Security Evaluation and Design of Structures Subjected to Blast Loading
João Miguel Pereira

Security Evaluation and Design of Structures Subjected to Blast Loading

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ABSTRACT

The work presented in this thesis was developed at the Department of Civil Engineering of University of Minho. This work involves experimental and numerical campaigns and intends to give a contribution for a better understanding of the effect of explosions.

Blast loading is a subject of much actuality and considerable lack of expertise. Europe has never been so rich and safe, where the violent years of the first half of the 20th century lead to an unprecedented period of peace and stability. Despite the terrorist decades, e.g. connected to ETA and IRA in Europe, the attacks of Madrid (2004), London (2005) and worldwide (New York, Oklahoma, Mumbai) had a major psychological effect in the societies. Clearly, the understanding about the effect of blast loading in structures and their subsystems saves lives and reduces damage in buildings.

The Buncefield explosion (2005) resulted in tremendous damage to the outlying area and huge fires involving 23 large oil fuel tanks. Experimental and finite element analyses are carried out for the static and dynamic response of lightweight metal boxes that are similar to the steel junction boxes on the site of this explosion. During the Buncefield Explosion Mechanism Phase I research, the lightweight steel junction boxes on the site located within the area covered by the gas cloud are compared with similar boxes tested under a range of different loading conditions using hydrostatic pressure, gas explosions and high explosive charges. The residual plastic deformations for these boxes are recorded and used to validate a finite element based modelling approach. The predicted pressure-time history is in reasonably good agreement with the measurements. Further parametric studies are then conducted to produce iso-deformation curves which can be used in accident investigations to back track the blast loading from structure deformations.

Investigation of the dynamic properties of construction materials is critical for structural engineering. The strain rate effect influences the properties on most constructions materials. This effect on materials such as concrete or steel has been intensively investigated. However, such studies on masonry materials such as clay bricks cannot be found in the open literature easily. Understanding the strain rate effect on masonry materials is important for proper modelling and design of masonry structures under high velocity impacts or blast loads. This work aims to study the behaviour of masonry and its individual components in compression at different strain rates. A Drop Weight Impact Machine is used at different heights and weights introducing different levels of strain rate. The strain rate effect on the compressive strength, Young’s modulus, strain at peak strength and compressive fracture energy is determined from the experimental results. Empirical relations of dynamic increase factors (DIF) for these material properties are also presented.
The vulnerability of the masonry envelop under blast loading is considered critical due to the risk of loss of lives. The dynamic behaviour of masonry infill walls subjected to dynamic out-of-plane loading is experimentally investigated in this work. In the present study water plastic containers, having in its centre a detonator inside a cylindrical explosive charge, are used as confined underwater blast wave generators (WBWG). Tests are performed in unreinforced walls with 1.7 by 3.5 m, which are 1:1.5 scaled, and the results presented. These results are used to calibrate numerical models using ABAQUS Explicit dynamics, allowing a detailed study on this kind of masonry panels under dynamic out-of-plane loading in the form of parametric studies. Two different reinforcement solutions are studied in the numerical model and the results are presented. The results are used to create pressure-impulse (P-I) diagrams which can help the designer to estimate the response of these elements under different loading conditions.

Protection is not an absolute concept and there is a level of protection where the cost of the protection provided with respect to the cost of potential loss is in balance. On one hand, protection cannot offer full guarantee of safety and, on the other hand, too much protection is a waste of resources with regard to what is expected to be saved. The purpose of protective construction is to improve the probability of survival of people and other contents in a given facility for a given threat, to an adequate level. In order to improve this probability, one must first understand the threat and accordingly analyse the facility. In this work risk assessment is addressed and applied to a large Public Transport Operator. This assessment allows identifying potentially critical infrastructure, which are studied and its structural security is evaluated for different scenarios regarding blast loading.

**Keywords:** Blast loading; Dynamic response; FE modelling; LS-DYNA; Masonry; Impact; Drop Weight; Strain rate; DIF; Infill walls; Out-of-plane loading; WBWG; Risk assessment; COUNTERACT; Structural security; ABAQUS.
RESUMO

O trabalho apresentado nesta tese foi desenvolvido no Departamento de Engenharia Civil da Universidade do Minho. Este trabalho envolve campanhas experimentais e numéricas, e pretende ser uma contribuição para uma melhor compreensão do efeito das explosões.

As explosões são um tema atual e de conhecimento muito reduzido. A Europa nunca esteve tão rica e segura, sendo que os anos violentos da primeira metade de século 20 conduziram a um período sem precedentes de paz e estabilidade. Apesar de décadas de terrorismo, por exemplo da ETA e do IRA na Europa, os ataques de Madrid (2004), Londres (2005) e em todo o mundo (Nova York, Oklahoma, Mumbai) têm tido um impacto psicológico muito elevado nas sociedades. Claramente, a compreensão sobre o efeito da ação das explosões nas estruturas e nos seus subsistemas salva vidas e reduz os danos nas construções.

A explosão de Buncefield em 2005 resultou num dano impressionante nas proximidades e fogos imensos em 23 grandes depósitos de combustível. Campanhas experimentais e numéricas são executadas para estudar o comportamento dinâmico de caixas metálicas semelhantes às encontradas em Buncefield. Durante a investigação do incidente de Buncefield, as caixas presentes na área coberta pela nuvem de vapor são comparadas com caixas semelhantes sujeitas a diferentes tipos de ações, usando pressão hidrostática, explosões de gás e explosivos de grau militar. As deformações permanentes são registadas e usadas para validar modelos numéricos com recurso ao Método dos Elementos Finitos. A previsão dos perfis de pressão é adequada aos dados obtidos durante a investigação do acidente. Os modelos numéricos obtidos são usados para criar curvas de iso-dano que podem ser usadas para auxiliar em investigações pós-acidente relacionando a deformação nessas subestruturas com o perfil de pressões atuante.

A investigação das propriedades dinâmicas dos materiais de construção é considerada crítica para a engenharia. O efeito das velocidades de deformação influencia as propriedades da maior parte dos materiais de construção. Este efeito tem sido estudado em betão e aço. No entanto, estudos sobre este efeito em alvenaria, e seus componentes, são reduzidos na literatura. A compreensão destes efeitos em alvenaria é crucial para uma modelação e dimensionamento adequado destes materiais quando sujeitos a impactos ou explosões. Este trabalho pretende estudar o comportamento da alvenaria, e dos seus componentes individuais, em compressão a diferentes velocidades de deformação. Para o efeito uma torre de queda é usada com diferentes alturas e pesos de impacto, introduzindo no sistema diferentes velocidades de deformação. O efeito das velocidades de deformação é determinado pelos resultados experimentais, e estudo para a resistência à compressão, módulo de elasticidade, extensão em compressão máxima e energia de fratura em compressão. Relações empíricas sob a forma de fatores de incremento dinâmico (FID) são apresentadas para estas propriedades mecânicas.
A vulnerabilidade da envolvente em alvenaria dos edifícios quando sujeita a explosões é considerada crítica devido ao risco de perdas humanas. O comportamento de paredes de enchimento quando sujeitas a ações dinâmicas fora do seu plano é investigado neste trabalho. São usados reservatórios de água com uma carga explosiva no seu centro para atuar como geradores de onda de choque. São executados testes em paredes de enchimento de alvenaria com 1.7×3.5 metros (escala de 1:1.5). Os resultados obtidos são usados para calibrar modelos numéricos com recurso ao software ABAQUS Explicit. Isto permite uma análise alargada do comportamento destes elementos quando sujeitos a ações dinâmicas fora do plano, sob a forma de estudos paramétricos. Duas soluções de reforço são estudadas numericamente e os resultados apresentados. Os resultados obtidos nas tarefas anteriores são usados para criar diagramas de pressão-impulso (P-I) que auxiliam os técnicos que pretendam estimar a resposta deste tipo de paredes quando sujeitas a diferentes tipos de ação.

A proteção não é um conceito absoluto e há um nível de proteção em que o custo da proteção em relação às perdas possíveis está em equilíbrio. Por um lado, não é possível garantir proteção e segurança absoluta, por outro lado, demasiada proteção pode ser um desperdício de recursos em relação ao que se espera proteger. O objetivo de proteção, no caso de estruturas, é melhorar a probabilidade de sobrevivência dos seus ocupantes e seu conteúdo, para uma ameaça específica, obtendo uma probabilidade adequada. Para isto ser possível é necessária uma comprensão detalhada da ameaça e da estrutura. Neste trabalho a avaliação de risco é estudada e uma metodologia é aplicada a um Operador de Transportes Públicos. Esta avaliação de risco permite identificar infraestruturas críticas, tendo sido avaliada a segurança estrutural de uma destas a diferentes cenários de explosão.

Palavras-chave: Alvenaria; Impacto; Torre de queda; FID; Paredes de enchimento; Carregamento fora do plano; WBWG; Avaliação de risco; COUNTERACT; Segurança estrutural; ABAQUS.
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Chapter 1

1 INTRODUCTION

Many countries have been victims of terrorism on a grand scale. Bombs have exploded inside and near buildings in several countries, causing human casualties and very high economic losses. As a result, these events have created a concern about the capacity of countries to protect themselves against such actions. Particularly relevant aspects are the ability to protect assets against car-bombs and the structural integrity of existing buildings.

Despite decades of terrorism in Europe, linked for example to ETA and IRA, the recent attacks in Madrid (2004) and London (2005) had a particularly severe psychological effect on societies. In fact, a perceived terrorist threat has become an important issue for EU citizens and, in an Eurobarometer survey in April/May 2007, 81% of the EU population would like to have a joint decision of the EU in the area of prevention against terrorism.
An explosion in the vicinity or inside a building can have catastrophic effects, damaging and destroying parts of the building, windows, and walls, forcing the closure of services, and causing fire and smoke that can injure or even cause the death of its occupants. The effects of an explosion in structural members can produce both local and global responses associated with different modes of failure. The type of structural response depends mainly on the explosive charge, the orientations of the structural element relative to the direction of the wave propagation and the support conditions. In general, the failure modes associated with this type of action can be through bending and shear.

Risk reduction has to include the public understanding of the need for preventive measures and structural strengthening for critical infrastructures. The perception of architects and engineers for measures of structural integrity against collapse must be incorporated in the early stage of the design process. This theme is perhaps important enough to be considered in many new projects, depending on the consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post-incident period, and on the social and economic consequences of collapse. Although there is some information and experience on the effect of blast loading on buildings it seems to be difficult to integrate this knowledge in the regulations and practices of construction.

1.1 Objectives

The present work intends to give insight on several topics related to blast loading and structural response to impulsive loading. The first objective of this work is to provide understanding on the explosion phenomenon and its interaction with structures, for both terrorist actions and accidental explosions. It is also intended to provide empirical tools to help assessing the blast load parameters for a post-disaster scenario regarding industrial accidents or terrorist attacks, allowing the research team responsible for the accident investigation to estimate the magnitude of the explosion through simple tools.

Impact and blast loading introduces in the materials high strain rates which affect the material properties. Quantifying this influence on masonry and its components is also
one of the objectives of this work. Regarding the response of structural elements under blast loading, it is intended to develop a new test setup able to test wall panels under out-of-plane blast loading. From the results obtained with the out-of-plane blast test it is also intended to create empirical tools to help practitioners to assess the behaviour and design masonry infill walls under blast loading.

Two different case studies on risk assessment and security evaluation are expected. It is intended to identify the elements, inside a public transportation network, with the highest risk associated to a selected range of threats. From the group identified previously it is intended to perform a structural evaluation under different explosion scenarios. Lastly, this work intends to introduce these topics at a national level, hoping they could lead to future developments in this field.

1.2 OUTLINE OF THE THESIS

Besides this introductory chapter, this thesis is composed of six additional chapters, each contributing to a particular subject related to this field.

Chapter 2 gathers the most relevant concepts and theories needed to understand the work done previously in this field. This chapter addresses definitions on explosions, explosives, concept of blast waves, and interaction between blast waves and structure. The chapter also presents available methods of analysis, together with the influence of impulsive loading in the material behaviour as well as the most important design codes available.

Chapter 3 intends to give insight on the determination of blast loading parameters. The work presented in this chapter was performed in collaboration with Kingston University London, UK and the Fire and Explosion Study Centre in London, UK in the framework of the Buncefield Major Incident Investigation. The most relevant aspect is to study the possibility of estimating the magnitude of explosions by using pressure indicators or sub-structures. Using common objects present within and in the neighbourhood of most of industrial buildings, such as steel switch boxes, cars or oil drums as reference samples, it is possible to estimate the overpressures in case of an explosion by building
a relation between the residual permanent deformation with pressure/impulse responsible for such deformation. Pressure-impulse (P-I) can help future accident investigations by backtracking the overpressures and impulses according to the observed damage in those specific sub-structures.

Chapter 4 presents an experimental campaign on the influence of the strain rate on the mechanical properties of masonry and its components. The work presented in this chapter was developed in collaboration with the Mechanical Engineering Department of University of Minho, Portugal. The most relevant issue is to study the influence of the strain rate effect on handmade clay brick and mortar. Compression tests were performed using a Drop Weight (DW) impact machine. Empirical relations using Dynamic Increase Factors (DIF) were developed and can be used to improve material models used by advanced non-linear analysis software for proper prediction of the structural response under impulsive loading.

Chapter 5 present a newly developed test setup for dynamic out-of-plane loading using underWater Blast Wave Generators (WBWG) as loading source. The work presented in this chapter was performed in collaboration with the Laboratory for Energetics and Detonics (LEDap), Condeixa-a-Nova, Portugal. This new test setup is presented including the development of sensors and acquisition systems. This chapter also studies the behaviour of masonry infill walls subjected to blast loading. Three different masonry walls are studied. Unreinforced masonry infill wall and two different reinforcement solutions. These solutions have been studied previously for seismic action mitigation. Lastly, empirical tools are presented to help designers to make informed decision on the use of these elements under blast loading.

Chapter 6 present two case studies. The work presented in this chapter was performed in collaboration with REFER, Portugal. A risk assessment due to terrorist actions involving explosions is performed. The selected infrastructure for performing this assessment involved one of the largest transportation operators in Portugal and the elements in the infrastructure with the highest associated risk are highlighted. From the group of elements with the highest associated risk, an element is selected for safety evaluation under blast loading. Through numerical analysis, different explosion scenarios are studied and the behaviour of the structure is presented.
Chapter 7 summarizes the final remarks and conclusions from the previous chapters. Some developments regarding future research work are also presented, together with suggestions regarding possible research topics in this field of knowledge.
Quite often we see on the news the effects of explosions. Explosions threaten people’s lives; threaten the integrity of buildings, industry, transportation, communications and services. Explosions can result from human action (Table 2.1) or unfortunate accidents (Table 2.2) and can vary from a nuclear explosion to a gun firing.

The technical community is, in general, not prepared to deal with the design or evaluation of structures when subjected to this kind of actions. This happens because the design codes very rarely include explosions as a design load and the dynamic effects of explosions on buildings have been studied in a small number of laboratories. The cost involved in researching and experimenting on these actions on structures is very high and in most countries left to the armed forces, which due to security restrictions do not publish their research. Due to the global increase of explosions, either by terrorism or accidents, the need of research and development in this area is obvious. However the focus of this research will probably shift from the military requirements, which is the
present situation, to civil requirements, in order to serve the needs of the present and the future.

Table 2.1 – Examples of recent terrorist attacks using explosions.

<table>
<thead>
<tr>
<th>Date</th>
<th>Location</th>
<th>Target</th>
</tr>
</thead>
<tbody>
<tr>
<td>March 2004</td>
<td>Madrid, Spain</td>
<td>Passenger trains and train stations</td>
</tr>
<tr>
<td>July 2005</td>
<td>London, UK</td>
<td>Passenger bus and underground trains</td>
</tr>
<tr>
<td>October 2008</td>
<td>Pamplona, Spain</td>
<td>University of Navarra</td>
</tr>
<tr>
<td>July 2011</td>
<td>Oslo, Norway</td>
<td>Norwegian Prime Minister’s office</td>
</tr>
<tr>
<td>November 2012</td>
<td>Tel Aviv, Israel</td>
<td>Passenger bus near the Defence Ministry</td>
</tr>
<tr>
<td>April 2013</td>
<td>Boston, USA</td>
<td>Finish line of the 2013 Boston Marathon</td>
</tr>
</tbody>
</table>

Table 2.2 – Examples of major industrial accidents.

<table>
<thead>
<tr>
<th>Date</th>
<th>Location</th>
<th>Type of fuel</th>
</tr>
</thead>
<tbody>
<tr>
<td>1975</td>
<td>Beek, Netherlands</td>
<td>Propylene</td>
</tr>
<tr>
<td>1989</td>
<td>Ufa, Russia</td>
<td>LPG</td>
</tr>
<tr>
<td>1991</td>
<td>Saint Herblain, France</td>
<td>Petrol</td>
</tr>
<tr>
<td>1992</td>
<td>Brenham, USA</td>
<td>C2-C4</td>
</tr>
<tr>
<td>2005</td>
<td>Buncefield, UK</td>
<td>Petrol</td>
</tr>
<tr>
<td>2005</td>
<td>Texas City, USA</td>
<td>C5-C7</td>
</tr>
</tbody>
</table>

This chapter gathers the most relevant concepts and theories needed to understand the work done in this field. This chapter addresses definitions on explosions, explosives, concept of blast waves, and interaction between blast waves and structure. Available methods of analysis will be presented, together with the influence of this load in the material behaviour as well as the most important design codes available.

### 2.1 EXPLOSION PHENOMENON

An explosion is defined as a sudden release of energy. In an explosion the gas expands and it is forced out of the volume it occupies, as consequence, a layer of compressed air – blast wave – is formed around that gas containing most of the energy released by the explosion (Ngo et al, 2007). An explosion can be categorized by their nature as:
• **Physical**, energy may be released from the catastrophic failure of a cylinder of compressed air, volcanic eruptions or even mixing of two liquids at different temperatures.

• **Chemical**, energy may be released from the rapid oxidation of fuel elements (carbon and hydrogen atoms)

• **Nuclear**, energy may be released from the formation of different atomic nuclei by the redistribution of the protons and neutrons within the interacting nuclei.

This work deals with physical explosion which can be caused by High Explosive (HE) and Vapour Cloud Explosions (VCE). Both explosives and bombs can be categorized as (Bangash and Bangash, 2006):

- **Small** explosive device, up to 5 kg TNT;
- **Medium** explosive device, up to 20 kg TNT;
- **Large** explosive device, up to 100 kg TNT;
- **Very large** explosive device, up to 2500 kg TNT.

### 2.1.1 High Explosives (HE)

An explosive material is a substance capable of releasing a great amount of energy after ignition. According to their physical state they can be classified as solid, liquid or gases. Explosive materials can be also categorized according to their speed of expansion as high explosive (detonation, the decomposition is propagated by the explosive shockwave), and low explosive (deflagration, the decomposition is propagated by a flame front).

An explosive can be ignited by impact, friction, heat, static electricity or electromagnetic radiation. According to the explosive sensitivity to ignition, they can be classified as primary explosives or secondary explosives. The first ones are very sensitive to ignite like ammonium permanganate or nitroglycerin, the secondary explosives are less sensitive and require more energy to be ignited like TNT (Trinitrotoluene) or RDX (cyclotrimethylenetrinitramine). This allows the secondary explosives to be used safely in a wider variety of applications.
All blast loading parameters are dependent on the quantity of energy present in the explosion. The charge mass, \( W \), is expressed in kilograms of TNT (TNT is used as a universal reference). The first step when analysing an explosion with materials other than TNT is to convert the mass of explosive into mass of TNT. Table 2.3 shows the conversion factors for a number of explosive materials according to their specific energy. From the table, it can be seen that 100 kg of HMX (nitroamine high explosive) converts into 125.6 kg of TNT since the ratio of their specific energies is 1.256 (5680/4520).

In case of terrorist-manufactured explosives (known as Home-Made Explosives or HME) it is difficult to determine the TNT equivalent due to the variability of their formulation and the quality control used in their manufacture. Conversion factor ranging from 0.4 (poor HME quality) up to the unity have been suggested (Mays and Smith, 1995). The case is similar for fuel-air or vapour cloud explosions where TNT equivalents between 0.4 and 0.6 have been used (SCI, 2009).

<table>
<thead>
<tr>
<th>Explosive</th>
<th>Mass specific energy, ( Q_x ) (kJ/kg)</th>
<th>TNT equivalent ( Q_x/Q_{TNT} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compound B</td>
<td>5190</td>
<td>1.148</td>
</tr>
<tr>
<td>RDX</td>
<td>5360</td>
<td>1.185</td>
</tr>
<tr>
<td>HMX</td>
<td>5680</td>
<td>1.256</td>
</tr>
<tr>
<td>Nitroglycerin (liquid)</td>
<td>6700</td>
<td>1.481</td>
</tr>
<tr>
<td>TNT</td>
<td>4520</td>
<td>1.000</td>
</tr>
<tr>
<td>Semtex</td>
<td>5660</td>
<td>1.250</td>
</tr>
</tbody>
</table>

2.1.2 Vapour Cloud Explosions (VCE)

One of the differences between vapour cloud explosions and conventional explosions is that the blast from this last one has a high peak pressure over a short period of time, when compared to a vapour cloud explosion which generates a lower peak but of longer duration. Another difference between these two types of explosion is that conventional explosions have a point source, whereas vapour cloud explosions occupy a large volume.
There are two known mechanisms for generating an explosion in a relatively unconfined vapour cloud. One is a deflagration, where the flame accelerates to high speed, which requires a mechanism for generating the flame acceleration. It has been shown in large-scale experiments (SCI, 2009) that this can be provided by turbulence generated as the explosion propagates through pipework congestion typical of a process plant.

The second mechanism is a detonation, which if sustained, can be much more damaging. It may arise from coalescence of a strong shock wave and a fast-moving chemically reacting front. Together, this can undergo a transition to propagation faster than the speed of sound and produce overpressures at the front in excess of 1 MPa (Strehlow, 1973). It can also arise from the high temperatures and pressure generated by a shock wave in a confined volume, high flame speed deflagration or directly from strongly focused shock waves in a very reactive mixture.

The physical properties of highly flammable liquids, such as petrol, are such that if they are mishandled or released there is a significant risk of fire or explosion. In this context, perhaps the most important properties of petrol is its high volatility. At normal ambient temperatures, the vapour released by simple evaporation from the fuel surface can readily be ignited as it mixes with air. It is classified legally as a Highly Flammable Liquid, meaning that it has a flashpoint below 32 °C (Wiekema, 1984). Flashpoint is used to define the hazard associated with liquid fuels and to help determining safe working conditions.

The flashpoint is defined as the minimum temperature at which a liquid fuel produces sufficient vapour to form a flammable or ignitable mixture with air. Petrol has a flashpoint of around -40 °C, well below normal (ambient) temperatures, and can be ignited very easily. It is the vapour that “burns”, releasing heat, some of which is transferred to the surface of the fuel, thus increasing the rate of evaporation that supplies the flame with fuel vapour. This flashpoint can be compared with a flashpoint over +40 °C for diesel fuel.

Not all concentrations of fuel vapour in air are flammable, meaning capable of being ignited by a small ignition source such as a spark or a flame. Ignition can only occur if the mixture of flammable vapour in air falls within certain concentrations, known as the
lower and upper explosion (or flammability) limits (Strehlow, 1973). For petrol vapour, concentrations below 1.4% in air are too lean to burn, while those above 7.6% are too rich. For all concentrations between these two limits, known as the flammable range, the mixtures will burn, meaning that a flame will propagate away from the ignition source. Whether an ignition can give rise to a fire, an explosion or a combination of both depends on a number of factors, including the conditions prevailing at the time and the immediate surroundings of the release, but particularly on the amount of vapour present and how it is able to mix or diffuse with air.

For an explosion involving petrol vapour to occur, the vapour/air mixture must be within its flammable range when it encounters a potential source of ignition. A flame will propagate rapidly, spreading spherically from the ignition source throughout the entire flammable mixture. The heat released as the fuel is consumed causes the gases to expand as a result of the associated temperature rise. If confined, the gases cannot expand freely and the pressure will rise until parts of the confining structure (such as windows of a building) fail and relieve the pressure. This is normally a violent event and may produce a shock wave that can cause remote damage. In general, overpressures are not developed if an unconfined petrol vapour/air mixture is ignited because the gases can expand freely (Roberts and Pritchard, 1982). However, the speed at which the flame travels through the flammable mixture can vary considerably, depending on a number of different factors, such as: a) the composition of the mixture; b) amount of vapour and size of the flammable cloud; c) strength of the ignition source; d) the partial confinement and e) the turbulence created within the mixture. If the flame speed is very high, some overpressures will be created as the expansion cannot occur rapidly enough.

The term partial confinement is used to describe a situation in which obstacles lie in the path of the advancing flame. As the unburnt mixture is forced past these obstacles, turbulence is produced. The flame will travel much more rapidly through a turbulent mixture than through a quiescent one. If the flame speed is greater than the speed of sound the explosion is termed a detonation. A cloud of petrol vapour in the open without any turbulence creating obstructions, such as complex pipe racks or congested plant, would on ignition produce a deflagration with a relatively slow flame speed, around 10 m/s. Such an event would give rise to overpressures of about 1.4 kPa. For a
flame to accelerate beyond this and create higher overpressures, some or all of the previous factors would have to play a part.

2.1.3 Blast wave

When a blast wave is formed it increases the pressure instantly to a value higher than the atmospheric pressure (side-on overpressure), which later decreases as the wave propagates outwards from the explosion source. After a short period of time the pressure behind the front drops below the atmospheric pressure. During this negative phase a vacuum is created and the air is sucked in creating high suction winds responsible for carrying debris for long distances.

Figure 2.1 shows a typical blast wave pressure–time history from an explosion. Initially, at arrival time \( t_A \), there is a peak side-on overpressure \( P_{SO} \) (UFC 3-340-02, 2008), higher than the atmospheric pressure \( P_0 \). The pressure then decays to the value of the atmospheric pressure after the positive phase duration \( t_d \), and continues to decay to a value lower than the atmospheric pressure \( P_{SO}^{-} \) (creating a vacuum). After \( t_d + t_{d^-} \) (duration of the positive phase plus the duration of the negative phase) the pressure returns to the atmospheric conditions. Two distinct phases are thus found on the pressure-time profile of the blast wave, the positive phase, while the pressure is higher than the atmospheric pressure, with the duration \( t_d \), and the negative phase, with the duration \( t_{d^-} \) while the pressure is lower than the atmospheric pressure. The negative phase has a higher duration and lower intensity than the positive phase. Increasing the distance to the explosion source, \( R \), results in a lower amplitude and higher duration. Explosive charges detonated extremely close to a structure impose a highly impulsive, high intensity pressure load over a localized region of the structure; explosive charges detonated further away from the target structure produce a lower intensity, longer duration pressure load over the entire structure (Ngo et al, 2007). Eventually, the entire structure will be engulfed in the blast wave; the reflection and diffraction effects will create focusing and shadow zones in a complex pattern around the structure. During the negative phase the structure already weakened may experience impact of debris carried by the suction winds present in this phase.
When the blast wave encounters a solid surface or an object denser than air it will reflect from it and, depending on its geometry and size, diffract around it. This reflection phenomenon will amplify the peak side-on overpressure by a reflection factor (Figure 2.2). These factors depend mostly on the intensity of the blast wave and the angle of incidence.

![Figure 2.1 – Pressure profile from external explosions.](image1)

![Figure 2.2 – Relation between the peak overpressure and the reflected pressure.](image2)
2.1.4 Blast loading

It is possible to identify three different interaction classes between the blast wave and the target structure: the first one is usually associated to large scale explosion, the structure is engulfed and compressed by the blast wave, there will be a translation force which will attempt to move the structure laterally (drag forces) but due to the size and nature of the structure it will not move (Figure 2.3a); the second one is associated to large scale explosions but when it interacts with smaller structures, for example a vehicle, here the structure will be engulfed and compressed by the blast wave, there will be drag forces too and due to the smaller size of the structure it will move laterally. In this case the drag forces will be responsible for most of the suffered damage (Figure 2.3b); the third interaction class will refer to the smaller blast charges, here the blast wave will have a lower intensity and in this case the structural elements should be analysed individually (Figure 2.3c).

For the first two interaction classes, and observing Figure 2.4, it can be noted that each structure facade will be subjected to two different types of load. First, each facade will experience the diffraction of the blast wave around the entire structure causing a compressing pressure around all facades. On a second instant, the structure will experience a drag force due to the dynamic pressures. In Figure 2.4a the peak overpressure applied to the frontal facade is the reflected pressure, $P_R$. This pressure will decay in the time interval ($t' - t_2$) until the value of pressure applied to the top and side facades (side-on overpressure, $P_{SO}$). The time $t'$ can be determined as (Mays and Smith, 1995):

$$t = 3 \times \frac{S}{U}$$  \hspace{1cm} (2.1)
Where $S$ is the smaller of: half of the structure breadth or the structure height; and $U$ is the blast front velocity, which can be determined as (Mays and Smith, 1995):

$$U = \sqrt{\frac{6P_{s0} + 7P_0}{7P_0}}a_0$$

(2.2)

Where $P_{s0}$ is the peak overpressure at the wavefront, $P_0$ is the atmospheric pressure and $a_0$ is the speed of sound at ambient conditions.

In Figure 2.4b the deviation from the linear decay of pressure is due to complex vortexes originated in the intersections of the frontal facade with the side and top ones. In Figure 2.4c there is a slower increase of pressure due to the time required by the blast wave to complete the diffraction process travelling down the rear of the structure. Figures 2.4d and 2.4e show the drag forces in the frontal and back facades due to the dynamic pressures. The resulting drag force is given by (Mays and Smith, 1995):

$$F_D = C_D \times q_s(t) \times A$$

(2.3)

Where $A$ is the loaded area, $C_D$ is the drag coefficient of the target which depends on the geometry and $q_s(t)$ is the dynamic pressure with a maximum of (Liepmann and Roshko, 1957):

$$q_s = \frac{5P_{s0}^2}{2(P_{s0} + 7P_0)}$$

(2.4)

UFC 3-340-02 (2008) suggests drag coefficients for the top and lateral facades according to the dynamic pressure.

An explosion is usually defined using two important parameters: charge of the explosion $W$, and the standoff distance $R$. With these two parameters and using the Hopkinson-Cranz scaling laws (Brode, 1955) it is possible to define a constant of proportionality capable of relating charges and distances for different explosions. This parameter $Z$ is called scaled distance and can be calculated as $Z = R/W^{1/3}$ [m/kg$^{1/3}$]. This
means that two different explosions with the same scaled distance value should produce the same overpressure, positive phase duration and impulse.

Figure 2.4 – Blast wave external loading on structures: a) front façade; b) top and side facades; c) rear façade; d) drag load on the front façade; e) drag load on the rear facade.

Since the middle of the last century many researchers study this phenomenon and several publications address the way to determine the overpressures derived from
explosions. Estimations of the peak overpressure due to spherical blast were introduced by Brode (1955) as:

\[
P_{S0} = \frac{6.7}{Z} + 1\text{bar} \quad (P_{S0} > 10 \text{bar})
\]

\[
P_{S0} = \frac{0.975}{Z} + \frac{1.455}{Z^2} + \frac{5.85}{Z^3} - 0.019 \text{bar} \quad (0.1 \text{bar} < P_{S0} < 10 \text{bar})
\]  \hspace{1cm} (2.5)

Later Newmark and Hansen (1961) introduced an expression to calculate the peak overpressure for high explosive charge detonating at ground surface:

\[
P_{S0} = 6784 \frac{W}{R^3} + 93 \left(\frac{W}{R^3}\right)^{1/2} \text{ (bar)}
\]  \hspace{1cm} (2.6)

Other empirical solutions were introduced later on, such as Kinney and Graham (1985) or Mills (1987) to estimate the side-on overpressure. To estimate the positive phase duration, Kinney and Graham (1985) proposed:

\[
\frac{t_d}{W^{1/3}} = \frac{980 \left(1 + \left(\frac{Z}{0.54}\right)^{10}\right)}{\left(1 + \left(\frac{Z}{0.02}\right)^3\right)\left(1 + \left(\frac{Z}{0.74}\right)^6\right)\left(1 + \left(\frac{Z}{6.9}\right)^2\right)} \quad (ms)
\]  \hspace{1cm} (2.7)

Later Bangash and Bangash (2006) introduced another equation to estimate the positive phase duration as a function of the charge of the explosion and the peak side-on overpressure:

\[
t_d = 10.23 \frac{W^{1/3}}{\sqrt{P_{S0}}} \hspace{1cm} (2.8)
\]

If the shock wave encounters an object it will reflect from it and, depending on the size and geometry, diffract around it. For an incident angle of 0º the reflected pressure will be given by:
Chapter 2 – Explosion and Structures: A State of the Art

\[ P_r = 2P_{S0} + (\gamma + 1)q_s \] \hspace{1cm} (2.9)

Where \( P_{S0} \) is the peak overpressure, \( \gamma \) is the specific heat ratio and \( q_s \) is the dynamic pressure defined as:

\[ q_s = \frac{1}{2} \rho_S u_s^2 \] \hspace{1cm} (2.10)

Here, \( \rho_S \) is the density of air and \( u_s \) is the particle velocity behind the wavefront defined as:

\[ u_s = \frac{a_0 P_{S0}}{\gamma P_0} \left( 1 + \left( \frac{\gamma + 1}{2\gamma} \frac{P_{S0}}{P_0} \right) \right)^{\frac{\gamma}{2}} \] \hspace{1cm} (2.11)

Knowing that for air \( \gamma \) is 1.4, introducing Eq. (2.11) and Eq. (2.10) into Eq. (2.9) gives:

\[ P_r = 2P_{S0} \left( \frac{7P_0 + 4P_{S0}}{7P_0 + P_{S0}} \right) \] \hspace{1cm} (2.12)

When the incident angle is 90º there is no reflection and the surface is loaded with the peak side-on overpressure. For values between 0º and 90º there is regular reflection and Mach reflection (Smith and Hetherington, 1994). Regular reflection occurs for angles between 0º and approximately 40º after which Mach reflection takes place. Figure 2.5 shows some reflection coefficients (relation between the reflected pressure and the peak side-on overpressure) for a range of peak overpressure versus the incident angle. Mach reflection is a much more complex phenomenon also known as ‘spurt’-type effect (Smith and Hetherington, 1994).

For design purposes the applied pressure can be idealised as a triangular shape load (Figure 2.6). As we can see, there are two different approaches, one is a more conservative approach (I) considering the maximum pressure value and the full positive phase duration; the second one keeps the same impulse (II) as the original curve keeping the same maximum pressure value but calculating the time after which the impulse
would be the same with the triangular shape load. In both cases the impulse is calculated as the area of the triangle representing the load, given as:

\[ i = \frac{1}{2} Pt_d \]  

(2.13)

![Figure 2.5](image)

Figure 2.5 – Influence of the angle of incidence on the reflected coefficients (UFC 3-340-02, 2008).

![Figure 2.6](image)

Figure 2.6 – Idealisation of the blast loading profile.
2.2 STRUCTURAL RESPONSE

The complexity in analysing blast loading lays on the high velocities of deformation, the non-linear response of the materials and the uncertainty in the forces and deformations calculations. To simplify the analysis, a number of assumptions related to the structural response to this kind of loading has been presented and accepted in the scientific community.

2.2.1 Methods of analysis

In assessing the behaviour of blast loaded structures it is often the case that the calculation of the final states is the principal requirement for a designer rather than a detailed knowledge of its displacement-time history. To establish the main aspects of this kind of analysis, the response of a Single Degree Of Freedom (SDOF) elastic structure is considered and the link between the duration of the blast load and the natural period of vibration of the structure is established.

In the SDOF approach the structure is idealised as an equivalent system with a concentrated mass, structure damping and a spring which represents the structural stiffness. The structural mass, $M$, is affected by an external force, $F(t)$, and the structural response, $R$, is expressed in terms of the vertical displacement, $y$, and the spring constant, $K$. The term $c$ represents the structure damping coefficient and it will attenuate the amplitude of vibration in time, see Figure 2.7.

As stated earlier the loading profile is simplified as a triangular shape loading with a peak $F_m$ and a positive duration $t_d$ (Figure 2.8). The Force function is now given by the general expression:

$$F(t) = F_m \left(1 - \frac{t}{t_d}\right)$$  \hspace{1cm} (2.14)
Considering $c=0$ (undamped structure) the general solution for the response can be expressed as displacement, Eq. (2.15), or as velocity, Eq. (2.16) (Biggs, 1964):

$$y(t) = \frac{F_m}{K}(1 - \cos \omega t) + \frac{F_m}{Kt_d} \left( \frac{\sin \omega t}{\omega} - t \right)$$

$$\dot{y} = \frac{F_m}{K} \left[ \omega \sin \omega t + \frac{1}{t_d} (\cos \omega t - 1) \right]$$

Here, $\omega$ is the natural circular frequency of vibration of the structure, which can be related to the natural vibration period of the structure, $T$. This relation is given by:

$$\omega = \frac{2\pi}{T} = \sqrt{\frac{K}{M}}$$

The maximum response is defined by the maximum dynamic deflection $y_m$ which occurs at time $t_m$. The maximum deflection can be determined by setting Eq. (2.16) equal to zero.
The dynamic load factor, DLF, is defined as the relation between the maximum dynamic deformation $y_m$ and the maximum static deformation $y_{st}$ which would have resulted of the static application of the peak load $F_m$.

$$ DLF = \frac{y_m}{y_{st}} \quad (2.18) $$

The structural response, $R(t)$, to blast loading is greatly influenced by the relation $t_d/T$ or $\omega t_d$. Three loading regimes are categorized as follows (Baker et al, 1980):

- $\omega t_d < 0.4$ : impulsive loading regime (Figure 2.9a).
  This occurs when the positive phase duration is small compared with the natural period of vibration. In this case the force has finish acting before the structure has had the time to respond significantly, meaning that most of the deformations will occur after $t_d$.

- $\omega t_d > 40$ : quasi-static loading regime (Figure 2.9b).
  This occurs when the positive phase duration is large compared with the natural period of vibration. In the limit the force may be considered as remaining constant whilst the structure attains the maximum deflection.

- $0.4 < \omega t_d < 40$ : dynamic loading regime (Figure 2.9c).
  This occurs when the positive phase duration is similar to the natural period of vibration. In this case the structural response is more complex to evaluate.

![Loading regimes](image)

Figure 2.9 – Loading regimes: a) impulsive regime; b) quasi-static regime; c) dynamic regime.

When the structure cannot be idealised as a single degree of freedom, a multiple degree of freedom (MDOF) analysis using numerical techniques must be carried out. Multiple degree of freedom systems such as multi-story frames are based on lumped-mass
assumptions where the number of degrees of freedom is equal to the number of types of motion possible within the system (Figure 2.10). For each degree of freedom, there is a corresponding equation of motion. These equations of motion are used to determine the natural frequencies of a structure and corresponding modes.

Structural elements when subjected to blast are expected to undergo large plastic deformations. Considering these effects as well as strain rate effects, strain hardening and temperature variation is possible through numerical simulations which require software able to perform non-linear explicit analysis, usually associated to the use of Finite Elements (FE). In practice Finite Element analysis techniques provide the most acceptable level of accuracy for the dynamic response of structures (Yandzio and Gough, 1999).

Table 2.4 shows most of the software used in the field of blast prediction and structural response. Computational methods in this area are usually divided into two large groups, the ones used to predict the loading profiles from explosions and the ones used to predict the structural response. Computational programmes for blast loading prediction can use both empirical solutions, based on equations from the literature, or more advanced numerical calculations, based on Computational Fluid Dynamics (CFD). Computational programmes for determining the structural response use, in general Finite Element Method (FEM). There are some programmes allowing both analyses in a coupled or uncoupled form. The uncoupled analysis calculates the blast load as if the structure was rigid and then applies these loads to a model of the structure in order to obtain the response. For the coupled analysis, the blast simulation is linked with the structural response module. In this type of analysis the computational fluid mechanics (CFD) model for blast-load prediction is solved simultaneously with the computational solid mechanics (CSM) model for structural response. Prediction of the blast induced pressure field on a structure and its response involve highly nonlinear behaviour.
Computational methods for blast-response prediction must therefore be validated by comparing calculations to experimental results.

In short, there are simpler methods for predicting the blast load, like ConWep, and simpler methods for predicting the structural response, like SDOF or MDOF available which can be used for the modelling of simple components, like beams, slabs, columns and walls. For more complex structural components or the entire building, nonlinear finite element analysis using explicit dynamics should be used.

Table 2.4 – Software for the blast prediction and structural response.

<table>
<thead>
<tr>
<th>Name</th>
<th>Type of analysis</th>
<th>Author</th>
</tr>
</thead>
<tbody>
<tr>
<td>BLASTX</td>
<td>Blast prediction, CFD code</td>
<td>SAIC</td>
</tr>
<tr>
<td>CTH</td>
<td>Blast prediction, CFD code</td>
<td>Sandia National Laboratories</td>
</tr>
<tr>
<td>FEFLO</td>
<td>Blast prediction, CFD code</td>
<td>SAIC</td>
</tr>
<tr>
<td>FOIL</td>
<td>Blast prediction, CFD code</td>
<td>Applied Research Associates, Waterways Experiment Station</td>
</tr>
<tr>
<td>SHARC</td>
<td>Blast prediction, CFD code</td>
<td>Applied Research Associates, Inc.</td>
</tr>
<tr>
<td>DYNA3D</td>
<td>Structural response + CFD (coupled analysis)</td>
<td>Lawrence Livermore National Laboratory (LLNL)</td>
</tr>
<tr>
<td>LS-DYNA</td>
<td>Structural response + CFD (coupled analysis)</td>
<td>Livermore Software Technology Corporation (LSTC)</td>
</tr>
<tr>
<td>Air3D</td>
<td>Blast prediction, CFD code</td>
<td>Royal Military of Science College, Cranfield University</td>
</tr>
<tr>
<td>CONWEP</td>
<td>Blast prediction (empirical)</td>
<td>US Army Waterways Experiment Station</td>
</tr>
<tr>
<td>AUTO-DYN</td>
<td>Structural response + CFD (coupled analysis)</td>
<td>Century Dynamics</td>
</tr>
<tr>
<td>ABAQUS</td>
<td>Structural response + CFD (coupled analysis)</td>
<td>ABAQUS Inc.</td>
</tr>
</tbody>
</table>

2.2.2 Material behaviour

Different loading conditions lead to different strain rates. Quasi-static loading produce strain rates of around $10^{-5}$ s$^{-1}$, while impacts and blast loading produce strain rates of well over $10^0$ s$^{-1}$ (Ngo et al, 2007). When subjected to dynamic loading conditions, materials can have a much different behaviour when compared with their static
behaviour (Meyers, 1994; Hiermaier, 2008; Ngo et al, 2004; Stavrogin and Tarasov, 2001). Current design on structural response and damage under impact and blast loading assumes typically static material properties (Baylot et al, 2005; Moreland et al, 2005). This can lead to an inaccurate prediction of structural damage and fragmentation.

Concrete under dynamic loading conditions shows quite different mechanical properties when compared with those under static loading. While the dynamic stiffness does not vary a great deal from the static stiffness, the stresses that are sustained for a certain period of time under dynamic conditions may gain values remarkably higher than the static compressive strength, see Figure 2.12 (Ngo et al, 2004). Grote et al (2001) reported strength magnification factors as high as four in compression up to six in tension for strain rates ranging from $10^2$ to $10^3/s$.

The increase in the peak compressive stress ($f_c$), is introduced in the CEB-FIP (1990) as a DIF (Dynamic Increase Factor, i.e. a ratio of the dynamic to static parameters’ values) and can be determined as:

$$DIF = \left( \frac{\dot{\varepsilon}}{30 \times 10^{-6}} \right)^{1.026 \alpha}, \quad \dot{\varepsilon} \leq 30 \text{ s}^{-1}$$  \quad (2.19)$$

$$DIF = \gamma \left( \frac{\dot{\varepsilon}}{30 \times 10^{-6}} \right)^{3}, \quad \dot{\varepsilon} > 30 \text{ s}^{-1}$$  \quad (2.20)$$

$$\alpha = \sqrt{5 + 9 \frac{f_{cs}}{f_{co}}}$$  \quad (2.21)$$

Where $\log \gamma = 6.156 \alpha - 2$ and $f_{co} = 10 \text{ MPa}$. 

![Figure 2.11 – Expected strain rates for different loading conditions (Ngo et al, 2007).](image-url)
Malvar and Ross (1998) proposed a DIF equation for tensile strength of concrete, $f_t$:

$$DIF = \left( \frac{\dot{\epsilon}}{10^{-6}} \right)^\delta, \quad \dot{\epsilon} \leq 1 \text{ s}^{-1}$$

$$DIF = \beta \left( \frac{\dot{\epsilon}}{10^{-6}} \right)^{\frac{1}{3}}, \quad \dot{\epsilon} > 1 \text{ s}^{-1}$$

$$\delta = \frac{1}{(5 + 8 f_{cs} / f_{co})}$$

$$\log(\beta) = 6\delta - 2$$

Where $f_{co} = 10 \text{ MPa}$.

Figure 2.12 – Stress-strain curves of concrete at different strain-rates (Ngo et al, 2004).

Asprone et al (2009a) verified the accuracy of CEB and Malvar formulations using a modified Hopkinson bar for tensile tests. It was shown that the CEB formulation underestimates the tensile strength of concrete, while the Malvar and Ross formulation overestimates the tensile strength. Ruiz et al (2010) studied the influence of the displacement rate, $\dot{\delta}$, in the fracture energy, $G_f$, of high-strength concrete. It was shown that the compressive fracture energy is enhanced with the increase of the displacement rate; and this effect is more significant after the displacement rate of $7.04 \times 10^2 \text{ mm/s}$. 


In reinforcement steel and due to its isotropic behaviour, their elastic and inelastic response to dynamic loading is easier to monitor and assess. Norris et al (1959) tested steel with two different static yield strengths of 330 MPa and 278 MPa under tension at strain rates ranging from $10^{-5}$ to 0.1 s$^{-1}$. An increase of strength was found in both steel types of 9-21% and 10-23% respectively. Dowling and Harding (1967) conducted tensile experiments using the tensile version of Split Hopkinton’s Pressure Bar on mild steel using strain rates ranging from $10^{-3}$ to 2000 s$^{-1}$, and it was shown that the yield strength of mild steel can be almost doubled, and the ultimate tensile strength can be increased by about 50%. Malvar and Ross (1998) also studied this phenomenon and described the strength enhancement of steel reinforcing bars under the effect of high strain rates in terms of DIF, which can be evaluated for different steel grades and for yield stresses, $f_y$, ranging from 290 MPa to 710 MPa:

$$DIF = \left( \frac{\dot{\varepsilon}}{10^{-4}} \right)^{a}$$  \hspace{1cm} (2.26)

$$a(yield\ stress) = 0.074 - 0.04 \left( \frac{f_y}{414} \right)$$  \hspace{1cm} (2.27)

UFC 3-340-02 (2008) also presents the stress-strain relations for concrete and reinforcement bars, as shown in Figure 2.13 and Figure 2.14 respectively. This normative document takes into account the effect of high strain rates in the form of a DIF. In Table 2.5 we can see the DIF values for concrete and reinforcement bars presented in UFC 3-340-02 (2008) according to the design range and the type of stress. Here, far and close-in design ranges are related to the location of the element relative to the explosion.

In case of masonry structures and its components, the available literature is even more scattered. Recently Hao and Tarasov (2008) conducted an experimental study under dynamic uniaxial compression using a Triaxial Static-Dynamic Testing Machine. These authors reported a DIF for the compressive strength of around 2.3 and 1.12 for the DIF of the ultimate strain, for a strain rate of 150 s$^{-1}$. For the Young’s modulus a DIF of 1.95 was reported for the same level of strain rate.
Burnett et al (2007) presented results from dynamic tensile experiments on mortar joint using a specially design Slipt Hopkinson pressure bar. Their results showed that there is a dynamic enhancement of the mortar strength at strain rate of 1 s\(^{-1}\), with a DIF of 3.1.

Asprone et al (2009b) studied this effect on a specific Italian stone using Hydro-Pneumatic Machine and a modified Hopkinson bar for tensile tests. The tensile strength of this type of stone showed an increase of three times the static reference.
2.3 **CODES AND STANDARDS**

Federal standards and criteria in USA are widely recognized as the primary source of guidelines for the design of buildings to resist blast loading. Because of the uniqueness of each building mission, functional requirements and physical security design objectives, there are limited codes and standards that apply to blast mitigation design. Although the majority of these design guidelines were focused on military applications such knowledge is relevant for civil design practice. A list of the most referenced guidelines includes:

- **Structures to Resist the Effects of Accidental Explosions, UFC 3-340-02 (Unified Facilities Criteria, 2008)** – This manual appears to be the most widely used publication by both military and civil organizations for designing structures to prevent the propagation of an explosion and to provide protection for personnel and valuable equipment.

- **Minimum Antiterrorism Standards for Buildings, UFC 4-010-01 (Unified Facilities Criteria, 2012)** – This document represents a significant commitment by USA Department of Defence (DoD) to seek effective ways to minimize the likelihood of mass casualties from terrorist attacks against DoD personnel in the buildings in which they work and live.

- **Design of Buildings to Resist Progressive Collapse, UFC 4-023-03 (Unified Facilities Criteria, 2009)** – This document provides the design requirements necessary to reduce the potential of progressive collapse for new and existing facilities that experience localized structural damage through normally unforeseeable events such as explosions.

- **A Manual for the Prediction of Blast and Fragment Loading on Structures, DOE/TIC-11268 (U.S. Department of Energy, 1992)** – This manual provide guidance to the designers of facilities subjected to accidental explosions and aids in the assessment of the explosion-resistant capabilities of existing buildings.

- **Protective Construction Design Manual, ESL-TR-87-57 (Air Force Engineering and Services Center, 1989)** – This manual provide procedures for the analysis and design of protective structures exposed to the effects of conventional (non-nuclear) weapons and is intended for use by engineers with basic knowledge of weapons effects, structural dynamics and hardened protective structures.
• Design and Analysis of Hardened Structures to Conventional Weapons Effects, UFC 3-340-01 (Unified Facilities Criteria, 2002) – This manual provides procedure for the design and analysis of protective structures subjected to the effects of conventional weapons. It is for use by engineers involved in designing hardened facilities, e.g. bunkers.

• Structural Design for Physical Security – State of the Practice Report (ASCE, 1999) – This document is a guideline for engineers who would like to incorporate security aspects in their projects.

2.4 Final Remarks

This chapter has provided a summary of blast related definitions, details on explosive mechanisms and explosives; methods to determine the pressure and impulse values originated by an explosion; and the interactions between the blast waves and the structure, as well as the forces applied to the structure when subjected to blast loading. It should be noted that the level of damage suffered by a structure cannot be determined solely from knowledge of the pressure and impulse values from a particular explosion. It is also of great importance to know the characteristics of the blasted-loaded building, in particular the dynamic properties of the used materials, the dynamic properties of the structure and the shape of the structure. There are methods available to determine the structure response when subjected to blast loading, ranging from simpler methods like SDOF up to the use of commercial software using FE to perform explicit dynamic analysis.

2.5 References


UFC 3-340-01, *Design and analysis of hardened structures to conventional weapons effects (FOUO)*. Department of Defence, USA 2002.


UFC 4-023-03, *Design of buildings to resist progressive collapse*, Department of Defence, USA 2009.


Chapter 3

3 ASSESSING BLAST LOADING PARAMETERS FROM POST-DISASTER SCENARIOS

The vulnerability of civilian buildings when exposed to explosions has been shown by the latest terrorist attacks or industrial accidents. A blast wave from an explosion acting directly on a building can cause major economic and human losses. As result, the number of studies in the structural response, retrofitting and repairing of structures has increased in the last years. As experimental full-scale test are expensive, numerical analysis plays a very important role in the development of knowledge in this field.

When dealing with blast loading problems the first issue to be addressed is the magnitude of the loading, meaning overpressures and impulses. This is important when trying to design or protect against explosions, when it is mandatory to know the magnitude of what to protect or design for; and, in a post-blast phase, when dealing with accident investigations or to be aware of magnitudes in past incidents to protect for possible future incidents.
3.1 ASSESSING BLAST LOADING PARAMETERS THROUGH PRESSURE INDICATORS

In Chapter 2 from this document different methods for assessing blast loading parameters were presented, being empirical or numerical. Empirical methods use mainly equations derived for military applications, assuming High Explosive (HE) as a point source explosive. Numerical methods through the use of advanced Computational Fluid Dynamics (CFD) formulation are able to determine the pressure distribution for both HE and Vapour Cloud Explosions (VCE), however these analysis are usually time consuming and require advanced training.

Another method to assess blast load parameters in the event of an explosion, either HE or VCE, is to assess the damage of the explosion and relate this damage with the overpressures and impulses that could have damaged the structure. This becomes easier if smaller, simpler structures are chosen to be studied. Metal switch boxes, oil drums, car, tanks, trees, pipes etc, makes good pressure indicators. Metal switch boxes in particular are small, ductile structures capable of withstanding large deformations. These pressure indicators or sub-structures can also provide data on the direction of the blast wave, making it easier to identify the source of the blast in case of HE, or the ignition source in case of vapour cloud explosion.

The objective is to relate the level of damage observed with the overpressures and impulses that could generate that level of damage. There are some commonly accepted damage figures from blast overpressures, such as Table 3.1, which can be used for comparative purposes. However, care has to be taken in using this data as it does not take into account the duration of the pressure wave (impulse), but only considers its maximum overpressure.

Dealing with small, simple structures makes it easier to represent a pressure-impulse ($P-I$) or a iso-damage curve, which provide the load-impulse combination that caused a specific level of damage to be assessed very readily, as it will be shown later in this document.
This chapter is intended to give insight on the determination of blast loading parameters. This work has been done in collaboration with Kingston University London, UK and the Fire and Explosion Study Centre in London, UK in the framework of the Buncefield Major Incident Investigation. The most relevant aspect is to study the possibility of estimating the magnitude of explosions by using pressure indicators or sub-structures. Using common objects present within and in the neighbourhood of most of industrial buildings, such as steel switch boxes, cars or oil drums as reference samples, it is possible to estimate the overpressures in case of an explosion by building a relation between the residual permanent deformation with pressure/impulse responsible for such deformation. Finally, pressure-impulse (P-I) diagrams can help future accident investigations by backtracking the overpressures and impulses according to the observed damage in those specific sub-structures.

Table 3.1 – Commonly accepted figures for damage and harm to people from blast overpressure (SCI, 2009).

<table>
<thead>
<tr>
<th>Pressure (kPa)</th>
<th>Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.7</td>
<td>Breakage of small windows under strain</td>
</tr>
<tr>
<td>6.3</td>
<td>Roof damage to oil storage tanks</td>
</tr>
<tr>
<td>10.3-20</td>
<td>People knocked down or thrown to the ground</td>
</tr>
<tr>
<td>21-28</td>
<td>Rupture of oil tanks</td>
</tr>
<tr>
<td>56-70</td>
<td>Brick walls demolished</td>
</tr>
<tr>
<td>206-240</td>
<td>Near 100% fatality from lung haemorrhage</td>
</tr>
<tr>
<td>490</td>
<td>Collapse of heavy masonry or concrete bridges</td>
</tr>
</tbody>
</table>

3.2 Buncefield Major Incident

The Buncefield oil storage and transfer depot is a large tank farm occupied by three companies (Figure 3.1): Hertfordshire Oil Storage Limited a joint venture between Total UK Limited and Shevron Limited; United Kingdom Oil Pipelines Limited and West London Pipeline and Storage Limited whose site is operated by British Pipeline Agency Limited; and British Petroleum Oil UK Limited.

The Buncefield explosion (11 December 2005) resulted in tremendous damage to the outlying area and huge fires involving 23 large oil fuel tanks. One important aspect of the incident was the severity of the explosion (Figure 3.2), which would not have been
anticipated in any major hazard assessment of the oil storage depot before the incident. The area covered by the vapour cloud was estimated to be around 120,000 m² and the average height of the cloud was around 2 m giving a volume of 240,000 m³. Overpressures within the area of the cloud were found to be consistently high. From a combination of damage assessment and comparative testing and analysis it was concluded that the overpressure within the cloud was generally greater than 200 kPa. Overpressure diminished rapidly with distance away from the edge of the cloud; evidence suggests overpressures in the region of 5-10 kPa within 150 m (SCI, 2009).

Evidence shows that the main explosion probably resulted from ignition of a vapour cloud emanating from Tank 912 in Bund A in the Hertfordshire Oil Storage Limited West side, see Figure 3.1, most likely resulting from an overfill of unleaded petrol.

Detailed modelling of the area immediately surrounding the emergency pump house supports the hypothesis that the trees and undergrowth along Three Cherry Lane caused flame acceleration to a velocity of several hundred m/s; at such high flame speeds a transition to detonation is considered possible.

![Figure 3.1 – Schematic of Buncefield site and surrounding area (SCI, 2009).](image-url)
The following timeline describes the incident in Buncefield Oil Depot from 10\textsuperscript{th} December to 15\textsuperscript{th} December 2005 (SCI, 2009):

**10 December 2005:**
- Around 19:00, Tank 912 in Bund A at the Hertfordshire Oil Storage Limited West site started receiving unleaded motor fuel from the T/K pipeline, pumping at about 550 m\textsuperscript{3}/hour.

**11 December 2005:**
- Around 00:00 the terminal was closed to tankers and stock check of products was carried out. When this was completed, at around 01:30, no abnormalities were reported.
- From approximately 03:00, the level gauge for Tank 912 recorded an unchanged reading. However, filling of Tank 912 continued at a rate of around 550 m\textsuperscript{3}/hour.
- Around 05:20 Tank 912 would have been completely full and starting to overflow. From this time onwards, continued pumping caused fuel cascade down the side of the tank and through the air, leading to the rapid formation of a rich fuel/air mixture in Bund A.
- At 05:38 vapour cloud from the escaping fuel is first visible in a CCTV footage from a camera looking down western edge of Bund A, flowing out of the north-west corner of Bund A towards the west.
- At 05:46 the vapour cloud had thickened to a depth of about 2 m and was flowing out of Bund A in all directions.
At 05:50 the vapour cloud had started flowing off site near the junction of Cherry Tree Lane and Buncefield Lane, following the ground topography. It spread west into Northgate House (n. 2 in Figure 3.1) and Fuji car parks (n. 1 in Figure 3.1) and towards Catherine House (n. 13 in Figure 3.1).

Between 05:50 and 06:00 the pumping rate down the T/K pipeline to Hertfordshire Oil Storage Limited West, and onwards to Tank 912, gradually rose to about 890 m$^3$/hour.

At 06:01 the vapour cloud extended to the west almost as far as Boundary Way in the gaps between the 3-Com, Northgate and Fuji buildings; to the north-west it extended as far as the nearest corner of Catherine House. It probably extended to the north of the Hertfordshire Oil Storage Limited site as far as Tank 12, and probably extended south across part of the Hertfordshire Oil Storage Limited site, but not as far as the tanker filling gantry. To the west it reached the British Pipeline Agency Limited site. At this same time the first explosion occurred, followed by further explosions and a large fire that engulfed over 20 large storage tanks. The main explosion event appears to have been centred on the car parks between the Hertfordshire Oil Storage Limited West site and the Fuji and Northgate buildings.

At 06:08 an emergency services major incident was declared and operational command and control was set up near the incident site within minutes.

At 09:00 an extensive plume of smoke from burning fuel dispersed over southern England and beyond. The plume could be seen from many kilometres away, and was also clearly identified in satellite images.

12 December 2005:

At 12:00 the peak of the fire was reached, 20 Hertfordshire pumps were on site with 20 support vehicles and 180 fire-fighters.

15 December 2005:

“Fire all out” was declared by the fire services after 786 000 litres of foam concentrate and 68 millions of water were used to contain the incident during the period of fire-fighting operations.

An explosion can be produced when a gas cloud is ignited within a confined volume such as a building. As the flame propagates through the gas cloud it produces hot
combustion products. The confinement prevents expansion of these combustion products and, as a consequence, the pressure increases. In general, this continues until the confining structure fails, in some cases catastrophically. This mechanism does not explain the type of explosion that occurred at Buncefield as the majority of the cloud was not confined. It is recognised that two “confined explosions” did occur, but these events alone could not explain the severity of the overall explosion. These finding lead to believe that in fact there was a detonation mechanism arising from the deflagration mechanism through the increase of the flame front velocity. A better description of these mechanism is given is Chapter 2.

The investigation was directed by Health and Safety Commission (HSC) which produced several reports of the investigation process and can be assessed in http://www.buncefieldinvestigation.gov.uk/.

3.3 ASSESSING BLAST LOAD PARAMETERS THROUGH SUB-STRUCTURES

With the objective of aiding the Buncefield Major Incident investigation experimental work was performed on sub-structures. These sub-structures, such as steel drums, cars and metal switch boxes, were subjected to HE (High Explosives) in air blast, gas explosion through VCE (Vapour Cloud Explosion) and hydrostatic pressure tests. The results are presented in the following sections.

3.3.1 Steel drums

Steel drums were located in various parts of the site. These included full, partially empty and empty drums. In many cases the prior location of the drums could be determined. Overall translational displacements during the explosion were relatively small (less than 5 meters).

The deformation characteristic of the top of the drum is a good overpressure indicator, including the end plate and the section of side wall near the end plate. Similar deformation of the top of a drum can be reproduced in a static hydraulic test. Figure 3.3
shows the result of a test at an imposed pressure of 300 kPa. In these tests, drums were filled with concrete up to the liquid level indicated by the undeformed drum part in Figure 3.3. This is necessary to simulate the same type of failure observed in the explosion due to the inertia of the liquid. The results indicate a static overpressure in excess of 200 kPa in this location.

Figure 3.3 – Oil drum after static hydraulic test with fully plastic longitudinal sidewall buckling in the ullage (SCI, 2009).

Gas explosion tests carried out on empty oil drums did not produce the level of deformation observed in both static tests and objects in Buncefield at overpressure of up to 180 kPa. The deformation in those tests was largely restricted to the side (Figure 3.4a). At the highest pressure test, the end plates of the drum exhibited a limited amount of distortion seen in Figure 3.4b, it did not extend through the entire circumference and the amount of deformation was small. The gas explosion test results on end plate distortion indicated an overpressure of over 180 kPa and are consistent with those from hydraulic tests.

Figure 3.4 – Oil drums after gas explosion tests: a) empty oil drum after 180 kPa gas explosion; b) drum cap end (SCI, 2009).
3.3.2 Cars

Around 20 cars were in the area covered by the vapour cloud in Buncefield. All of them were badly damaged (Figure 3.5).

Figure 3.5 – Typical car damage at Buncefield (SCI, 2009).

Enclosed gas explosion tests were carried out by Health and Safety Laboratory (HSL) and Figure 3.6 shows some of the cars after the tests, sorted by increasing overpressure. The results of these tests show that pressure of well over 100 kPa and a rapid rise in overpressure are required to cause anything like the damage observed at Buncefield. Unfortunately during the final test it was not possible to record the overpressures so no better estimate can be made of the minimum overpressure.

There were also tests carried out by British Petroleum involving five cars positioned at various distances and various orientations (Figure 3.7) and subjected to a HE charge of about 170 kg equivalent TNT. They were positioned so that they would be exposed to overpressures ranging from 170 kPa to 1 MPa peak side-on overpressure for a duration of the positive phase between 10 to 12 ms.

As shown in Figure 3.8a one of the cars was positioned in front of a wall which provided a more even pressure load on the vehicle as well as increased the pressure impulse the car experienced. The damage observed in many cases was limited to panel deformation while the underlying supporting structure remained intact. Figure 3.8a shows the side of cars which had been exposed to a peak side-on overpressure of 200 to 300 kPa.
Figure 3.6 – View of cars exposed to gas explosion, sorted by increasing overpressure exposure (SCI, 2009).

Figure 3.7 – Arrangement of the cars before the test explosion (SCI, 2009).

Similar pattern of damage was observed for other cars. Figure 3.8b shows a Ford Sierra exposed to a peak side-on overpressure of 400 kPa. Major structural damage was observed at a peak side-on overpressure of 1 MPa, like shown in Figure 3.9 where the damage was caused by a peak side-on overpressure of about 1 MPa with a positive duration of 11 ms.

Figure 3.8 – Cars after HE explosion tests: a) nose oriented; b) side oriented (SCI, 2009).
There were other tests carried out by the Centre for the Protection of National Infrastructure (CPNI) in the UK and Engineering Research Development Centre (ERDC) in the USA on vehicles exposed to high blast overpressures from a range of explosive charges including ANFO. Details of these tests are classified. But the results of the BP tests described here are consistent with those by CPNI and ERDC. The results showed that for significant structural damage similar to the one observed in Buncefield, a peak side-on overpressure of well over 200 kPa is required.

Cars are complex structures, the response of which to blast loading could be different. The test results showed that fairly high explosion overpressure is required to deform a vehicle causing its structural damage. The BP tests described were only able to deform the vehicle predominantly on one side, whereas, the Buncefield vehicles were uniformly crushed indicating that the pressure loading was uniform around the vehicles. From damage observed in the BP tests, the overpressure required to cause the level of damage observed in Buncefield was estimated to be around 1 MPa with duration of the positive phase of 11 ms. However, due to the different overpressure characteristics between gas explosion and HE, from other available data, it is likely that overpressures experienced by cars at Buncefield exceeded 200 kPa for durations typical of gas explosion in the open (around 50 ms).

3.3.3 Steel switch boxes

A number of switch boxes (Figure 3.10) were tested under a different range of loading conditions trying to reproduce the damage observed in Buncefield. A series of tests with
metal switch boxes in enclosed gas explosions was carried out. The switch boxes showed no sign of deformation or any damage except in the last test where the maximum overpressure was 180 kPa. The damage suffered by the box was relatively minor, when compared with those observed in Buncefield, with a maximum deformation of 11 mm.

Some static hydraulic testing of small boxes (300×300×130 mm³) has been carried out in a diving equipment test chamber at HSL. Pressurisation used air in a cylinder reservoir and was completed in approximately 2 seconds. An applied overpressure of 100 kPa caused minimal damage to the box as shown in Figure 3.11.

Application of a pressure of 400 kPa over a period of 2 seconds caused substantial plastic deformation of the box back and sides (Figure 3.12). The front was only slightly deformed. The pressurisation was slow enough to allow very large amplitude deformation via the first observed failure. This caused an internal pressure rise and tore the welding, allowing rapid inflow of water. The lack of large amplitude deformation of the front of the box in Figure 3.12 does not mean this could not have occurred if the box had been exposed to a rising rapidly to 400 kPa pressure.

Switch boxes were also exposed to blast overpressures from HE in a number of tests. Two box sizes were used, made from carbon steel. HE produces pressure waves of different characteristics to those from gas explosions – blast wave from HE has a much shorter duration when compared with a gas explosion and the wave profile is also different. Peak side-on overpressures ranged from 100 to 800 kPa. It was observed that
the panel of the box facing the blast could reproduce the damage observed in Buncefield. A more detailed description of this HE tests can be found in Section 3.4.1 of this document.

![Images of damaged small boxes after 100 kPa and 400 kPa static hydraulic tests](image)

Figure 3.11 – Damaged small box after 100 kPa in the static hydraulic test: a) closed box; b) open box (SCI, 2009).

The switch boxes were found to be unexpectedly strong. The smaller boxes (300×300×130 mm$^3$) were much stronger than the larger ones (600×600×600 mm$^3$) in resisting overpressures. Static pressure tests showed that the box can withstand 100 kPa overpressure with minor damage. A 400 kPa static loading was capable of crushing a small switch box. Both the gas explosion and the HE tests indicated that damage at about 200 kPa, for a range of explosion duration of about 50 ms, is minor. Results indicated that the pressure required to cause the level of damage at Buncefield exceeds 200 kPa.

![Images of damaged small boxes after 400 kPa static hydraulic tests](image)

Figure 3.12 – Damaged small box after 400 kPa in the static hydraulic test: a) side view; b) back view (SCI, 2009).
3.4 ISO-DAMAGE CURVES

The damage of a structure when subjected to blast loading is dependent on the impulse and the pressure. It is possible to plot P-I (pressure-impulse) diagrams or Iso-damage curves (Figure 3.13) which allow to define in an easy way the pressure impulse combination that will cause a specific level of damage.

![Figure 3.13 – Generic P-I (Pressure-Impulse) diagram.](image)

These plots establish a relation between the pressure/impulse and the permanent deformation on a structure. To do so, either experimental tests or FE (Finite Element) modelling are used to predict the structural response; several experimental or numerical simulations are required for a selected range of overpressures and durations (impulses).

In the following section, the experimental work done to calibrate the FE model will be shown together with the FE modelling used to generate P-I diagrams for metal switch boxes under blast loading from High Explosives.

3.4.1 Experimental analysis

A number of switch boxes were exposed to blast overpressure from HE in a number of tests. These boxes were carbon steel boxes. In each test, boxes were placed at locations to receive specific overpressure loading for the explosive charge used. Figure 3.14 shows a typical general arrangement. It can be seen that, for each test, two boxes (one
large box and one small box) had the front face facing the charge and another two (one large box and one small box) had its side face facing the charge.

Peak side-on overpressures, $P_{SO}$, in tests ranged from 100 kPa to 800 kPa. The reflected pressure, $P_r$ (pressure experienced by the side of the box that faced the charge) ranged from 320 kPa to 5 MPa. The duration of the highest peak side-on overpressure was about 8 ms and the duration of the lowest peak side-on overpressure was about 17 ms for the positive phase of the blast wave. These are within the range of pressure durations expected in detonations.

The characteristic damage was predominantly by crushing of the side facing the blast. This is different from the damage observed in Buncefield where boxes were crushed on all faces. One of the characteristics of these HE tests is that pressure decayed rapidly at these levels of overpressure. Thus, the peak side-on pressure on the face closer to the charge is larger than the face at the back or the lateral faces. It is noted that in some cases the back face was pushed outwards (rather than crushed inwards). This is due to the effect of adiabatic compression. The crushing of the front face caused an increase in internal pressure which deformed the back face outwards. These factors contributed for the difference in observed damage patterns between the boxes tested and those in Buncefield. However the deformation suffered by the side facing the blast provided indicative overpressures required to crush boxes observed in Buncefield.

![Figure 3.14 – General scheme of the experimental study: a) two different box sizes; b) box orientation facing the blast (SCI, 2009).]
Figure 3.15 shows the deformation of the panels which faced the blast wave for the large boxes. These four boxes were exposed to the values of peak side-on overpressure and reflected pressure registered in Table 3.2 for the front face and the side face.

Figure 3.16 shows the deformation of the panels which faced the blast wave for the small boxes. No significant deformation was registered for the lower values of peak side-on overpressure. In Table 3.2 it is possible to identify the tests carried out and the respectively pressures and impulses. After the test, the boxes were measured in terms of residual deformation and the results were recorded.

Figure 3.15 – Large boxes after HE tests: a) boxes hit on the front panel; b) boxes hit on the side panel (SCI, 2009).

Figure 3.16 – Small boxes after HE tests: a) boxes hit on the front panel; b) boxes hit on the side panel (SCI, 2009).
3.4.2 FE modelling

Previous studies have shown different methods to predict the structure response to blast loading. Studies lead to satisfactory results in the prediction of the permanent deformations of structures subjected to blast loading using SDOF (Single Degree of Freedom) models according to Yang et al (1997), Liew et al (2008), Fisher et al (2009), Jones et al (2009) and Schleyer et al (2009). This approach has however several limitations. With the increase of computers capacity and the constant development of software, the FE (Finite Elements) method has been getting the attention of the scientific community in the past few years.

Several studies in metal structures modelled using FE subjected to blast loading are available in order to give insight on the particularities of the modelling in terms of software and material response. Yuen et al (2005) studied the response of quadrangular stiffened plates subjected to uniform blast loading. They have also investigated the deformation of mild steel plates subjected to large-scale explosions. According to their results the use of the Hopkinson-Cranz scaling laws have proven to be useful to evaluate pressures, time durations and impulses; and the use of proper explicit dynamic codes can lead to a reasonable agreement with experimental results.

Baldeen and Nurick (2005), Theobald and Nurick (2007) and Bonorchis and Nurick (2009) studied the influence of boundary conditions of the loading of rectangular plates subjected to localized blast loading. They showed that axial crushing of tubes sandwiched between steel panels could be used to absorb significant energy from a blast load and studied the post-failure motion of steel plates subjected to blast loading. Sabuwala et al (2005) analysed beam to column connections subjected to blast loads, showing that the UFC 3-340-02 (2008) over-designs these elements. Krauthammer (1999) proposed a model to predict the behaviour of structural concrete and structural

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Table 3.2 – Reflected pressures and impulses for the experimental tests.

<table>
<thead>
<tr>
<th>Test</th>
<th>Reflected Pressure (MPa)</th>
<th>Reflected Impulse (kPa.s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>0.65</td>
<td>4.70</td>
</tr>
<tr>
<td>T2</td>
<td>0.32</td>
<td>2.60</td>
</tr>
<tr>
<td>T3</td>
<td>2.39</td>
<td>13.80</td>
</tr>
<tr>
<td>T4</td>
<td>5.20</td>
<td>20.80</td>
</tr>
</tbody>
</table>
steel connections subjected to blast loading conducting a series of numerical simulations and concluded that the current design procedures should be modified for a better prediction under these loading conditions. Børvic et al (2008a, 2008b) managed to predict the structural response of a protective structure subjected to blast loading using LS-DYNA achieving a good relation with the deformations verified in the experimental studies. Hanssen et al (2002) investigated the behaviour of aluminium foam panels subjected to blast loading, using an analytical approach and the non-linear finite element analysis provided the same conclusions. Jama et al (2009) through numerical modelling studied square tubular steel beams subjected to transverse blast loading using LS-DYNA and concluded that these elements undergo local cross-sectional deformation followed by global beam bending deformation and highlighted the importance of the strain-rate hardening for proper detail in both local and global deformation.

Børvic et al (2009) compared the response given using the coupled formulation and the pure Lagrangian formulation. The results showed a good agreement obtaining the pressure-time profiles of the blast when using two different approaches: a) specialized software, in this case ConWep (ConWep, 1991) or; b) empirical equations available in the literature (UFC 3-340-02, 2008; Mays and Smith, 1995; Bangash and Bangash, 2006). To model loading conditions on the switch boxes the pressure-time recorded in the experimental studies was applied to the structure using a pure Lagrangian formulation. The steel material was modelled using an isotropic hardening model, pertaining to the Von Mises yield condition, and strain rate-dependent dynamic yield stress based on Cowper and Symonds model (Børvic et al, 2009).

In this work, the Finite Element (FE) software LS-DYNA was used to model this blast loading problem. This software allows three different approaches: (i) Pure Lagrangian formulation, in which the load is idealised as a pressure-time curve applied directly to the surface; (ii) Running an Eulerian simulation before the Lagrangian simulation, the objective is to obtain the pressure-time load on all faces around the structure; (iii) The Eulerian formulation can be applied with the Lagrangian formulation to have a full coupling between the blast waves and the structure deformations, even if the use of the coupled Eulerian-Lagrangian formulation increases considerably the computational time (LS-DYNA User’s Manual, 1997).
To model the switch boxes the pressure-time recorded in the experimental studies was applied to the structure and a pure Lagrangian formulation for the explicit simulation was used. The box was modelled with thin-shell elements specific for explicit dynamic analysis, with a thickness of 1.5 mm; the dimensions are presented in Figure 3.17 for the small box (for the large box the design is similar but with 600×600×230 mm³). The FE model was based on Belytschko-Lin-Tsay quadrilateral shell elements (LS-DYNA User’s Manual, 1997).

The steel was modelled using a piecewise linear hardening law, pertaining to the Von Mises yield condition with isotropic strain hardening, and strain rate-dependent dynamic yield stress based on the Cowper and Symonds model. According to previous researches on modelling metal structures under these extreme actions, the results obtained in the experimental work and the expected strain rates for this kind of action, the mechanical properties to be adopted are: a Young’s modulus of 210 GPa, a yield stress of 1000 MPa and a Poisson’s ratio 0.28, shown in Figure 3.18. For practical purposes, the behaviour is elastic/perfectly plastic, with a very low Young’s modulus in the second branch.

The applied load was modelled as a pressure time curve with a triangular shape as shown in Figure 3.19. In the side facing the blast the reflected pressure was applied and also in all other faces the incident pressure was applied. In all cases the pressure was applied inwards as seen in Figure 3.19 for the situation where the box has the side face towards the explosion.
After defining the material properties and the geometric model of our boxes, several simulations were carried out. The first step was to reproduce in FEM the experimental results produced in the previous section. At this stage four situations were defined:

- small box with the side panel facing the blast (SBS);
- small box with the front panel facing the blast (SBF);
- large box with the side panel facing the blast (LBS);
- large box with the front panel facing the blast (LBF).

Figure 3.20 represents one example of a numerical simulation of the large box with the front face facing the blast. The deformed shape evolution in time is such that, although all faces are subjected to inwards pressure, the highest value of the reflected pressure related to the incident pressure forces, in some cases, the side faces with a deformed shape in the opposite direction of the loading. This is consistent with the experimental tests.
Figure 3.20 – Deformed shape of the FE model for the large box hit on the front.

Figure 3.21 shows the comparison between the numerical study of the small box hit on the side with the respective experimental specimen, with the evolution in time of the deformed shape. The same damage pattern in both numerical and experimental tests can be found.

Condensing the results of numerical and experimental studies, a comparison can be established in Table 3.3. There were situations where the comparison had to be made in a panel other than the one facing the blast and some cases where it was not possible to compare at all; this is due to severe damage in some experimentally tested boxes.

The results show a difference, in terms of percentage ranging from 3.8% to 32.7% and range of 0.2 to 32.4 mm in terms of maximum displacement. As expected, the greater the pressure, greater the deformation in both numerical and experimental studies. Also the higher differences are registered for the large boxes when compared with the small boxes. The boxes hit on the front also showed larger differences when compared with those hit on the side face. When the deformation increases, the difference between the numerical and experimental tests also increases.
Table 3.3 – Summary of the comparison between the numerical and experimental analyses.

<table>
<thead>
<tr>
<th>Test</th>
<th>Residual Deformation (mm)</th>
<th>Difference (mm)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Exp.</td>
<td>Num.</td>
<td></td>
</tr>
<tr>
<td>$I_r=2.6$ kN.s (T2)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SBF</td>
<td>15.7</td>
<td>19.0</td>
<td>3.3 (17.1%)</td>
</tr>
<tr>
<td>LBS</td>
<td>20.0</td>
<td>23.9</td>
<td>3.9 (16.4%)</td>
</tr>
<tr>
<td>LBF</td>
<td>94.1</td>
<td>126.5</td>
<td>32.4 (25.6%)</td>
</tr>
<tr>
<td>$I_r=4.7$ kN.s (T1)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SBF</td>
<td>1.6</td>
<td>1.4</td>
<td>0.2 (10.4%)</td>
</tr>
<tr>
<td>LBS</td>
<td>25.5</td>
<td>17.2</td>
<td>8.3 (32.7%)</td>
</tr>
<tr>
<td>LBF</td>
<td>32.9</td>
<td>30.2</td>
<td>2.7 (8.2%)</td>
</tr>
<tr>
<td>$I_r=13.8$ kN.s (T3)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SBS</td>
<td>56.0</td>
<td>53.9</td>
<td>2.1 (3.8%)</td>
</tr>
<tr>
<td>SBF</td>
<td>7.4</td>
<td>6.6</td>
<td>0.8 (11.7%)</td>
</tr>
<tr>
<td>LBS</td>
<td>124.9</td>
<td>142.4</td>
<td>17.5 (12.3%)</td>
</tr>
<tr>
<td>$I_r=20.8$ kN.s (T4)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SBS</td>
<td>16.4</td>
<td>13.2</td>
<td>3.2 (19.6%)</td>
</tr>
<tr>
<td>SBF</td>
<td>29.8</td>
<td>40.2</td>
<td>10.4 (25.9%)</td>
</tr>
<tr>
<td>LBF</td>
<td>27.1</td>
<td>30.2</td>
<td>3.1 (10.1%)</td>
</tr>
</tbody>
</table>

3.4.3 Final plots

In order to produce the final P-I diagrams from these sub-structures when subjected to blast loading, a new set of numerical simulations took place. Several similar simulations were made by ranging both the peak side-on overpressure/reflected pressure and the positive phase duration (side-on impulse/reflected impulse). The maximum permanent deformation on the side facing the blast was recorded for each simulation and P-I
diagrams were created. The following plots represent all four situations for a selected range of reflected impulse of 0 kPa.s to 12 kPa.s:

- small box with the side panel facing the blast (SBS);
- small box with the front panel facing the blast (SBF);
- large box with the side panel facing the blast (LBS);
- large box with the front panel facing the blast (LBF).

Analysing the P-I diagrams plotted it can be seen that there are two well defined regions (Figure 3.26). The “Impulsive region” representing the region with the high pressures and lower durations and the “Quasi-static region” representing the region with the lower pressures and longer durations.

![Figure 3.22 – P-I diagram for the small box hit on the side face. Iso-damage curves: 20, 30, 40, 50, 60, 70, 80 and 90 mm.](image-url)
Figure 3.23 – P-I diagram for the small box hit on the front face. Iso-damage curves: 40, 50, 60, 70, 80, 90 and 100 mm.

Figure 3.24 – P-I diagram for the large box hit on the side face. Iso-damage curves: 30, 50, 70, 90, 110, 130, 150 and 170 mm.
Knowing that a vapour cloud explosion creates pressure waves with longer duration when compared with HE, the region representing the situation at Buncefield is the Quasi-static region, with longer durations. In the particular case of Buncefield, it was only possible to extract one junction metal box. As shown in Figure 3.27 the box had a deformation of about 70 mm on the front. Using the P-I diagram for the small box hit on the front (Figure 3.23) and knowing that VCEs are in the Quasi-static region, the pressure required to damage the box at that level would be around 270 kPa. This shares
the same conclusions as the CFD analysis and the other pressure indicators studies. This indicates that the presented methodology to study the blast loading parameters through pressure indicators should be able to predict the range of overpressures and impulses capable of damaging these sub-structures.

Figure 3.27 – Crushed junction box at Buncefield.

3.5 Final Remarks

This chapter provided a new methodology for predicting the blast loading parameters through pressure indicators. Taking Buncefield Major Incident as a case study, sub-structures were adopted to explain the severity of the explosion and the magnitude of the experienced overpressures. The obtained results showed a good agreement with the empirical and numerical solutions for estimating the blast loading parameters.

Finite Element Analyses (FEA) is one of the most used techniques to predict the behaviour of structures. FE software like LS-DYNA has been proved capable of, with good accuracy, model explicit dynamic situations. In the present work the geometric and material properties of switch boxes were calibrated using the results from the experimental study. It is shown that for higher values of pressure/impulse the difference between the experimental and numerical becomes slightly larger.

Four P-I (pressure-impulse) diagrams are presented allowing to relate the overpressures and the reflected impulse with the obtained deformation. It is possible to see a quasi-static domain and an impulsive domain in all plots. These tools can help assessing the
magnitude of overpressures and impulses in the event of accidental explosion. This is possible by backtracking the loading characteristics from the damage observed in steel switch boxes under similar loading conditions.

Explicit dynamic simulations are still in an early stage of development as many of the modelling possibilities for the implicit analysis are not yet available for the explicit analysis. Blast loading simulating still needs to concentrate the attention of researchers in terms of the material properties for high strain-rate situations. The commercial codes should be improved too in terms of available element types and material models.

3.6 REFERENCES


ConWep, *Conventional weapons effects*. US army Engineers Waterways Experiment Station, USA, 1991.


Chapter 3 – Assessing Blast Loading Parameters from Post-Disaster Scenarios


UFC 3-340-02, **Structures to resist the effects of accidental explosions.** Department of Defence, USA 2008.


Different loading conditions lead to different strain rates. Quasi-static loading produce strain rates of around $10^{-5}$ s$^{-1}$, while impacts and blast loading produce strain rates of well over $10^{0}$ s$^{-1}$. When subjected to dynamic loading conditions, materials can have a much different behaviour when compared with their static behaviour (Meyers, 1994; Hiermaier, 2008; Ngo et al, 2004; Stavrogin and Tarasov, 2001). Current research work on structural response and damage under impact and blast loading assumes typically static material properties (Baylot et al, 2005; Moreland et al, 2005). This can lead to an inaccurate prediction of structural damage and fragmentation.

Construction materials such as concrete or reinforcement bars have been studied under strain rate effects (Grote et al, 2001; Malvar, 1998), phenomenon already introduced into some standards such as CEB-FIP (1990) or UFC 3-340-02 (2008). However, very limited studies can be found in the literature on masonry materials, such as clay bricks
or mortar. Recently Hao and Tarasov (2008) conducted an experimental study under dynamic uniaxial compression using a Triaxial Static-Dynamic Testing Machine. These authors reported a DIF for the compressive strength of around 2.3 and 1.12 for the DIF of the ultimate strain, for a strain rate of 150 s⁻¹. For the Young’s modulus a DIF of 1.95 was reported for the same level of strain rate.

In this chapter, an experimental campaign on the influence of the strain rate on the mechanical properties of masonry and its components is described. The tests were conducted with a Drop Weight (DW) tower available at the Mechanical Engineering Department in the University of Minho. This equipment consists of a “hammer” with a given mass being released at a chosen height. Authors like Islam and Bindiganavile (2011), Zhang et al (2010) and Banthia et al (1998) have used this kind of testing apparatus to investigate the influence of the strain rate effect on different materials.

4.1 TESTING EQUIPMENT AND PROCEDURE

A Drop Weight Impact Machine (DW) was used to perform the compression tests at different strain rates. This equipment is available in the Mechanical Engineering Department at the University of Minho (Figure 4.1). It allows drop heights up to 9 m and the weight of the hammer ranges from 40 kg to 150 kg.

The load profile was measured at the base of the test specimen using a load cell specifically for dynamic applications – VETEK c2s (Figure 4.2a). This load cell is connected to a National Instruments Acquisition System (Figure 4.2b). This acquisition system is composed of a SCXI-1000DC chassis, a SCXI-1600 data acquisition and control card for PC connection and a generic input module SCXI-1520 with a SCXI-1314 mount. The SCXI-1600 limits the sampling speed to 200 kS/s (200 samples per millisecond), which was found to be enough even at a later stage where 5 channels where used at the same time, allowing an acquisition frequency of 40 kHz per channel. Due to previous inexperience with acquiring such high frequency acquisitions at the Department of Civil Engineering, the LabView acquisition software had to be developed as well. Here the main concern was to keep the software as light as possible,
in order to minimize possible errors or data transfer failures. The software consists of a simple loop where the data is stored during a predetermined period of time (Figure 4.3).

Figure 4.1 – DW tower at the Mechanical Engineering Department.

Figure 4.2 – Load acquisition hardware: a) Vetek c2s; b) National Instruments acquisition system.

Figure 4.3 – Acquisition software in LabVIEW 2010.
The deformation behaviour of the specimen was measured in two different ways. First, a FastCam video camera was used. It is a PHOTRON FastCam APX – RS (Figure 4.4) with a maximum frame rate of 250 000 frames per second. This equipment allowed the visualization of the test in slow motion and the measuring of the strain in one face of the test specimen. This strain measurement was possible using targets in the specimen at a specific location and performing a tracking sweep of those targets in the video. To perform the tracking sweep, the TEMA Tracking Software (v: 3.1-005) was used. With the relative position of the targets, the strain at each instance was calculated. The second methodology used to get the strain time history was using strain gauges. The strain gauges used were PFL-30-11-3L from TML and in this case, one strain gauge was placed in each face of the specimen. In the quasi-static tests, strain gauges were used to measure the strains in the specimen. In the dynamic tests, the FastCam was used to measure the strains. However, in order to validate and compared the results, some dynamic tests were also performed using strain gauges. These gauges were connected to the same acquisition equipment of the load cell.

![PHOTRON FastCam APX-RS](image1.jpg)

**Figure 4.4 – PHOTRON FastCam APX-RS.**

Figure 4.5 shows the schematic of the test setup. Varying the weight and the drop height of the impact hammer, different strain rates will be introduced in the specimen. Knowing the load/stress time history (Figure 4.6a) and the strain time history (Figure 4.6b) it is possible to plot the stress-strain relations (Figure 4.7) for different strain rates. From these relations all properties can be derived: a) compressive strength; b) strain at peak strength; c) Young’s modulus and d) fracture energy in compression. As can be seen in Figure 4.6b the strain rate of each specimen is not necessarily constant, varying in time. In this case the strain rate for each specimen is taken as constant and equal to the gradient of the strain-time curve. This procedure was previously used also by Hao and Tarasov (2008).
Figure 4.5 – Schematic of the test setup: 1) DW tower; 2) additional masses; 3) hammer; 4) test specimen; 5) load cell; 6) acquisition system; 7) fastcam video.

Figure 4.6 – Examples of time histories obtained in the present work: a) stress-time; b) strain-time.

Figure 4.7 – Examples of stress-strain relations obtained in the present work for different strain rates.
4.2 Test specimens description

When testing at high strain rates, using dynamic loading, it is difficult to avoid resonances and inertial effects. Specimens are often kept small in order to retain the assumption of stress equilibrium within the deforming specimen. Lateral friction may also provide some confinement at the two ends of the specimen under compression tests. Therefore, the dimensions of specimens must be a compromise between maximizing the size to ensure a complete representation of the material, proper height to base width / thickness ratio to reduce the friction effects at the ends, and minimizing the sample size to reduce the effect of inertia and non-uniform stress and strain in the specimen (Harding, 1989 and Dioh et al, 1995).

In this work, handmade clay bricks and mortar were studied. The following sections present the procedure for preparing the specimens and the selected dimensions for each test. The clay brick and mortar were studied independently and, together, in masonry specimens.

4.2.1 Brick specimens

With the objective of reproducing old Portuguese masonry construction, the brick used was from Galveias, a village located in the central part of Portugal where handmade bricks can still be found. Brick specimens were prepared from a number of solid handmade clay bricks. The Galveias brick (Figure 4.8a) measured 20x10x5 cm in dimensions and the test specimen (Figure 4.8b) measured 7x3x3 cm.

![Figure 4.8 – Handmade clay brick: a) Galveias brick; b) test specimen.](image)
From each brick, five specimens were prepared. The test specimens were cut from the original brick by means of a disk cutting machine (Figure 4.9a). After cutting the specimens, their edges were ensured to be intact and the loadbearing surfaces at both ends of the specimen were ground flat and parallel to each other. This was achieved using a grinding machine (Figure 4.9b).

When the necessary preparation works were completed the specimens were left to dry in a ventilated oven at 105ºC until reaching constant mass. When constant mass was achieved they were moved to a non-ventilated oven at 40ºC and were kept there almost until testing, removed only 1 hour before testing. This procedure followed the recommendations of the EN 772-1 (2002).

### 4.2.2 Mortar specimens

A commercial ready-mix mortar was used, MAPEI MAPE-ANTIQUE MC (Figure 4.10a). Several mixtures were prepared to evaluate the proper amount of water to use, keeping a good workability. The ratio was kept at 25 kg of product for 3.9 litres of water, resulting in 16 cm flow (Figure 4.10b).

The test specimens’ dimensions for the mortar were kept the same as the brick specimens (7x3x3 cm), in order not to adapt the testing rig. After the mixture was placed in the moulds (Figure 4.11a), these were placed in a climatic chamber at 25ºC and 65% humidity for five days. After these five days, the specimens were taken out of their moulds and tested in that same day. The compressive strength of mortar was
intended to be similar to the values of old mortars and, as can be seen in Figure 4.11b, after five days of curing the compressive strength of this mortar is about 3 MPa.

Figure 4.10 – Mortar: a) Mape-Antique MC; b) flow table.

Figure 4.11 – Mortar: a) specimens’ moulding; b) compressive strength at different ages.

4.2.3 Masonry specimens

The masonry specimens were composed of clay bricks and mortar and were prepared according to the European standard EN 772-1 (2002). Test setup limitations lead to the final dimensions of 23x8x8 cm for the masonry specimens. Again, each Galveias brick was cut to match the required dimensions (Figure 4.12). Each masonry specimen had four brick units and three layers of mortar with one centimetre thickness.
In order to facilitate the placement of the masonry specimens in the test setup, they were built on top of an aluminium plate which connects to the load cell. There is a thin layer of mortar between the first brick and the aluminium plate (Figure 4.13a) making sure that there is full contact between those two. The specimens were built in a flat surface, but because the brick faces on top and bottom are irregular, on top of the specimens a layer of self-levelling mortar was used (Figure 4.13b). The treatment regarding the brick units used in the masonry specimens were the same as the specimens for the brick specimens, and the time of curing for the masonry specimens was the same as the mortar specimens.

4.3 QUASI-STATIC REGIME

In order to have the quasi-static reference for comparison with the results subjected to the dynamic regime, an experimental campaign on the behaviour of these materials under quasi-static uniaxial compression was performed. A physical characterization was
also performed on clay brick and mortar, with the objective of understanding possible characteristics that could affect the results under dynamic conditions.

### 4.3.1 Compression tests

For the quasi-static compression tests, the test setup consisted of a steel frame which supported a servo-controlled actuator, with a 25 kN (50 kN for the masonry specimens) load capacity. The strain in the test specimen was obtained with three LVDTs and four strain gauges (Figure 4.14). These tests were performed according to the EN 772-1 (2002).

![Quasi-static acquisition apparatus.](image)

The mechanical properties under study were: a) compressive strength ($\sigma_{\text{max}}$); b) strain at peak strength ($\varepsilon_u$); c) Young’s modulus ($E$); and d) compressive fracture energy ($G_c$). Figure 4.15 shows the typical stress-strain relation and stress-displacement relation with information on how to determine these mechanical properties. The compressive strength is the maximum value of the stress-strain curve (Figure 4.15a). At that point the corresponding strain is the strain at peak strength. The Young’s modulus is calculated as the slope of the stress-strain diagram between 30% and 80% of the maximum stress. The compressive fracture energy is determined from the stress-displacement curve and corresponds to the marked area in Figure 4.15b. This procedure, to calculate the post-peak fracture energy in compression has been used before by Vasconcelos (2005) and is similar to the one indicated by Jansen and Shah (1997).

A total of 5 (five) specimens of handmade clay brick, 9 (nine) specimens of mortar and 4 (four) specimens of masonry were tested under quasi-static uniaxial compression and
the results are presented in Table 4.1, Table 4.2 and Table 4.3, respectively. The value between brackets is the coefficient of variation (CoV). The brick specimens have the highest compressive strength with 13.59 MPa, where the mortar has strength of 4.46 MPa. This resulted in masonry specimens with 7.94 MPa of compressive strength. Also important, the Young’s modulus is 2.32 GPa for the brick specimens and 0.80 GPa for the masonry specimens. The fracture energy in compression is similar for both brick and mortar specimens, with 1.56 N/mm and 1.43 N/mm respectively, while for masonry the fracture energy is much larger, reaching 7.64 N/mm. The reason for this is that the masonry specimens showed higher deformation capacity due to the interaction between the masonry components. The CoV for the handmade clay brick under quasi-static compression is slightly high when compared to the other tests performed. This is a material made by hand and where the quality control of the product and the manufacturing procedure is not as high as those from newer materials.

![Typical relations: a) stress – strain; b) stress – displacement.](image)

**Figure 4.15** – Typical relations: a) stress – strain; b) stress – displacement.

<table>
<thead>
<tr>
<th>Table 4.1 – Quasi-static mechanical properties of clay brick.</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1</td>
</tr>
<tr>
<td>---</td>
</tr>
<tr>
<td>σ&lt;sub&gt;max&lt;/sub&gt; [MPa]</td>
</tr>
<tr>
<td>ε&lt;sub&gt;u&lt;/sub&gt; [mm/m]</td>
</tr>
<tr>
<td>E [GPa]</td>
</tr>
<tr>
<td>G&lt;sub&gt;c&lt;/sub&gt; [N/mm]</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 4.2 – Quasi-static mechanical properties of mortar.</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1</td>
</tr>
<tr>
<td>---</td>
</tr>
<tr>
<td>σ&lt;sub&gt;max&lt;/sub&gt; [MPa]</td>
</tr>
<tr>
<td>ε&lt;sub&gt;u&lt;/sub&gt; [mm/m]</td>
</tr>
<tr>
<td>E [GPa]</td>
</tr>
<tr>
<td>G&lt;sub&gt;c&lt;/sub&gt; [N/mm]</td>
</tr>
</tbody>
</table>
Figure 4.16 shows the stress-strain envelopes for the three materials under study. As it can be seen, the post-peak behaviour of mortar is more ductile than those of brick or masonry.

### Table 4.3 – Quasi-static mechanical properties of masonry.

<table>
<thead>
<tr>
<th>Property</th>
<th>#1</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
<th>Average (CoV)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{\text{max}}$ [MPa]</td>
<td>8.25</td>
<td>7.13</td>
<td>8.78</td>
<td>7.59</td>
<td>7.94 (9%)</td>
</tr>
<tr>
<td>$\varepsilon_{u}$ [mm/m]</td>
<td>11.21</td>
<td>8.87</td>
<td>10.66</td>
<td>12.97</td>
<td>10.93 (15%)</td>
</tr>
<tr>
<td>$E$ [GPa]</td>
<td>0.81</td>
<td>0.85</td>
<td>0.90</td>
<td>0.64</td>
<td>0.80 (14%)</td>
</tr>
<tr>
<td>$G_c$ [N/mm]</td>
<td>7.95</td>
<td>7.53</td>
<td>7.68</td>
<td>7.40</td>
<td>7.64 (3%)</td>
</tr>
</tbody>
</table>

The values presented in Table 4.1 to Table 4.3 are assumed next as the static properties for these materials and were taken as the static reference when determining the dynamic increase factors.

#### 4.3.2 Physical properties

In order to have a deeper characterization of the materials under study, tests to obtain selected physical properties for clay brick and mortar were also performed. These results can help to justify differences in the behaviour of these materials in quasi-static and dynamic conditions. The following properties were studied here: a) water absorption coefficient due to capillarity action; b) water absorption by immersion; c) density; and d) porosity.
The tests performed to obtain the water absorption coefficient due to capillarity action followed the requirements in the standard EN 1015-18 (2002). Six specimens for each material were used. The specimens were placed in a climatic chamber at 40ºC until reaching constant mass. When constant mass was achieved the specimens were prepared for the test, which consisted in allowing only one surface to be in contact with water, using water resistant ink in all surfaces but the one in contact with water (Figure 4.17a). The specimens were then placed in a closed container during a specific amount of time. After 10 minutes and 90 minutes the specimens were taken out of the container and the mass was recorded. With the mass of each specimen after 10 and 90 minutes it is possible to determine the water absorption coefficient due to capillarity action $C$, using the following equation:

$$C = 0.1 \times (M2 - M1) \text{ kg} / (\text{m}^2 \times \text{min}^{0.5})$$  \hspace{1cm} (4.1)

Where:

$M1$ – mass of the specimen after 10 minutes in grams

$M2$ – mass of the specimen after 90 minutes in grams

Figure 4.17b shows the results obtained for clay brick and mortar. The average water absorption coefficient in the brick specimens was $0.55 \text{ kg} / (\text{m}^2 \cdot \text{min}^{0.5})$, which compares to the a much smaller coefficient for the mortar, equal to $0.11 \text{ kg} / (\text{m}^2 \cdot \text{min}^{0.5})$.

Figure 4.17 – Water absorption coefficient: a) preparing specimens; b) results obtained.
The tests performed to obtain the water absorption by immersion followed the requirements in the standard EN 1097-6 (2000). Six specimens for each material were used. The specimens were placed inside a container with water (Figure 4.18a) and their masses were recorded every 24 hours until reaching their saturated mass. Afterwards, the specimens were moved to a climatic chamber at 40ºC until reaching constant mass. Knowing the difference in mass between the water saturated mass and the dry mass it is possible to determine the water absorption percentage \( WA \) for each material using the following equation:

\[
WA = \frac{(M_1 - M_2)}{M_2} \times 100 \, (\%) \tag{4.2}
\]

Where:
- \( M_1 \) – Saturated mass
- \( M_2 \) – Dry mass

Figure 4.18b shows the results obtained for clay brick and mortar. Again, the average water absorption percentage is much higher for the clay brick specimens (12.6 %) than the percentage for the mortar specimens (5.0 %).

![Figure 4.18 – Water absorption: a) specimens submersed; b) results obtained.](image)

Density is the relation between the mass and the volume of the specimens. Six specimens for each material were used. The specimens were placed in a climatic
chamber at 40ºC until reaching constant mass. Knowing the mass and the dimensions of the specimens it is possible to determine their density \( \rho \) according to the following equation:

\[
\rho = \frac{M}{V} \text{ (Kg/m}^3) \tag{4.3}
\]

Where:

- \( M \) – Dry mass in kilograms
- \( V \) – Volume in cubic meters

Figure 4.19 shows the results obtained for clay brick and mortar. Here, the results of clay brick and mortar are similar, having average densities of 1795 kg/m³ and 1853 kg/m³, respectively.

Porosity is the relation between the volume of voids inside the specimens and their bulk volume. The procedure followed to determine the porosity of the clay brick and mortar specimens is described in the Standard LNEC E 394 (1993). Again, six specimens of each material were used. The specimens were introduced in water until reaching their saturated mass. Afterwards, the saturated mass and the hydrostatic mass was recorded. The specimens were then moved to a climatic chamber at 40ºC until reaching constant dry mass. Knowing the masses for these specimens, it is possible to determine the porosity \( A_i \) according to the following equation:
\[ A_i = \frac{m_1 - m_3}{m_1 - m_2} \times 100 \% \]  \hspace{1cm} (4.4)

Where:

- \( m_1 \) – Saturated mass
- \( m_2 \) – Hydrostatic mass
- \( m_3 \) – Dry mass

Figure 4.20 shows the results obtained for both materials. As expected the percentage of voids in the clay brick specimens (23 \%) is much higher than the percentage for the mortar specimens (9 \%).

![Figure 4.20 – Porosity for brick and mortar specimens.](image)

Table 4.4 summarizes the measured physical properties for the materials components. The value between brackets is the coefficient of variation (CoV). As expected, the clay brick, due to manufacturing process, has higher percentage of voids inside its volume and this is reflected on the water absorption properties. The pore structure is certainly also much different, given the enormous difference in the water absorption coefficient due to to capillarity action.

<table>
<thead>
<tr>
<th></th>
<th>( C ) (kg/(m(^2).min(^{0.5}))</th>
<th>WA (%)</th>
<th>( \rho ) (kg/m(^3))</th>
<th>Ai (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Brick</strong></td>
<td>0.55 (17%)</td>
<td>12.6 (2%)</td>
<td>1795 (2%)</td>
<td>23 (2%)</td>
</tr>
<tr>
<td><strong>Mortar</strong></td>
<td>0.11 (9%)</td>
<td>5.0 (5%)</td>
<td>1853 (2%)</td>
<td>9 (4%)</td>
</tr>
</tbody>
</table>
4.4 DYNAMIC REGIME

For the dynamic regime testing, the Drop Weight (DW) impact machine was used. Several impact tests under uniaxial compression were performed. As presented previously, the objective is to obtain the stress-strain relations from the data recorded at the load cell acquisition and the fastcam video. To facilitate the treatment of the data obtained, the stress-time curve was approximated to a second degree polynomial, while the strain-time curve was approximated to a linear function. As a result, the final shape of the stress-strain curves is a second degree polynomial and can be seen in Figure 4.21.

![Figure 4.21 – Final stress-strain curves at different strain rates for clay brick.](image)

With the stress-strain relations for each test, the mechanical properties could be determined and the dynamic increase factor (DIF) could be calculated as a function of the strain rate, according to the following equation:

\[
DIF = \frac{\text{Property (Dynamic)}}{\text{Property (Static)}} \cdot f(\dot{\varepsilon})
\]  \hspace{1cm} (4.5)

These results were used to establish the relations between the mechanical properties and the strain rate. These relations are usually described as bi-log-linear relations, meaning that they can be written with two log-linear functions, low slope for the quasi-static regime and high slope for the dynamic regime. In order to simplify these relations, the first log-linear function for the quasi-static regime was considered constant and set as DIF equals to 1 (one) until the point where the regime changes to dynamic.
4.4.1 Brick dynamic properties

A total of 99 specimens of handmade clay brick were tested under uniaxial compression, with strain rate ranging from $4 \, \text{s}^{-1}$ to $199 \, \text{s}^{-1}$. Due to errors in the acquisition system only 58 tests gave good results and were used to characterize the behaviour of clay brick under increasing strain rates. Figure 4.22 shows some typical failure modes of the specimens under impact testing.

![Figure 4.22 – Typical failure modes for brick specimens under impact testing.](image)

For these tests the acquisition frequency for the strain-time curve was set at 20 kHz and the acquisition frequency for the stress-time curve was set at 40 kHz. The acquisition frequency on the video equipment was greatly dependant on the lighting conditions and was set as the maximum possible at the time of the tests. In order to facilitate the convergence of both acquisitions, the acquisition frequency on the load cell was set as a multiple of the frequency on the video equipment. Figure 4.23 shows a typical sequence of the impact test recorded with the fast video equipment.

Table 4.5 shows the results obtained on the impact tests for clay brick specimens. Only some of the results are shown here, however, to establish the DIF relations all 58 tests were used and can be seen in Annex A.1. Five additional tests using strain gauges were performed in order to cross-check the results obtained with the targets and video equipment. These five tests actually represent 20 tests, meaning that strain gauges were placed in all four faces of the test specimen and the average was taken for each test. In the video equipment, only one face of the test specimen was measured, meaning that rotation of the specimen is not captured and the single strain value has to be considered carefully.
Chapter 4 – Mechanical Characterization of Traditional Materials

Figure 4.23 – Typical test sequence for clay brick.

Table 4.5 – Impact tests on clay brick.

<table>
<thead>
<tr>
<th>Strain rate (s⁻¹)</th>
<th>Compressive Strength</th>
<th>Young’s Modulus</th>
<th>Strain at Peak Strength</th>
<th>Fracture Energy</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>σ_u (MPa)</td>
<td>E (GPa)</td>
<td>ε_u (mm/m)</td>
<td>G_c (N/mm)</td>
</tr>
<tr>
<td></td>
<td>DIF</td>
<td>DIF</td>
<td>DIF</td>
<td>DIF</td>
</tr>
<tr>
<td>Static</td>
<td>13.31</td>
<td>2.32</td>
<td>6.95</td>
<td>1.56</td>
</tr>
<tr>
<td>Targets</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>14.45</td>
<td>2.69</td>
<td>7.27</td>
<td>2.98</td>
</tr>
<tr>
<td>6</td>
<td>19.85</td>
<td>3.23</td>
<td>8.39</td>
<td>4.48</td>
</tr>
<tr>
<td>10</td>
<td>21.28</td>
<td>3.83</td>
<td>7.76</td>
<td>4.96</td>
</tr>
<tr>
<td>21</td>
<td>21.92</td>
<td>4.86</td>
<td>5.55</td>
<td>3.70</td>
</tr>
<tr>
<td>23</td>
<td>22.02</td>
<td>3.22</td>
<td>8.85</td>
<td>5.57</td>
</tr>
<tr>
<td>29</td>
<td>22.80</td>
<td>3.82</td>
<td>7.76</td>
<td>4.72</td>
</tr>
<tr>
<td>33</td>
<td>26.07</td>
<td>6.27</td>
<td>4.29</td>
<td>2.89</td>
</tr>
<tr>
<td>34</td>
<td>27.13</td>
<td>6.75</td>
<td>8.78</td>
<td>6.65</td>
</tr>
<tr>
<td>40</td>
<td>27.62</td>
<td>6.22</td>
<td>5.83</td>
<td>4.86</td>
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<tr>
<td>46</td>
<td>27.81</td>
<td>6.30</td>
<td>5.85</td>
<td>5.00</td>
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<tr>
<td>73</td>
<td>28.86</td>
<td>5.87</td>
<td>7.25</td>
<td>6.76</td>
</tr>
<tr>
<td>176</td>
<td>30.59</td>
<td>5.04</td>
<td>8.34</td>
<td>9.10</td>
</tr>
<tr>
<td>Strain Gauges</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>15.60</td>
<td>2.85</td>
<td>6.45</td>
<td>2.54</td>
</tr>
<tr>
<td>10</td>
<td>21.70</td>
<td>3.47</td>
<td>7.30</td>
<td>5.18</td>
</tr>
<tr>
<td>11</td>
<td>18.71</td>
<td>3.06</td>
<td>8.25</td>
<td>3.92</td>
</tr>
<tr>
<td>20</td>
<td>24.08</td>
<td>3.28</td>
<td>9.46</td>
<td>5.73</td>
</tr>
<tr>
<td>29</td>
<td>25.51</td>
<td>4.66</td>
<td>7.46</td>
<td>4.90</td>
</tr>
</tbody>
</table>

Figure 4.24 shows the relation between the dynamic increase factor and the strain rate for the compressive strength of clay brick. As expected the strain rate influences the compressive strength of this material and the dynamic increase factor for a strain rate of...
200 s\(^{-1}\) is 2.5, meaning that the compressive strength is two and a half times the static value at that strain rate.

![Diagram](image)

Figure 4.24 – DIF (compressive strength) for clay brick.

The equations that describe these relations for the compressive strength as function of the strain rate can be written as:

\[
DIF(\sigma_u) = \begin{cases} 
1 & \text{if } 1E^{-5} s^{-1} < \dot{\varepsilon} < 2 s^{-1} \\
0.3344 \ln(\dot{\varepsilon}) + 0.7682 & \text{if } 2 s^{-1} < \dot{\varepsilon} < 200 s^{-1} 
\end{cases}
\] (4.6)

This log-linear trend-line has a R\(^2\) higher than 70%, which for this material was considered good. The results obtained with the strain gauges also showed a good agreement with the results from the targets. Comparing the results with Hao and Tarasov (2008), who tested clay brick with different equipment, there is also a good agreement. Those authors obtained a compressive strength DIF of 2.31 at a strain rate of 150 s\(^{-1}\), and with the proposed relations a DIF of 2.44 is obtained for the same strain rate.

Figure 4.25 shows the relation between the dynamic increase factor and the strain rate for the Young’s modulus of clay brick. Again, the strain rate influences the Young’s modulus of this material and the Young’s modulus for a strain rate of 200 s\(^{-1}\) is 2.4 times greater than the static reference.
The equations that describe these relations for the Young’s modulus as function of the strain rate can be written as:

\[
\text{DIF}(E) = \begin{cases} 
1 & \text{if } 1E - 5 \text{ s}^{-1} < \dot{\varepsilon} < 2 \text{ s}^{-1} \\
0.3105 \ln(\dot{\varepsilon}) + 0.7848 & \text{if } 2 \text{ s}^{-1} < \dot{\varepsilon} < 200 \text{ s}^{-1}
\end{cases}
\] (4.7)

For this mechanical property the results are more scattered, with a very low determination coefficient, but the results obtained with the strain gauges seems to be in agreement with the obtained trend-line. The Young’s modulus seems to be less affected by the strain rate when compared with the compressive strength. The relation between these two properties was also reported by Hao and Tarasov (2008), although slightly more pronounced. These authors reported a DIF for the Young’s modulus of 1.95 for a strain rate of 150 s\(^{-1}\), and with the proposed relations a DIF of 2.21 is obtained for the same strain rate.

Figure 4.26 shows the relation between the dynamic increase factor and the strain rate for the strain at peak strength of clay brick. Here, the results show that this property is less dependent on the strain rate. At a strain rate of 200 s\(^{-1}\) the strain at peak strength is only 1.31 times greater than its static reference.
Figure 4.26 – DIF (strain at peak strength) for clay brick.

The equations that describe these relations for the strain at peak strength as function of the strain rate can be written as:

\[
DIF(\varepsilon_u) = \begin{cases} 
1 & \text{if } 1 \text{ s}^{-1} < \dot{\varepsilon} < 2 \text{ s}^{-1} \\
0.0673 \ln(\dot{\varepsilon}) + 0.9533 & \text{if } 2 \text{ s}^{-1} < \dot{\varepsilon} < 200 \text{ s}^{-1}
\end{cases}
\]  (4.8)

Again, these results are more scattered when compared with the results obtained for the compressive strength, and the results obtained with the strain gauges seems to be in agreement with the obtained trend-line. The strain at peak strength seems to be least affected by the strain rate of the studied properties. This was also reported by Hao and Tarasov (2008). These authors reported a DIF for the strain at peak strength of 1.12 for a strain rate of 150 s\(^{-1}\), and with the proposed relations a DIF of 1.29 is obtained for the same strain rate.

Figure 4.27 shows the relation between the dynamic increase factor and the strain rate for the compressive fracture energy of clay brick. As the results show, the fracture energy is greatly influenced by the strain rate, and again a large scatter was found. For strain rates of 200 s\(^{-1}\) the fracture energy is 5.95 times greater than the static reference.
The equations that describe these relations for the compressive fracture energy as function of the strain rate can be written as:

\[
DIF(G_c) = \begin{cases} 
1 & \text{if } 1E - 5 \text{ s}^{-1} < \dot{\varepsilon} < 5 \text{ s}^{-1} \\
1.3419 \ln(\dot{\varepsilon}) - 1.1597 & \text{if } 5 \text{ s}^{-1} < \dot{\varepsilon} < 200 \text{ s}^{-1}
\end{cases}
\] (4.9)

Although it was not possible to find similar results in the literature for comparison, the results obtained with strain gauges are close to those obtained with the targets and the video equipment.

Figure 4.28 shows a summary of the results obtained for the handmade clay brick specimens. As it can be seen, the strain at peak strength appears to be the least affected by the strain rates. The compressive strength and the Young’s modulus have similar behaviour, which is consistent with the fact that the strain at peak strength is lightly affected. The fracture energy have a high increase with the strain rate, more than twice the increase of the compressive strength, meaning that the post-peak behaviour of this material is greatly influenced by the increase of strain rate.
4.4.2 Mortar dynamic properties

A total of 63 specimens of mortar were tested under uniaxial compression, with strain rate ranging from 2 $s^{-1}$ to 224 $s^{-1}$. Again, due to errors in the acquisition system only 54 tests gave good results and were used to characterize the behaviour of mortar under increasing strain rates. Figure 4.29 shows some typical failure modes of the specimens under impact testing.

For these tests the acquisition frequency for the strain-time curve was set at 15 kHz and the acquisition frequency for the stress-time curve was set at 30 kHz. The reason for different acquisition frequencies was already explained previously. At the time of
testing the lighting conditions were not the same as the ones available at the time of the
tests on clay brick. Figure 4.30 shows a typical sequence of the impact test recorded
with the fast video equipment.

![Typical test sequence on mortar.](image)

Table 4.6 shows the results obtained on the impact tests for mortar specimens. Only a
few selected results are shown here, however, as stated previously, to establish the DIF
relations all 54 tests were used and can be seen in Annex A.2. Six additional tests using
strain gauges were performed in order to cross-check the results obtained with the
targets and video equipment. These six tests actually represent 24 tests, meaning that
strain gauges were placed in all four faces of the test specimen and the average was
taken for each test. In the video equipment, only one face of the test specimen was
measured, meaning that rotation of the specimen is not captured.

Figure 4.31 shows the relation between the dynamic increase factor and the strain rate
for the compressive strength of mortar. As expected, the strain rate influences the
compressive strength of this material and the dynamic increase factor for a strain rate of
200 s\(^{-1}\) is 4.13, which means that for this level of strain rate the compressive strength of
this kind of mortar is four times greater than its static reference.
Table 4.6 – Impact tests on mortar.

<table>
<thead>
<tr>
<th>Strain rate (s⁻¹)</th>
<th>Compressive Strength</th>
<th>Young's Modulus</th>
<th>Strain at Peak Strength</th>
<th>Fracture Energy</th>
<th>Specimen (♯)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>σᵤ (MPa)</td>
<td>DIF</td>
<td>E (GPa)</td>
<td>DIF</td>
<td>εᵤ (mm/m)</td>
</tr>
<tr>
<td>Static</td>
<td>4.46</td>
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<td>1.10</td>
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</tr>
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<td>--</td>
<td>1.00</td>
<td>0.91</td>
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</tr>
<tr>
<td>17</td>
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<td>1.72</td>
<td>1.32</td>
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<td>9.34</td>
<td>2.09</td>
<td>1.63</td>
<td>1.48</td>
<td>7.20</td>
</tr>
<tr>
<td>27</td>
<td>11.61</td>
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<td>2.24</td>
<td>2.04</td>
<td>5.17</td>
</tr>
<tr>
<td>30</td>
<td>12.33</td>
<td>2.76</td>
<td>2.81</td>
<td>2.55</td>
<td>7.10</td>
</tr>
<tr>
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<td>2.85</td>
<td>2.49</td>
<td>2.26</td>
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<td>2.69</td>
<td>2.45</td>
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</tr>
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<td>3.57</td>
<td>3.25</td>
<td>7.50</td>
</tr>
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<td>177</td>
<td>16.99</td>
<td>3.81</td>
<td>3.21</td>
<td>2.92</td>
<td>7.80</td>
</tr>
<tr>
<td>193</td>
<td>19.53</td>
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<td>3.25</td>
<td>2.95</td>
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<td>1.39</td>
<td>1.31</td>
<td>1.19</td>
<td>6.72</td>
</tr>
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</tr>
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</tr>
<tr>
<td>74</td>
<td>14.01</td>
<td>3.14</td>
<td>2.88</td>
<td>2.62</td>
<td>5.52</td>
</tr>
</tbody>
</table>

Figure 4.31 – DIF (compressive strength) for mortar.
The equations that describe the relations for the compressive strength as function of the strain rate can be written as:

\[
DIF(\sigma_u) = \begin{cases} 
1 & \text{if } 1E - 5 \text{ s}^{-1} < \dot{\varepsilon} < 4.35 \text{ s}^{-1} \\
0.8166 \ln(\dot{\varepsilon}) - 2.005 & \text{if } 4.35 \text{ s}^{-1} < \dot{\varepsilon} < 200 \text{ s}^{-1}
\end{cases} 
\] (4.10)

This log-linear trend-line has a \( R^2 \) of 85 %, which for this material was considered good. In addition, the results obtained with strain gauges have a good agreement with this log-linear relation.

Figure 4.32 shows the relation between the dynamic increase factor and the strain rate for the Young’s modulus of mortar. Just as for the brick specimens, the Young’s modulus is also influenced by the increase in strain rate. For a strain rate of 200 s\(^{-1}\) the Young’s modulus should be 3 times greater than its static reference.

![Figure 4.32 – DIF (Young’s modulus) for mortar.](image)

The equations that describe the relations for the Young’s modulus as function of the strain rate can be written as:

\[
DIF(E) = \begin{cases} 
1 & \text{if } 1E - 5 \text{ s}^{-1} < \dot{\varepsilon} < 4.35 \text{ s}^{-1} \\
0.5275 \ln(\dot{\varepsilon}) + 0.2245 & \text{if } 4.35 \text{ s}^{-1} < \dot{\varepsilon} < 200 \text{ s}^{-1}
\end{cases} 
\] (4.11)
The Young’s modulus appears to be less sensitive to the strain rate when compared with the compressive strength. The results from the strain gauges have a good agreement with this function. At this time it is possible to notice that the results for this kind of mortar have a smaller dispersion when compared with the results obtained for the handmade clay brick.

Figure 4.33 shows the relation between the dynamic increase factor and the strain rate for the strain at peak strength of mortar. Like previously, for the brick specimens, the strain at peak strength is the least affected property by the strain rate. For a strain rate of 200 s\(^{-1}\) the strain at peak strength has a dynamic increase factor of 1.11.

![Figure 4.33 – DIF (strain at peak strength) for mortar.](image)

The equations that describe the relations for the strain at peak strength as function of the strain rate can be written as:

\[
\text{DIF}(\varepsilon_u) = \begin{cases} 
1 & \text{if } 1 \text{E} - 5 \text{ s}^{-1} < \dot{\varepsilon} < 4.35 \text{ s}^{-1} \\
0.0286 \ln(\dot{\varepsilon}) + 0.9579 & \text{if } 4.35 \text{ s}^{-1} < \dot{\varepsilon} < 200 \text{ s}^{-1}
\end{cases}
\]  

(4.12)

Similar to the results obtained for the clay brick specimens, the strain rate has a little influence on this property and the results are more scattered when compared with the compressive strength. However, the results obtained with the strain gauges were able to confirm this log-linear relation.
Figure 4.33 shows the relation between the dynamic increase factor and the strain rate for the compressive fracture energy of mortar. Unlike the results obtained for clay brick, the strain rate has a smaller influence in the fracture energy of this kind of mortar. The results show that for strain rates of 200 s\(^{-1}\) the dynamic increase factor, for the fracture energy, is 2.73.

\[
DIF(G_c) = \begin{cases} 
1 & \text{if } 1 E - 5 \text{ s}^{-1} < \dot{\varepsilon} < 9 \text{ s}^{-1} \\
0.5582 \ln(\dot{\varepsilon}) - 0.2269 & \text{if } 9 \text{ s}^{-1} < \dot{\varepsilon} < 200 \text{ s}^{-1}
\end{cases}
\] (4.13)

Although it was not possible to find similar results in the open literature for comparison the relation here presented shows similar results for the strain gauges and the video equipment.

Figure 4.35 shows a summary of the results obtained for the mortar specimens. As it can be seen, the strain rate seems to have little influence on the strain at peak strength. There is a more pronounced difference between the dynamic increase factor of the compressive strength and the Young’s modulus. The results obtained for the fracture
energy show that the influence of the strain rate in the post-peak behaviour is smaller when compared with the handmade clay brick.

![Image](image.png)

Figure 4.35 – DIFs for mortar mechanical properties.

### 4.4.3 Masonry dynamic properties

A total of 12 specimens of masonry were tested under uniaxial compression, with strain rate ranging from $2 \text{ s}^{-1}$ to $54 \text{ s}^{-1}$. Figure 4.36 shows some typical failure modes of the specimens under impact testing with increasing strain rates.

For these tests the acquisition frequency for the strain-time curve was set at 10 kHz and the acquisition frequency for the stress-time curve was set at 40 kHz. The reason for the lower acquisition frequency for the masonry specimens was the size of the specimens. There is a relation between the resolution of the video and the maximum frame rate available, due to the memory buffer of the equipment. Figure 4.37 shows a typical sequence of the impact test recorded with the fast video equipment.

Table 4.7 shows the results obtained for all impact tests on masonry specimens. Although four targets can be seen in the masonry specimens, the uniaxial strain was calculated between the two centered targets. For these tests only video tracking was used to calculate the strain during each test.
Figure 4.36 – Typical failure modes for masonry specimens under impact testing.

Figure 4.37 – Typical test sequence on masonry.

Figure 4.38 shows the relation between the dynamic increase factor and the strain rate for the compressive strength of masonry. As expected, the compressive strength of masonry increases with the increase of strain rate. According to these results and for a strain rate of 200 s$^{-1}$, the compressive strength of these masonry specimens is more than two times greater than its static reference.
Table 4.7 – Impact tests on masonry.

<table>
<thead>
<tr>
<th>Strain rate (s⁻¹)</th>
<th>Compressive Strength</th>
<th>Young's Modulus</th>
<th>Strain at Peak Strength</th>
<th>Fracture Energy</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>σ_u (MPa)</td>
<td>E (GPa)</td>
<td>ε_u (mm/m)</td>
<td>G_c (N/mm)</td>
</tr>
<tr>
<td>Static</td>
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<td>--</td>
<td>10.93</td>
<td>7.64</td>
</tr>
<tr>
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<td>10.73</td>
<td>6.29</td>
</tr>
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<td>5.0</td>
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</tr>
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</tr>
<tr>
<td>10.6</td>
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<td>13.95</td>
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</tr>
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<td>12.96</td>
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</tr>
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<td>9.71</td>
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</tr>
<tr>
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<td>1.57</td>
<td>10.66</td>
<td>18.77</td>
</tr>
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<td>1.48</td>
<td>12.05</td>
<td>13.89</td>
</tr>
<tr>
<td>54.0</td>
<td>16.61</td>
<td>2.09</td>
<td>14.80</td>
<td>18.75</td>
</tr>
</tbody>
</table>

Figure 4.38 – DIF (compressive strength) for masonry.

The equations that describe the relations for the compressive strength as function of the strain rate can be written as:

\[
DIF(\sigma_u) = \begin{cases} 
\frac{1}{0.2798 \ln(\dot{\varepsilon}) + 0.6863} & \text{if } 1E - 5 \text{ s}^{-1} < \dot{\varepsilon} < 3 \text{ s}^{-1} \\
10^{-5} & \text{if } 3 \text{ s}^{-1} < \dot{\varepsilon} < 200 \text{ s}^{-1} 
\end{cases} \tag{4.14}
\]
These log-linear relations have a $R^2$ of 80%. Which is good considering the type of materials involved. However these results are limited to the amount of specimens tested. Comparing this results with the results obtained for its components, it can be seen that these are closer to the results presented for the handmade clay brick.

Figure 4.39 shows the relation between the dynamic increase factor and the strain rate for the Young’s modulus of masonry. The Young’s modulus is also influenced by the increase in strain rate. For a strain rate of 200 $s^{-1}$ the Young’s modulus should be two times greater than its static reference.

The equations that describe the relations for the Young’s modulus as function of the strain rate can be written as:

$$DIF(E) = \begin{cases} 
\frac{1}{0.2409 \ln(\dot{\varepsilon}) + 0.8701} & \text{if} \quad 1E - 5 s^{-1} < \dot{\varepsilon} < 1.7 s^{-1} \\
1.7 s^{-1} < \dot{\varepsilon} < 200 s^{-1} 
\end{cases}$$

(4.15)

Comparing this results with the results obtained for its components, it can be seen that these are closer to the results presented for the handmade clay brick, similar to the observation made for the compressive strength.
Figure 4.40 shows the relation between the dynamic increase factor and the strain rate for the strain at peak strength of masonry. Similar to what was observed for its components, the strain at peak strength is the least influenced by the strain rate. For a strain rate of 200 s\(^{-1}\) the strain at peak strength is 1.26 times its static reference.

![Figure 4.40 – DIF (strain at peak strength) for masonry.](image)

The equations that describe the relations for the strain at peak strength as function of the strain rate can be written as:

\[
DIF(\varepsilon_{pu}) = \begin{cases} 
1 & \text{if } 1 \times 10^{-5} \text{ s}^{-1} < \dot{\varepsilon} < 4 \text{ s}^{-1} \\
0.0678 \ln(\dot{\varepsilon}) + 0.9036 & \text{if } 4 \text{ s}^{-1} < \dot{\varepsilon} < 200 \text{ s}^{-1}
\end{cases}
\]  

(4.16)

The observations regarding this properties are similar to the ones already presented for the other materials. The strain rate has little influence in the strain at peak strength for masonry, as well as for its components.

Figure 4.41 shows the relation between the dynamic increase factor and the strain rate for the fracture energy of masonry. Again, similar to the components of these masonry specimens, the compressive fracture energy has a considerable increase with the increase of strain rates. For a strain rate of 200 s\(^{-1}\) the fracture energy should be three times greater than its static reference.
The equations that describe the relations for the compressive fracture energy as function of the strain rate can be written as:

\[
DIF(G_c) = \begin{cases} 
  1 & \text{if } 1 \times 10^{-5} \text{ s}^{-1} < \dot{\varepsilon} < 2 \text{ s}^{-1} \\
  0.4716 \ln(\dot{\varepsilon}) + 0.5968 & \text{if } 2 \text{ s}^{-1} < \dot{\varepsilon} < 200 \text{ s}^{-1}
\end{cases}
\]  (4.17)

Comparing these results with the results obtained for its components, it can be seen that these are closer to the results presented for the mortar specimens.

Figure 4.42 shows a summary of the results obtained for the masonry specimens. As it can be seen, the strain rate seems to have little influence on the strain at peak strength. The reported behaviour for the compressive strength and the Young’s modulus is similar. The fracture energy have a high increase with the strain rate, greater than the compressive strength, meaning that, similar to the brick observations, the post-peak behaviour of this material is greatly influenced by the increase of strain rates.
4.5 COMPARISON AND DISCUSSION

Figure 4.43 shows the influence of the strain rate on the three different materials tested. At a strain rate of around 200 $s^{-1}$ the DIF regarding the compressive strength for brick, mortar and masonry are 2.54, 4.13 and 2.17 respectively. The results show that the behaviour of the masonry specimens is similar to the behaviour of the clay brick. For the mortar specimens this influence is more pronounced with a DIF for mortar of around double of the brick. This difference could be explained with the existence of synthetic fibres in the mortar composition. Authors such as Zhou et al (2013) and Tran et al (2014) suggested that the presence of fibres in the composition of cementitious composites have a direct influence in the strain rate effect of these materials.

Figure 4.44 summarizes the results obtained for the Young’s modulus for all three materials. For a strain rate of around 200 $s^{-1}$ the DIF regarding the Young’s modulus for brick, mortar and masonry are 2.43, 3.02 and 2.15 respectively. Although less pronounced than the compressive strength, the comparison between all three materials is similar.
Figure 4.43 – DIF for compressive strength of brick, mortar and masonry.

Figure 4.44 – DIF for Young’s modulus of brick, mortar and masonry.

Figure 4.45 shows the results obtained for the strain at peak strength for all three materials. At a strain rate of around 200 s$^{-1}$ the DIF regarding the strain at peak strength for brick, mortar and masonry are 1.31, 1.11 and 1.26 respectively. As can be seen, the influence of the strain rate in this property is almost negligible, which is consistent with the behaviour observed for other geo-materials, such as rock and concrete (Zhao et al, 1999 and Hao & Tarasov, 2008).
Figure 4.45 – DIF for strain at peak strength of brick, mortar and masonry.

Regarding the compressive fracture energy, the results are summarized in Figure 4.46. Here the results obtained for mortar and masonry are closer with DIF of 2.73 and 3.10 respectively, for a strain rate of 200 s\(^{-1}\). The DIF for brick at the same strain rate is 5.95. These results suggest that the post-peak behaviour of the handmade clay brick is more influenced by the strain rate. One possible explanation is related to the porosity of both materials. The porosity of these clay bricks (23 %) is higher than the porosity of this kind of mortar (9 %). In fact, Vasconcelos (2005) suggested that the porosity of materials could be one of the physical properties with direct influence in its post-peak behaviour.

A large experimental campaign was performed on different loading regimes and different materials. Almost 250 impact tests were performed during this research and more than 60 handmade clay brick were cut in order to prepare the specimens. Masonry specimens and its components, clay brick and mortar, were tested under quasi-static regime – strain rate of \(10^{-5}\) s\(^{-1}\) – and dynamic regime with strain rates ranging from 2 s\(^{-1}\) up to 200 s\(^{-1}\). It was found that the mechanical properties of these materials increase with the increase in strain rate, having DIFs ranging from 2 to 6 for a strain rate of 200 s\(^{-1}\). Equations 4.6 to 4.17 represent the empirical relations obtained and they can be used to estimate the response of this kind of materials under different strain rates.
Figure 4.46 – DIF for compressive fracture energy of brick, mortar and masonry.

4.6 REFERENCES


Chapter 4 – Mechanical Characterization of Traditional Materials


The vulnerability of the masonry envelop under blast loading is considered critical due to the risk of loss of lives. Very few numerical or experimental studies have been conducted on impact and blast on structural components of building structures, characterized by strain rates well over $1 \text{s}^{-1}$, with quasi-static tests characterized by strain rates in the range $10^{-5}$ to $10^{-7} \text{s}^{-1}$. There is a need for more research to obtain an accurate representation of the effect of blasts, as high nonlinear behaviour and possible brittle failure has been observed. The out of plane vulnerability of the masonry envelop under dynamic loading is considered critical due to the risk of loss of lives, emphasized by many studies, particularly in the case of earthquakes (Calvi and Bolognini, 2001), and explosion debris (Wu and Hao, 2007). Still, only a few laboratory experimental investigations are available, simulating vehicles impacts on parapets (Gilbert et al, 2002) and air-blasting (Mayrhofer, 2002).

A key issue on the mechanical behaviour under blast is the strength increase due to high-strain rate. Explosions produce very high strain rates, usually around $10^2 – 10^4 \text{s}^{-1}$. 
Reinforced concrete structures, for example, are highly affected by this phenomenon; its resistance can increase greatly due to the high strain rate effect, dynamic increase factors as high as 4 in compression and 6 in tension have been reported (Grote et al, 2001; Ngo et al, 2004). In the case of masonry and its components the available studies are very limited in number. Recently, dynamic increase factors higher than 2 in compression for clay brick were reported (Hao and Tarasov, 2008) and Chapter 4 from the present document specifically addresses this issue.

This work intends to present a newly developed test setup for dynamic out-of-plane loading using underWater Blast Wave Generators (WBWG) as loading source. Underwater blasting operations have been, during the last decades, subject of research and development of maritime blasting operations (including torpedo studies), aquarium tests for the measurement of blasting energy of industrial explosives and confined underwater blast wave generators. WBWG allow a wide range for the produced blast impulse and surface area distribution. It also avoids the generation of high velocity fragments and reduces atmospheric sound wave (Tavares et al, 2012; Ambrósio et al, 2013).

One objective of this work is to study the behaviour of masonry infill walls subjected to blast loading. Three different masonry walls are to be studied, namely unreinforced masonry infill walls and two different reinforcement solutions. These solutions have been studied previously for seismic action mitigation (Pereira, 2013). Finally, there is an intention to create tools to help designers to make informed decisions on the use of infills under blast loading.

### 5.1 TEST SETUP FOR DYNAMIC OUT-OF-PLANE LOADING

This work was performed in collaboration with LEDap (Laboratory of Energetics and Detonics) in Condeixa-a-Nova, Figure 5.1a. The developed test setup was constructed at LEDap facilities and comprises several elements. A support steel structure holds the specimen in place and provides sufficient reaction to the wall’s reinforced concrete frame, Figure 5.1b. Additional details on the design of the support structure can be found in Annex B.1.
On one side of the wall a number of large (one cubic metre) water containers are placed to act as WBWG and apply the desired load. On the other side of the wall, measuring equipment is placed in order to characterize the behaviour of the wall. The maximum deflection is measured using laser equipment and high speed video cameras are used to record the wall during the test. All the area is surrounded by protection walls and a safe area was defined to provide safe hosting for the acquisition equipment and personal during the tests, Figure 5.2.

![Figure 5.1 – Test setup: a) LEDap facilities; b) support steel structure.](image1)

![Figure 5.2 – Final configuration of the test setup.](image2)

### 5.1.1 Blast wave generator

The original blast wave generators (BWG), from the direct application of an explosion in air with high explosives, have the inconvenient of producing hot polluted gases, providing a reduced area of induced pressure, allowing the possibility of generation of
high velocity fragments and producing a very intense sound wave. Using confined underwater blast wave generators (WBWG), applying the extremely high rate conversion of the explosive detonation energy into the kinetic energy of a thick water confinement, allows a surface area distribution. This also avoids the generation of high velocity fragments and reduced the atmospheric sound wave (Plaksin and Campos, 2007).

Since the physical properties of water and air are different, the characteristics of the shock waves (in air and water) are also different, mainly due to density and shock wave velocity (shock impedance) of the materials. The density of water is about 800 times greater than the density of air, while the sound velocity in water is 1500 m/s and the sound velocity in air is about 330 m/s (4.5 times faster). Therefore, the shock wave in water is 4.5 times faster than in air, and the pressure impulse for the shock wave in water is 15-20 times higher than in air (Tavares et al, 2012). After the detonation of an explosive charge under water, the detonation products expand generating shock wave in water and forming a gas bubble. This gas bubble expands and the pressure inside the bubble decreases. Because of inertia of water flow in front of the bubble, the expansion of the gas bubble continues even after the pressure inside the bubble decreases slightly below the pressure of the surrounding water. Afterwards, the pressure inside the gas bubble drops below the pressure of the surrounding water and the gas bubble movement stops (Plaksin and Campos, 2007). However, the phenomenon does not fully stop as the gas bubble contracts under the action of surrounding pressure. The contractions and expansions continue for several cycles, which generates pulsating movement in the gas bubble and additional compression waves in the water (Tavares et al, 2012).

In the tests carried out here, the explosive charge was place in the centre of the water container (Figure 5.3). The explosive used in the tests was PETN (pentaerythritol tetranitrate), a highly explosive organic compound. For the first level of loading, 7.2 g of PETN were used in each water container.

The generic one cubic metre containers have a metallic protection mesh surrounding them, which needed to be cut on the side facing the wall, in order to have full contact. The metallic mesh on the remaining sides was left in place to help keeping the water volume (Figure 5.4a). Due to the size of the wall under study (3.5×1.7 m) six water
containers were used, being two rows of 3 water containers on top of each other. The procedure for placing the water containers was the following: a) prepare all metallic meshes as indicated; b) place the first row of containers in their final position; c) place the explosive charge in the centre of the container (using a thin tube to guide and to keep the charge in place); d) fill the first row of containers with 3000 litres of water (Figure 5.4b); e) place a wooden board on top of the first row to help distributing the weight of the second row; f) place the second row of containers (Figure 5.4c); repeat steps c) and d).

Figure 5.3 – Water container: a) schematic view; b) photo.

Figure 5.4 – Preparing the WBWG: a) cutting the metallic mesh; b) filling the first containers; c) both levels in place.

5.1.2 Pressure/deflection acquisition

One of the main issues regarding dynamic testing, in this case blast loading, is the proper acquisition of signals. The measuring equipment needs to have capacity for high acquisition frequencies. In this work there were two signals that needed to be recorded: a) the pressure profile acting on the wall; and b) the deflection profile of the wall.
For the pressure acquisition, a new sensor was developed. The mechanism used to measure the pressure consists of an assembled instrumental stainless steel plate between the wall and the water container. The pressure is measured using a tube connected to a sensor. This tube contains thin oil and is connected in a closed loop, Figure 5.5. The pressure device works like a force multiplier that provides hydrostatic pressure transmission. The pressure sensors used were 4-20 mA Gems™ Sensors and Controls 3100B0016G01B and 3100B0010B01B. The connection of the sensor to the peripheral equipment can be seen in Figure 5.6. In order to plot the acquired pressure signal, these sensors were connected to a Tektronix TDS 320 oscilloscope, Figure 5.7. This sensor was previously tested and calibrated (Tavares et al, 2012 and Ambrósio et al, 2013).

Figure 5.5 – Pressure sensor schematics (left) and construction (right).

Figure 5.6 – Pressure sensors and connections to the peripheral equipment.

Figure 5.7 – Tektronix TDS 320 oscilloscope and printer.
Chapter 5 – Masonry Infill Walls Under Blast Loading

For the deflection acquisition, a Keyence CMOS Multi-Function analogue laser sensor IL-2000 with a signal amplifier IL-1000 (Figure 5.8a) was used. This sensor was connected to a National Instruments acquisition system composed of a SCXI-1000DC chassis, a SCXI-1600 data acquisition and control card for PC connection and a generic input module SCXI-1520 with a SCXI-1314 mount, Figure 5.8b. In this case the sampling rate was limited by the laser sensor and was set at 3 kHz. With this system, it is possible to measure only the deflection of one point in the wall. The selected point was the centre point of the wall, Figure 5.9.

![Laser sensor and amplifier](image1)
![Acquisition system](image2)

Figure 5.8 – Deflection acquisition: a) laser sensor and amplifier; b) acquisition system.

Besides the usage of pressure and displacement transducers, high speed video equipment was used to study the behaviour of the wall during the test. Three different cameras were used, marked in Figure 5.1b as camera A, B and C. Camera A (Figure 5.10) is a PHOTRON APX-RS and was placed to have a full view of the wall. This camera was set with an acquisition frequency of 1 kHz. Cameras B and C (Figure 5.11) are Casio EX-FH25 and were placed with different angles. Camera B was placed on the side of the wall in order to capture the profile of the wall. Camera C was placed in order to capture the WBWG and their behaviour during the test. Both cameras were set with an acquisition frequency of 0.4 kHz. To help having a better view of the movement of the wall, a regular mesh was drawn in the wall using black tape, Figure 5.9.
Figure 5.9 – Laser sensor: a) final position; b) support system.

Figure 5.10 – Camera A: a) PHOTRON APX-RS; b) rear view of the protection case; c) front view of the protection case.

Figure 5.11 – Camera B and C: a) Casio EX-FH25; b) rear view of the protection case; c) front view of the protection case.
5.1.3 Test specimens

The test setup was developed for testing the dynamic out-of-plane behaviour of infill masonry walls. The adopted solutions for the infill masonry represent the single leaf infill walls in modern construction. The masonry infill is built inside a reinforced concrete (RC) frame that provides the boundary conditions. The final dimensions of the test specimens can be seen in Figure 5.12a, which are made in a 1:1.5 scale. Three different solutions (Figure 5.12b) were planned: a) unreinforced single leaf infill with 30×20×15 cm brick (label URM); b) single leaf infill with 30×20×15 cm brick with bed joint reinforcement (label JAR); c) single leaf infill with 30×20×15 cm brick with external reinforcement in the plaster (label RAR). All three solutions have 15 mm cover on each side. M5 plaster is used on one side and projected lime is used on the other side, with the exception of the solution with the external reinforcement which has plaster on both sides. Here, due to the difficulties in preparing this experimental set-up, only one test is presented. Additional details regarding the design of the masonry specimens can be found in Annex B.2.

The construction process of the walls consists of the following steps: 1st) construction of the RC frames; 2nd) construction of the masonry infill panel with or without reinforcement; 3rd) execution of plaster with or without reinforcement. The placement of the masonry is done by successive horizontal rows, always from the columns. For the first masonry unit, mortar is applied on the bed and head faces. The unit is then pressed against lower beam and column. The last unit in each horizontal row is usually cut in order to ensure dimensional compatibility. In situations where the panels' geometry makes the cut unreasonable, the spaces are filled with mortar (Pereira et al, 2011).
5.2 UNREINFORCED INFILL WALL UNDER BLAST LOADING

The traditional construction process for building these masonry walls was already described. In this specific work, additional steps had to be taken because the wall could not be built already in its final position. First, the reinforced concrete frame was built outside the testing site (Figure 5.13a). After curing, the frame was transported and placed in front of the support structure (Figure 5.13b). At that location, the masonry infill was built with the process described before (Figure 5.13c). After curing the wall, the plaster and the projected lime were applied to the wall (Figure 5.13d). With the masonry wall built, it had to be slightly moved towards the support structure using heavy machinery (Figure 5.13e). With the wall in its final position, the reinforced concrete frame was bolted to the steel support structure in 11 marked places along its perimeter (Figure 5.13f).

With the wall specimen ready to be tested, the loading containers and data acquisition equipment needed to be put in place. As shown before, the water container were placed and filled in two phases. Meanwhile all the sensors and acquisition systems are mounted and tested (Figure 5.14a). The final step before the test itself is connecting the detonators (Figure 5.14b). Due to the dangerous nature of these tests, all systems need to be triple checked before this final step. After placing the detonators, no one is allowed into the test site and every system is controlled from the designated safe area. The acquisition systems start and a countdown is set until detonation.

After the test, the acquired signals need to be processed. The oscilloscopes provided the applied pressure on the wall and the final pressure profile was plotted, Figure 5.15. The pressure arises to 149 kPa in the first 6 ms, then decays and reaches 119 kPa at 17.5 ms and stops acting after 29 ms. From the laser sensor, the deflection on the central point of the wall was obtained, Figure 5.16. The deflection on the wall has an expected profile, increasing until its maximum of 14.6 mm after 24 ms and has a residual value of around 11 mm.
Figure 5.13 – Building process and aspect of the specimen in the test setup.

Figure 5.14 – Unreinforced wall test: a) all systems mounted; b) connecting the detonators.
Besides these profiles, the video acquired with the high speed cameras was also analysed. Figure 5.17 and Figure 5.18 show some pictures taken from those videos at specific times. Due to the magnitude of the deformations (small when compared with the dimensions of the wall) it is difficult to perceive the evolution of deformations in just a few static pictures. However, analysing the slow motion video, it is possible to see the full behaviour of the wall. The maximum displacement is achieved at the centre point of the wall, which behaves as a plate supported in its four edges.

After the test, a visual inspection of the wall was also performed. The cracks were marked in order to have a view of the crack pattern (Figure 5.19a). There is a large concentration of large horizontal cracks at the centre of the wall (Figure 5.19b) and these spread to the corner, as they move away from the centre. There are also some large cracks at the top support edge.
Figure 5.17 – Pictures from video at Camera A at different instances.
Figure 5.18 – Pictures from video at Cameras B and C at different instances.
5.3 Numerical Modelling

The global field of structural analysis of masonry structures encompasses several different approaches and a review is given by Lourenço (2008). In general, the approach towards the numerical representation of masonry can address the micro modelling of the individual components: unit (brick, block, etc.) and mortar; or the macro modelling of masonry as a composite. Depending on the level of accuracy and the simplicity desired, it is possible to use the following modelling strategies: a) detailed micro-modelling, where units and mortar in the joints are represented by continuum elements whereas the unit-mortar interface is represented by discontinuum elements; b) simplified micro-modelling, where expanded units are represented by continuum elements whereas the behaviour of the mortar joints and unit-mortar interface is lumped in discontinuum elements; c) Macro-modelling, where units, mortar and unit-mortar interface are smeared out in a homogeneous continuum. Many approaches involving different approximations and ingenious assumptions have been sought, e.g. Gambarotta and Lagomarsino (1997), Massart et al (2004), Calderini and Lagomarsino (2006), where simplified non-linear homogenization techniques were used.

This numerical analysis was performed using a Finite Element Model (FEM). The geometry model was based on the description provided in section 5.1.3. Only the infill panel was modelled and a perfect connection was considered between the infill panel and the reinforced concrete frame (Figure 5.20a). A macro-modelling strategy was adopted, where the panel is considered a homogeneous continuum.
The FEM model was built in the ABAQUS software (ABAQUS User Manual, 2010), where the Explicit solver was used. This software has been used successfully in previous situation regarding similar loading conditions (Cabello, 2011; Jacinto et al, 2001; Heidarpour et al, 2012) and similar materials (Zheng et al, 2010; Al-Gohi et al, 2012). The wall was discretized with 8-node solid elements (C3D8R) with reduced integration and hourglass control (ABAQUS User Manual, 2010). The final mesh was automatically generated by ABAQUS, and is rather refined. The edges were considered constrained in all degrees of freedom. The thickness of the wall was set as 180 mm (brick plus plaster on both sides). The final mesh has 4872 elements and 6844 nodes (Figure 5.20b).

5.3.1 Material model

The CDP (Concrete Damaged Plasticity) model used in ABAQUS software is a modification of the Drucker-Prager model by Lubliner et al (1989), Lee and Fenves (1998). In particular, the shape of the failure surface in the deviatoric plane (Figure 6.12) needs not to be a circle and it is governed by parameter $K_c$. This parameter can be interpreted as a ratio of the distances between the hydrostatic axis and, respectively, the compression meridian and the tension meridian in the deviatoric plane. This ratio is always higher than 0.5 and when it assumes the value 1, the deviatoric cross section of the failure surface becomes a circle (Kmiecik and Kaminski, 2011). The CDP model requires four additional parameters to be defined: a) the dilatation angle, the flow potential eccentricity, the ratio of initial equibiaxial compressive yield stress to initial...
uniaxial compressive yield stress and the viscosity parameter. For all these five parameters the default values suggested in ABAQUS User’s Manual were used (Table 5.1). Additional information regarding this model can be found in ABAQUS User Manual (2010), Kmiecik and Kaminski (2011), and Jankowiak and Lodygowsky (2005).

![Figure 5.21 – Failure surface in CDP model, represented in the deviatoric plane S1, S2 and S3 (Kmiecik and Kaminski, 2011).](image)

<table>
<thead>
<tr>
<th>Table 5.1 – Default parameters of CDP model.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter</td>
</tr>
<tr>
<td>----------------------</td>
</tr>
<tr>
<td>Dilatation angle ($\Psi$)</td>
</tr>
<tr>
<td>Eccentricity ($\varepsilon$)</td>
</tr>
<tr>
<td>$f_{so}/f_{c0}$</td>
</tr>
<tr>
<td>$K_c$</td>
</tr>
<tr>
<td>Viscosity parameter ($\mu$)</td>
</tr>
</tbody>
</table>

The CDP model assumes that the failure for tensile cracking and compressive crushing of the material is characterized by damage plasticity. The model uses the concept of isotropic damage evolution in combination with isotropic tensile and compressive plasticity to represent the inelastic and fracture behaviour of the material. The model also allows the definition of strain hardening in compression and strain softening in tension. The adopted stress-strain curves in tension and compression can be seen in Figure 5.22.

As stated previously, these masonry walls were tested and mechanically characterized under quasi-static conditions. The data collected from Pereira (2013) regarding the
quasi-static mechanical properties for the single leaf infill wall with plaster on both sides can be seen in Table 5.2.

![Stress-strain relations](image)

(a) in tension; (b) in compression

(Lubliner et al, 1989).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Static</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_t [MPa]$</td>
<td>0.125</td>
</tr>
<tr>
<td>$f_c [MPa]$</td>
<td>1.24</td>
</tr>
<tr>
<td>$E_0 [GPa]$</td>
<td>3.6</td>
</tr>
<tr>
<td>$G_f [N/mm]$</td>
<td>0.012</td>
</tr>
</tbody>
</table>

The CDP model allows the definition of the mode-I fracture energy instead of the definition of the stress-strain relation in tension. The quasi-static properties obtained from the tests serve as a base for the calibration of the numerical model. The mechanical properties under dynamic conditions will be subsequently obtained by matching the deflection of the numerical model with the experimental data.

### 5.3.2 Explicit analysis

ABAQUS Explicit was used to solve the non-linear equations of this problem. In order to keep this problem within a pure Lagrangian formulation, the blast loading was defined as a pressure profile. The blast loading applied was derived from the obtained experimental data (Figure 5.15b). The calibration process started with the application of the static reference mechanical properties. After realizing that the displacement obtained was much higher than the obtained experimentally, the mechanical properties were
increased gradually until there was a good agreement between the numerical model and the experimental data. The final dynamic properties and the respective dynamic increase factor (DIF) can be seen in Table 5.3. This process is not fully objective but, although these materials are not the same as the materials studied in Section 4, the obtained results are in agreement with the conclusions obtained for the masonry specimens. Dynamic increase factors between 2 and 3 for the compressive strength and the Young’s modulus. The tensile behaviour was not studied in Section 4.

Table 5.3 – Dynamic mechanical properties.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Static</th>
<th>Dynamic</th>
<th>DIF</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_t$ [MPa]</td>
<td>0.125</td>
<td>0.375</td>
<td>3</td>
</tr>
<tr>
<td>$f_c$ [MPa]</td>
<td>1.24</td>
<td>3.78</td>
<td>3</td>
</tr>
<tr>
<td>$E_0$ [GPa]</td>
<td>3.6</td>
<td>7.2</td>
<td>2</td>
</tr>
<tr>
<td>$G_f$ [N/mm]</td>
<td>0.012</td>
<td>0.025</td>
<td>2</td>
</tr>
</tbody>
</table>

Figure 5.23a shows the result from the numerical model in terms of displacement vs. time and compares it with the experimental data. There is a good initial agreement up to 12 mm in deformation. At this instant the experimental curve changes its slope, probably due to appearing cracks. The maximum displacement has a difference of 3%. In the post-peak behaviour there is some difference between the experimental and the numerical model. In the experimental test the wall was able to set its residual deformation at 76% of the maximum deformation. In the numerical model the residual deformation was 91%. In the experimental test, when the blast wave from the WBWG reaches the wall it generates an expansion wave that travels through the thickness of the wall. When this expansion wave reaches the opposite edge of the wall it will start moving in the opposite direction creating a “negative” wave profile, which was not considered in the numerical model. This can justify part of the difference. It is possible that there is some sliding in the test, particularly, in the top crack, which is not considered in the model.

A proper definition of the mechanical properties through adoption of dynamic increase factors is crucial in this type of analysis. As can be seen in Figure 5.23b, where a comparison of the dynamic increase factor is performed, using the static properties (DIF=1) in this model results in excessive deformation, which would lead to the
collapse of the wall. On the other hand, using a dynamic increase factor of 5 in all mechanical properties means that the wall behaves mostly in its elastic regime, having a neglectable residual deformation.

![Figure 5.23](image1)

Figure 5.23 – Numerical results: a) comparison between the numerical model and the experimental data; b) DIF’s influence in the response.

Besides the comparison of the deflection profile, the damage on the wall was also compared. The maximum principal plastic strains are a reasonable indicator of cracking and were plotted in Figure 5.24 for the face on the back of the explosion. As expected, according to this model, there is a concentration of cracks at mid height of the wall that will start to spread to the corners as we move further from the centre point. There is also some damage at the bottom and top edge of the wall. These results are in agreement with the observed damage in the experimental test (Figure 5.19).

![Figure 5.24](image2)

Figure 5.24 – Maximum principal plastic strain in the external face of the wall
5.3.3 Reinforcement

Although it was not possible to test experimentally reinforced solutions for the infill masonry walls, as done by Pereira (2013), numerical models for the planned reinforced solutions were prepared. The first reinforcement solution under study – JAR: bed joint reinforcement – has BEKAERT MURFOR RND .4/100 every two horizontal joints. The adopted geometry for the numerical model can be seen in Figure 5.25 and comprises the previous model with the addition of 8 reinforcement bars embedded in the masonry region. Each reinforcement bar has a cross section area of 12.57 mm$^2$.

![Figure 5.25 – JAR solution: a) geometry; b) assembly view from the numerical model.](image)

The second solution for reinforcement under study – RAR: external mesh reinforcement – has BEKAERT – ARMANET Ø1.05 mm 12.7×12.7 mm in both sides of the wall, embedded in the plaster. An equivalent reinforcement grid with 87.55×87.55 mm openings was added to the unreinforced model (Figure 5.26). Each reinforcement bar has a cross section area of 5.97 mm$^2$. The reason for changing the grid size and not using the original one is the computational time to run the analysis. With the original grid, the final FE mesh was composed of more than 150 000 elements and with this equivalent grid have only 21472 elements, from the masonry region and the truss elements from the reinforcement.

The reinforcement elements, for both models, were truss elements T3D2 which is a 2 node linear 3D truss for explicit analysis (ABAQUS User Manual, 2010). For the
material properties an elastic – ideal plastic model was adopted. The static mechanical properties for the reinforcement were collected from the product datasheets, available in Annex B.3. Because there was no experimental work performed on the material properties at high strain rates for these reinforcement bars, the recommended DIF in UFC 3-340-02 (2008) was used. This standard indicates a DIF of 1.23 for the tensile strength of reinforcement steel in bending. The dynamic properties for this material can be seen in Table 5.4.

Figure 5.26 – RAR solution: a) geometry model; b) reinforcement grid.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Static</th>
<th>Dynamic</th>
<th>DIF</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_y ,[MPa]$</td>
<td>320</td>
<td>394</td>
<td>1.23</td>
</tr>
<tr>
<td>$E_0 ,[GPa]$</td>
<td>210</td>
<td>210</td>
<td>1</td>
</tr>
</tbody>
</table>

Figure 5.27 shows the result obtained for the displacement at the centre point of the wall considering both reinforcement solutions and compares it with the unreinforced model. As can be seen, both solutions show an improvement in the response of the structure. The solution considering bed joint reinforcement decreases the maximum displacement of the wall in 25% and the solution with the grid reinforcement decreases the maximum displacement in 50%. These solutions have been studied for seismic action mitigation and the results reported for that specific application suggest that both reinforcement solutions improve the response of the out-of-plane behaviour of masonry walls (Pereira et al, 2011). However, the obtained response seems to be similar for both reinforcement
solutions under cyclic combined in-plane and out-of-plane loading using airbags (Pereira et al, 2011). The results are hardly comparable as there is no in-plane action in the present case. The numerical analysis for impulsive loading carried out suggests a considerable difference between both reinforcement solutions but it must be kept in mind that the results were not validated with experimental data.

Certainly, higher improvements in the response of the wall could be obtained by increasing the amount of reinforcement added. Figure 5.28 shows the maximum displacement for the URM wall and compares it with three different solutions for the bed joint reinforcement: JAR 4mm – bed joint reinforcement with 4/100 in every two horizontal joint; JAR 5mm – bed joint reinforcement with 5/100 in every two horizontal joint; and JAR all joints – bed joint reinforcement with 4/100 in every horizontal joint.

Figure 5.27 – Influence of reinforcement in the response of the wall.

Figure 5.28 – Influence of the amount of reinforcement in the response of the wall.
As can be seen in Figure 5.28 the improvement with increasing the amount of reinforcement isn’t proportional to the reinforcement ratio. Thus, it is reasonable to assume that the behaviour is rather dependant on the compressive behaviour of masonry. This can be seen in Figure 5.29, where another analysis was performed with the minimum JAR reinforcement and a masonry with double compressive strength ($f_c = 7.5\ MPa$). Doubling the compressive strength of masonry, the response (in terms of maximum displacement) is less than half.

![Figure 5.29 – Influence of the compressive strength of masonry in the reinforced solutions.](image)

In order to have a better grasp on the influence of these minimum reinforcement solutions, additional model were studied for different wall thickness. Thickness of 140 mm and 230 mm were studied, the selected values for the thickness of the wall were determined assuming the use of 30×20×11 cm$^3$ and 30×20×20 cm$^3$ bricks plus same plaster on both sides, even if plaster is usually not considered for design purposes. Figure 5.30 shows the obtained results for the minimum reinforcement solutions for these two different thickness walls. The grid reinforcement was the same for both models, but due to construction restrictions, the bed joint reinforcement needed to be different. RND .4/80 and RND .4/150 trusses were used for the 140 mm and the 230 mm respectively. It can be seen that the grid reinforcement has a higher improvement in the response of the wall for both models. These results show that the response of the wall to these impulse loadings is highly influenced by its thickness. The same loading profile resulted in maximum deformations of about 400 mm, 15 mm and 3 mm for increasing thickness of 140 mm, 180 mm and 230 mm respectively.
5.4 Parametric Study

In order to discuss the influence of the mechanical and geometric properties of masonry infill panels on the blast response, a parametric study was performed. For this kind of analysis it is important to understand this influence as a function of the impulsive loading. This can be obtained by varying the applied load according to the scaled distance \( Z (Z = R/W^{1/3}) \) which depends on the weight of the explosive \( (W) \) and the standoff distance \( (R) \).

Knowing the weight of the explosive and its standoff distance it is possible to determine the applied reflected pressure for different loading scenarios using equations 5.1 – 5.3 developed for point source explosions.

\[
P_{50} = 6784 \frac{W}{R^3} + 93 \left( \frac{W}{R^3} \right)^{1/2} \quad (5.1)
\]

\[
P_r = 2P_{50} \left( \frac{7P_0 + 4P_{50}}{7P_0 + P_{50}} \right) \quad (5.2)
\]

\[
t_D = 10.23 \frac{W^{1/3}}{P_{50}} \quad (5.3)
\]

Here \( P_{50} \) is the side-on overpressure, \( P_r \) is the reflected pressure and \( t_D \) is the positive duration. A detailed description of these equations is given in Chapter 2.
Table 5.5 shows the range of properties selected for this parametric study. The tensile and the compressive strength range from the static reference value up to a DIF equal to 5. The Young’s modulus ranges from its static reference up to a DIF equal to 3 and the fracture energy ranges from its static reference up until a DIF equal to 4. The selected values for the thickness of the wall were determined assuming the use of 30×20×11 cm³ and 30×20×20 cm³ bricks plus same plaster on both sides, even if plaster is usually not considered for design purposes. The reinforcement can either be absent (URM), in the bed joint (JAR) or in the plaster (RAR). Unless stated otherwise, when varying a selected property the remaining properties are kept at their mid values.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Min</th>
<th>Mid</th>
<th>Max</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_t$ [MPa]</td>
<td>0.125</td>
<td>0.375</td>
<td>0.650</td>
</tr>
<tr>
<td>$f_c$ [MPa]</td>
<td>1.26</td>
<td>3.78</td>
<td>6.30</td>
</tr>
<tr>
<td>$E_0$ [GPa]</td>
<td>3.6</td>
<td>7.2</td>
<td>10.8</td>
</tr>
<tr>
<td>$G_f^I$ [N/mm]</td>
<td>0.012</td>
<td>0.025</td>
<td>0.050</td>
</tr>
<tr>
<td>Thickness [mm]</td>
<td>140</td>
<td>180</td>
<td>230</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>JAR</td>
<td>URM</td>
<td>RAR</td>
</tr>
</tbody>
</table>

The compressive strength, Figure 5.31a, has a considerable influence on the maximum displacement of the wall, for smaller scaled distances. This influence appears to fade once a certain level of compressive strength is achieved, which means that from a certain point there is no real advantage on increasing the compressive strength. The Young’s modulus, Figure 5.31b, influences the maximum displacement of the wall at all levels of scaled distance. When analysing the tensile strength, Figure 5.31c, it is possible to see the same behaviour of that the compressive strength, with a similar conclusion. When varying the tensile strength, the fracture energy was also changed in the same proportion as the tensile strength. The Mode I-fracture energy, Figure 5.31d, only influences the maximum displacement at smaller scaled distances. Here, the tensile strength was kept the same for all models. The thickness of the wall, Figure 5.31e, is one of the parameters with larger influence on the maximum displacement of the wall. As seen before, the use of reinforcement solutions decreases the maximum displacement of the wall, but only to moderate extent for the (low) amounts of reinforcement used. Figure 5.31f shows that this influence is inversely proportional to the scaled distance.
Figure 5.31 – Parametric study on the properties of infill walls subjected to blast loading: a) compressive strength; b) Young’s modulus; c) tensile strength; d) mode-I fracture energy; e) thickness of the wall; f) reinforcement solution.
5.5 PRESSURE-IMPULSE DIAGRAMS

Pressure-Impulse diagrams (Figure 5.32) are empirical tools that allow a given load-impulse combination, which will cause a specific level of damage, to be assessed readily (Cormie et al, 2009). These diagrams can be used to assess a specific loading profile which caused certain damage to an element, in a post-disaster scenario, as shown in Chapter 3. On the other hand, these tools can be used at an early design stage to get an approximation of the damage to an element given a specific loading profile.

Figure 5.32 – Generic pressure-impulse (P-I) diagram.

In order to make it easier for the designer to use these tools, for the structural elements under study, it is better to have damage criteria (Table 5.6) instead of pure deflection curves. For the present work, the criteria defined by UFC-3-340-02 (2008) will be applied, meaning that instead of iso-deflection curves, the P-I diagrams were plotted with two levels of damage, reusable and non-reusable. With the FE model calibrated, several simulations were performed for different levels of overpressures and impulses. For these numerical models a 1:1 scale was used, meaning that the masonry infill panels have an area of 5250 by 2550 mm². Two different masonry infill panels were studied, with 180 mm thickness and 230 mm thickness.

<table>
<thead>
<tr>
<th>Element</th>
<th>Yield pattern</th>
<th>Maximum support rotation</th>
</tr>
</thead>
<tbody>
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<td><strong>Masonry Reusable</strong></td>
<td>One-way</td>
<td>0.5°</td>
</tr>
<tr>
<td></td>
<td>Two-way</td>
<td>0.5°</td>
</tr>
<tr>
<td><strong>Masonry Non-reusable</strong></td>
<td>One-way</td>
<td>1.0°</td>
</tr>
<tr>
<td></td>
<td>Two-way</td>
<td>2.0°</td>
</tr>
</tbody>
</table>

Table 5.6 – Masonry damage criteria (UFC-3-340-02, 2008).
Figure 5.33 to Figure 5.35 present the obtained pressure-impulse diagrams for the three constructive solutions under study. As expected, the reinforced solutions are able to accommodate somewhat larger loading profiles and have the non-reusable and the reusable curves further away. This becomes clear when analysing Figure 5.36 for the 180 mm thickness and Figure 5.37 for the 230 mm thickness, where a comparison for the three constructive solutions under study is performed for both levels of damage. Of course, higher percentages of reinforcement can be used to obtain a specified performance but, here, the focus is given to the minimum amounts of reinforcement.

Figure 5.33 – P-I diagram for unreinforced masonry infill panel: a) 180 mm; b) 230 mm.

Figure 5.34 – P-I diagram for masonry infill panel minimum with bed joint reinforcement: a) 180 mm; b) 230 mm.
Figure 5.35 – P-I diagram for masonry infill panel with minimum grid reinforcement in the plaster: a) 180 mm; b) 230 mm.

Figure 5.36 and Figure 5.37 show that if the damage level required is the reusable stage, there is no real advantage in using the minimum reinforcement solutions, for weak masonry infills and large panels. Only at the non-reusable stage the minimum reinforcement solutions have a relevant contribution for the wall’s response.

Figure 5.36 – P-I diagrams comparing three solutions for 180 mm: a) reusable stage; b) non-reusable stage.

These P-I diagrams can be used to select the proper constructive solution regarding a specific level of blast loading under design. As can be seen from Figure 5.33 to Figure 5.37, the thickness of the wall is one important aspect to account for. The grid
reinforcement is the solution with the highest mechanical improvement regarding the maximum displacement of the wall. Another important aspect regarding this reinforcement solution is that it also protects against the appearance of flying debris into, possibly, occupied areas.

![Image: P-I diagrams comparing three solutions for 230 mm: a) reusable stage; b) non-reusable stage.]

Figure 5.37 – P-I diagrams comparing three solutions for 230 mm: a) reusable stage; b) non-reusable stage.

5.6 Final Remarks

A newly developed test setup for dynamic out-of-plane testing on walls was presented, including the developed sensors and acquisition apparatus. Using underwater blast wave generators (WBWG) it was possible to have a surface area distribution of pressure avoiding the generation of high velocity fragments and reducing atmospheric sound wave. Also the required test site area can be greatly reduced using these WBWG as opposes to traditional air blast, where to have a full surface distribution the charge need to be far away from its target.

One unreinforced masonry infill panel was tested under blast loading using underwater blast wave generators and the results were presented. The obtained results were used to calibrate a numerical model using ABAQUS Explicit dynamics software. A good agreement between the numerical model and the experimental data was obtained, allowing a detailed study on this kind of masonry panels under dynamic out-of-plane
loading in the form of a parametric study for different loading conditions and different properties of masonry. This parametric study showed that there is a point where the increase of the compressive and tensile strength is no longer effective (as the response becomes elastic), while the Young’s modulus and the wall thickness are the parameters with the higher influence on the behaviour of the wall panel. Two different reinforcement solutions were analysed numerically and the results show that both solutions improve the response of the wall, to a moderate extent, as the amounts of reinforcement are close to the minimum values (only for crack control). The reinforcement solution with the best performance under blast loading was the grid reinforcement in the plaster of the wall, as the amount of reinforcement is slightly higher.

These results were used to create empirical tools – Pressure-Impulse diagrams – which can help the designer to estimate the response of the element under different loading conditions. It was shown that the use of these (low percentage) reinforcement solutions is more effective considering the non-reusable stage of the element. If the requirement is the reusable stage there is no real advantage in the use of these (low percentage) reinforcement solutions, and the best way to improve the response of the wall would be increasing its thickness or designing the reinforcement according to the performance sought. More experimental data is required to confirm these findings and additional masonry infill walls should be tested.

5.7 REFERENCES


Protection is not an absolute concept and there is a level of protection where the cost of the protection provided with respect to the cost of potential loss is in balance. On one hand, protection cannot offer full guarantee of safety and, on the other hand, too much protection is a waste of resources with regard to what is expected to be saved. The purpose of protective construction is to improve the probability of survival of people and other contents in a given facility for a given threat. In order to improve this probability, one must first understand the threat and accordingly analyse the facility. In this chapter Risk Assessment will be addressed and applied to a large Public Transport (PT) Operator. This assessment will allow identifying potentially critical infrastructure, which will be studied and its security will be evaluated for different scenarios regarding blast loading.
6.1 TERRORISM

Terrorism is defined by the U. S. Department of State in the United States Code as “premeditated, politically motivated violence perpetrated against non-combatant targets by subnational groups or clandestine agents, usually intended to influence an audience” (DoS, 2007). The United Nations Security Council Resolution 1566 (2004) defines Terrorism as “criminal acts, including against civilians, committed with the intent to cause death or serious bodily injury, or taking of hostages, with the purpose to provoke a state of terror in the general public or in a group of persons or particular persons, intimidate a population or compel a government or an international organization to do or to abstain from doing any act, which constitute offences within the scope of and as defined in the international conventions and protocols relating to terrorism, are under no circumstances justifiable by considerations of a political, philosophical, ideological, racial, ethnic, religious or other similar nature”. There are four key distinguishing elements of terrorism: (a) It is premeditated – planned in advance and not conducted as an impulsive act of rage; (b) It is political – designed to change the existing political order; (c) It is aimed at civilians – not military personnel or facilities; (d) It is carried out by subnational groups – not a country’s army (TTSRL, 2008).

In terrorism, physical assets including people, products, services, information, and property are viewed as targets. Terrorist attacks are often spectacular, designed to disturb and influence a wide audience beyond the victims of the attack itself. Terrorists can operate individually or in large groups and can perpetrate their attacks in different ways for different goals: (a) Causing casualties; (b) Damaging or destroying critical infrastructure; (c) Disrupting the economy; (d) Harassing, weakening, or embarrassing the government; (e) Discouraging tourism or investments due to perceived insecurity (Bennett, 2007).

Armed attacks and bombings constituted nearly 80% of all terrorist attacks in 2011 (Figure 6.1a). Suicide attacks accounted for just 2.7% of terrorist attacks in 2011 but 21% of all terrorism-related fatalities, a fact that underscores their extreme lethality. IEDs (Improved Explosive Devices) were the most frequently used and deadliest terrorist weapon employed (Figure 6.1b). The number of bombing attacks has remained
relatively consistent over the past five years, ranging between approximately 4000 and 4500 annually (NCC, 2012).

Over 12000 people were killed by terrorist attacks in 2011. More than half of the people killed in 2011 were civilians and 755 were children (Figure 6.2). Although civilians were the largest single group of victims killed in terrorist attacks, their numbers, between 2007 and 2011, in proportion to the total number of deaths have decreased. The number of government employees and contractors, government officials and police has increased from the previous year (NCC, 2012).

Figure 6.1 – Number of attacks and deaths by terrorism in 2011: a) by attack type and b) by weapon type (NCC, 2012).

Figure 6.2 – Deaths by victim categories in 2011 (NCC, 2012).
In the year of 2011, over two-thirds of all terrorist attacks struck infrastructure or facilities. Of those, transportation assets and public places were the most frequently targeted. Transportation facilities – such as vehicles, buses and transportation infrastructure – incurred damage in about 39% of the attacks, while public places – including communal areas, markets, polling stations, religious institutions, schools and residences – incurred damage in about 28% of the attacks (Figure 6.3).

![Figure 6.3 – Attacks damaging facilities by facilities category in 2011 (NCC, 2012).](image)

In summary, bombing is the weapon of choice for terrorist attacks, being used in 40% of the terrorist attacks worldwide (in the year of 2011) and responsible for most of fatalities and injuries related to terrorism. Looking at the damaged infrastructure it is possible to see transportation infrastructure as the most targeted type of infrastructure.

### 6.2 Risk Assessment

Terrorism has been described as the deliberate use of violence to create a sense of shock, fear and outrage in the mind of the target population. One of the reasons that make this easy to achieve is that developed societies have become very dependent on complex and fragile systems (railways, airlines, gas pipelines, electricity infrastructure, large shopping areas and business centres, etc.) which are both vulnerable and critical to society’s functions, and provide the terrorist with many suitable targets. Attackers can use various weapon systems in different combinations and such events cannot be
predicted. However, reliable information and objective threat and risk assessment can produce effective estimates of such incidents.

Risk Assessment must be understood as a step or a process inside the Risk Management model. Figure 6.4 shows a simplified representation of the Risk Management model. Risk Management is a systematic and analytical process by which an organization identifies, reduces, and controls its potential risks and losses (Homeland Security, 2011). This process allows organizations to determine the magnitude and effect of the potential loss, the likelihood of such loss actually happening, and the countermeasures that could lower the probability or magnitude of loss.

The first step is to conduct a threat assessment where the threat or hazard is identified, defined and quantified. The next step of this model is to identify the values of the asset that need to be protected. After performing the asset values assessment, the next step is to conduct a vulnerability assessment where there is an evaluation of the potential vulnerability of the critical assets against the identified threats or hazards. The next step is the risk assessment: analysing the threat, asset value, and vulnerability it is possible to ascertain the level of risk for each critical asset against applicable threat. The final step of this model is to consider mitigation options. Risk assessment on its own incorporates the first four steps described: threat, impact and vulnerability assessment which allow assessing the risk. In the following topics each of these steps will be addressed briefly.

There are a number of methods available to conduct an organization’s risk assessment, and the steps can be accomplished in different sequences. Examples are given in:
• FEMA 452, from the Federal Emergency Management Agency, USA, How-To Guide to Mitigate Potential Terrorist Attacks Against Buildings – considers a comprehensive methodology to prepare a risk assessment, providing means to assess the risk to the assets and to make risk-based decisions on how to mitigate those risks. The primary use of this methodology is for buildings, although it could be adapted for other types of critical infrastructure (FEMA 452, 2005).


• DEMA (The RVA model), from the Danish Emergency Management Agency, Approach to Risk and Vulnerability Analysis for Civil Contingency Planning – has developed a generic scenario-based model for risk and vulnerability analysis. The model is developed for government agencies with responsibilities for society’s critical functions (DEMA, 2006).

• UFC 4-020-01, from the Department of Defence, USA, DoD Security Engineering Facilities Planning Manual – it includes a procedure for risk analysis as a part of a preliminary design criterion (UFC 4-020-01, 2008).

All the previous methodologies have one common objective: to apply a quantitative assessment process that identifies the assets at highest risk. One key factor that can lead to an effective risk assessment is the selection of the entities/people brought into the process. These assessments should have inputs from, but not limited to, police agencies, intelligence agencies, structural engineers and national emergency management agencies.

### 6.3 TRANSPORTATION INFRASTRUCTURE

Modern society is heavily dependent on transportation networks. In the past 60 years, these networks have been seen as an appropriate target for terrorists (Muhlhausen and McNeil, 2011). They allow easy access and they provide suitable cover for escape. They also provide concentrations of civilians and their slaughter never fails to generate high
levels of public interest, both national and international. The attacks on the Spanish railway network in Madrid in March 2004, the Metro and Bus strikes in London in July 2005, the Mumbai train attacks in July 2006 and the bombing in the international arrival hall of Moscow's busiest airport in January 2011, among many others, all point to public passenger transport networks as being suitable terrorist targets.

COUNTERACT (Cluster Of User Networks in Transport and Energy Relating to anti-terrorist ACTivities) was an European research project set up to improve security against terrorist attacks aimed at public passenger transport, intermodal freight transport and energy production and transmission infrastructure (COUNTERACT, 2006). The project focused on the protection of critical transport infrastructures, public transport passengers and goods. It reviewed the existing security policies, procedures, methodologies and technologies to identify the best practices, which in turn have been promoted throughout the relevant security community in the EU. One of the main objectives of the project was to develop generic guidelines for conducting risk assessment in public transport networks. These guidelines will be presented briefly in the next sections.

### 6.3.1 Identify key infrastructure

In order to identify possible targets for attacks it is necessary to structure the whole PT (Public Transport) system in an operational diagram. To structure the system it is important to address, namely, the following aspects:

a) How attractive is the city/region for terrorists compared to others?

b) How attractive is the PT system for terrorists compared to other potential targets in the city/region?

c) Which system elements are most attractive for terrorists?

d) Which parts of the network are most critical to the operation?

e) What is the number of passengers in interchange/stations/stops, vehicles (at peak times)?

f) Is there special/ large events organized nearby that could temporarily raise the risk level?
g) Are there institutions/organizations nearby that generate a group of passengers which is at special risk?

By structuring the PT system taking into consideration the aspects above, it will be possible to assess the key infrastructure in the system. The combination of the two components, Probability of Occurrence and Impact/Severity, in a portfolio allows the user to identify risks easily.

### 6.3.2 Probability of occurrence

The probability of occurrence is the possibility of a threat being executed, which is measured in escalating categories. COUNTERACT suggest a 5-level scale (Table 6.1), where the criteria for differentiation between the different steps focus mainly on the frequency that the threat has been executed in their own or in other PT operations.

<table>
<thead>
<tr>
<th>Probability</th>
<th>Level</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very High</td>
<td>5</td>
<td>The threat can be executed at any time and/or has been executed within the organization repeatedly.</td>
</tr>
<tr>
<td>High</td>
<td>4</td>
<td>It has to be reckoned with the threat being executed repeatedly. The threat has been executed within the own organization once.</td>
</tr>
<tr>
<td>Possible</td>
<td>3</td>
<td>An execution of the threat has to be reckoned with. The threat has been executed repeatedly within other PT operations world-wide, or at least once within a PT operation in the own/neighbouring country.</td>
</tr>
<tr>
<td>Low</td>
<td>2</td>
<td>The threat is executed rarely, but has been executed in isolated cases in other organizations (world-wide).</td>
</tr>
<tr>
<td>Very Unlikely</td>
<td>1</td>
<td>An execution of the threat is extremely unlikely, and the threat has never been executed in other PT operations before.</td>
</tr>
</tbody>
</table>

### 6.3.3 Severity of occurrence

Impact/Severity stands for the damage to an asset arising from the execution of a threat, which is measured in escalating categories. COUNTERACT suggest a 4-level scale (Table 6.2), where the criteria for differentiation between the different steps focus mainly on the consequences of the various threats for persons, property and PT
operator. The final classification for the Impact would be the maximum of the three consequences.

Table 6.2 – Impact/Severity (COUNTERACT, 2009).

<table>
<thead>
<tr>
<th>Consequences for Persons</th>
<th>Consequences for Property/Environment</th>
<th>Consequences for PT Operator and Services</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Disastrous</strong> 4</td>
<td>Several deaths and/or numerous severe injuries</td>
<td>Most severe damage to property and/or environment</td>
</tr>
<tr>
<td><strong>Critical</strong> 3</td>
<td>Low number of deaths and/or severely injured</td>
<td>Severe damage to property and/or environment</td>
</tr>
<tr>
<td><strong>Marginal</strong> 2</td>
<td>Light casualties</td>
<td>Notable damage to property and/or environment</td>
</tr>
<tr>
<td><strong>Uncritical</strong> 1</td>
<td>Possibility of few light casualties</td>
<td>Small damage to property and/or environment</td>
</tr>
</tbody>
</table>

6.3.4 Risk categories

The combination of Probability of Occurrence and Impact/Severity results in the Risk categories applying the following formula:

\[
Risk = Probability\ of\ Occurrence \times Impact/Severity
\]  \hspace{1cm} (6.1)

COUNTERACT suggests four risk categories according to their score (Table 6.3) and the subsequent required action.

Table 6.3 – Risk categories (COUNTERACT, 2009).

<table>
<thead>
<tr>
<th>Categories</th>
<th>Score</th>
<th>Action Required</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Intolerable</strong></td>
<td>15 – 20</td>
<td>Must be avoided or Impact must be mitigated as far as possible</td>
</tr>
<tr>
<td><strong>Precarious</strong></td>
<td>8 – 12</td>
<td>Shall only be accepted if the efforts for prevention and/or mitigation of impact is unreasonably high</td>
</tr>
<tr>
<td><strong>Tolerable</strong></td>
<td>4 – 6</td>
<td>Shall be accepted, but threat needs to be assessed regularly</td>
</tr>
<tr>
<td><strong>Negligible</strong></td>
<td>1 – 3</td>
<td>Shall be accepted</td>
</tr>
</tbody>
</table>

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6.4 CASE STUDY – “LISBON DISTRICT”

As stated previously, transportation infrastructure is the most targeted type of infrastructure when it comes to terrorist attacks. A risk assessment was performed on the largest Public Transportation Operator in Portugal – REFER. REFER was created in 1997, through Law-Decree 104/97, of 29 April, as a public company responsible for providing the public service of managing the national railway network infrastructure in Portugal. REFER manages an operational network of 2800 km with a daily average of trains of 1776 and comprises 561 train stations, Administration Offices (AO) and Operational Command Centres (OCC) (REFER, 2012).

Performing a risk assessment analysis allow to identify the elements in the network with the highest risk regarding a specific threat and provides the decision makers with essential tools to prioritize possible interventions. The COUNTERACT methodology was chosen for its specific character regarding Public Transportation Operators. Because of time constrains only the District of Lisbon was considered in this study. In this part of Portugal REFER serves a population of two million residents plus commuters, including the Portuguese Capital.

6.4.1 Key infrastructure

The first step in the analysis is to identify all the elements present in this geographic area and, following predefined criteria, identify those where the probability of occurrence and the severity will be determined. In this geographic area, REFER has five railways (Figure 6.5), one Operational Command Centre (OCC) and one Administrative Office (AO). The railways being: a) “Linha de Sintra”; b) “Linha de Cascais”; c) “Linha de Cintura”; d) part of “Linha do Norte” and e) part of “Linha do Oeste”. Identifying all the elements resulted in 71 train stations plus an OCC and an AO.

The next step consists in an element classification following five distinct criteria. The choice of the selected criteria took into account the internal methodology for characterization of stations used by REFER. The criteria selected were: C1 – Passenger flow; C2 – Service provided; C3 – Mobility; C4 – Significance; and C5 – Location.
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Figure 6.5 – Railways in the selected area for the risk assessment.

C1 – Passenger flow:
The number of passenger starting or ending a journey at a station is an important indicator. This influences the design and maintenance of the element, and regarding this specific study, provides a direct relation with the potential victims of a possible attack. For each station a value of $CI$ is assigned based on the following equations:

\[
CI_i = \begin{cases} 
V_i < 10000 & C1_i = \left( \frac{V_i - V_{min}}{10000 - V_{min}} \right) \times 25 \\
10000 \leq V_i < 50000 & C1_i = \left( \frac{V_i - 10000}{50000 - 10000} \right) \times 25 + 25 \\
50000 \leq V_i < 250000 & C1_i = \left( \frac{V_i - 50000}{250000 - 50000} \right) \times 25 + 50 \\
V_i \geq 250000 & C1_i = \left( \frac{V_i - 250000}{V_{max} - 250000} \right) \times 25 + 75 
\end{cases}
\]

Where:
- $CI_i$ – Value of criteria $CI$ for the station $i$;
- $V_i$ – Monthly passenger flow for the station $i$;
- $V_{min}$ – Minimum monthly passenger flow for all stations under analysis;
- $V_{max}$ – Maximum monthly passenger flow for all stations under analysis.

The values defined for each interval were defined according to the internal manuals of the PT Operator.

C2 – Service provided:
The service provided at a specific element affects the distance covered during the journey, the amount of time a passenger stays in the station, and the frequency of trains, among other aspects. The service reflects the area of influence associated with the
station. The services considered are: Suburban, Regional, Inter-Regional, Inter-City, Alfa Pendular (faster strains North-South connecting Lisbon to Porto and other main cities) and International. For each station a value of $C_2$ is assigned based on the following equation:

$$C_{2i} = \sum S_{pi}$$

(6.3)

Where,

- $Suburban = 0.225$
- $Regional = 0.100$
- $Inter – Regional = 0.100$
- $Inter – City = 0.225$
- $Alfa Pendular = 0.225$
- $International = 0.125$

The weights of each service were defined according to the internal manuals of the PT Operator.

**C3 – Mobility:**

The complementary services and the offered transport conditions at a specific station should take into account the mobility regarding other transport modes (soft modes, highway, railway, sea, aerial, etc.). The connection with other services affects the station capacity and construction layout. This criterion takes into account all the different services provided at each station. For each station a value of $C_3$ is assigned based on the following equation:

$$C_{3i} = \sum M_i$$

(6.4)

Where,

- $Soft mode = 0.04$
- $Private cars = 0.08$
- $Taxis = 0.08$
- $Buses = 0.15$
- $Tram = 0.15$
- $Boats or Ferries = 0.25$
- $Underground Metro = 0.25$
C4 – Significance:
This criterion reflects the significance of each station according to its nature, and it is one of the most difficult criteria to define. However a few parameters (Table 6.4) are defined to help verifying if a station gathers the necessary conditions to be labelled as significant for each level. Five levels of significance were defined, namely: National, Regional, Touristic, Architectural and Rail network. For each station a value of $C4$ is assigned based on the following equation:

$$C4_i = \sum S_i$$  \hspace{1cm} (6.5)

Where,

- **National** 0.30
- **Regional** 0.25
- **Touristic** 0.15
- **Architectural** 0.15
- **Rail network** 0.15

C5 – Location:
This criterion takes into account the location of the station. The selected stations for this study are located in 12 municipalities of the Lisbon district. The value of this criterion is proportional to the population of each municipality and ranges from 1.0 to 0.0 for the highest population to the lowest population, respectively. The number of inhabitants for each municipality was taken from the national census of the year 2011 (INE, 2012).

Final score:
Other criteria could be selected in this first step. Events such as large concerts, sports events, every outdoor activity that implies a large volume of people using a specific element at a specific time could lead to a different classification of the element. This dynamic property of this process is an important feature and shows how “real time” monitoring is so important. In order to achieve the final ranking score of each element in the network the following equation was applied:

$$C_i = 0.51 \times C1 + 0.13 \times C2 + 0.13 \times C3 + 0.13 \times C4 + 0.10 \times C5$$  \hspace{1cm} (6.6)
The weights selected for each criterion can be different according to the specific network under study or taking into account different selected criteria. It should be noted that the passenger flow is taken as the most important parameter, with about 50% of the weight.

Table 6.4 – Significance justification parameters.

<table>
<thead>
<tr>
<th>Significance</th>
<th>Justification parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>National</td>
<td>Highlighted station with a high hierarchical national level. Placed with special relevance as an image of the Portuguese rail network for the Portuguese and foreigner passenger.</td>
</tr>
<tr>
<td>Regional</td>
<td>Highlighted station at regional level. Placed with special relevance as representative of its region. A station with National significance has cumulatively Regional significance. Also applies if the station in a district capital.</td>
</tr>
<tr>
<td>Touristic</td>
<td>Station with particular interest from the tourist point of view as it provides accessibility to the tourist area in which it operates. Station belonging to a particular railway with a touristic character. Connections with touristic routes, namely with other transport modes.</td>
</tr>
<tr>
<td>Architectural</td>
<td>Station with recognised historical and architectural significance. Station with relevant aesthetics or cultural elements.</td>
</tr>
<tr>
<td>Rail network</td>
<td>Station with unique conditions or services regarding an operational point of view. Station with historic value regarding the development of the national railway network.</td>
</tr>
</tbody>
</table>

These five criteria were applied to all elements of the network and a ranking score was achieved. An acceptable threshold should be defined by the team selected to perform the risk assessment, in order to proceed with the analysis. Here, a threshold of 0.6 was selected, meaning that every element with this score and higher was taken to the next steps of the COUNTERACT process. Table 6.5 shows the individual scores for selected examples of elements in this network. The full list of scores can be seen in Annex C.1.

As a result of the first step 12 stations were selected with scores higher than 0.6 (Table 6.7). These stations will join the CCO and the AO for the next steps. These last two elements were immediately moved to the following steps due to their specificity.
Chapter 6 – Protecting Infrastructure

Table 6.5 – Selected examples of key infrastructure scores.

<table>
<thead>
<tr>
<th>#</th>
<th>Railway</th>
<th>Station</th>
<th>C1</th>
<th>C2</th>
<th>C3</th>
<th>C4</th>
<th>C5</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>Cascais</td>
<td>Oeiras</td>
<td>0.78</td>
<td>0.23</td>
<td>0.20</td>
<td>0.00</td>
<td>0.30</td>
<td>0.49</td>
</tr>
<tr>
<td>15</td>
<td>Cascais</td>
<td>Estoril</td>
<td>0.68</td>
<td>0.88</td>
<td>0.20</td>
<td>0.15</td>
<td>0.37</td>
<td>0.54</td>
</tr>
<tr>
<td>17</td>
<td>Cascais</td>
<td>Cascais</td>
<td>0.78</td>
<td>0.23</td>
<td>0.20</td>
<td>0.15</td>
<td>0.37</td>
<td>0.51</td>
</tr>
<tr>
<td>23</td>
<td>Cintura</td>
<td>Chelas</td>
<td>0.18</td>
<td>0.23</td>
<td>0.12</td>
<td>0.40</td>
<td>1.00</td>
<td>0.29</td>
</tr>
<tr>
<td>30</td>
<td>Norte</td>
<td>Bobadela</td>
<td>0.43</td>
<td>0.23</td>
<td>0.12</td>
<td>0.00</td>
<td>0.36</td>
<td>0.32</td>
</tr>
<tr>
<td>35</td>
<td>Norte</td>
<td>Vila Franca de Xira</td>
<td>0.75</td>
<td>0.65</td>
<td>0.20</td>
<td>0.15</td>
<td>0.24</td>
<td>0.54</td>
</tr>
<tr>
<td>43</td>
<td>Oeste</td>
<td>Telhal</td>
<td>0.01</td>
<td>0.10</td>
<td>0.12</td>
<td>0.15</td>
<td>0.68</td>
<td>0.12</td>
</tr>
<tr>
<td>46</td>
<td>Oeste</td>
<td>Mafra</td>
<td>0.01</td>
<td>0.20</td>
<td>0.12</td>
<td>0.15</td>
<td>0.12</td>
<td>0.08</td>
</tr>
<tr>
<td>58</td>
<td>Sintra</td>
<td>Lisboa Rossio</td>
<td>1.00</td>
<td>0.23</td>
<td>0.60</td>
<td>1.00</td>
<td>1.00</td>
<td>0.85</td>
</tr>
<tr>
<td>62</td>
<td>Sintra</td>
<td>Amadora</td>
<td>0.89</td>
<td>0.23</td>
<td>0.20</td>
<td>0.60</td>
<td>0.31</td>
<td>0.62</td>
</tr>
</tbody>
</table>

### 6.4.2 Probability of occurrence

In this step, each element is crossed with the selected threats. For the purpose of this study, only threats involving explosions were selected. Five levels were selected according to the capacity of the delivery system:

- Suicide vest (9 kg TNT)
- Luggage (20 kg TNT)
- Car (500 kg TNT)
- Van (1 500 kg TNT)
- Truck (25 000 kg TNT)

The calculation of the probability of occurrence for each threat and each element implies a research on previous and similar attacks and attempts. Table 6.6 shows some of the previous attacks on PT Operators after the year 2000.

After crossing the information of previous attacks on similar PT Operators and their delivery systems, a value of Probability of Occurrence (Table 6.1) is assigned to each threat and for each element, with the results shown in Table 6.7. As can be seen the highest value for Probability of Occurrence is 3. No threat has been executed within the own organization, but similar threats has been executed repeatedly within other PT Operators worldwide, including neighbouring countries.
Table 6.6 – Previous terrorist attacks on PT Operators using explosives (examples).

<table>
<thead>
<tr>
<th>Place</th>
<th>Year</th>
<th>Fatalities</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angola</td>
<td>2001</td>
<td>252</td>
<td>Used an explosive device to derail a train and attack the passengers with fire weapons.</td>
</tr>
<tr>
<td>Refiganj, India</td>
<td>2002</td>
<td>130</td>
<td>Derail a train over a bridge.</td>
</tr>
<tr>
<td>Stavropol Krai, Russia</td>
<td>2003</td>
<td>46</td>
<td>Suicide bomber in a train.</td>
</tr>
<tr>
<td>Moscow, Russia</td>
<td>2004</td>
<td>41</td>
<td>Suicide bomber at the subway station Avtozavodskaya.</td>
</tr>
<tr>
<td>Madrid, Spain</td>
<td>2004</td>
<td>191</td>
<td>Several explosions in the railway system.</td>
</tr>
<tr>
<td>London, UK</td>
<td>2005</td>
<td>56</td>
<td>Three explosions at subway stations and one explosion in a bus.</td>
</tr>
<tr>
<td>Mumbai, India</td>
<td>2006</td>
<td>209</td>
<td>Several explosions in the suburban system.</td>
</tr>
<tr>
<td>Moscow, Russia</td>
<td>2010</td>
<td>40</td>
<td>Two suicide bombers at subway stations, Lubyanka and Park Kultury.</td>
</tr>
<tr>
<td>Moscow, Russia</td>
<td>2011</td>
<td>35</td>
<td>Suicide bomber at Domodedovo airport.</td>
</tr>
</tbody>
</table>

Table 6.7 – Probability of occurrence.

<table>
<thead>
<tr>
<th>#</th>
<th>Railway</th>
<th>Station</th>
<th>Vest</th>
<th>Luggage</th>
<th>Car</th>
<th>Van</th>
<th>Truck</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Cascais</td>
<td>Cais do Sodré</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>Cascais</td>
<td>Algés</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>19</td>
<td>Cintura</td>
<td>Campolide</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>20</td>
<td>Cintura</td>
<td>Sete Rios</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>21</td>
<td>Cintura</td>
<td>Entrecampos</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>22</td>
<td>Cintura</td>
<td>Roma-Areeiro</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>25</td>
<td>Norte</td>
<td>Santa Apolónia</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>27</td>
<td>Norte</td>
<td>Oriente</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>58</td>
<td>Sintra</td>
<td>Rossio</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>59</td>
<td>Sintra</td>
<td>Benfica</td>
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<td>3</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>62</td>
<td>Sintra</td>
<td>Amadora</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>66</td>
<td>Sintra</td>
<td>Agualva-Cacém</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>72</td>
<td>CCO</td>
<td>CCO-Lisboa</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>73</td>
<td>AO</td>
<td>Administration</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

6.4.3 Severity of occurrence

As shown before in Table 6.2, the consequences can be separated in three categories: consequences for persons, property and PT Operations. In the case of consequences for persons, the impact/severity is measured by the numbers of injured people and fatalities. This was estimated according to previous attacks with similar delivery systems and the number of passengers at peak time for each station. The consequences for property were
estimated studying the layout of the station. Figure 6.6 shows the maximum pressure for the threats under study for different standoff distances. Each threat has a minimum standoff distance for each station layout, meaning that, as an example, it is not possible for a truck carrying 25 000 kg of TNT equivalent to get closer than 10 meters from the Cais do Sodré main building, while a vest carrying 9 kg of TNT equivalent can get virtually anywhere. Following this procedure for each station layout it is possible to establish minimum standoff distances for each threat and each station. With this information and the data from Figure 6.6 it is possible to determine the maximum pressure each threat can develop for every station. There are reference charts, such as FEMA 426 (2003) or Elsayed and Atkins (2008), where pressure thresholds are presented for different construction materials. In reality, the damage from blast loading depends on the maximum pressure and the positive duration (impulse), however, for this simple estimation only the maximum pressure was considered sufficient.

![Figure 6.6](image)

Figure 6.6 – Maximum pressure as a function of the standoff distance.

The consequences for the PT Operator were estimated according previous attacks on similar size stations. Studying the time while the attacked PT Operators ceased functions on a similar size station due to similar threats, it is possible to have an estimation of the required time for this PT Operator.
Table 6.8 – Severity of occurrence.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
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<tbody>
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<td>Cais do Sodré</td>
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<td>4 2 2</td>
<td>3 3 2</td>
<td>3 3 3</td>
<td>4 4 4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Algés</td>
<td>2 1 1</td>
<td>3 2 2</td>
<td>3 3 2</td>
<td>3 3 2</td>
<td>3 4 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Campolide</td>
<td>2 1 1</td>
<td>3 2 2</td>
<td>3 2 2</td>
<td>3 2 2</td>
<td>3 3 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sete Rios</td>
<td>3 1 1</td>
<td>3 2 2</td>
<td>3 3 2</td>
<td>3 3 2</td>
<td>3 4 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Entrecampos</td>
<td>3 1 1</td>
<td>4 2 2</td>
<td>3 3 2</td>
<td>3 3 2</td>
<td>4 4 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roma-Areeiro</td>
<td>2 1 1</td>
<td>3 2 2</td>
<td>3 2 2</td>
<td>3 2 2</td>
<td>3 3 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Santa Apolónia</td>
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<td>3 2 2</td>
<td>3 2 2</td>
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<td>3 4 4</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oriente</td>
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<td>3 2 2</td>
<td>3 3 3</td>
<td>3 4 3</td>
<td>4 4 4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rossio</td>
<td>3 1 1</td>
<td>4 2 2</td>
<td>3 3 2</td>
<td>3 4 2</td>
<td>4 4 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Benfica</td>
<td>2 1 1</td>
<td>3 2 2</td>
<td>3 3 2</td>
<td>3 3 2</td>
<td>3 4 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Amadora</td>
<td>3 1 1</td>
<td>4 2 2</td>
<td>3 3 2</td>
<td>3 3 2</td>
<td>3 4 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Agualva-Cacém</td>
<td>3 1 1</td>
<td>4 2 2</td>
<td>3 3 3</td>
<td>3 3 2</td>
<td>4 4 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CCO-Lisboa</td>
<td>2 1 3</td>
<td>2 2 3</td>
<td>1 2 3</td>
<td>2 3 4</td>
<td>2 4 4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Administration</td>
<td>2 1 2</td>
<td>2 2 2</td>
<td>1 2 2</td>
<td>2 3 3</td>
<td>2 4 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

6.4.4 Risk matrix

With the scores for the Probability of Occurrence and the Severity of Occurrence is possible to plot the Risk Matrix shown in Table 6.9. Because the Portuguese PT Operator has no previous occurrences of attacks, there is no combination with disastrous classification. Some combinations scored a critical classification (8 – 12) due to similar attacks on neighbouring countries and the respectively delivery systems (9 – 20 kg TNT), with easy “infiltration” and possibility to achieve low standoff distances.

This methodology is relatively easy to apply and provides the PT Operator with tools to quantify the relative risk for its elements. It must be kept in mind that this is a dynamic process and requires “real time” updates whenever there is a change in the network. A public event nearby one of the elements could lead to a higher risk value on that element at that specific time.
### Table 6.9 – Risk matrix.

<table>
<thead>
<tr>
<th>#</th>
<th>Railway</th>
<th>Station</th>
<th>Vest</th>
<th>Luggage</th>
<th>Car</th>
<th>Van</th>
<th>Truck</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Cascais</td>
<td>Cais do Sodré</td>
<td>9</td>
<td>12</td>
<td>6</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>Cascais</td>
<td>Algés</td>
<td>6</td>
<td>9</td>
<td>6</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>19</td>
<td>Cintura</td>
<td>Campolide</td>
<td>6</td>
<td>9</td>
<td>6</td>
<td>6</td>
<td>3</td>
</tr>
<tr>
<td>20</td>
<td>Cintura</td>
<td>Sete Rios</td>
<td>9</td>
<td>9</td>
<td>6</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>21</td>
<td>Cintura</td>
<td>Entrecampos</td>
<td>9</td>
<td>12</td>
<td>6</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>22</td>
<td>Cintura</td>
<td>Roma-Areeiro</td>
<td>6</td>
<td>9</td>
<td>6</td>
<td>6</td>
<td>3</td>
</tr>
<tr>
<td>25</td>
<td>Norte</td>
<td>Santa Apolónia</td>
<td>6</td>
<td>9</td>
<td>6</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>27</td>
<td>Norte</td>
<td>Oriente</td>
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<td>9</td>
<td>6</td>
<td>8</td>
<td>4</td>
</tr>
<tr>
<td>58</td>
<td>Sintra</td>
<td>Rossio</td>
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<td>12</td>
<td>6</td>
<td>8</td>
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</tr>
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<td>59</td>
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<td>Benfica</td>
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<td>9</td>
<td>6</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>62</td>
<td>Sintra</td>
<td>Amadora</td>
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<td>12</td>
<td>6</td>
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</tr>
<tr>
<td>66</td>
<td>Sintra</td>
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<tr>
<td>72</td>
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<td>CCO-Lisboa</td>
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</tr>
<tr>
<td>73</td>
<td>AO</td>
<td>Administration</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
</tbody>
</table>

Where,

<table>
<thead>
<tr>
<th>Intolerable: 15-20</th>
<th>Precarious: 8-12</th>
<th>Tolerable: 4-6</th>
<th>Negligible: 1-3</th>
</tr>
</thead>
</table>

As stated before, Risk Assessment in only one of the steps in the Risk Management model and the following step should be a detailed analysis of the highest risk elements, where prevention and mitigation measures would be studied. Comparing the risk values with and without those prevention and mitigation measures and the required investment costs, the PT Operator could make informed decisions on where and how to act. If a more detailed study on the behaviour of a specific element is required, a security assessment could be performed. In the next section a Security assessment, regarding the structural behaviour, will be performed on one of highest risk elements.

### 6.5 Protecting Critical Infrastructure

Rossio station (Figure 6.7a) was selected for this study. Formerly known as Central Station, this building was design between 1886 and 1887 by the Portuguese architect José Luís Monteiro. This building is classified since 1971 as Property of Public Interest by IGESPAR. The three-story building is constructed in limestone stonework. The building “L” shape can be seen in Figure 6.7b. This is a high value element in the PT
Operator Network, not only because of its effect on public opinion but due to its high passenger flow.

Figure 6.7 – Rossio station: a) front façade (East) and side façade (North); b) top view schematic.

Two different scenarios (Figure 6.8) were studied in this analysis:

- Scenario A – corresponds to an explosion at a square on the South side of the building (at 4 meters from the building). The luggage size IED (around 20 kg TNT) was the selected delivery system for this scenario.
- Scenario B – corresponds to an explosion at the East façade of the building (at 5 meters from the centre of the façade). The vehicle size IED (around 1500 kg TNT) was the selected delivery system for this scenario. Another situation was analysed – Scenario B’ – where the access to vehicles up until 25 meters from the East façade was closed. The same delivery system (1500 kg TNT) would still be possible but only at 25 meters from the centre of the façade.

Figure 6.8 – Different explosion scenarios: scenario A on the South side and scenario B on the East side.
6.5.1 FEM model

The FEM model was built in the ABAQUS software, where the Explicit solver was used. The definition of the geometric model was based on available drawings but without access to the detailed project of the building. This lack of information leads to some assumptions, which will be presented in this section.

Figure 6.10 shows the adopted geometry of the building. It is an “L” shaped building with around 2300 m$^2$ per floor and external walls having a thickness of 1.0, 0.8 and 0.6 m for the 1$^{\text{st}}$, 2$^{\text{nd}}$ and 3$^{\text{rd}}$ floor, respectively. The stone columns were assumed with 0.8×0.8 m$^2$ and 0.4×0.8 m$^2$, for the front section and side section, respectively. The dimensions used to construct this model can be seen in Figure 6.10a. The story heights are about 7.7, 6.8 and 6.8 m, from the ground level to the top (Figure 6.10b). The lower ends of the walls at the 1$^{\text{st}}$ floor are considered fixed to the ground (0.0 m level). Due to lack of information regarding the floors of the building, different models were prepared, neglecting and considering the contribution of pavements.

![Figure 6.9 – Building schematic.](image-url)

The walls were modelled as shell elements and the columns were modelled as beam elements. The model was discretized in several parts creating a mesh (Figure 6.11). This mesh was automatically generated by ABAQUS, and then manipulated and controlled in order to obtain a good quality mesh. The walls are discretized with quadrilateral 4 nodes (S4R) and 3 nodes (S3R) shell elements. These are three-dimensional, iso-parametric, doubly curved thin or thick shell element. These elements have five degrees of freedom at each node, reduced integration, hourglass control, and finite membrane...
strain (ABAQUS User Manual, 2010). The columns are discretized with 2-node linear beam elements (B31). The final mesh has 27968 nodes and 24491 elements.

Figure 6.10 – Adopted geometry: a) first floor; b) wall section.

Figure 6.11 – FEM mesh of the building.
2.5.1.1 Material model

The material model adopted is the CDP (Concrete Damaged Plasticity) model available in ABAQUS software and it is a modification of the Drucker-Prager model. A more detailed description of this model is given in section 5.3.1 of this document.

The mechanical properties for the masonry are presented in Table 5.2 and were collected from Oliveira (2003) and Tassios (2010). The data collected from these two sources corresponds to the static properties of limestone stonework (Static label). It should be noted that for this kind of analysis a DIF must be introduced. UFC-3-340-02 (2008) suggests a DIF of 1.19 for the compressive strength of masonry and a DIF of 1.0 for the other properties. The work presented in Chapter 4 of this document and other researches lead to assume that the suggested value in the UFC-3-340-02 for the DIF of masonry could be far from reality. Another set of properties was introduced with a DIF of around 1.7 (DIF1.7 label).

Table 6.10 – Mechanical properties.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Static</th>
<th>UFC-3-340-02</th>
<th>DIF1.7</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_0$ [MPa]</td>
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<td>0.2</td>
<td>0.4</td>
</tr>
<tr>
<td>$\sigma_0$ [MPa]</td>
<td>4.5</td>
<td>5.2</td>
<td>7.5</td>
</tr>
<tr>
<td>$\sigma_{cu}$ [MPa]</td>
<td>6.0</td>
<td>7.0</td>
<td>10.0</td>
</tr>
<tr>
<td>$E_0$ [GPa]</td>
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<td>4.5</td>
<td>7.5</td>
</tr>
<tr>
<td>$\rho$ [kN/m$^3$]</td>
<td>24</td>
<td>24</td>
<td>24</td>
</tr>
</tbody>
</table>

Due to the lack of information regarding the constitution of the pavements, two different situations were considered: a) neglecting the contribution of the pavements, meaning that the masonry panels are only constrained at ground level and at the connections with the other panels; and b) considering a generic pavement assuming perfect connections to the walls, introducing intermediate constrains at the masonry panels. This pavement was assumed as a reinforced concrete slab recent addition, modelled as elastic, with a Young’s Modulus of 30 GPa and a density of 2400 kg/m$^3$. 
2.5.1.2 Blast loading

In order to keep this problem as a pure Lagrangian formulation, the blast loading was defined as pressure profiles. Knowing the position and the weight, in TNT equivalent, it is possible to estimate the pressure profile acting on a specific surface. Chapter 2 provides the interaction between the blast wave and the structure. Figure 6.12 shows the blast loading distribution for Scenario B. Due to the size of the East façade, three zones of loading were defined (L1, L2 and L3) each having different standoff distances (R1, R2 and R3). Regarding the North and South sides as well as the roof, the standoff distance was measured at one meter distance from the edge (Cormie et al, 2009) into the surface itself, and the pressure profile was considered constant throughout all the façade (L4).

![Figure 6.12 – Blast loading distribution.](image)

Knowing the weight of the explosive and the distances for each loading zone, it is possible to plot each pressure profile, using equations 6.7 – 6.10:

\[
P_{50} = 6784 \frac{W}{R^3} + 93 \left( \frac{W}{R^3} \right)^{\frac{1}{2}} 
\]  
(6.7)

\[
P_r = 2P_{50} \left( \frac{7P_0 + 4P_{50}}{7P_0 + 2P_{50}} \right) 
\]  
(6.8)

\[
t_D = 10.23 \frac{W^{\frac{1}{3}}}{\sqrt{P_{50}}} 
\]  
(6.9)
In Chapter 2 there is a more detailed description of the previous equations. As an example, Figure 6.13 shows the pressure profiles for a Scenario B’ situation with 1500 kg @ 25 m. The same procedure was applied to Scenario A, in which eleven different pressure profiles were developed.

As expected, the maximum pressure is highly dependent on the distance to target. For a building with such high dimensions, the effects from both scenarios will be mostly localized. In order to decrease the computational time on the analysis, the whole structure was divided into two parts (Figure 6.14): a) Front section, regarding Scenario B and B’; and b) Side section, regarding Scenario A.

Figure 6.13 – Pressure profiles acting on the building for 1500 kg @ 25 m.

Figure 6.14 – Sections of the building for different scenarios.
6.5.2 Explicit analysis

ABAQUS Explicit was used to solve the non-linear equations of this problem. This software has been used successfully in previous situations regarding similar loading conditions (Cabello, 2011; Jacinto et al, 2001; Heidarpour et al, 2011) and similar materials (Zheng et al, 2010; Al-Gohi et al, 2012). It must be noted that this analysis focuses only on the structural response of the building. Non-structural parts of the building, door frames, glazing systems, or occupants were not taken into consideration in the present analysis.

For this kind of analysis it is necessary to define a damage criterion that can be applied to categorize the damage on the masonry panels. UFC-3-340-02 (2008) classifies the damage to unreinforced masonry walls according to the support rotation (Table 5.6). Other authors (Doherty et al, 2002; Zapata and Weggel, 2008; UFC-3-340-01, 2002) state that collapse would occur if the maximum deflection reaches the wall thickness. Varma et al (1996) reported a 4-level qualitative damage criterion based on observation of the wall. For the present work, the criteria defined by UFC-3-340-02 (2008) will be applied, meaning that the support rotations will be checked in order to categorize the damage on the masonry panels.

Table 6.11 – Unreinforced masonry damage criteria (UFC-3-340-02, 2008).

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<thead>
<tr>
<th>Element</th>
<th>Yield pattern</th>
<th>Maximum support rotation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry Reusable</td>
<td>One-way</td>
<td>0.5°</td>
</tr>
<tr>
<td></td>
<td>Two-way</td>
<td>0.5°</td>
</tr>
<tr>
<td>Masonry Non-reusable</td>
<td>One-way</td>
<td>1.0°</td>
</tr>
<tr>
<td></td>
<td>Two-way</td>
<td>2.0°</td>
</tr>
</tbody>
</table>

2.5.1.3 Scenario A

Scenario A corresponds to an explosion at a square on the South side of the building (Figure 6.8). It is a place with possible high concentration of people due to the presence of outdoor cafes. An explosion with 20 kg TNT at 4 meters from a surface will create a reflected pressure of around 1.5 MPa with duration of 1.4 ms (Figure 6.15). The adopted methodology for applying the load was already described previously and resulted in eleven different pressure profiles applied to the masonry panels according to
the distance from the explosion centre. The reflection angle was considered constant at 90° for all panels. For the present analysis, the initial instant corresponds to the moment when the blast wave first touches the building.

Scenario A was studied with material properties labelled as UFC-3-340-02 (Table 5.2) and considering the contribution of pavements. As it will be shown this scenario represents a low impact loading in the structure, and in order to easily see the results, only part of the structure (the closest part to the explosion), will be presented (Figure 6.16).

![Figure 6.15 – Pressure profiles for scenario A.](image)

![Figure 6.16 – Side section final mesh.](image)
Table 6.12 shows the time histories for the deformation and the maximum principal plastic strains for this part of the building. The panel on the left, which is closest to the explosion, is the first to be loaded. Then the blast wave reaches the panel on the right. At this time, the first panel is already unloaded and it is still moving due to the structure inertial forces.

Although the structure has small displacements, the loading is enough to reach the nonlinear behaviour of the masonry. As given in Table 6.12 there is a concentration of plastic strains on the right side of the panel on the left. It is possible to have in that area some cracking, although it should be negligible.

This level of loading is very low for this structure. The closest panel to the explosion has a maximum displacement of around 2.75 mm, keeping a 1.5 mm permanent displacement after the loading (Figure 6.17a). The analysis of the support rotations (Figure 6.17b) shows that these are still far away from the failure criteria described before. Although only the results from these two panels are shown, the rest of the structure was analysed and, as we move further away from the explosion, the maximum deformation of each panel decreases. In fact, apart from the area described above where there is a concentration of plastic strains, the rest of the structure stay in its elastic regime.

![Figure 6.17 – Time histories: a) displacement; b) rotations at supports.](image)
Table 6.12 – Scenario A: deformed mesh and location of the maximum principal plastic strains.
2.5.1.4 Scenario B

Scenario B corresponds to an explosion at the East façade of the building, at 5 meters from the centre of the façade (Figure 6.8). An explosion with 1500 kg TNT at 5 meters from a surface will create a reflected pressure of around 34.5 MPa with duration of 1.7 ms in the L1 region and around 2.5 MPa in the L2 region (Figure 6.18). The adopted methodology for applying the load was described previously and resulted in four different pressure profiles applied to this building.

![Pressure profile for scenario B in the L1 and L2 regions of the front section.](image)

Figure 6.18 – Pressure profile for scenario B in the L1 and L2 regions of the front section.

Scenario B is a close-range large blast, and it will generate very high strain rates in the masonry, for this reason this scenario was studied with the material properties labelled as DIF1.7 (Table 5.2), which are assumed closer to the actual physical characteristics. Due to the presence of large span masonry panels, both situations regarding the pavements (neglecting and considering its contribution) were considered and the results were compared. Only the first 30 ms of analysis are presented here. Although being possible to capture the complete behaviour of the structure, 30 ms are enough to reach the collapse of the structure considering the damage criteria defined previously.

Table 6.13 shows the evolution of deformation for this model. As can be seen, regions L1 to L3 are loaded in order due to their proximity to the explosion. The global response of the structure changes if we neglect or consider the contribution of
pavements. In the first case, the East facade panel behaves as one large masonry panel being supported at ground level and on its side edges. In the second case, considering the contribution of the pavements, the East façade behaves with intermediate supports along its height, similar to three “independent panels”. Due to the dimensions of these panels (very long) it is almost as if they were only supported at the bottom and at the top.

Table 6.13 – Deformed mesh time history for scenario B: with and without pavements.
The load resulting from this explosion is quite high and the structure response is quite fast. In the first 30 ms the masonry reaches a velocity of around 10 m/s resulting in around 300 mm of maximum displacement in the L1 region after 30 ms (Figure 6.19). The difference, in the maximum displacement for the L1 region after 30 ms, neglecting or considering pavements in the model is around 17%. When we increase the distance from the explosion this difference increases.

![Figure 6.19](image)

Figure 6.19 – Displacement time histories for scenario B: a) neglecting pavements; b) considering pavements.

Analysing the support rotations (Figure 6.20) it is clear that, in both situations, the masonry panel rotates beyond the non-reusable state defined in UFC-3-340-02 (2008). At this point, it was considered that this part of the structure would have collapsed. The contribution of pavements in the model leads to lower values of rotations at ground level (Figure 6.20b). However, the behaviour of the panel in the first floor is closer to one-way yield pattern which lowers the limit to 1.0°.

Table 6.14 shows the evolution of the maximum principal plastic strains. In both cases the collapse would occur close to the boundaries of the L1 region. The supports at ground level sustain high levels of strains in both cases, but considering pavements, the area at the 2nd floor pavement also presents itself with large plastic strains. This is due to the intermediate support originated by that pavement.
Figure 6.20 – Rotations time histories for scenario B: a) neglecting pavements; b) considering pavements.

Table 6.14 – Time history of the location of the maximum principal plastic strains for scenario B: with and without pavements.
2.5.1.5 Scenario B’

Scenario B’ corresponds to an explosion at the East façade of the building, at 25 meters from the centre of the façade (Figure 6.8). This simulates the possibility of closing to traffic the road right in front of this façade and the application of bollards preventing vehicles to get closer to the building. An explosion with 1500 kg TNT at 25 meters from a surface will create a reflected pressure of around 0.45 MPa with duration of 9.5 ms in the L1 region (Figure 6.21). The adopted methodology for applying the load was already described and resulted in four different pressure profiles applied to this building.

In this scenario all three sets of material properties were studied and compared for both, neglecting and considering the pavements contribution. Different analysis times were considered due to the different behaviour of neglecting or considering the contribution of pavements. The analysis neglecting the contribution of pavements had duration of 1000 ms (1 s). Considering the contribution of pavements, 150 ms are enough to capture the complete behaviour of the structure.

Table 6.15 and Table 6.16 show the evolution of deformation and the maximum principal plastic strains, neglecting and considering the contribution of pavements. These results were plotted with the material properties labelled as DIF1.7. The behaviour of the masonry panels is similar to the one observed in Scenario B. Without pavements, the east façade behaves as one large masonry panel supported at ground
level and on its sides. With pavements, it is clear the “independent panel” behaviour at the 3rd floor (Table 6.16).

Figure 6.22 shows the displacement in the L1 region and at 3rd floor. The behaviour for both possibilities regarding pavements is different. In the first case (Figure 6.22a) the maximum displacement at 3rd floor is achieved at around 0.75 seconds after the blast wave reaching the structure while in the second case (Figure 6.22b) the maximum displacement at 3rd floor is reached at around 0.055 s after the arrival of the blast wave.

![Figure 6.22 – Displacement time histories for scenario B: a) neglecting pavements; b) considering pavements.](image)

In both models the support rotations (Figure 6.23) are kept under the Reusable limit established by UFC-3-340-02 (2008). In the model considering the contribution of pavements an additional point was analysed. As can be seen in Table 6.16 the maximum displacement will take place in the 3rd floor, meaning that the maximum rotation is at the 3rd floor level. This last rotation is still under the reusable limit (Figure 6.23b). Although it is not shown here, the rotations at the side edges of the East façade were also analysed and its value are also under safe levels.
Table 6.15 – Scenario B’ neglecting pavements: deformed mesh and location of the maximum principal strains.

<table>
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<th>Time (ms)</th>
<th>Deformed mesh</th>
<th>Location of the max. principal strains</th>
</tr>
</thead>
<tbody>
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<td>t = 0 ms</td>
<td><img src="image1" alt="Deformed mesh" /></td>
<td><img src="image2" alt="Location" /></td>
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<td>t = 200 ms</td>
<td><img src="image1" alt="Deformed mesh" /></td>
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<tr>
<td>t = 400 ms</td>
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<td><img src="image2" alt="Location" /></td>
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</tbody>
</table>
Table 6.16 – Scenario B’ considering pavements: deformed mesh and location of the maximum principal strains.

<table>
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<th>Location of the max. principal strains</th>
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</table>
The distribution of stresses and strains for both models is quite different. Table 6.15 and Table 6.16 show the maximum principal plastic strains for both models. While in the model neglecting the contribution of pavements there is a concentration of plastic strain at ground level, along the horizontal support (Table 6.15), in the other model that is not observed. In the second model, because the maximum deformation occurs at the 3rd floor, there is a concentration of plastic strain at the 3rd floor mid-level (Table 6.16).

In order to evaluate the influence of the suggested material properties, a comparison was made and the results can be seen in Figure 6.24. As expected the maximum displacement is achieved with the static properties. The maximum displacement with DIF1.7, which represents an increase of 70% in the strength and modulus of the material, is around 68% of the static reference. The dynamic increase factor suggested by UFC 3-340-02 (2008) leads to a maximum displacement of around 92% of the static reference. The selection of material properties has a large influence on the structural response. Clearly, the UFC 3-340-02 standard wants to suggest an underestimated dynamic increase factor for materials, although recent researches suggest otherwise.
6.6 **Final Remarks**

A specific risk assessment model for public transport networks was applied to a case study in a Portuguese region and the elements with the highest risk due to external explosions were identified. It was argued that this model is dynamic and highly dependable on the risk assessment team responsible for conducting it.

From the highest risk group, one element was selected for a detailed analysis, Rossio Station. This structure was modelled using explicit non-linear dynamics and the results were presented for different explosion scenarios. It was shown that a small package explosion would have a small impact on the structure while a large package explosion would lead to the collapse of the structure. Increasing the standoff distance, as a measure for mitigating the impact of the explosion, was analysed and proven to be an effective measure.

Considering or neglecting the contribution of the pavements affects the behaviour of the structure and assessing the real conditions of the pavements and their connections to the wall panels is important in order to have quality results. A comparison was also made for different material properties. Selecting the material properties also has an important role, due to the large impact on the final results. If possible, in situ assessment of
material properties should take place to properly grasp the condition of existing buildings. Recent research, including the one in this work, suggests that the available codes could be outdated regarding the dynamic increase factor for material properties.

6.7 REFERENCES


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UFC 3-340-01, *Design and analysis of hardened structures to conventional weapons effects (FOUO)*. Department of Defence, USA 2002.


Chapter 7

7 CONCLUSIONS

This thesis studied several topics related to blast loading and the response of structures under such impulsive loading. To do so, a literature review was performed on the explosion phenomenon and its interaction with structures, for both terrorist actions and accidental explosions. The main issues regarding explosions with High Explosives (HE) and Vapour Cloud Explosions (VCE) were addressed and the available methodologies to assess the blast loading parameters from these two different types of explosions were presented. In addition, the available methods for analysing structures under blast loading were presented and research on the material behaviour under high strain rates was performed. This developed background supported the research work performed. The main achievements found in each chapter are summarized below.
7.1 SYNTHESIS AND RESULTS

In Chapter 3 a new methodology for predicting the blast loading parameters through pressure indicators was provided. This methodology was applied to lightweight switch boxes and Pressure-Impulse (P-I) diagrams were presented. These tools, developed using ANSYS/LS-DYNA, can help assessing the magnitude of overpressures and impulses in the event of accidental explosions, by backtracking the loading characteristics from the damage observed in lightweight switch boxes under similar loading conditions. Taking Buncefield Major Incident as a case study, sub-structures were adopted to explain the severity of the explosion and the magnitude of the experienced overpressures. The obtained results showed a good agreement with the empirical and numerical solutions for estimating the blast loading parameters. The lack of powerful material models and element types available in explicit dynamics analysis was also highlighted.

In Chapter 4 a large experimental campaign was performed on different loading regimes and different materials. Masonry specimens and masonry components, clay brick and mortar, were tested under quasi-static regime – strain rate of $10^{-5}$ s$^{-1}$ – and dynamic regime with strain rates ranging from 2 s$^{-1}$ up to 200 s$^{-1}$. Almost 250 impact tests allowed finding that most mechanical properties under compression of these materials increase with the strain rate, having Dynamic Increase Factors (DIFs) ranging from 2 to 6 for a strain rate of 200 s$^{-1}$. It was found that the strain at peak strength can be considered constant and unchanged with the strain rate. For the compressive strength and the Young’s modulus, both masonry and clay brick presented similar behaviour while the mortar specimens were more sensitive to the strain rate. The presence of fibres in the composition of the mortar adopted in the tests could help explain this difference. For the compressive fracture energy, both masonry and mortar presented similar results, while the handmade clay brick specimens, probably due to their high porosity, presented a much higher increase of fracture energy with the strain rate. Lastly, empirical relations of Dynamic Increase Factors (DIF) for each property and material were developed and presented.

In Chapter 5 a newly developed test setup for dynamic out-of-plane testing on walls was presented, including the developed sensors and acquisition apparatus. Advantages such
as, having a surface area distribution, avoiding the generation of high fragments and reducing the atmospheric sound wave, lead to the adoption of underwater blast wave generators (WBWG) as opposed to the traditional air blast. The results obtained from the test on one unreinforced masonry infill panel allowed calibrating numerical models using ABAQUS Explicit software. Parametric studies showed that there is a point where the increase of the compressive and tensile strength is no longer effective (as the response becomes elastic), and the Young’s modulus and the wall thickness are the parameters with the higher influence on the behaviour of the wall panel. Although there was no experimental test performed on reinforced masonry infill panels, two different (low percentage) reinforcement solutions were studied with the numerical models. Finally, these results were used to create empirical tools – Pressure-Impulse diagrams – which can help the designer to estimate the response of the element under different loading conditions. It was shown that the use of these (low percentage) reinforcement solutions for crack control purposes is more effective considering the non-reusable stage of the element. If the requirement is the reusable stage there is no real advantage in the use of these (low percentage) reinforcement solutions, and the best way to improve the response of the wall would be increasing its thickness or designing reinforcement according to the sought performance level.

In Chapter 6 a specific risk assessment model for public transportation networks was applied to a case study in a Portuguese region and the elements of the infrastructure with the highest risk due to external explosions were identified. Risk assessment methodologies proved to be highly dependent on the assessment team which should be composed of professionals with different expertise. One of the infrastructure elements identified previously was selected for a detailed structural safety analysis using non-linear dynamics available in ABAQUS Explicit. Different external explosion scenarios were studied and the behaviour of the structure under blast loading was presented. It was shown that increasing the standoff distance for an explosion is always a good measure to improve the structural response, more so in the case of historical buildings, where the possibility of structural retrofitting is more limited. The importance of proper material characterization under dynamic loading was also highlighted.
The work presented in this thesis is composed of experimental and numerical campaigns, which gave a contribution for better understanding the effects of impact and blast loading on civil engineering structures and materials.

7.2 FUTURE DEVELOPMENTS

In the scope of material behaviour under dynamic loading some aspects that deserve further attention are highlighted:

- The empirical relations obtained for the material properties under different strain rates can and should be used to improve the available material models for numerical analysis. Regarding this aspect, different researchers have been implementing strain rate dependent non-linear material models in explicit dynamic structural analysis software.
- The evaluation of the effect of the strain rates for the selected materials in this thesis under tension is needed. The DIF characteristics of these materials under tension are likely to be different from the behaviour under compression. This study requires changes in the adopted test set-up or the use of different testing equipment, such as a large modified Split-Hopkinson pressure bar.
- The evaluation of the effect of the strain rates on strengthening materials for masonry walls. Regarding this aspect a new drop-weight tower was designed and built (Figure 7.1) to study the influence of the strain rates on the bond behaviour of FRP sheets and clay brick.

![Figure 7.1 – New drop-weight tower for pull-out dynamic testing: a) schematics; b) building process.](image)
In the scope of structural behaviour under dynamic loading some aspects that deserve further attention are highlighted:

- The results obtained numerically for the reinforced masonry infill panels should be validated experimentally with the developed test setup. Additional wall panels for testing are part of a current research project at University of Minho.
- Additional methodologies for modelling these masonry infill panels, such as micro-modelling, should be investigated.
- Strain rate dependent non-linear material models should be tested and validated with the experimental results from these masonry panels, for different impacts.

In the scope of risk assessment and structural safety evaluation some aspects that deserve further attention are highlighted:

- For increasing the value of the developed risk matrix, additional detailing of the transportation network and related infrastructure should be provided and the incorporation of knowledge from additional sources, such as national crisis management agencies and police is advised.
- Additional geometric and mechanical detailing of the structures is required for a more reliable safety assessment.
- The use of different modelling techniques, such as Discrete Element (DE) or Applied Element (AE) should also be investigated. These techniques allow for proper visualization of the crack propagation and progressive collapse, and their performance can be compared with explicit finite elements.
Annexes

ANNEXES
ANNEX A.1

ANNEX A.1 – Complete list of impact tests on clay brick

<table>
<thead>
<tr>
<th>Strain rate (s⁻¹)</th>
<th>Compressive Strength</th>
<th>Young’s Modulus</th>
<th>Strain at Peak Strength</th>
<th>Fracture Energy</th>
<th>Specimen (#)</th>
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### ANNEX A.2

**ANNEX A.2 – Complete list of impact tests on mortar**

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ANNEX B.1

Annex B.1.1 – Schematic of the test setup.

Label:
1 - RC frame
2 - Wall specimen
3 - IPE 300
4 - HEA 180
5 - L 200x200x18
Annex B.1.2 – Schematic of the test setup.

Annex B.1.3 – Schematic of the test setup.
Annex B.1.4 – Steel support structure [mm].

Annex B.1.5 – Steel support structure [mm].
Annex B.1.6 – Steel support structure, details [mm].
Annex B.2.1 – Masonry construction details [mm].
DATA SHEET

Murfor® RND
FOR MASONRY WITH MORTAR JOINTS

POSITION OF Murfor™ RND IN THE MORTAR JOINTS

GEOMETRY

COATING

Epoxy  Zn  Fe  ASI 302  Zn  Fe

Width (mm)  Diameter (mm)  Diameter (mm)
30  3.4  5.5  3.5  7.5
30  3.4  5.5  3.5  7.5
30  3.4  5.5  3.5  7.5
30  3.4  5.5  3.5  7.5
300  3  3.7

RND/E (ISO 1449)
RND/S (AS 1400)
RND/Z (C3)

PACKAGING

25 pieces per bundle (75 m)
40 bundles per pallet (3050 m)

STORAGE

KEEP DRY
RIGHT
WRONG

ISO  CE  KOMO  ATG

APPROVALS

CE LABEL: Murfor® is CE marked according to EN 845-3: masonry bed joint reinforcement. For detailed info, “EC declaration of conformity” available on request.

OTHER APPROVALS: Bekaert is close to the market. Bekaert seeks conformance to every necessary national product quality standard. In case one or more requirements are not covered by the CE or ISO 14001 certifications, other Murfor® approvals are Zulassung DIB, Vauenetraum and ITB.

Bekaert will advise on the most suitable Murfor® type for your application. Ask for our recommendations on type, amount and positioning at:
www.bekaert.com/building
info.building@bekaert.com

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All details concerning our products are general information. For detailed information, product specifications are available on request. © 2006 Bekaert

Annex B.3.1 – MURFOR RND Datasheet.
Annex B.3.2 – ARMANET Datasheet.
ANNEX C.1

ANNEX C.1.1 – Full list of key infrastructure scores.

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ANNEX C.1.1 – Full list of key infrastructure scores (cont.).

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