eosid    hgid    grav    adpopt    tmid
      1      1      1      0      0      0      0      0
*PART
MORTAR
      2      1      2      0      0      0      0      0
*PART
FRP
      3      1      3      0      0      0      0      0
*PART
Adhesive
      4      1      4      0      0      0      0      0
$-----1-----2-----3-----4-----5-----6-----7-----8

```

*SECTION_SOLID_TITLE

P-1

\$	secid	elform	aet
	1	1	0

\$-----1-----2-----3-----4-----5-----6-----7-----8

*MAT_ADD_EROSION

\$CardName:Used by Matl: 1

\$	mid	excl
	1	999999.0

\$	pfail	sigp1	sigvm	epsp1	epssh	sigth	impulse	failtm
	999999.0	999999.0	999999.0	0.02000	999999.0	999999.0	999999.0	999999.0

*MAT_ADD_EROSION

\$CardName:Used by Matl: 2

\$	mid	excl
	2	999999.0

\$	pfail	sigp1	sigvm	epsp1	epssh	sigth	impulse	failtm
	999999.0	999999.0	999999.0	0.02500	999999.0	999999.0	999999.0	999999.0

\$-----1-----2-----3-----4-----5-----6-----7-----8

*MAT_SOIL_AND_FOAM_FAILURE_TITLE

Brick

\$	mid	ro	g	bulk	a0	a1	a2	pc
	1	2400.000	.2200E+11	.1800E+12	.5555E+14	.2112E+8	2.008	-.710E+7

\$	vcr	ref
	0.0	0.0

\$	eps1	eps2	eps3	eps4	eps5	eps6	eps7	eps8
	0.0	-.7500E-4	-.1880E-3	-.3040E-3	-0.00949	-0.09490	0.0	0.0

\$	eps9	eps10
	0.0	0.0

\$	p1	p2	p3	p4	p5	p6	p7	p8
	0.0	3950000.	9880000.	.1600E+8	.5000E+9	.5000E+10	0.0	0.0

\$	p9	p10
	0.0	0.0

*MAT_SOIL_AND_FOAM_FAILURE_TITLE

Mortar

\$	mid	ro	g	bulk	a0	a1	a2	pc
	2	2400.000	.2200E+9	.1328E+10	.4156E+12	1827000.	2.008	-800000.0

\$	vcr	ref
	0.0	0.0

\$	eps1	eps2	eps3	eps4	eps5	eps6	eps7	eps8
	0.0	-0.00940	-0.01650	-0.23700	-2.370	0.0	0.0	0.0

\$	eps9	eps10
	0.0	0.0

\$	p1	p2	p3	p4	p5	p6	p7	p8
	0.0	3950000.	6140000.	.9880E+8	.9880E+9	0.0	0.0	0.0

```

$      p9      p10
      0.0      0.0

*MAT_ELASTIC_TITLE
GFRP
$      mid      ro      e      pr      da      db
      3 1500.000 .1930E+11 0.33000      0.0      0.0

*MAT_ARUP_ADHESIVE
$      mid      ro      e      pr  TENMAX  GCTEN  SHRMAX  GCSHR
      4 2400.000 0.065E+09 0.29000 1.84E+06 1.0E+20 5.87E+06 1.3E+03
$      PWRT      PWRS      SHRP      SHT_SL      EDOT0      EDOT2
      2      2      0      0      1      0
$-----1-----2-----3-----4-----5-----6-----7-----8

*HOURGLASS_TITLE

$      hgid      ihq      qm      ibq      q1      q2      qb      qw
      1      6 0.05000      0      1.500 0.06000      0.0      0.0
$-----1-----2-----3-----4-----5-----6-----7-----8

*DAMPING_GLOBAL
$      lcld      valdmp      stx      sty      stz      srx      sry      srz
      0 30.000      0.0      0.0      0.0      0.0      0.0      0.0
$-----1-----2-----3-----4-----5-----6-----7-----8

*SET_NODE_LIST_TITLE
NODE SET_1
$      sid      da1      da2      da3      da4
      1
$      nid1      nid2      nid3      nid4      nid5      nid6      nid7      nid8
      1      4      3      2      10      9      14      13
.
.
.
$-----1-----2-----3-----4-----5-----6-----7-----8

*SET_PART_LIST_TITLE
PART SET_1
$      sid      da1      da2      da3      da4
      1
$      pid1      pid2      pid3
      1      2      3
$-----1-----2-----3-----4-----5-----6-----7-----8

*SET_SEGMENT_TITLE
SEGMENT SET_1
$      sid      da1      da2      da3      da4
      1
$      N1      N2      N3      N4
      16764 16765 16766 16767

```

```

16767    16766    16768    16769
16769    16768    16770    16771
16771    16770    16772    16773
$-----1-----2-----3-----4-----5-----6-----7-----8
*BOUNDARY_SPC_SET_ID
$      id
      1
$      nsid      cid      dofx      dofy      dofz      dofrx      dofry      dofrz
      1          0          1          1          1          0          0          0
*BOUNDARY_SPC_SET_ID
$      id
      2
$      nsid      cid      dofx      dofy      dofz      dofrx      dofry      dofrz
      2          0          1          1          1          0          0          0
*BOUNDARY_SPC_SET_ID
$      id
      3
$      nsid      cid      dofx      dofy      dofz      dofrx      dofry      dofrz
      3          0          1          1          0          0          0          0
$-----1-----2-----3-----4-----5-----6-----7-----8
*LOAD_SEGMENT_SET
$      ssid      lcid      sf      at
      1          1      -1.000      0.0
$-----1-----2-----3-----4-----5-----6-----7-----8
*DEFINE_CURVE_TITLE
$      lcid      sidr      sfa      sfo      offa      offo      dattyp
      1          0      1.000      1.000      0.0      0.0          0
$      abscissa (time)      ordinate (value)
      0.000000E+00      0.000000E+00
      0.460000E-01      0.264000E+09
      1.000000E-01      0.574000E+09
$-----1-----2-----3-----4-----5-----6-----7-----8
*NODE
$      nid      x      y      z      tc      rc
      1 0.000000000E+00 1.099999994E-01 0.000000000E+00
      .
      .
      .
$-----1-----2-----3-----4-----5-----6-----7-----8
*END

```

2. NSM interface element model

```

$-----1-----2-----3-----4-----5-----6-----7-----8
$ LS-DYNA(970) DECK WAS WRITTEN BY: eta/VPG VERSION 3.2
$
$ ENGINEER:
$ PROJECT:
$ UNITS:  M,  KG,  SEC,  N
$ DATE:  Jul  5, 2007 at 14:07:24
$-----1-----2-----3-----4-----5-----6-----7-----8
*KEYWORD
$-----1-----2-----3-----4-----5-----6-----7-----8
*TITLE

$-----1-----2-----3-----4-----5-----6-----7-----8
*CONTROL_BULK_VISCOSITY
$      q1      q2      type
      1.500   0.06000      1
*CONTROL_CONTACT
$  slsfac  rwpnal  islchk  shlthk  penopt  thkchg  orien  enmass
      0.10000      0.0      1      0      1      0      1      0
$  usrstr  usrfrc  nsbcs  interm  xpene  ssthk  ecdt  tiedprj
      0      0      10      0  4.000      0      0      0
$  sfric  dfrc  edc  vfc  th  th_sf  pen_sf
      0.0      0.0      0.0      0.0      0.0      0.0      0.0
$  ignore  frceng  skiprwg  outseg  spotstp  spotdel
      0      0      0      0      0      0
*CONTROL_ENERGY
$  hgen  rwen  slnten  rylen
      2      2      1      1
*CONTROL_HOURLASS
$  ihq  qh
      6  0.05000
*CONTROL_OUTPUT
$  npopt  neecho  nrefup  iaccop  opifs  ipnint  ikedit  iflush
      1      3      0      0      0.0      0      100  5000
$  iprtf
      0
*CONTROL_TERMINATION
$  endtim  endcyc  dtmin  endeng  endmas
      0.02980      0      0.0      0.0      0.0
*CONTROL_TIMESTEP

```

```

$  dtinit    tssfac    isdo    tslimt    dt2ms    lctm    erode    ms1st
      0.0    0.90000      0      0.0 -2.000E-6      0      0      0
$-----1-----2-----3-----4-----5-----6-----7-----8
*DATABASE_GLSTAT
$      dt      binary
      .1000E-2      0
*DATABASE_MATSUM
$      dt      binary
      .1000E-2      0
$-----1-----2-----3-----4-----5-----6-----7-----8
*DATABASE_BINARY_D3PLOT
$  dt/cycl    lcdt      beam      npltc
      .500E-3      0      0      0
$  ioopt
      1
*DATABASE_BINARY_D3DUMP
$  dt/cycl
      100000.0
$-----1-----2-----3-----4-----5-----6-----7-----8
*DATABASE_EXTENT_BINARY
$CardName:EXTENT_1
$  neiph      neips      maxint      strflg      sigflg      epsflg      rltflg      engflg
      2      0      3      1      1      1      1      1
$  cmpflg      ieverp      beamip      dcomp      shge      stssz      n3thdt
      0      0      0      0      0      0      2
$  nintsld
      1
$-----1-----2-----3-----4-----5-----6-----7-----8
*PART
BRICK
$      pid      secid      mid      eosid      hgid      grav      adpopt      tmid
      1      1      1      0      0      0      0      0
*PART
MORTAR
      2      1      2      0      0      0      0      0
*PART
FRP
      3      1      3      0      0      0      0      0
*PART
Adhesive material
      4      1      4      0      0      0      0      0
$-----1-----2-----3-----4-----5-----6-----7-----8
*SECTION_SOLID_TITLE
P-1

```

```

$      secid      elform      aet
      1          1          0
$-----1-----2-----3-----4-----5-----6-----7-----8
*MAT_ADD_EROSION
$CardName:Used by Matl: 1
$      mid      excl
      1 999999.0
$      pfail      sigp1      sigvm      epsp1      epssh      sigth      impulse      failtm
      999999.0 999999.0 999999.0 0.04000 999999.0 999999.0 999999.0 999999.0
*MAT_ADD_EROSION
$CardName:Used by Matl: 2
$      mid      excl
      2 999999.0
$      pfail      sigp1      sigvm      epsp1      epssh      sigth      impulse      failtm
      999999.0 999999.0 999999.0 0.03000 999999.0 999999.0 999999.0 999999.0
$-----1-----2-----3-----4-----5-----6-----7-----8
*MAT_SOIL_AND_FOAM_FAILURE_TITLE
Brick
$      mid      ro      g      bulk      a0      a1      a2      pc
      1 2400.000 .2200E+11 .1800E+12 .5555E+14 .2112E+8 2.008 -.710E+7
$      vcr      ref
      0.0      0.0
$      eps1      eps2      eps3      eps4      eps5      eps6      eps7      eps8
      0.0 -.7500E-4 -.1880E-3 -.3040E-3 -.000949 -.009490 0.0 0.0
$      eps9      eps10
      0.0      0.0
$      p1      p2      p3      p4      p5      p6      p7      p8
      0.0 3950000. 9880000. .1600E+8 .5000E+9 .5000E+10 0.0 0.0
$      p9      p10
      0.0      0.0
*MAT_SOIL_AND_FOAM_FAILURE_TITLE
Mortar
$      mid      ro      g      bulk      a0      a1      a2      pc
      2 2400.000 .2200E+9 .1328E+10 .4156E+12 1827000. 2.008 -800000.0
$      vcr      ref
      0.0      0.0
$      eps1      eps2      eps3      eps4      eps5      eps6      eps7      eps8
      0.0 -0.00940 -0.01650 -0.23700 -2.370 0.0 0.0 0.0
$      eps9      eps10
      0.0      0.0
$      p1      p2      p3      p4      p5      p6      p7      p8
      0.0 3950000. 6140000. .9880E+8 .9880E+9 0.0 0.0 0.0
$      p9      p10
      0.0      0.0

```

*MAT_ELASTIC_TITLE

CFRP

\$	mid	ro	e	pr	da	db
	3	1500.000	.1600E+12	0.15000	0.0	0.0

*MAT_ARUP_ADHESIVE

\$	mid	ro	e	pr	TENMAX	GCTEN	SHRMAX	GCSHR
	4	2400.000	0.28E+09	0.25000	0.85E+06	1.0E+20	14.50E+06	5.0E+03

\$	PWRT	PWRS	SHRP	SHT_SL	EDOT0	EDOT2
	2	2	0.1	0	1	0

\$	1	2	3	4	5	6	7	8

*HOURLGLASS_TITLE

\$	hgid	ihq	qm	ibq	q1	q2	qb	qw
	1	6	0.05000	0	1.500	0.06000	0.0	0.0

\$	1	2	3	4	5	6	7	8

*DAMPING_GLOBAL

\$	lcid	valdmp	stx	sty	stz	srx	sry	srz
	0	30.000	0.0	0.0	0.0	0.0	0.0	0.0

\$	1	2	3	4	5	6	7	8

*SET_NODE_LIST_TITLE

NODE SET_1

\$	sid	da1	da2	da3	da4
	1				

\$	nid1	nid2	nid3	nid4	nid5	nid6	nid7	nid8
	1	4	3	2	10	9	14	13

.

.

.

\$	1	2	3	4	5	6	7	8

*SET_PART_LIST_TITLE

PART SET_1

\$	sid	da1	da2	da3	da4
	1				

\$	pid1	pid2	pid3	pid4
	1	2	3	4

\$	1	2	3	4	5	6	7	8

*SET_SEGMENT_TITLE

SEGMENT SET_1

\$	sid	da1	da2	da3	da4
	1				

\$	N1	N2	N3	N4
	12225	12226	12227	12228
	12229	12225	12228	12230
	12231	12229	12230	12232

\$-----1-----2-----3-----4-----5-----6-----7-----8

*BOUNDARY_SPC_SET_ID

\$ id

1

\$	nsid	cid	dofx	dofy	dofz	dofrx	dofry	dofrz
	1	0	1	1	1	0	0	0

*BOUNDARY_SPC_SET_ID

\$ id

2

\$	nsid	cid	dofx	dofy	dofz	dofrx	dofry	dofrz
	2	0	1	1	1	0	0	0

*BOUNDARY_SPC_SET_ID

\$ id

3

\$	nsid	cid	dofx	dofy	dofz	dofrx	dofry	dofrz
	3	0	1	1	0	0	0	0

\$-----1-----2-----3-----4-----5-----6-----7-----8

*CONTACT_AUTOMATIC_SINGLE_SURFACE_ID

\$ cid

1

\$	ssid	msid	sstyp	mstyp	sboxid	mboxid	spr	mpr
	1	0	2	0	0	0	0	0

\$	fs	fd	dc	vc	vdc	penchk	bt	dt
	0.60000	0.40000	100.000	0.0	20.000	1	0.0	1.000E+21

\$	sfs	sfm	sst	mst	sfst	sfmt	fsf	vsf
	1.000	1.000	0.0	0.0	1.000	1.000	1.000	1.000

\$	soft	sofscf	lcidab	maxpar	sbopt	depth	bsort	frcfreq
	2	0.50000	0	1.200	5.000	5	0	1

\$	penmax	thkopt	shlthk	snlog	isym	i2d3d	slidthk	slidstf
	0.40000	0	0	0	1	1	0.0	0.0

\$ igap ignore

1 0

\$-----1-----2-----3-----4-----5-----6-----7-----8

*LOAD_SEGMENT_SET

\$	ssid	lcid	sf	at
	1	1	-1.00	0.0

\$-----1-----2-----3-----4-----5-----6-----7-----8

*DEFINE_CURVE_TITLE

\$	lcid	sidr	sfa	sfo	offa	offo	dattyp
	1	0	1.00	1.00	0.0	0.0	0

\$	abscissa (time)	ordinate (value)
	0.000000E+00	0.000000E+00
	1.800000E-03	2.125000E+08

2.500000E-03	2.500000E+08
3.300000E-03	3.208300E+08
1.030000E-02	8.583300E+08
1.730000E-02	1.500000E+09
2.080000E-02	1.795800E+09
2.140000E-02	1.695800E+09
2.400000E-02	1.900000E+09
2.680000E-02	2.083300E+09
2.730000E-02	1.979200E+09
2.890000E-02	2.020800E+09
2.920000E-02	1.929200E+09
2.980000E-02	1.887500E+09

\$-----1-----2-----3-----4-----5-----6-----7-----8

*NODE

\$	nid	x	y	z	tc	rc
	1	0.000000000E+00	1.099999994E-01	0.000000000E+00		
.						
.						
.						

\$-----1-----2-----3-----4-----5-----6-----7-----8

*END

APPENDIX C: NUMERICAL MODELS OF THE ALUMINIUM FOAM PROTECTED URM WALLS

```

$-----1-----2-----3-----4-----5-----6-----7-----8
$ LS-DYNA(971) DECK WAS WRITTEN BY: eta/VPG VERSION 3.3
$
$ ENGINEER:
$ PROJECT:
$   UNITS:  M, KG, SEC, N
$   DATE:  Aug  5, 2008 at 18:55:08
$-----1-----2-----3-----4-----5-----6-----7-----8
*KEYWORD
$-----1-----2-----3-----4-----5-----6-----7-----8
*TITLE
Distinct_W4_1Foam_40mm_A356SiC040_SD2.7_Interface
$-----1-----2-----3-----4-----5-----6-----7-----8
*CONTROL_BULK_VISCOSITY
$      q1      q2      type
      1.500    0.06000      1
*CONTROL_CONTACT
$  slsfac  rwpnal  islchk  shlthk  penopt  thkchg  orien  enmass
      0.80000      0.0      1      0      1      0      1      0
$  usrstr  usrfrc  nsbcs  interm  xpene  ssthk  ecdt  tiedprj
      0      0      10      0  4.000      0      0      0
$  sfrc  dfrc  edc  vfc  th  th_sf  pen_sf
      0.0      0.0      0.0      0.0      0.0      0.0      0.0
$  ignore  frceng  skiprwg  outseg  spotstp  spotdel  spothin
      0      0      0      0      0      0      0.0
$  isym  nserod  rwgaps  rwgdt  rwksf  icov  swradf  ithoff
      0      0      0      0.0      0.0      0      0.0      0
*CONTROL_ENERGY
$  hgen  rwen  slnten  rylen
      2      2      2      2
*CONTROL_HOURLASS
$  ihq  qh
      6  0.05000
*CONTROL_OUTPUT
$  npopt  neecho  nrefup  iaccop  opifs  ipnint  ikedit  iflush
      1      3      0      0      0.0      0      100      5000
$  iptf  ierode  tet10  msgmax  ipcurv
      0      0      2      0      0
*CONTROL_TERMINATION

```

```

$   endtim   endcyc   dtmin   endeng   endmas
      0.04000       0       0.0       0.0       0.0

*CONTROL_TIMESTEP
$   dtinit   tssfac   isdo   tslimt   dt2ms   lctm   erode   ms1st
      0.0   0.90000       0       0.0 -.2700E-6       0       0       0

$   dt2msf   dt2mslc   imslc
      0.0       0       0

$-----1-----2-----3-----4-----5-----6-----7-----8
*DATABASE_GLSTAT
$   dt   binary   lcur
  .1000E-2       1       0

*DATABASE_MATSUM
$   dt   binary   lcur
  .1000E-2       1       0

$-----1-----2-----3-----4-----5-----6-----7-----8
*DATABASE_BINARY_D3PLOT
$   dt/cycl   lcdt   beam   npltc
  .1000E-2       0       0       0

$   ioopt
      1

*DATABASE_BINARY_D3DUMP
$   dt/cycl
      200000.0

$-----1-----2-----3-----4-----5-----6-----7-----8
*DATABASE_EXTENT_BINARY
$CardName:EXTENT_1
$   neiph   neips   maxint   strflg   sigflg   epsflg   rltflg   engflg
      0       0       3       1       1       1       1       1

$   cmpflg   ieverp   beamip   dcomp   shge   stssz   n3thdt   ialemat
      0       0       0       0       0       0       2       0

$   nintsld   pkp_sen   sclp   blank   msscl   therm
      1       0       0.0       0       0

$-----1-----2-----3-----4-----5-----6-----7-----8
*PART
BRICK
$   pid   secid   mid   eosid   hgid   grav   adpopt   tmid
      1       1       1       0       1       0       0       0

*PART
MORTAR
      2       1       2       0       2       0       0       0

*PART
Foam
      3       2       3       0       3       0       0       0

*PART

```

Steel

4	1	4	0	4	0	0	0
---	---	---	---	---	---	---	---

*PART

Interface

6	1	6	0	6	0	0	0
---	---	---	---	---	---	---	---

\$-----1-----2-----3-----4-----5-----6-----7-----8

*SECTION_SOLID_TITLE

P-1

\$	secid	elform	aet
	1	1	0

*SECTION_SOLID_TITLE

S0000003

\$	secid	elform	aet
	2	9	0

\$-----1-----2-----3-----4-----5-----6-----7-----8

*MAT_ADD_EROSION

\$CardName:USED BY MATL: 1

\$	mid	excl	mxpres	mnpeps
	1	999999.0	0.0	0.0

\$	pfail	sigp1	sigvm	epsp1	epssh	sigth	impulse	failtm
	999999.0	999999.0	999999.0	0.03000	999999.0	999999.0	999999.0	999999.0

*MAT_ADD_EROSION

\$CardName:USED BY MATL: 2

\$	mid	excl	mxpres	mnpeps
	2	999999.0	0.0	0.0

\$	pfail	sigp1	sigvm	epsp1	epssh	sigth	impulse	failtm
	999999.0	999999.0	999999.0	0.03000	999999.0	999999.0	999999.0	999999.0

\$-----1-----2-----3-----4-----5-----6-----7-----8

*MAT_SOIL_AND_FOAM_TITLE

Brick

\$	mid	ro	g	bulk	a0	a1	a2	pc
	1	2400.000	.2200E+11	.1800E+12	.5555E+14	.2112E+8	2.008	-7100000.

\$	vcr	ref
	0.0	0.0

\$	eps1	eps2	eps3	eps4	eps5	eps6	eps7	eps8
	0.0	-.7500E-4	-.1880E-3	-.3040E-3	-0.00949	-0.09490	0.0	0.0

\$	eps9	eps10
	0.0	0.0

\$	p1	p2	p3	p4	p5	p6	p7	p8
	0.0	3950000.	9880000.	.1600E+8	.5000E+9	.5000E+10	0.0	0.0

\$	p9	p10
	0.0	0.0

*MAT_SOIL_AND_FOAM_TITLE

Mortar

```

$      mid      ro      g      bulk      a0      a1      a2      pc
      2 2400.000 .2200E+9 .1328E+10 .4156E+12 1827000. 2.008 -800000.0
$      vcr      ref
      0.0      0.0
$      eps1      eps2      eps3      eps4      eps5      eps6      eps7      eps8
      0.0 -0.00940 -0.01650 -0.23700 -2.370 0.0 0.0 0.0
$      eps9      eps10
      0.0      0.0
$      p1      p2      p3      p4      p5      p6      p7      p8
      0.0 3950000. 6140000. .9880E+8 .9880E+9 0.0 0.0 0.0
$      p9      p10
      0.0      0.0
*MAT_MODIFIED_HONEYCOMB_TITLE
Foam_15%
$      MID      RO      E      PR      SIGY      VF      MU      BULK
      3 4.069E+02 7.10E+10 0.33 3.22E+08 0.23418 0.05 0.0
$      LCA      LCB      LCC      LCS      LCAB      LCBC      LCCA      LCSR
      6      6      6      7      7      7      7
$      EAAU      EBBU      ECCU      GABU      GBCU      GCAU      AOPT
4.266E+08 4.266E+08 4.266E+08 1.5997E+8 1.5997E+8 1.5997E+8 0.0
$      XP      YP      ZP      A1      A2      A3
$      D1      D2      D3      TSEF      SSEF
      0.77 0.77
*MAT_PLASTIC_KINEMATIC_TITLE
Steel
$      mid      ro      e      pr      sigy      etan      beta
      4 7830.000 .2070E+12 0.28000 .3100E+9 .7630E+9 0.0
$      src      srp      fs      vp
      40.000 5.000 0.75000 0.0
*MAT_ARUP_ADHESIVE_TITLE
M0000006
$      mid      ro      e      pr      tenmax      gcten      shrmax      gcshr
      6 2400.000 0.08E+09 0.29000 1.84E+06 2.0E+03 5.87E+06 0.25E+03
$      pwrt      pwrs      shrp      sht_sl      edot0      edot2      blank      xedge
      2 2.000 0.0 0.0 1.000 0.0 0 0.0
$-----1-----2-----3-----4-----5-----6-----7-----8
*HOURGLASS_TITLE
$      hgid      ihq      qm      ibq      q1      q2      qb/vdc      qw
      1      6 0.01000 0 1.500 0.06000 0.0 0.0
*HOURGLASS_TITLE
$      hgid      ihq      qm      ibq      q1      q2      qb/vdc      qw

```

```

      2      6  0.01000      0    1.500  0.06000      0.0      0.0
*HOURLASS_TITLE

$   hgid    ihq      qm      ibq      q1      q2    qb/vdc      qw
      3      2  0.10000      0    1.500  0.06000      0.0      0.0
*HOURLASS_TITLE

$   hgid    ihq      qm      ibq      q1      q2    qb/vdc      qw
      4      5  0.10000      0    1.500  0.06000      0.0      0.0
*HOURLASS_TITLE

$   hgid    ihq      qm      ibq      q1      q2    qb/vdc      qw
      6      6  1.000      0    1.500  0.06000      0.0      0.0
$---+---1---+---2---+---3---+---4---+---5---+---6---+---7---+---8
*DAMPING_GLOBAL
$   lcid    valdmp      stx      sty      stz      srx      sry      srz
      0    30.000      0.0      0.0      0.0      0.0      0.0      0.0
$---+---1---+---2---+---3---+---4---+---5---+---6---+---7---+---8
*SET_NODE_LIST_TITLE
NODE SET_1
$   sid      da1      da2      da3      da4
      1
$   nid1      nid2      nid3      nid4      nid5      nid6      nid7      nid8
    20443      20446      20448      20450      20452      20454      20444      20447
.
.
.
$---+---1---+---2---+---3---+---4---+---5---+---6---+---7---+---8
*BOUNDARY_SPC_SET_ID
$   id
      1
$   nsid      cid      dofx      dofy      dofz      dofrx      dofry      dofrz
      1      0      0      1      0      0      0      0
$---+---1---+---2---+---3---+---4---+---5---+---6---+---7---+---8
*CONSTRAINED_GLOBAL
$   tc      rc      dir      x      y      z
      1      5      1      0.0      0.0      0.0
      3      4      3      0.0      0.0      0.0
$---+---1---+---2---+---3---+---4---+---5---+---6---+---7---+---8
*CONTACT_AUTOMATIC_SINGLE_SURFACE_ID
$   cid
      1
$   ssid      msid      sstyp      mstyp      sboxid      mboxid      spr      mpr
      0      0      5      0      0      0      0      0

```

```

$      fs      fd      dc      vc      vdc      penchk      bt      dt
      0.90000      0.0      0.0      0.0      0.0      1      0.0 .1000E+21
$      sfs      sfm      sst      mst      sfst      sfmt      fsf      vsf
      1.000      1.000      0.0      0.0      1.000      1.000      1.000      1.000
$      soft      sofscf      lcidab      maxpar      sbopt      depth      bsort      frcfrq
      2      0.80000      0      1.200      0.0      5      0      1
$      penmax      thkopt      shlthk      snlog      isym      i2d3d      sldthk      sldstf
      4.000      0      0      0      0      1      0.0      0.0
$      igap      ignore      dprfac      dtstif      blank      blank      flangl
      1      0 .1000E-2      0.0      0.0
$-----1-----2-----3-----4-----5-----6-----7-----8
*LOAD_SEGMENT_SET_ID
$      id
      1
$      ssid      lcid      sf      at      dt
      1      1      1.000      0.0      0.0
$-----1-----2-----3-----4-----5-----6-----7-----8
*DEFINE_COORDINATE_NODES_TITLE
COORDINATE 00000001
$      cid      n1      n2      n3      flag      dir
      1      156601      61249      173539      0      X
$-----1-----2-----3-----4-----5-----6-----7-----8
*DEFINE_CURVE_TITLE
LCur_1
$      lcid      sidr      sfa      sfo      offa      offo      dattyp
      1      0      1.000      1.000      0.0      0.0      0
$      abscissa (time)      ordinate (value)
      0.000000E+00      3.000000E+06
      0.020000E-01      0.000000E+00
*DEFINE_CURVE_TITLE
LCA
$      lcid      sidr      sfa      sfo      offa      offo      dattyp
      2      0      1.000      1.000      0.0      0.0      0
$      abscissa (time)      ordinate (value)
      -0.266667E-02      -0.320000E+07
      0.000000E+00      0.000000E+00
      0.800000E-02      0.420000E+07
      0.200000E-01      0.200000E+07
      0.680000E+00      0.400000E+07
      0.850000E+00      0.120000E+11
*DEFINE_CURVE_TITLE
LCB
$      lcid      sidr      sfa      sfo      offa      offo      dattyp
      3      0      1.000      1.000      0.0      0.0      0

```

```

$   abscissa (time)   ordinate (value)
      -0.284615E-02    -0.370000E+07
      0.000000E+00     0.000000E+00
      0.714000E-02     0.500000E+07
      0.120000E-01     0.500000E+07
      0.680000E+00     0.500000E+07
      0.850000E+00     0.120000E+11
*DEFINE_CURVE_TITLE
LCC
$   lcld      sidr      sfa      sfo      offa      offo      dattyp
      4        0      1.000      1.000      0.0      0.0      0
$   abscissa (time)   ordinate (value)
      -0.333333E-02    -0.500000E+07
      0.000000E+00     0.000000E+00
      0.706000E-02     0.600000E+07
      0.120000E-01     0.600000E+07
      0.680000E+00     0.600000E+07
      0.850000E+00     0.120000E+11
*DEFINE_CURVE_TITLE
LCS
$   lcld      sidr      sfa      sfo      offa      offo      dattyp
      5        0      1.000      1.000      0.0      0.0      0
$   abscissa (time)   ordinate (value)
      -0.300000E-02    -0.210000E+07
      -0.228200E-02    -0.210000E+07
      0.000000E+00     0.000000E+00
      0.113000E-02     0.100000E+07
      0.228200E-02     0.210000E+07
      0.300000E-02     0.210000E+07
*DEFINE_CURVE
$   LCID      SIDR      SCLA      SCLO      OFFA      OFFO
      6        0      1.0      1.0
$           A1           O1
      0.000000      2.531531E+06
      0.033184      2.531531E+06
      0.100000      3.705721E+06
      0.150000      4.328555E+06
      0.200000      4.538712E+06
      0.250000      4.512386E+06
      0.300000      4.533725E+06
      0.350000      4.854431E+06
      0.400000      5.611365E+06
      0.450000      6.802155E+06
      0.500000      8.318809E+06

```

```

0.550000      1.003933E+07
0.600000      1.197732E+07
0.650000      1.448963E+07
0.700000      1.854196E+07
0.765820      2.959121E+07
*DEFINE_CURVE
$   LCID      SIDR      SCLA      SCLO      OFFA      OFFO
      7        0      1.0 0.5773503
$           A1           O1
0.000000      2.531531E+06
0.033184      2.531531E+06
0.100000      3.705721E+06
0.150000      4.328555E+06
0.200000      4.538712E+06
0.250000      4.512386E+06
0.300000      4.533725E+06
0.350000      4.854431E+06
0.400000      5.611365E+06
0.450000      6.802155E+06
0.500000      8.318809E+06
0.550000      1.003933E+07
0.600000      1.197732E+07
0.650000      1.448963E+07
0.700000      1.854196E+07
0.765820      2.959121E+07
$-----1-----2-----3-----4-----5-----6-----7-----8
*NODE
$   nid      x      y      z      tc      rc
10430 6.250000000E-01 2.749999985E-02 0.000000000E+00
.
.
.
$-----1-----2-----3-----4-----5-----6-----7-----8
*END

```