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Building Cladding Subject to Explosive Blast: A Study of Its Resistance and Survivability, with Particular Reference to Architectural Aspects and Multi-Panel Glazing Systems

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ABSTRACT

Cladding, especially glazing with large pane sizes in the form of curtain walling, is a popular architectural feature in modern city centres. Unfortunately, the present day terrorist realised the potential for causing not only loss of life but also significant economic loss. Governments and others cannot justify spending large sums on buildings to protect them against all likely events, even though an attack could cause considerable personal injury both inside and outside buildings. The air blast phenomenon is examined and its impact on structures and structural elements. Architectural forms are reviewed and descriptions are given of different cladding systems. The damage tolerance of different cladding materials is examined and compared. Glass cladding is selected for particular examination because of its wide spread use in modern architecture and the particular threat that it imposes upon people and building systems. If we can take measures that would reduce casualties for minimal cost and even enhance the appearance of a building, this would clearly be worthwhile. One option is to consider reducing the size of unsupported glazing with respect to the overall architectural aspect, while balancing the visual impact and attractiveness. Glass is a fragile material and, when exposed to blast, the broken glass can injure people both inside and outside the building. Glass curtain walls are normally constructed of large glass panes supported by a structural system consisting of horizontal and vertical mullions which transfer their own weight and imposed loading via fixings to the main structure. The widespread popularity of glass is discussed and analysis is performed to demonstrate that reducing the unsupported area of glazing and using high length to breadth ratios increases its resistance to blast loading. Based on these principles, architectural design concepts are presented that can both improve the resistance to fracture of the glass and have a visual impact. This thesis considers ways to improve the facade behaviour when exposed to the positive and negative pulse exerted by explosive blast wave. Finite element analysis is used to compare a typical current design of glass curtain wall with new concepts which can improve the resistance of the glass to blast loading. The concepts use smaller panes of glass supported by their own mullion system.

ACKNOWLEDGEMENTS

Acknowledgement is made to the Royal Military College of Science, Cranfield University, Engineering Systems Department in Shrivenham and to the highly outstanding staff members who devoted much of their precious time to me and to the current research. The recent terrorist attacks against the United States of America's embassies in Nairobi and Dar-Elsalam highlights the need for this research, as an appropriate shield against their despicable attacks that killed and injured so many innocent people.

I wish to express my deep gratitude to Mr. **L.J.Kennedy**-Supervisor of the current research - for his kind collaboration, active support, continuous encouragement over the last three and half years, his valuable advice during the preparation of this thesis, including the critical review of the manuscript, and all his persistent efforts to make the end of this thesis possible. It was a real privilege to work under his supervision.

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Chapter 1

Introduction

1.1 Background

Blast impact on structures and their response to blast loading are major fields of interest within the Engineering Systems Department of Cranfield University. **"Building Cladding subject to Explosive Blast: a study of its resistance and survivability, with particular reference to architectural aspects and multi-panel glazing systems"** was selected as a PhD. project to bring an architectural view-point to the department's area of interest.

Early meetings with various experts helped to focus on the area needing study. In July 1995, at a meeting with Mr. C. Veale of the UK Ministry of Defence, the importance of studying the impact of blast upon building cladding systems was discussed. Mr. Veale explained that much work had been done on glazing but none had been done on cladding. He also mentioned some important considerations about cladding surface design and how curved and angled surfaces affect response to blast loading.

In August 1995, a meeting was held with Mr. I. Thirlwall and Mr. P.Jackson of R. T. James and Partners Consulting Engineers. They agreed with Mr.Veale that no consideration was given in the early stages of building design to the behaviour of cladding that might be exposed to blast loading. They talked about their experience of the National Westminster Tower project. They mentioned how an explosion in 1992 affected most of the cladding and the curtain walls within a radius of 200 metres from the explosion. The National Westminster Tower was within that radius.

All the precast concrete cladding panels used had to be changed completely due to damage to the cladding panels on the lower floors. They added that due to the method of fixing the complete height run of the panels had to be removed. They also talked about anti-shatter film and laminated glass as solutions for protecting traditional windows and curtain walls against blast. However, they felt that there was still a good reason for examining methods to improve the blast resistance of glazing.

...mb attacks have been increasingly directed against commercial, financial and ...ntres consisting of buildings of conventional soft constructions was the subject of ... presentation of Mr. M. Farlong from the Security Team of the Union Bank of ...land in our meeting at the bank in London in October 1995. In November 1996 ... gave the same presentation at the Institution of Civil Engineers. He talked about his ... experience of terrorist attack and the use of cladding in the protection of the ...ding.

... noted that the scale of terrorist attacks has increased in recent years. The most ... objective of terrorists is to cause shock, fear and outrage in the minds of the ... victims by the deliberate use of violence. Modern buildings, public and corporate, ... been built and are still being built with little concern for the destructive effects of ... attacks, and as a result they offer the occupants, and contents of buildings, too ... protection.

...number of events and casualties we have heard about due to blast loading effects on ... buildings is catastrophic. In his discussion at the Institution of Civil Engineers in ... 1996, Mr. Roy Kinnear of Sandberg and Partners noted that blast loading causes ... damage to the buildings within its radius of effect. Penetrating the cladding, the ... disrupts the structure and reduces the glazing to sharp fragments, flying in all ...tions. He also added that the force exerted by the blast loading of an explosion ...ed not only the cladding panels but also their fixings. Fixings must be easy to ... so they should not be inaccessible.

...2 Influences

... of commerce and finance are now being built in capital cities already rich in ...ectural character and features. If one had to design such buildings, one would think ... the concept that makes the building unique. Curtain walling is one of the concepts the ...er selects to provide his building with either harmony or contrast with the ...ding buildings.

... 4 explores the architectural aspects of the different forms of cladding as an ... skin to the building. Cladding provides it with texture and acts as the wedding which make the building aesthetically attractive to both occupants and visitors. The materials of the cladding are either heavy, like brick, stone and precast concrete or light, such as stainless steel, aluminium and glass reinforced polyester (GRP). In between, there is glass which is neither heavy nor light.

However, glass makes the building transparent and provides it with light. This transparency gives the material the advantage that it can be combined with any other material. There is no substitute for glass, since no normal building is without windows. Using glass together with solid materials, forms a relationship between the solid and the opaque.

The areas of glass surface and the rhythm of repetition within the surface area of an elevation are relevant to the form order of the designer's concept. Providing the building with a degree of protection is limited by economic factors. It would be very expensive and illogical in the case of commercial buildings to protect them against an event which has a low risk factor.

Also, what would a commercial building look like when it was fully protected? One simply does not know how the terrorist will think, and what or where his next target will be. We have to consider both the use of the building cladding as a means of protection to the whole structure, and the enhancement of the cladding receptivity against blast.

The use of cladding as a primary method of protection means that the cladding system should be designed as an armoured wall, integrated with the structure to behave together as one unit. When exposed to blast loading it should provide the building with adequate means of protection against a high risk threat.

Alternatively, consideration must be given to a low risk building, giving the cladding a degree of resistance against blast loading, which has a low chance of taking place. It was stated by Smith and Hethrington, 1994 that designing a structure to have a significant resistance to blast loading will add from 2% - 3% to the overall project costs. Also, one must take into account the aesthetic requirements.

There should be a balance between giving the building cladding a limited blast resilient stiffness and the cladding's visual impact. Forming ordered patterns of small areas of

... using mullions, or strapped glass rather than big panels, could be aesthetically ... rather than detrimental to the environment.

... The Threat

The threat discussed in this research is the catastrophic event which results from a high ... charge detonated by terrorists in order to evoke violence. We have to decide ... threat we are going to deal with. We have to decide whether it is a far field or a ... field threat. Then we can consider the charge we expect to explode and its distance ... the building. The various fields can be described as follows:

... Near field

A near field blast is one which occurs very close to the structure and in which, Z (the ... distance from the charge centre in m/kg$^{1/3}$), is given by:

$Z = R/(W)^{1/3}$ where $0.05 < Z < 0.3$ (0.05 is the surface of the charge)

R is the distance from the charge centre in metres.

W is the charge mass expressed in kilograms of TNT.

... example of a near field blast effect is the damage caused to the Alfred P Murrah ... in Oklahoma City in April 1995.

... Medium field

The medium field is defined as that within the scaled range as follows:

$$0.3 < Z < 1$$

... Far field

The far field is defined as follow:

$$1 < Z < 10$$

When Z exceeds 10 the overpressure is generally less than 0.1 bar (= 10 kPa)

It is assumed that the building lies in the far field when the range is 50m or more and the charge is between 50 kg. and 500 kg. of TNT.

1.4 The Aim of the Research

The main objective of this thesis is to consider modern building cladding materials and systems and their potential resistance to blast effects. The study led to the consideration of the following aspects.

- Far field effects, where main structural elements would survive but weaker claddings, like glass, would cause serious injury and damage.
- A safe collapse concept for cladding, accepting that economic and aesthetic considerations necessitate less than complete protection.
- The performance of glazing due to its relative weakness when compared with other cladding materials.

1.5 The requirements of cladding

Cladding has various general requirements to achieve its function:

a. Visual impact.

b. The effect of the panel's own weight on the cladding material selection due to its impact on the following:
   - Building height, which will be limited if the panel is too heavy.
   - When using heavy materials in high rise buildings, the cost will increase in comparison to light materials. This could be due to higher capacity foundations i.e pile foundations.

c. The materials must be sufficiently durable to withstand the different weather conditions and the effects of pollution.

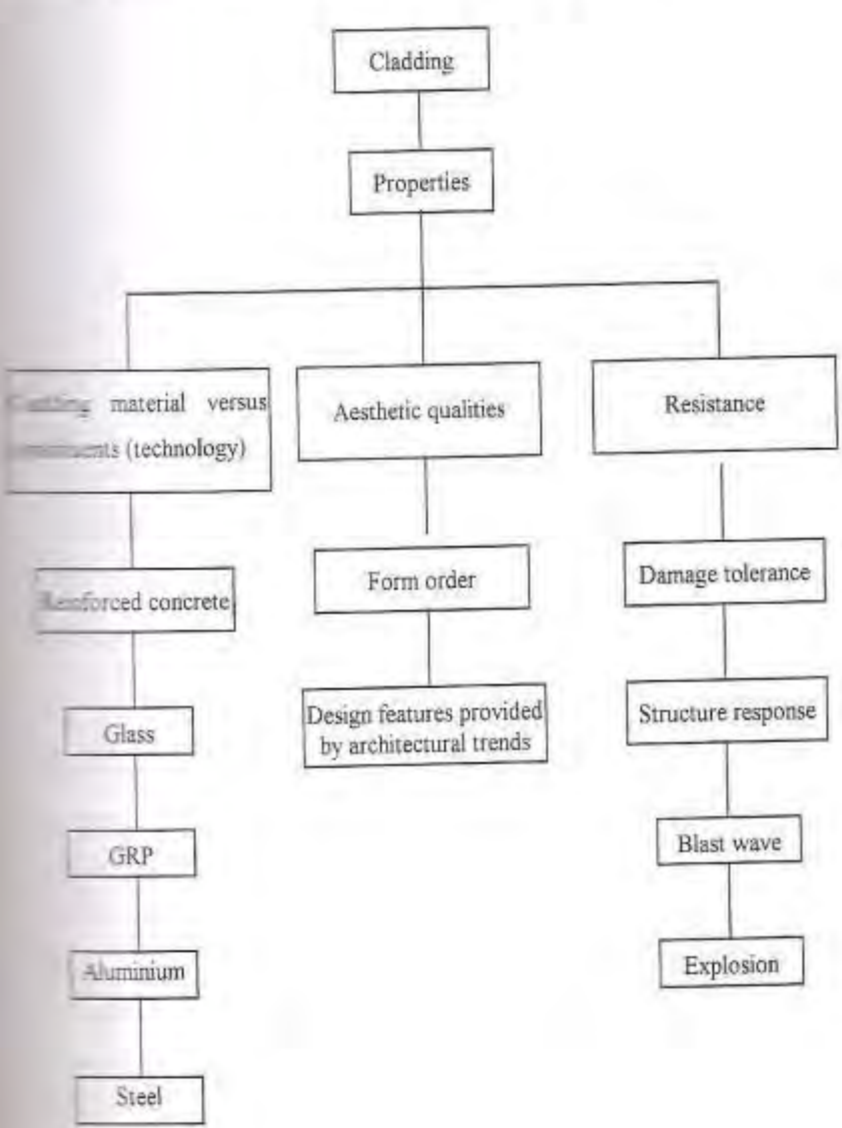
d. The panels must be impermeable to rain and have controllable balance between both heat loss and heat gain.

When considering the additional effects of blast, cladding must have sufficient resistance to damage and be easily repaired after an attack.

1.6 Research methodology

In order to achieve the above goals, and to balance the cladding resistance and its aesthetic qualities, the research will use initially three case histories to illustrate the results of terrorist attacks on city centres. Cladding as a wrapping element for the interior and exterior of buildings is vulnerable to the effects of blast. This is especially true for the glazing where there is often increased injury to people and damage to services as well as air conditioning. The case histories demonstrate the value of investigating the properties of cladding.

The following scheme gives a better understanding of the scope of cladding.



1.7 The scope of the research

The research is intended to cover the following in nine chapters

- *Chapter 1*

  Introduction

- *Chapter 2*

  Case histories.

The case histories of chapter 2 show how the fragile nature of glass makes it dangerous when it is exposed to explosive blast. It always turns into fragments and spreads all over the area of explosion threatening people inside and outside the building.

- *Chapter 3*

  The blast wave mechanism and the structure response.

  The blast wave behaviour and its interaction with structures is discussed in this chapter.

- *Chapter 4*

  The influence of architectural form order on protection from explosive blast

  In this chapter, the architecture and aspects of different cladding forms are explored in terms of form order.

- *Chapter 5*

  Description of cladding classification, curtain walling systems and their constituents, and their behaviour when exposed to explosive blast.

The different cladding materials and their constituents are discussed in this chapter concentrating on the characteristics that are highly affected by explosions, such as the method of fixing the panel and its geometry. Moreover, the secure failure concept of cladding is briefly introduced.

- *Chapter 6*

  Damage tolerance of different cladding panels.

The impact of blast upon specific types of cladding materials is discussed in this chapter. The cladding resistance is represented by Iso-damage curve.

- *Chapter 7*

  The influence of architectural design and features on the glazing overall resistance.

  This chapter focuses on the prime importance of glass as a cladding material possessing unique properties.

- *Chapter 8*

  The response of glazing to explosive blast.

  In this chapter a numerical technique is presented for the behaviour of glass under applied equivalent static load, together with an examination of the glazing response to statically distributed applied load conducted on typical samples of glass

- *Chapter 9*

  Discussion, Conclusions and Recommendations for further Studies.

  This chapter discusses the main conclusions from the research and the recommended required treatments to enhance cladding resistance while ensuring that aesthetic qualities are not sacrified. Additionally, this chapter includes the main points that are recommended to be thoroughly investigated in separate future research.

Chapter 2

Case Histories

CHAPTER 2

Case Histories

[illegible]

[illegible] defined as the planned use of violence in order to create a feeling of [illegible] to the public, with the purpose of imposing a political or religious [illegible] terrorists have failed to achieve through legitimate channels of [illegible]. Terrorists have increased their effectiveness by attacking different targets [illegible] life of people will be disrupted. Targets have included railroads, [illegible] centres.

[illegible] decades, the number of terrorist operations against individuals and [illegible] has escalated. The methods and tools of these operations have been [illegible] increasing the losses to property and individuals. Famous operations on an [illegible] scale were: the bombing of St Mary Axe in London, in April, 1992, the [illegible] the premises of the Egyptian Embassy in Pakistan by means of a luxury [illegible] packed with explosives, the leakage of poisonous gases into the Tokyo [illegible], the taking of hostages for about four months in the residence of the [illegible] ambassador in Lima and the destruction of the American headquarters in Al-[illegible] Saudi Arabia.

[illegible] has several methods of operation, the most important of which is the [illegible] explosive packages of different types and sizes, with the objective of [illegible] establishments. The effects of these explosions vary from destroying [illegible] glass to the full collapse of an establishment, such as happened in the attack on [illegible] Marine base in Beirut in 1983.

[illegible] engineers were assigned to examining how to protect establishments and [illegible] were made to confront this new type of danger. The expertise of military [illegible] was applied in attempts to secure civil sector buildings. Cranfield University [illegible] connections was a pioneer in this domain, *Elliot C., 1989.*

[illegible] of the results of attacks on establishments found that, in addition to damage [illegible] elements, there have been dramatic increases in human losses inside and outside establishments. These are the result of the damage of exterior walls and the widespread scattering of fragments. These effects have been accentuated through the use of materials of low resistance to explosive wave pressure such as glass.

In pre-constructed surface units, it has been noted that the resistance and construction nature of panels are so weak that it is necessary to undo the whole surface to change the defective panels, *Thirlwall L., Kinnear R., 1995.*

The increasing threat provided the momentum for Cranfield University and its staff to study the effects of explosions on the external surfaces of buildings. This was seen as an addition to their on-going research into the general effects of explosions on buildings. This present research is an attempt to extend this area of interest and, in addition, to integrate the visual and aesthetic elements of the building into any considerations for protection

To set the background for this research, it was worth summarising three of the most severe terrorist explosions to have occurred in the United Kingdom during the last six years. These caused tremendous commercial disruption and the two bombs in London threatened its position as Europe's leading financial centre. The case histories include details of charge weights and damage to cladding and glazing of buildings in the vicinity of the detonation.

2.2 Case history 1

<u>London's financial district bombing</u>

On Saturday 24th April 1993 at 1027 hours at Bishopsgate in the City of London, an explosion devastated the area shown in Figure 2.1. One ton of TNT equivalent was hidden in a parked dumper truck outside the Hong Kong & Shanghai Bank. The explosives were packed in boxes under a tarpaulin. The Irish Republican Army (IRA) gave a telephone warning at 9.30 a.m., indicating that an explosion would occur.
During the week, this area has several thousand banking and office workers; for example the Natwest Tower alone normally has 2500 workers during the week. But on Saturday the area is less crowded than during weekdays, though there were a number of shoppers on the streets. People were evacuated from the area and the police cordoned it off.

Figure 2.2 This view was taken of the area adjacent to the blast (*Times, 25.4.1993*) looking along Bishopgate. The severe structural damage in the near field is shown. Cladding strengthening would clearly bring no benefit close to the blast site.

2. *Pertinent details of damage*

Despite police precautions, one man was killed in the street and 45 were wounded; mainly from falling glass up to a mile from the charge.

The overall damage was estimated at £1.5 to £2 billion. This was covered by insurance for the cost of building repairs, glass and cladding replacement. In addition, there was an estimated £8 billion worth of business losses and consequent losses were not covered by the insurance companies.

As a result, many companies (estimated 20%) closed down. Windows at the European Bank for Reconstruction almost one mile away were broken. All the glass was blown out in the eastern side of the 52 storey Natwest Bank Tower after (*Times, 25.4.1993*)

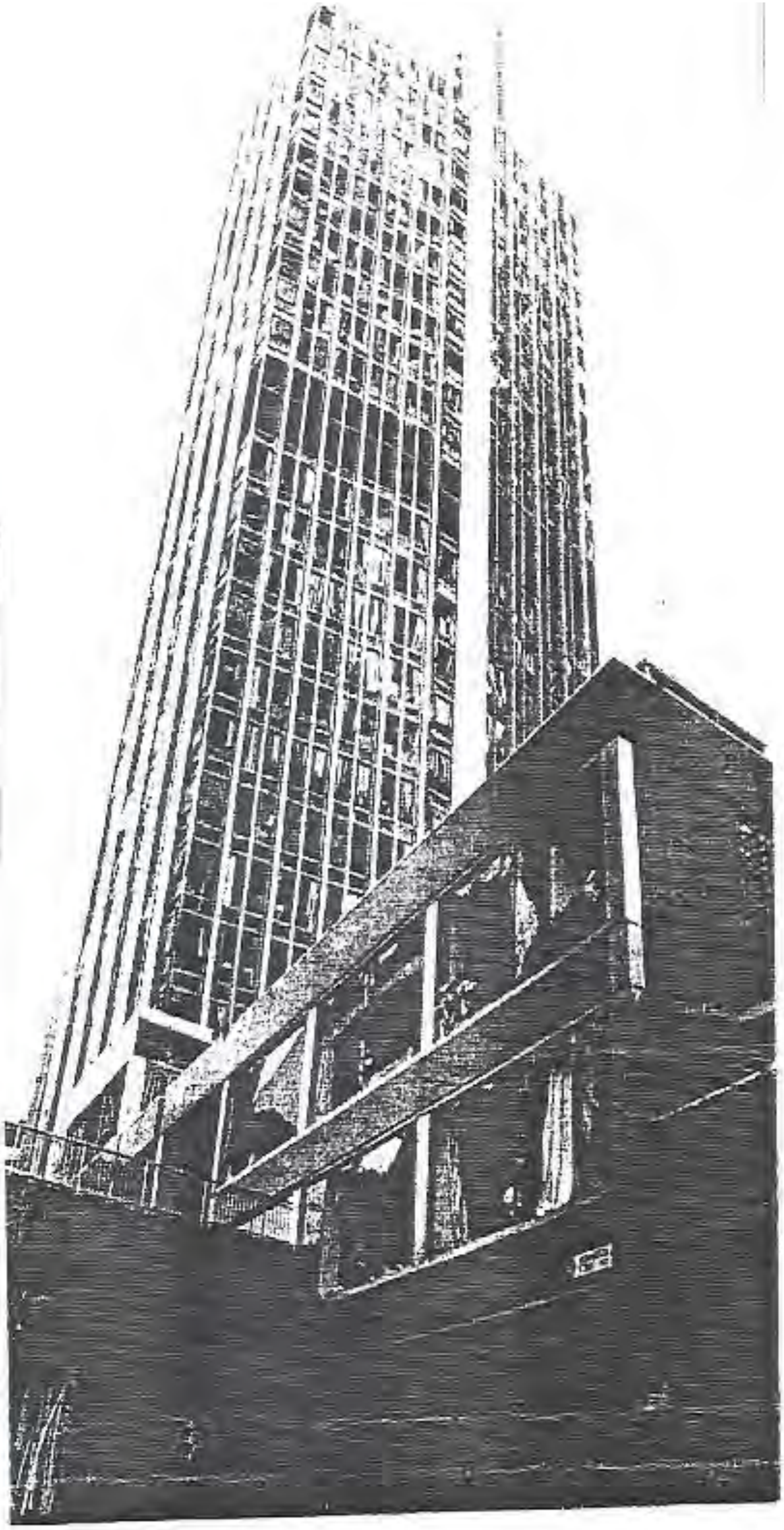
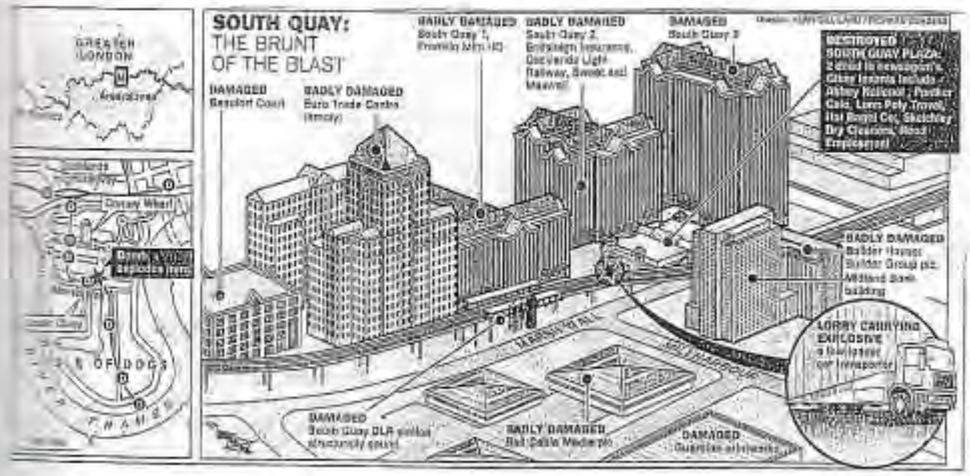


Figure 2.3 Double blow. Hong Kong & Shanghai Bank has been hit twice, *(Times, 27.4.1993).*

South Quay Docklands bomb

the 9th of February 1996 at 1830 hours a bomb exploded at South Quay Street in London Docklands in the area , shown in Figure 2.4.



2.4 A map and isometric diagram for the Docklands site showing where the bomb exploded, *(Daily Telegraph, 10.2.1996)*.

home made explosive of about 500 kg of equivalent TNT exploded. The home made explosive (HME) is classified as a high explosive, depending on the mix used. It is usually more powerful than commercial dynamite and is about 75% of the power of TNT. When detonated on the open ground it would produce a hemispherical blast wave. The bomb detonation released gas pressure up to 300 kilobars at a temperature of up to 4000°C. The blast wave consists of a layer of compressed air, resulting from the violent expansion of this gas. Security experts believed that the low loader used was to reduce the risk of detection. There was no warning but at 1800 hours the IRA announced that their cease-fire had ended. There was no time to take any precautionary action.

Blast results

devastation was caused because, even after the IRA's 1992 and 1993 massive bomb attacks on St Mary Axe and Bishopsgate, few buildings in Docklands had their blast resistance increased. Institution of Civil Engineers (ICE) vice-president, Millington, said that the experience gained from blast-resistant design in Northern Ireland had been made available via seminars and books, *(New Civil* eer, 15.2.1996). But he did not remember any specific requests for design help the Docklands was being re-developed following the IRA's attacks. The best ition was to keep potential bombs well away from buildings. Windows should plastic film on them and be mounted in concealed frames. He said it was not mic to design a complete building so that it could not be damaged. A security t's comment about the Dockland upgrading was that "It was cheap and cheerful". the IRA cease-fire in 1994, Loss Prevention Council Development Director Roy g said:

e was a perception that the threat was disappearing and extra security just 't worth the candle. It's very difficult to improve blast resistance significantly ut excessive costs", (*New Civil Engineer, 15.2.1996*). Due to the blast, 2 people killed and hundreds were wounded by flying debris as the blast swept through the ings. The total losses of up to £150 million were paid out by the insurance anies for the cost of repairs to the damaged buildings.

e 2.5 The glass curtain walls severely damaged due to blast fragments everywhere. *(New Civil Engineer, 15.2.1996)*

Figure 2.6 Most of the cladding materials within the bomb site need to be changed, ((New Civil Engineer, 15.2.1996).

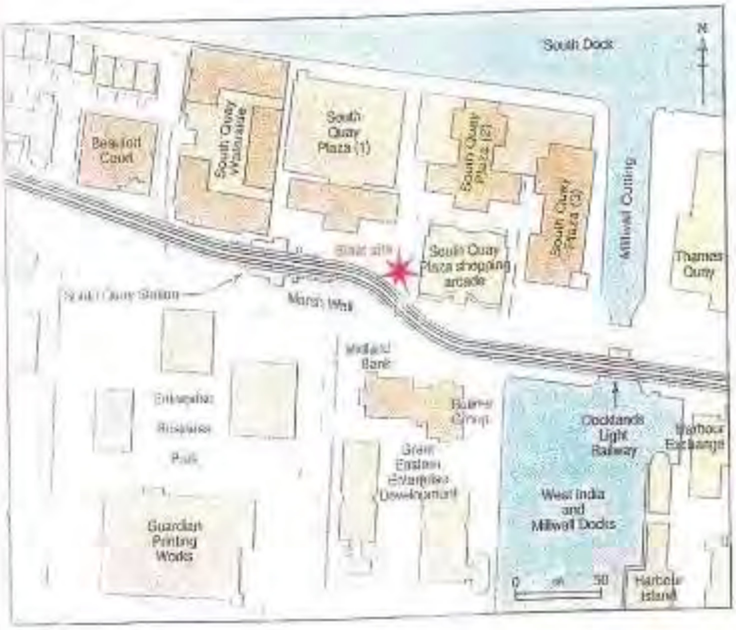


Figure 2.7 General layout and general description of the building losses within the bomb range. (*New Civil Engineer, 15.2.1996*).

South Quay Waterside.

Three 19 storey blocks and a 24 storey tower. Steel framed main structure with composite deck floors sitting on RC framed basement. Clad in granite and tinted glass with no protective film. Most glass lost but cladding appears solid and undamaged.

South Quay Plaza 1, 2, 3 Arcade.

Shopping arcade and three office blocks building framed with pitched roofs and mirrored glass and granite cladding SQP1 is seven storeys.

SQP2 and SQP3.

SQP2 is 14 storeys. Extensive damage to cladding of SQP1, with granite slabs stripped from backing frames at corners of building. Roof damaged. Virtually all glass lost and east side of building stripped of its frame. Almost all glass lost from facades facing explosion at SQP2 and SQP3. Much lost from other sides.

Thames Quay

Consists of three interconnecting blocks with stepped facades. Windows have plastic film coating. Cladding panels dimpled by explosion.

Great Eastern Enterprise.

The two steel framed buildings are structurally separated by 100 mm but joined cosmetically. Clad with aluminium panels and semi-reflective film. Total destruction of front and west facade. Some structural damage appears likely. Surprisingly little damage to side remote from blast.

2.4 Case history 3

Manchester bomb 15th. June 1996

The explosion occurred on Saturday 15th. June 1996 at 1040 hours in Manchester City Centre. The area is shown in Figure 2.8

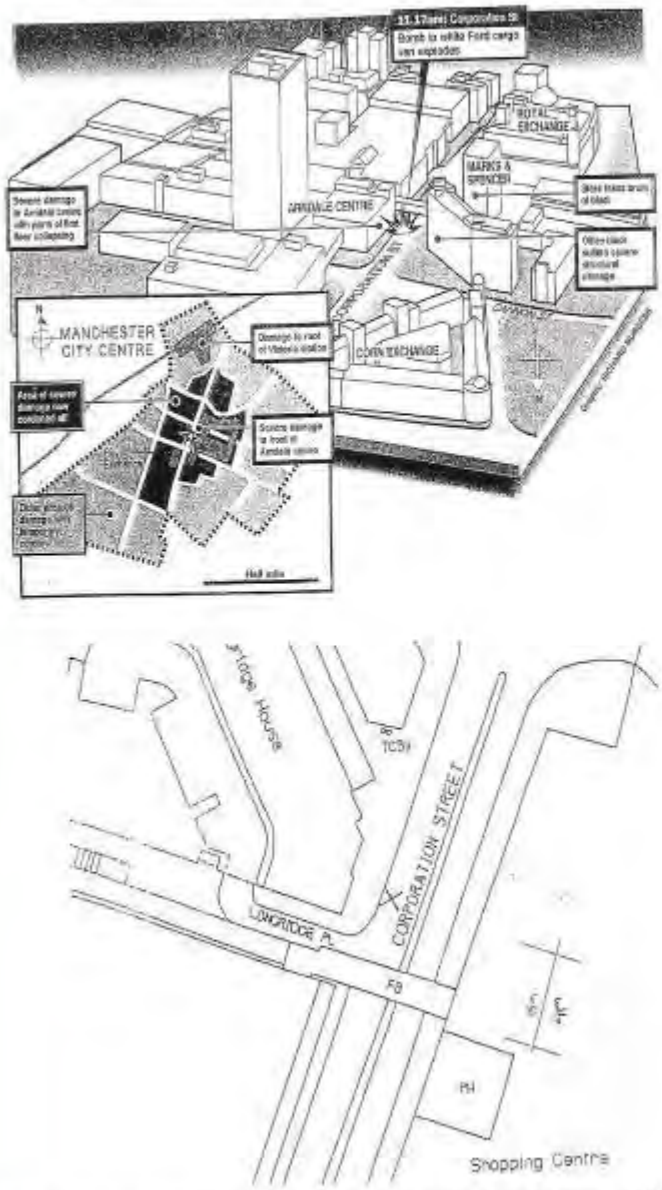


Figure 2.8 A map and isometric diagram of the site of the bomb, *(Daily Telegraph, 16.6.1996)*.

A terrorist attack was made by the IRA. They mined a van with home made explosives, with about 500 kg TNT equivalent charge. There was a warning. The police emptied all the shops immediately and made a cordon around the whole town centre.

At about 1000 hours a van displaying a parking ticket was parked in a double yellow lined area. Between 1020 and 1030 there were some attempts to stop the detonation of the bomb but these failed, *Daily Telegraph, 16.6.1996.*

*1. **Blast results***

Due to the blast two people were killed. The rebuilding of the damaged properties is expected to cost the insurance companies £300 million.

The van was parked near a pedestrian bridge. The blast lifted the bridge upwards and moved it from its place. Also, Figure 2.9 shows how the blast caused severe damage to the Marks and Spencer's building structure and its cladding system.

Figure 2.9 The scene of devastation after the explosion, 200 people were hurt in the blast, *(Daily Telegraph, 15.6.1996).*

2. Pertinent details of damage to cladding and glazing damage:

The blast caused damage to window glass up to a radius of 1/2 mile. The damage is indicated by views shown in Figures 2.10, 2.11.

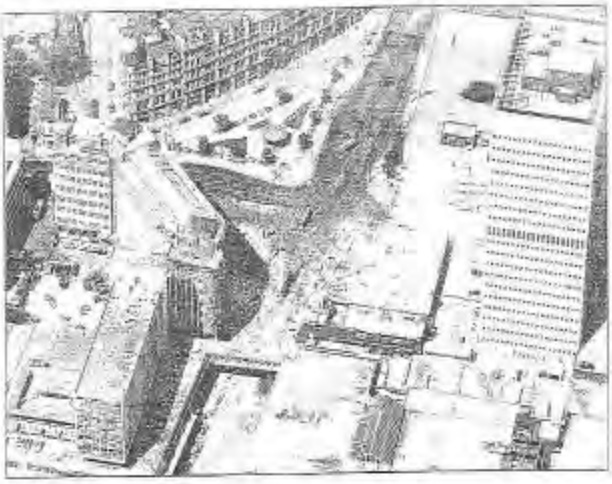


Figure 2.10 A view from the air of Manchester centre after being wrecked by the force of the bomb. *(Daily Telegraph, 15.6.1996).*

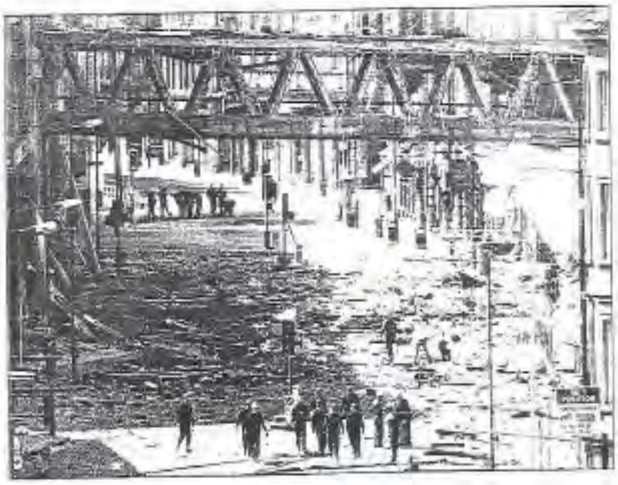


Figure 2.11 Devastation in Corporation Street, with the Arndale Centre on the left and Marks & Spencer to the right. *(Daily Telegraph, 15.6.1996).*

3. **Bomb damage recovery report**

In the New Civil Engineer, dated 20[th] of June 1996, in an article titled "Manchester engineers lead bomb damage recovery" it was reported that, after a cursory examination by Manchester City Council engineers and building surveyors, they reported that:

- A 30 m long steel truss footbridge providing high level pedestrian access across the street could collapse at any time. The 200 ton bridge was lifted right off its bearings by the explosive blast, leaving it dangerously unsafe as stated by the council's Divisional Bridge Engineer, Steve Tart.
- 1000 city centre buildings of widely varying sizes sited immediately outside the central zone were structurally checked.
- Intense pressure from building owners and the police to reopen quickly outer sections of the total 36 ha of the city centre originally cordoned off has allowed time for only preliminary examination of most buildings.
- Acting City Engineer Mr Derek Robinson said: "As engineers, we would have liked more time for detailed checks but it was important to get city centre businesses back to work quickly". He also added that "We believe only the four Corporation Street buildings have suffered structurally and even they, with propping and strutting, should allow us to soon reopen the central zone".
- It took the City Council engineers from 3 to 4 weeks to inspect a 100 m length of road either side of the bomb site to decide whether or not these buildings could be reused again. These costs of financial losses are not covered by insurance companies.
- A decision on the footbridge and on the partial re-build of the structurally damaged buildings may take several weeks longer.
- The central area cordoned off contained a typical large town mix of high street properties and functions.
- Heavy mid-Victorian stone buildings lie alongside eight to twelve storey steel or concrete framed 1960's shop frontages which house multi-tenancy shop units, along with street corner department stores with the usual large window frontages.

- Around 40% of the buildings lost windows, with fallen non-structural cladding and roof covering creating a scene of devastation which appeared more structurally serious than it was.
- Mr Robinson from the City Council said: "Virtually all the damage is superficial, though even a building with shattered upper storey glass windows is still a dangerous structure to street users until all the glass is removed".
- The main frontages of four major buildings in Corporation Street had no windows. Instead, there was a double row of 7 m tall heavy concrete panels hung onto the structure above ground level brickwork infill.
- Dozens of the tile-covered cladding panels along about half the 100 m long frontage had been blasted off by the bomb, placed just few meters away, and there was evidence that the frame itself had buckled.
- Engineers also feared that frame deflection caused by the blast could have over stressed fixed column connections with the Shopping Centre's pad foundation slab leaving some of the structure unstable. But the vast majority of the centre, including a 20 storey office block in its midst, remains sound and only local rebuilding may be needed.
- The building closest to the bomb, the 10 storey Longridge House looks devastated but again it appears that only the large non-structural concrete infill panels on the lower floors of the steel frame structure have collapsed.
- Adjacent to the Arndale Centre on the other side, the heavy stone mid-Victorian four floors Royal Exchange building has lost most windows and could be structurally damaged. The Marks & Spencer store is devoid of both its glazed shop front and the heavy concrete coping. The council engineers said that initial investigations shows that structural damage appears relatively light.
- Extensive glazing damage appeared random. Ground floor windows relatively close to the blast remain intact while those high up a 26 storey office block some 500 m away are shattered. Complete shop fronts several streets away from the blast area were blown out, but adjacent windows and those on the opposite side of the road were not even cracked.

Figure 2.12 The cladding of Marks & Spencer was severely damaged due to blast. The pedestrian bridge lifted up the two victims who were walking across the bridge.

Figure 2.13 The multi storey garage cladding was severely damaged.

Figure 2.14 Precast concrete panels damaged, Marks and Spencers, Manchester, UK.

Figure 2.15 The glazing distroyed due to blast, Marks and Spencers, Manchester, UK.

"Despite glass showering down into the street as the area was hurriedly evacuated, only two persons were killed by the blast but several hundred were injured", *New Civil Engineer, 20th June, 1996.*

The blast ripped right through some buildings with windows blown out on one side and in on the other, as shown in Figures 2.12 to 2.15.

2.5 Lessons learnt

A summary of the details of the three case histories is given in Table 2.1.

Table 2.1 Summary of case histories

Case History number	Date	Weight of charge	Pertinent details of damage to cladding and glazing	Cost of damages
1 London Bishopsgate	4/1993	one ton of TNT	Falling glass over a mile radius was observed.	£ 1.50 to 2.5 billion , one man was killed and 45 were injured.
2 London South the Quarry Dockland bomb	2/1996	500 kg of TNT	Radius of influence was about 1/2 mile causing destruction to the cladding panels.	£ 150 millions - 2 civilians were killed and hundred were wounded
3 Manchester bomb	6/1996	500 kg of TNT	Damage to window glass to a radius of 1/2 mile	£ 300 million.

These case histories highlight a number of issues which it is hoped to cover in this research. The blast wave and explosion sequence produced by terrorist explosive material are discussed. The structure response and resistance function of cladding are analysed in order to establish their damage tolerance, using iso-damage diagrams. Cladding materials and its constituents are also discussed, both from a technical and aesthetic point of view.

The main points to note are:

- Public buildings and civic centres are terrorist targets.
- Building cladding, designed to withstand normal wind pressure conditions, has weak resistance against blast.

- Explosions can happen anywhere. In a few seconds buildings can become worthless.
- Billions of pounds have been spent in repairing or replacing damaged cladding and glazing.
- Most of the injuries are to people walking on the street who are hurt by falling glass.
- Care must be taken in the future to enhance the cladding resistivity so that losses will decrease.

Spending a few million pounds for the strengthening of glazing could save several billions.

Chapter 3

The blast wave mechanism and the structure response

Chapter 3

The blast wave mechanism and the structure response

3.1 Scope

This chapter considers the explosives used by terrorists and the threat they pose when they are directed towards public buildings with their conventional soft constructions. The consequences of explosions, how the blast wave is produced and its behaviour until it engulfs the target will be analysed.

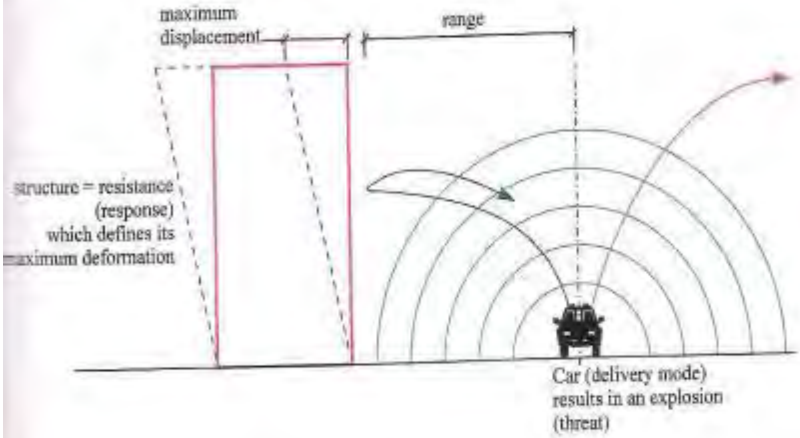


Figure 3.1 A car carrying an amount of explosive will cause the explosion which creates the blast wave which has a duration and affects any structure with specific resistance with its different parameters

This chapter is divided into three major sections. In the first section the explosive classification is reviewed and also the nature of the explosion. In the second section the blast wave mechanism will be investigated. In the third section the structure response to blast loading will be illustrated.

3.2 The range of delivery modes used by terrorists

The method of attack or the method of charge delivery to the target is important. The charge size is limited by the container in which the initiation is going to take place, and its effect will be limited by distance from the target. The following classification of delivery modes is provided by *Elliot, 1989*.

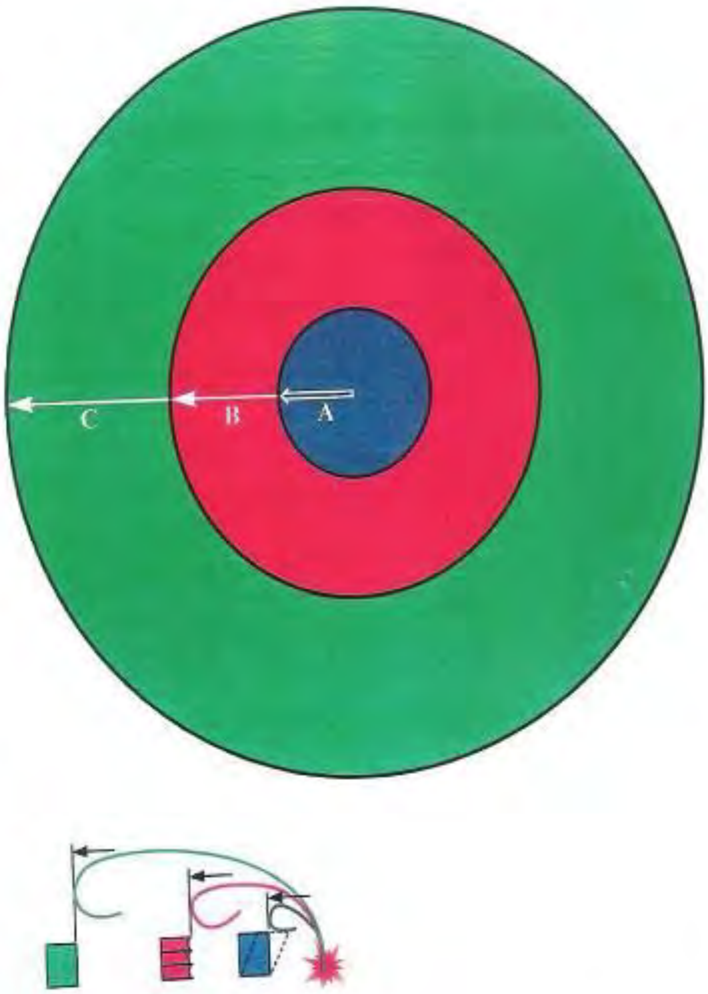


Figure 3.3 An estimated configuration for the creation of blast wave

2. The damaging aspects of a blast wave

The damage caused by the blast wave is clearly related to the range of the target from the explosion as well as the charge size. This is illustrated in Figure 3.3. This shows the relative effects by zoning the area around the event. Region A represents the total destruction of the structures that exist in this region, whereas at region B the structure might expect a partial failure of the structure elements and a total failure of cladding. Region C stands for glass failure caused to structures at this region.

This estimation is an illustrative example but it is not precise as these regions are proportioned according to the charge weight and the ranges to the targets. It is usually appropriate to use scaled ranges to allow consideration of varying charge sizes. However, to highlight the considerable potential in protecting against cladding, including glazing, damage, Table 3.2 shows the scale of such potential benefits in the outer zones.

Table 3.2 Blast Damage Effects with range.

Zone	Radius m	Ratio to Zone A	Area m^2	Ratio to Zone A
A	50	1	7853	1
B	150	3	70685	9
C	1000	20	3141592	400

Note : Ranges assumed for illustration.

A further example,illustrated in Figure 3.4, shows how an explosion does not give its target any early warning of approaching devastation. The first symptom of an explosive blast is a potent blow due to a sudden pressure rise in its shock front. This evolves as a swift crushing effect in the form of a blast over-pressure wave (pressure above atmospheric) combined with a blast wind of super hurricane velocity.

Figure 3.4 shows the effect of the pressure of a typical blast wave and is systematically illustrated in Figure 3.5. At time A on both figures, the atmosphere is still undisturbed. Whereas at time B, it is immediately after the blast wave shock arrives at the structure. The associated blow is with a force proportional to the pressure rise and the impact area. In addition there is the force exerted by the blast wind. These blast waves decrease quasi exponentially with time until the pressure becomes equal to the atmospheric (zero overpressure) after which there is a slight negative phase, as at point C, along with a reversed blast wind, (*Kinney and Graham, 1985*).

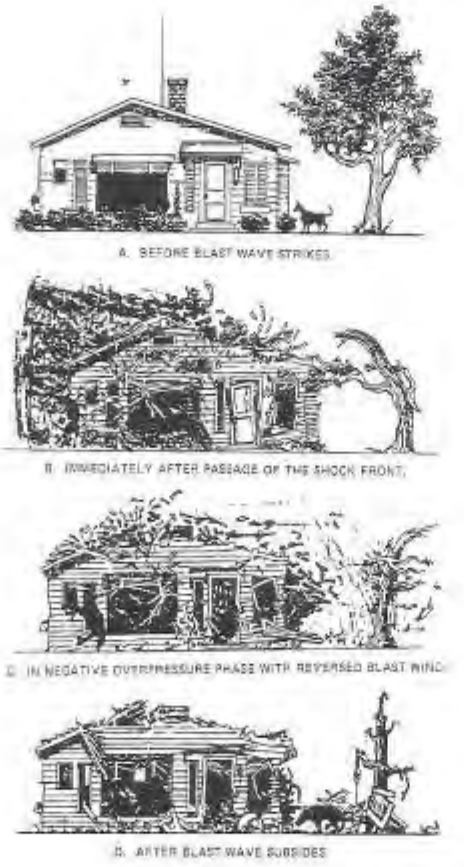


Figure 3.4 Sequential photographic representation of blast wave effects. Times A to D in figure 3.5 correspond to those in this figure. (*Kinney and Graham, 1985*).

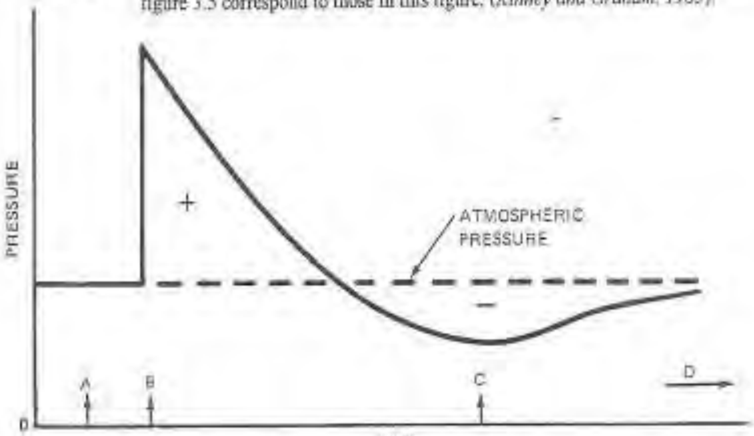


Figure 3.5 Blast wave pressures plotted against time, (*Kinney and Graham, 1985*).

Peak positive incident pressure p_s-(psi)

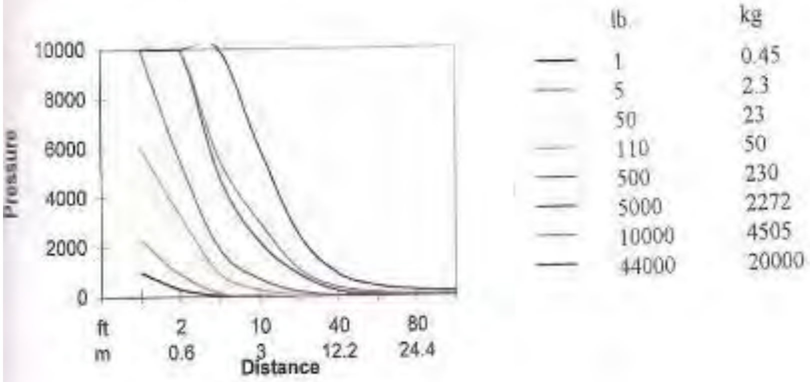


Figure 3.10 The relation between the incident pressure and distance, *Elliot, 1989.*

Table 3.4 Pressure vs distance in free air for a surface burst of different weights of TNT explosive, *after [TM5-1300, chapter 2, 1985].*

		Charge Weight							
		1 lb. 0.45 kg	5 lb. 2.3 kg	50 lb. 23 kg	110 lb. 50 kg	500 lb. 230 kg	5000 lb. 2272 kg	10000 lb. 4505 kg	44000 lb. 20 tons
Meters	Feet	Peak positive incident pressure - p_{so} (psi)							
0.3	1	1000	2300	4500	6000	-	-	-	-
0.6	2	300	900	2300	3300	5500	-	-	-
1.5	5	40	140	650	900	1900	4800	5500	-
3	10	9.5	34	160	280	700	2200	3000	6000
6.1	20	3	7	33	70	200	900	1100	2500
12.2	40	1.3	2.5	7	12	40	230	360	900
18.3	60	0.7	1.3	4	6	18	80	160	400
24.4	80	0.5	1	3	3.6	9.5	50	90	250
30.5	100	0.37	0.75	2.3	3	6	26	55	160

3.5 Blast wave mechanism

When a high explosive material is caused to detonate, a series of events take place. Due to explosion, hot gases of temperatures about 3000 - 4000°C are created. These gases can reach pressures from 100 - 300 kilobars. The gases will expand violently forcing the surrounding air out of the space it occupies. The blast wave will be produced taking a form of pressurised gas in front of the released gases.

As the gases formed by the explosion expand, the pressure decays until it becomes atmospheric as the blast wave moves out of its source. The pressure of the pressurised air at the blast wave front decays due to increasing distance. As the gases expand they lose their temperature and their pressure becomes a little under atmospheric pressure.

The gases became over-expanded and consequently they flow back towards their start point pushed by the difference between the atmospheric pressure and the explosive gas pressure.

As a result, the blast wave forms a region of under pressure. This region is known as the negative phase. Afterwards there is a return to equilibrium as the air gases pushed away from the source return, (*Smith and Hetherington, 1994*).

The following Figures illustrate both the reactions inside and outside the charge, (*Smith and Hetherington, 1994*).

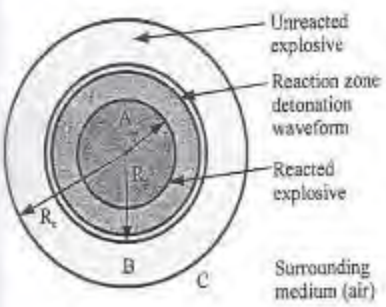


Figure 3.6.a Inside the charge, (*Smith and Hetherington, 1994*)

r = radial distance from charge centre.
R_c = radius of spherical charge.
R_d = radius of detonation wave front.

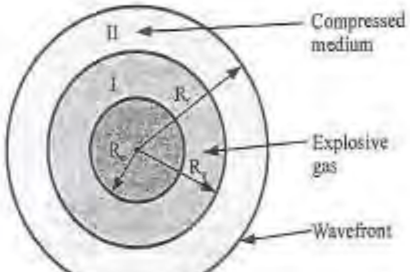


Figure 3.6.b Outside the charge, (*Smith and Hetherington, 1994*).

R_c = radius of spherical charge.
R_f = radius of blast wave front.
R_g = radius of explosive gases.

Blast waves from sources other than TNT are quantified by the conversion of the actual mass of the charge into a TNT equivalent mass. Guidance on conversions for different standard explosives is given in standard texts, such as *TM5-1300* and *Smith and Hetherington 1994*.

Parameters required to quantify blast wave negative phase

Δp_{min} is the rarefaction component of the blast wave which is given by *Brode, 1955* by the following equation:

$$\Delta p_{min} = -\frac{0.35}{Z}(Z > 1.6) \qquad 3.7$$

Where Δp_{min} is the peak under pressure in bars

Z is the scaled distance

The specific impulse of the negative phase $\bar{i}$ is given by

$$\bar{i} \approx i_s[1-\frac{1}{2Z}] \qquad 3.8$$

Where $\bar{i}$ is the negative phase specific impulse

i_s is the positive specific impulse

Z is the scaled distance

3.7 Blast wave reflection

When a blast wave is delivered to a target denser than its surrounding space it will reflect from it. The reflection is one of three types, normal, oblique angle or mach stem. These events are illustrated in Figure 3.11.

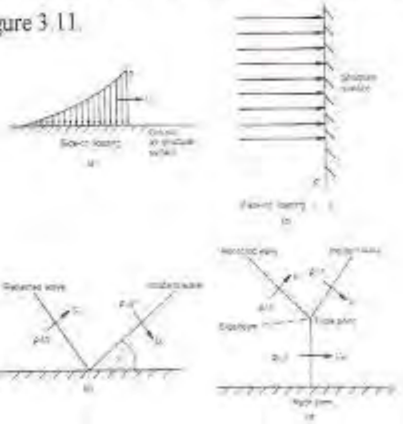


Figure 3.11 Side-on and face-on pressure loading, regular and Mach reflection, *(Mays and Smith, 1995)*.

1. Normal reflection

When a blast wave hits a target at a right angle (zero angle of incidence), the air particles will be caused to stop for a short time. These particles at the shock front will then have a relative velocity which is proportional to that of the particles still moving towards it. The particles will be compressed and then move in the opposite direction with a velocity of the same magnitude. *Rankine, 1870* in his studies into shocks in ideal gases developed a relationship involving the peak overpressure, dynamic pressure and specific heat parameters, that has been further developed to give (*Smith and Hetherington, 1994*):

$$p_r = 2p_s\left[\frac{7p_o + 4p_s}{7p_o + p_s}\right] \tag{3.9}$$

Where p_r is the peak reflected overpressure

p_s is the peak side-on overpressure

p_o is the atmospheric pressure

By observation, when p_s is a lot less than p_o (e.g. a small charge at long range), p_r tends towards a value of two times p_s. On the other hand, when p_s is much greater than p_o, the value of p_r tends towards eight times p_s. Figure 3.12 shows reflected overpressure and impulse i_r for normally reflected blast waves plotted against scaled distance Z.

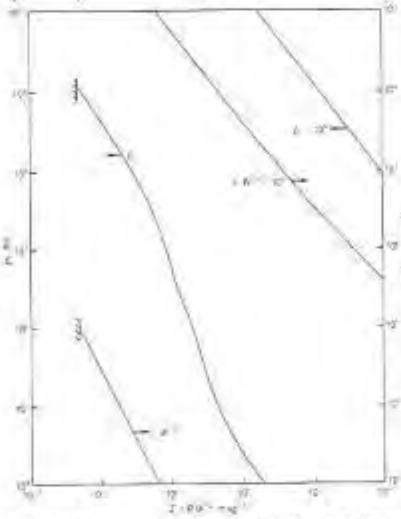
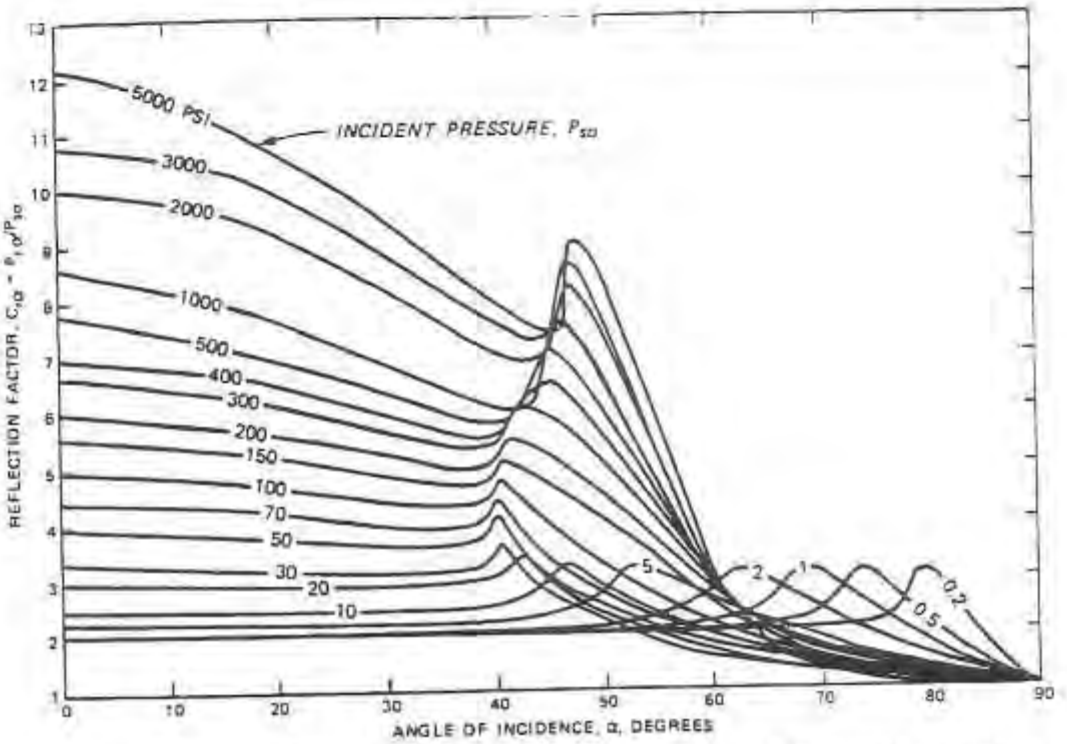


Figure 3.12 Normally reflected blast wave parameters for spherical charges of TNT. *(after Baker et al, 1983)*.

Reflection with an oblique angle takes place when a shock impinges on a surface with an angle between the shock front and the plane of the reflecting surface, see Figure 3.11.

Figure 3.14 shows an example of a 500 lb bomb exploded at 60 ft away from a structure located at three different angles of incidence, *(Elliot, 1989)*. From Table 3.4 and Table 3.5 the value of p_{s^o} and p_r of a 500 lb. bomb at a range of 60 feet are p_{s^o} = 18 psi and p_r = 46 psi.

Using the plot of the reflected pressure coefficient ($C_{r\alpha} = p_{r\alpha} / p_{s^o}$) versus angle of incidence *(after TM 5-855-1)*, (Figure 3.14) it is possible to derive the reflected pressures as shown.

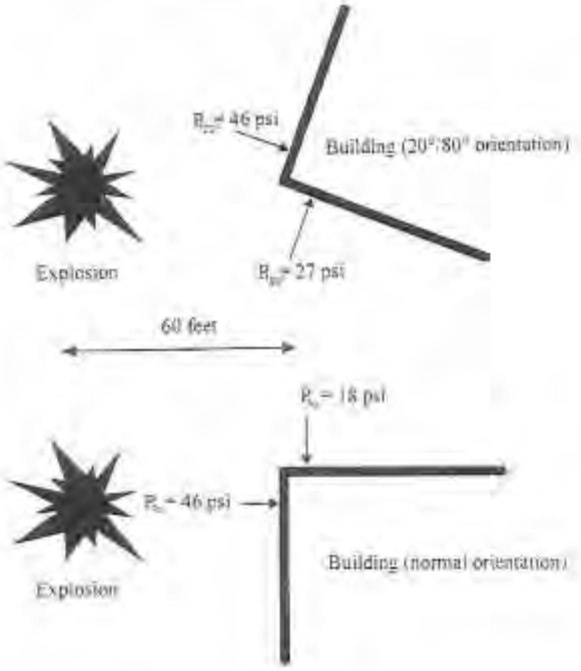


SOURCE: Air Force Weapons Laboratory, "Protection from Nonnuclear Weapons," AFWL TR 70-127.

Figure 3-14 Reflected pressure coefficient versus angle of incidence *(after TM 5-855-1), 1987* for $p_{so} = 18$ psi and $\alpha = 45^\circ$, $C_{r\alpha} = 3$

and $p_{r\alpha} = 3 \times 18 = 54$ psi (on both face).

Hence, it is clear that the intuitive solution for minimum loading of angling the building at 45° to the blast direction does not produce the least loading.



a. Building at 20°/80° orientation.

b. Building (normal orientation)

c. Building (45° orientation)

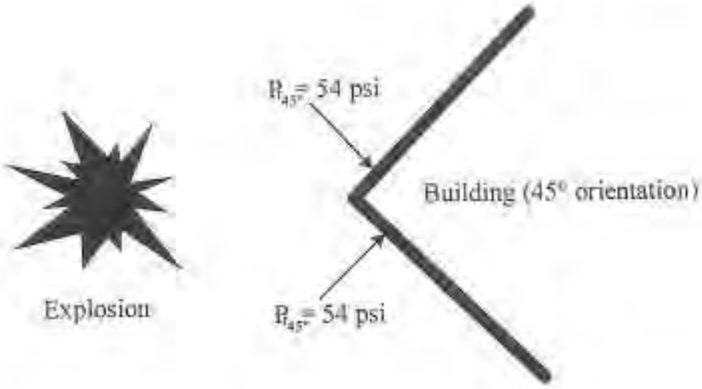


Figure 3.15 The effect of building orientation on the peak pressure value, *(Elliot, 1989)*.

a. The reflecting surface

When a surface is exposed to a blast wave, its strength does not affect the reflected wave strength. For example, two surfaces, one of concrete and the other of glass, will reflect similar waves, even though it seems that the pane of glass would be unlikely to do so. This is because the shock reflection occurs before the glass pane has time to respond, even though it is very likely to break subsequently. It is also possible for blast waves to fly over small openings and perform as if those openings did not exist, *(Elliot, 1989)*.

b. Multiple reflection

Figure 3.16 illustrats that, when the blast wave is scattered due to surface reflection, the pressure from an explosion might enlarge many times at a point where the incident and reflected waves come simultaneously together to form an interacted pattern. When they come together, it is in stages that might cause the incident and reflected waves to form a single larger blast shock wave. If there is a segregation in time between the two pulses, the target will receive multiple shocks. All of these shocks will be completed before the building starts its structural response to the loading. Reflection could be enhanced if the building has re-entrant walls or where the explosions take place between close buildings, particularly tall ones close together. This can be seen if a few of the wave paths are drawn in plan or elevation around the building, *(Elliot, 1989)*.

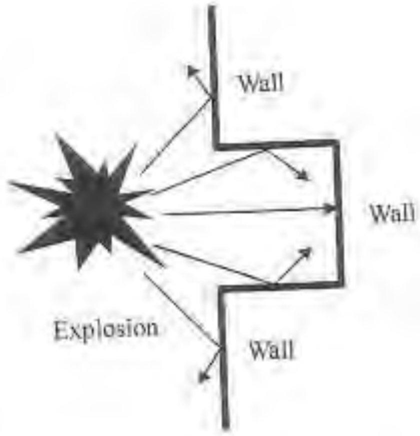


Figure 3.16.a Shock waves focused into a re-entrant wall, *(Elliot, 1989)*.

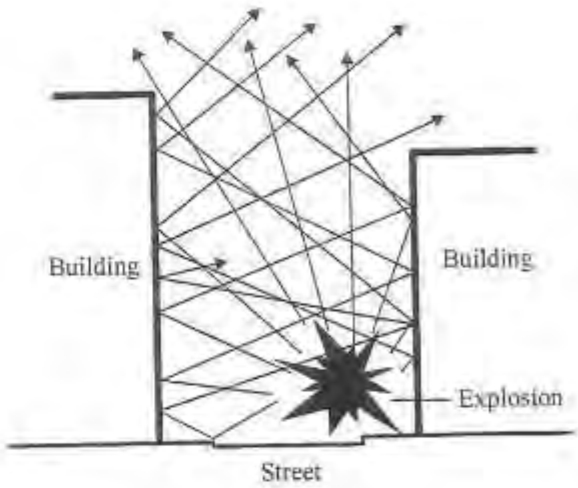


Figure 3.16.b Shock waves reflected off buildings in a narrow street, *(Elliot, 1989)*.

3. Mach stem reflection

The third case of reflection is the Mach stem, (see Figure 3.12). In such a case the blast wave front hits the target surface with a "near grazing incidence". The shock does not "bounce" immediately but it deflects taking "spurt" along over the surface. The wave front travels approximately perpendicular to the surface and in this case it is known as a Mach stem. This effect occurs when the angle of incidence exceeds about 40°. This explain's the apparent anomaly of higher than expected pressure in Figure 3.15

3.8 Blast loading of structures

When a blast wave of an explosive charge interacts with a target the loading pattern will depend on the size of both the charge and the target.

Firstly, if a huge scale blast wave interacts with a medium size target it will tend to engulf and squash the target, and due to the target size and its structure characteristics, it may deflect under load.

Secondly, when a small charge interacts with a large scale target the response of each structure element needs to be analysed individually since they are going to be loaded sequentially.

In the first case of loading, two types of load components will be produced at the target; diffraction and drag load components. When a blast wave diffracts around a structure, it will engulf it, creating a squashing force to its surfaces. When loading on one side of the target, it will be pushed one way, followed by a lower intensity push in the opposite direction as the diffraction is completed. Also, drag loading will produce a push as well and this will be followed by a suction force on the right hand side as the blast wave dynamic pressure (the blast wind) passes over and round the target. *(Smith and Hetherington, 1994).*

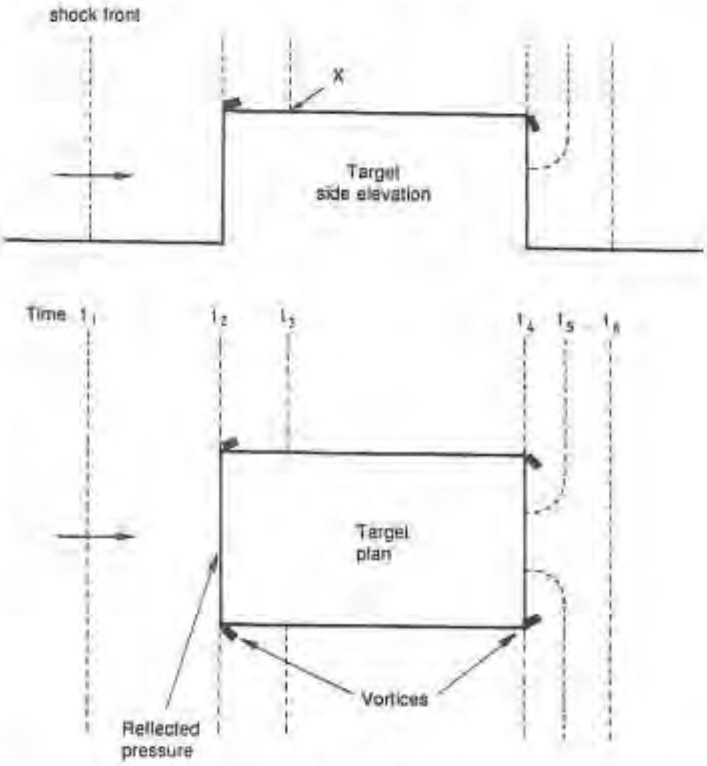


Figure 3.17 Diffraction of blast wave round simple geometry target, *(Smith and Hetherington, 1994).*

Smith and Hetherington, 1994 analysed the loads exerted when a blast wave engulfs a structure. Figure 3.17 shows the diffraction process as a shock front passes across a structure. Figure 3.18 shows the pressure and drag loadings as they vary with time. The times shown in Figure 3.17 relate to those shown in Figure 3.18.

Figure 3.18.a shows the peak pressure when it encounters the front face of the target, at time t_2 and becomes the peak reflected overpressure p_r. During the period of time $(t'-t_2)$, this pressure will decay. This is due to the pressure of the blast wave when passing over the top and round the sides of the structure being less than p_r. The peak

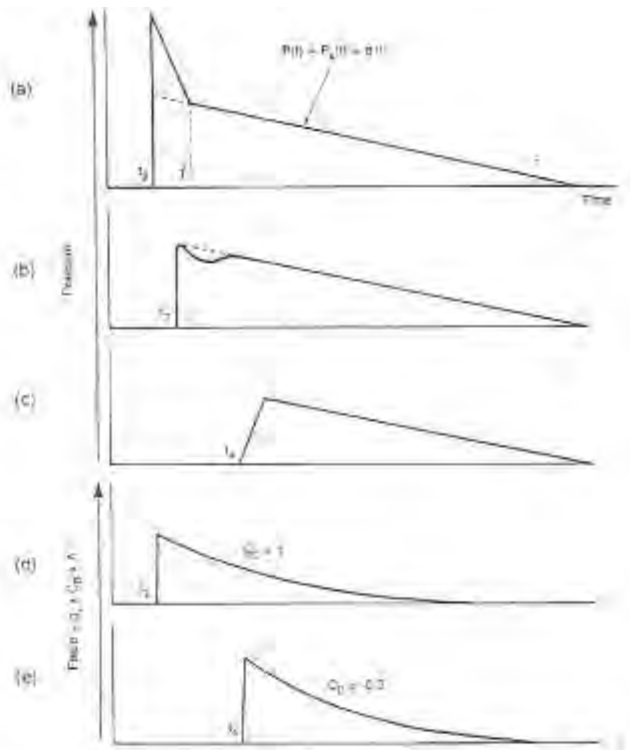


Figure 3.18 Pressure vs time and drag force vs time for diffraction loading of simple geometry target (Smith and Hetherington, 1994)

$$p_{stag}(t) = p_s(t) + q_s(t)$$ 3.1

Where p_{stag} is the stagnation pressure

p_s is peak side-on overpressure

q_s is peak dynamic pressure

The pressure reduction time t' may be approximated as:

$$t' = \frac{3s}{U_s}$$ 3.1

Where t' is the pressure reduction time

s is the target dimension, (It is the smaller dimension of $B/2$ or structure height H given in Figure 3.19)

U_s is the blast wave front speed

After time t' the pressure at the front face will be a time-varying function of the static and dynamic overpressure parameters of the wave. Figure 3.18.b shows the deviation from linear decay of pressure on top and sides after time t_1 because of the vortices caused at the intersection of the top and the sides with the front. Figure 3.18.c shows the load contour on the back end elevation which is affected by the finite rise time as the front diffracts. Figure 3.18.d and 3.18.e show the forces applied on the front and rear faces of the target by dynamic, drag or wind forces. The drag force F_D is given by:

$$F_D = C_D \times q_s(t) \times A \qquad 3.12$$

Where F_D is the drag force

C_D is the drag coefficient

q_s is the peak dynamic pressure

A is the area of target loaded by blast

These aspects are covered in more detail in Kinney and Graham, including the diffraction of blast loads around a structure and the effects of drag forces acting on it. Examples of drag coefficient C_D are given in Table 3.6

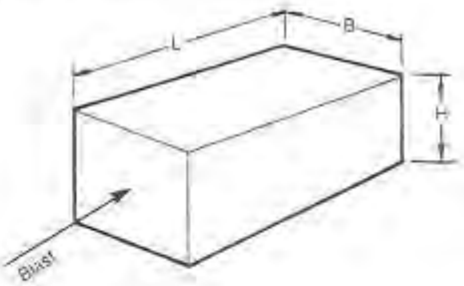


Figure 3.19 Dimensions of simple geometry target. *(Smith and Hetherington, 1994)*.

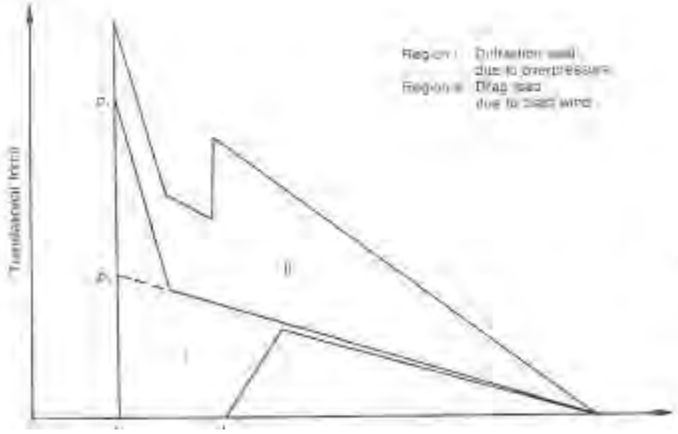
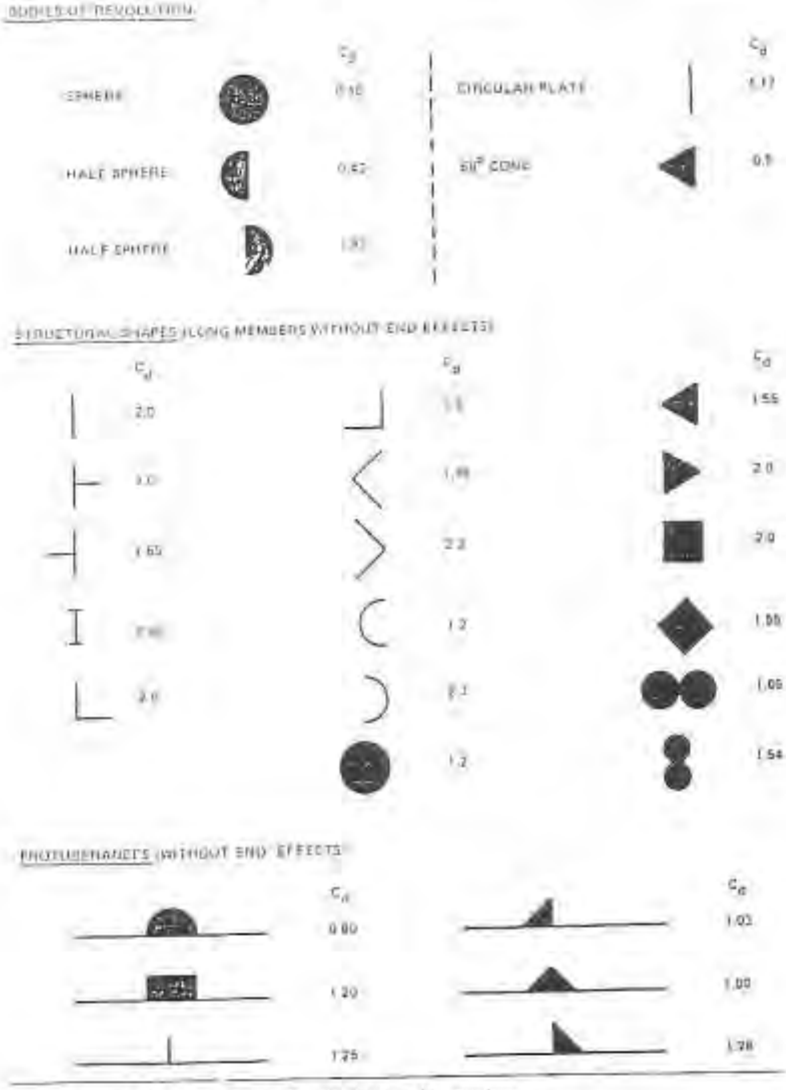


Figure 3.20 Translational force vs time for diffraction of blast wave round simple geometry target, *(Smith and Hetherington, 1994)*.

Table 3.6 Drag coefficient *(Kinney and Graham, 1985).*



*Flow direction from left to right. For high Reynolds numbers.

3.9 The response of structures to explosive blast

Two phenomena characterise the blast wave; the peak pressure and the impulse. The peak pressure defines the maximum force that is created against a structure, since force is the product of pressure multiplied by the area upon which it is exerted.

Impulse is a measure of the product of force, which may vary with time, and its duration and specific impulse is the impulse per unit area (i.e. specific impulse is the product of pressure, over time, and duration).

The way an element behaves under a blast load is decided by two factors: the element's ability to absorb applied stress and the element's natural period of vibration.

Impulse is a measure of the product of force, which may vary with time, and its duration and specific impulse is the impulse per unit area (i.e. specific impulse is the product of pressure over time, and duration).

The way an element behaves under a blast load is decided by two factors: the element's ability to absorb applied stress and the element's natural period of vibration.

Structural vibration due to types of disturbance

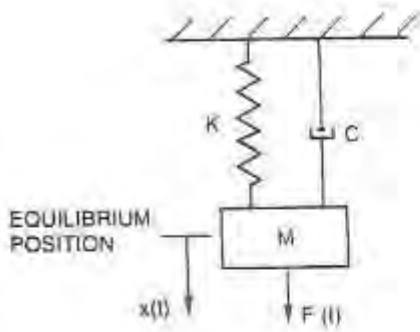
A structure can be disturbed from its normal position of static equilibrium in a number of ways. It can either be given an initial displacement or be given an initial velocity i.e. by moving it from its static equilibrium position and releasing it.

Such movement will cause it to vibrate with a period of oscillation. This is dependent on its mass and a factor characterising the resistance that it develops.

The establishment of a single degree of freedom system

When a blast wave hits a target structure it causes it to deform. Such deformation ranges from simple non-effectual deformation to a total destructive one. When a blast wave impinges on a structure, it forms a dynamic system of loading with the blast wave acting as a forcing function.

This forcing function causes deformation in a form of displacement, which is to be resisted by the inherent strength of the structure and by its dynamic inertial resistance.



Where c is a damping coefficient
$F(t)$ is a forcing function
K is the structure elastic resistance.
M is the structure mass.
$x(t)$ is the time dependent displacement

However the displacement-time relation is important when analysing the behaviour of a blast loading on a structure. To do so, it is convenient to consider a Single Degree Of Freedom system (SDOF), when one needs to study the interaction between the blast loading as an applied force and the structure resistance, such as the one shown in Figure 3.20 for an elastic system.

In Figure 3.20, $F(t)$ is a time varying applied loading. A free body diagram of the structure showing the acting forces is shown in Figure 3.22.

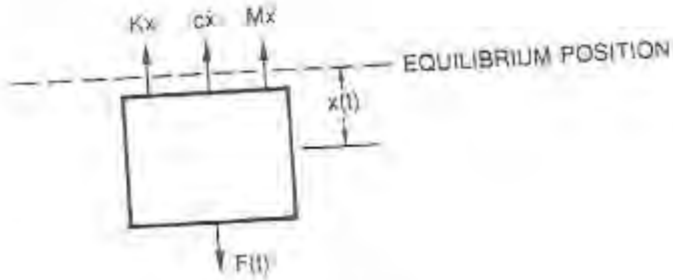


Figure 3.22 Free body diagram of single degree of freedom system. *(Smith and Hetherington, 1994)*

The equation of motion of this structure is:

$$M\ddot{x}+c\dot{x}+Kx=F(t) \qquad 3.13$$

3. Rigid target elastically supported

When an elastic structure is subjected to an idealised triangular pulse, such a system can be idealised as a SDOF system.

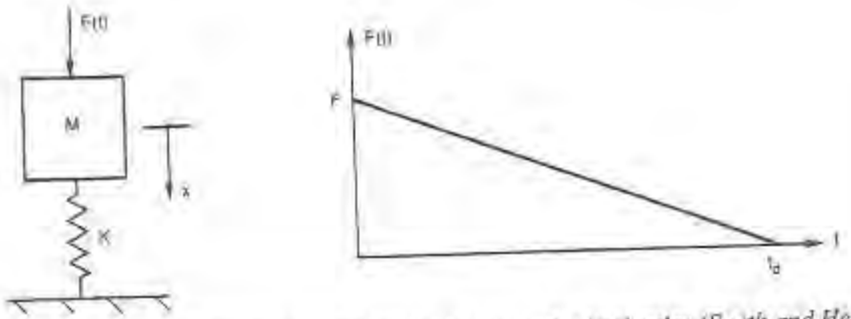


Figure 3.23 Single degree of freedom system loaded by triangular load pulse *(Smith and Hetherington, 1994)*.

In case of a structural response time less than the positive phase duration, the displacement is given by:

$$x(t) = \frac{F}{K}(1 - \cos\omega t) + \frac{F}{K}\left(\frac{\sin\omega}{\omega} - t\right) \qquad 3.17$$

Where ω is the natural circular frequency of vibration given by: $\omega = \sqrt{\left(K/M\right)}$.

Taking the worst case, the maximum displacement x_{max} occurs, when the velocity of the structure is zero. Differentiating the equation and setting dx/dt to zero gives:

$$o = \omega\sin(\varpi t_m) + \frac{1}{t_d}\cos(\omega t_m) - \frac{1}{t_d} \qquad 3.18$$

$$\frac{x_{max}}{F/K} = \psi(\omega t_d) = \psi'\left(\frac{t_d}{T}\right) \qquad 3.20$$

Where T is the natural period of vibration of the structure $= \frac{2\pi}{\omega}$

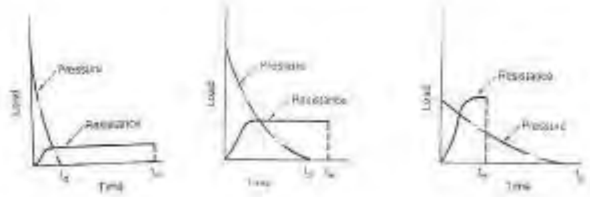
ψ is a function of ωt_d

ψ' is a function of $\frac{t_d}{T}$

4. The classification of loading

Loads which last a short time compared with the structures natural period of vibration are known as impulsive loading.

However if the applied load duration lasts a longer period of time than the natural period of vibration of the structure such type of loading is called quasi-static. In between the two types is dynamic loading, which has a load duration similar to the structures natural period of vibration.



Pressure design range	High	Intermediate	Low
Design load	Impulse	Pressure-time or dynamic	Pressure or quasi-static
Typical Pressure levels	>> 14 bar	<14 bar	<.7 Bar
Pressure duration	Short	Intermediate	Long
Response time	Long	Intermediate	Short
Relationship of t_m to t_d	$t_m/t_d > 3$	$3 > t_m/t_d > 0.1$	$t_m/t_d < 0.1$

Figure 3.24 Load regime summary after (*Baker et. al, 1980*)

The strain energy acquired by the structure, U, is the area under the resistance displacement curve which is given by:

$$U = \frac{1}{2} K x_{max}^2 \qquad 3.23$$

Equating the WD and U:

$$\frac{x_{max}}{(F / K)} = 2 \qquad 3.24$$

Which could be written as:

$$\frac{x_m}{x_{st}} = 2 \qquad 3.25$$

Where x_{st} is that static displacement achieved due to the application of force F statically. Equation 3.26 is known as the Dynamic Load Factor (DLF). In this case DLF takes the maximum of 2 to represent an upper boundary of response and is called the the quasi-static asymptote.

b. When analysing the response of a structure to an impulsive load, one has to know that when the impulse approaches the structure it provides it with instantaneous velocity change. Consequently, the structure will gain a kinetic energy, which is converted, to strain energy.

The impulse, I, that causes an initial velocity change $\dot{x}_o$ is given by the following equation:

$$\dot{x}_o = \frac{I}{M} \qquad 3.26$$

The kinetic energy *(KE)*the structure will receive is given by:

$$KE = \frac{1}{2} M \dot{x}_o^2 = \frac{I^2}{2M} \qquad 3.27$$

The structure will acquire the same strain energy U as before, because it displaces by x_{max}. Thus the KE and U are equated and the resulting equation rearranged to get

The three regimes are summarised by *Smith and Hetherington, 1994* in terms of the product of natural frequency and positive phase duration, which is proportional to the ratio of T to t_d as indicated by equation 3.21.

$$0.4 > \omega t_d \quad \left[\alpha \frac{t_d(short)}{T(long)}\right] \quad \text{Impulsive}$$

$$40 < \omega t_d \quad \left[\alpha \frac{t_d(long)}{T(short)}\right] \quad \text{Quasi-Static}$$

$$0.4 < \omega t_d < 40 \quad \left[\frac{t_d}{T} \approx 1\right] \quad \text{Dynamic} \qquad 3.21$$

Also the three regimes are represented in terms of time taken to reach maximum displacement t_m as indicated in Figure 3.24 after *Baker et.al, 1983.*

5. Evaluation of the limits of response

a. When evaluating the response of a quasi-static load on a structure, the load pulse can be idealised as shown in Figure 3.25.a where the structure resistance can be represented by the graph of Figure 3.25.b.

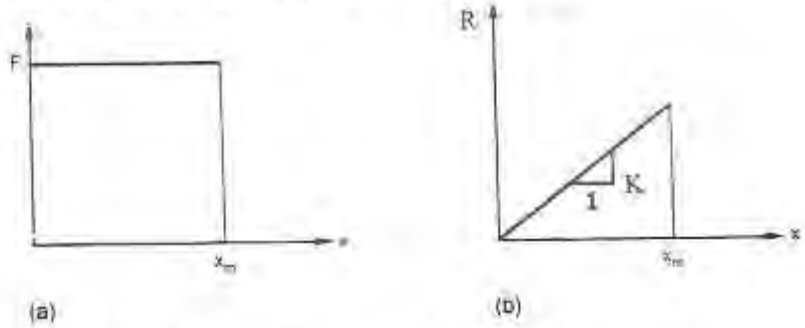


Figure 3.25.a Idealised quasi-static load pulse.
b Elastic resistance vs displacement,
(Mays and Smith 1995).

The basic principal when analysing a quasi-static loading is to know that the work done on the structure to cause it to deform is converted to strain energy. The work done *(WD)* by the load to cause the structure to deform with the displacement x_{max} is given as:

$$WD = Fx_{max} \qquad 3.22$$

$$\frac{I^2}{2M} = \frac{1}{2} K x_{max}^2 \qquad 3.28$$

Then

$$\frac{x_{max}}{F/K} = \frac{I}{\sqrt{KM}\left(F/K\right)} = \frac{\frac{1}{2} F t_d}{\sqrt{KM}\left(F/K\right)} = \frac{1}{2} \omega t_d \qquad 3.29$$

This equation is the equation of the impulsive asymptote of response. Figure 3.25 gives the two asymptotes which can be drawn on a response curve with abscissa of ωt_d and an ordinate of $\frac{x_{max}}{F/K}$. The response of the structure is given by the curve drawn in Figure 3.26 relative to the asymptotes.

The three regimes shown in Figure 3.26 of quasi-static, impulsive and dynamic response are identified on the resulting graph as regions I, II and III respectively.

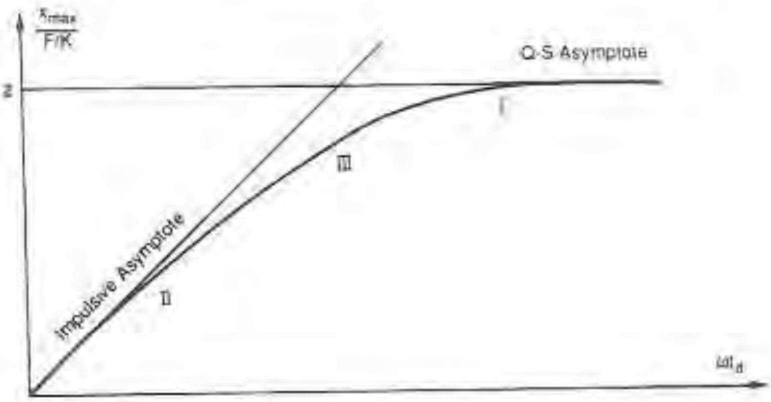


Figure 3.26 Response of elastic SDOF system for all load regimes (*Smith and Hetherington, 1994*).

6. Pressure – impulse or iso-damage curves

A (P-I) pressure-impulse or iso-damage curve is that curve which gives the pressure and impulse combinations that can cause damage to the target. The P-I diagram characterises the bounds on behaviour of a target structure under pressure and impulse (which can be a total impulse I or (as here) a specific impulse i_s or i_r), (*Mays and Smith*

$$\frac{x_{max}}{F/K} = \frac{Kx_{max}}{F} = 2 \qquad 3.30$$

or:

$$\frac{2F}{Kx_{max}}[\alpha \frac{Maximum \quad Load}{Maximum \quad resistance}] = 1 \qquad 3.31$$

This is the equation of the modified quasi-static asymptote. It could be plotted on a graph with ordinate $\frac{2F}{Kx}$. Also, multiplying the abscissa ωt_d of Figure 3.26 by the inverse of the ordinate of Figure 3.25 and noting that $\omega = \sqrt{\frac{K}{M}}$:

$$\frac{Ft_d}{Kx_{max}}\sqrt{\left(K/_M\right)} = \frac{2I}{x_{max}\sqrt{(KM)}} \qquad 3.32$$

As the impulsive asymptote of Figure 3.26 given by equation 3.29. When using the abscissa: $\frac{I}{x_{max}\sqrt{(KM)}}$ means the new impulsive asymptote is given by: $\frac{I}{x_{max}\sqrt{(KM)}} = 1$.

Now the response can be re-plotted using axes: $\frac{2F}{Kx_{max}}$ and $\frac{I}{x_{max}\sqrt{(KM)}}$ to get Figure 3.27.

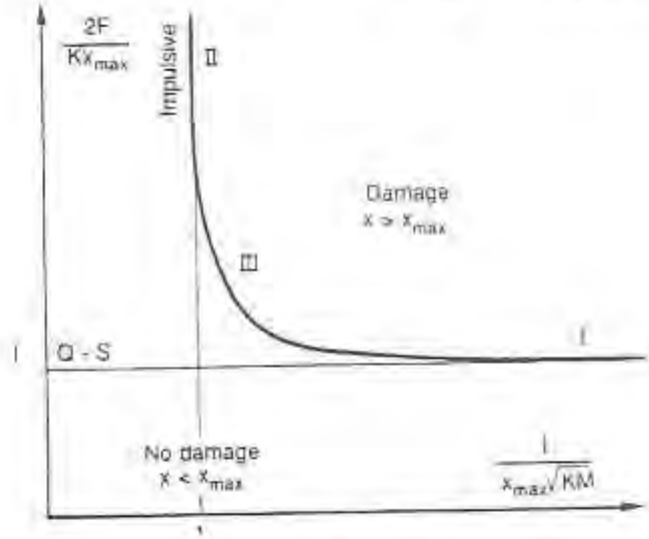


Figure 3.27 Pressure impulse diagram for elastic SDOF. (*Baker et. al. 1983*).

The form of this graph allows easier assessment of response to a specified load. Once a minimum displacement is defined (i.e. the damage criterion has been specified) this curve then indicates the combinations of load and impulse that will cause failure. Combinations of pressure and impulse that fall to the left of and below the curve will not induce failure while those to the right and above the graph will produce failure due to stresses in excess of the allowable limit, (*Smith and Hetherington 1994*). Another structural model for a building we have to consider is that where the structure is idealised as a rigid plastic structure which undergoes yield when it reaches a resistance force R as shown in Figure 3.28.

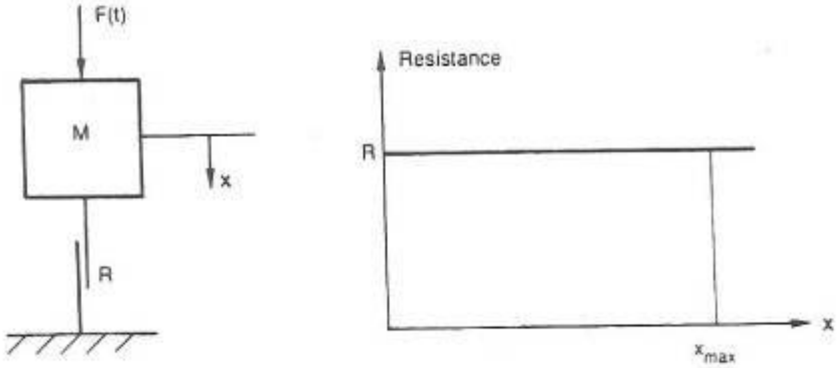


Figure 3.28 Rigid Plastic structure loaded by blast wave, (*Baker et . al, 1983*).

The upper and lower bounds of response, the quasi-static and impulse asymptotes respectively are established by analysis. The work done by a constant blast force F to achieve a displacement x_{max} is equated to the strain energy acquired by the structure. Hence:

$$\begin{aligned} Fx_{max} &= Rx_{max} \\ \therefore \frac{F}{R} &= 1 \end{aligned} \qquad 3.33$$

This idealisation (*Smith and Hetherington 1994*) shows that the quasi-static asymptote is the zero displacement asymptote but for a force to resistance ratio that just exceeds unity, a large displacement can take place.

When equating the kinetic energy caused by the blast load impulse I with strain energy the impulse asymptote is obtained. Thus:

$$\begin{aligned} \frac{I^2}{2M} &= Rx_{max} \\ \therefore \frac{I^2}{x_{max}MR} &= 2 \end{aligned}$$

$$\therefore \frac{I}{\sqrt{x_{max} MR}} = \sqrt{2} \qquad 3.34$$

Baker et al 1983 used the axes, $\frac{I}{\sqrt{x_{max} MR}}$ *and* $\frac{F}{R}$ for the iso-damage curve as shown in Figure 3.29

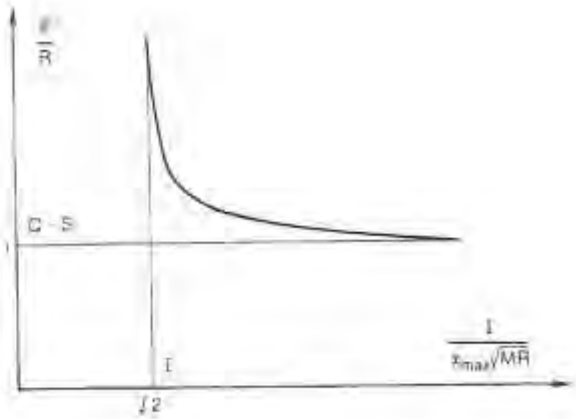


Figure 3.29 Pressure – impulse diagram for rigid plastic structure (*Baker et al, 1983*)

7. *Studying the structure response by using equivalent systems as elastic - plastic SDOF*

Smith and Hetherington 1994 show the analysis of a structure with an elastic-plastic resistance and elastic stiffness K. At yield point, the displacement is x_{el}, and the maximum displacement is x_{max}. Figure 3.30 shows that system.

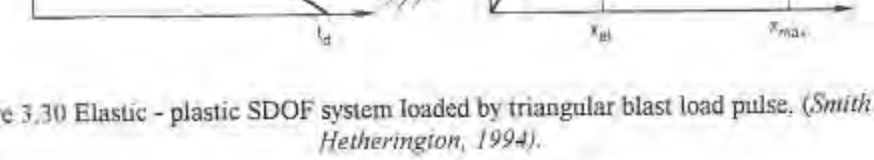


Figure 3.30 Elastic - plastic SDOF system loaded by triangular blast load pulse. (*Smith and Hetherington, 1994*).

4.4 Cladding protective design configuration

1. Some layout considerations

As *Mays and Smith, 1995* said, any architectural concept or feature influences the behaviour of the building when exposed to the shock wave.

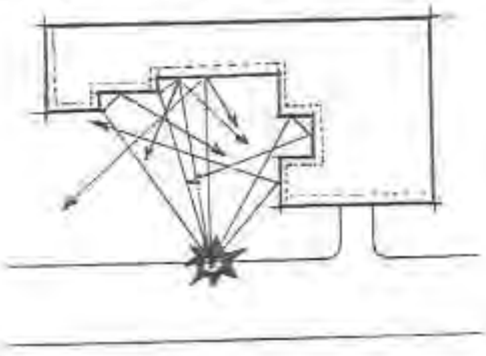


Figure 4.10 a. A building layout with inward staggered treatments. The dotted line indicates where special cladding hardening treatments are required.

Figure 4.10.b If the threat is directed from a direction opposite to the above for the same structure such location provides a highly protected region.

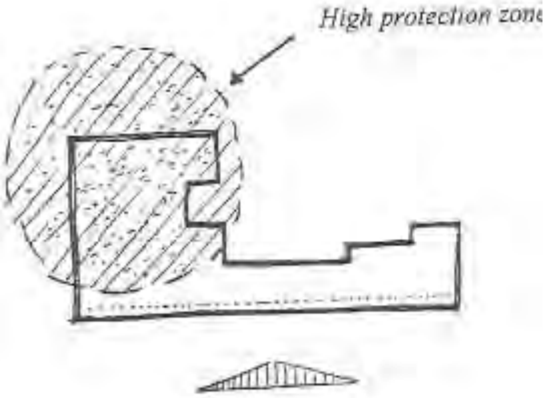
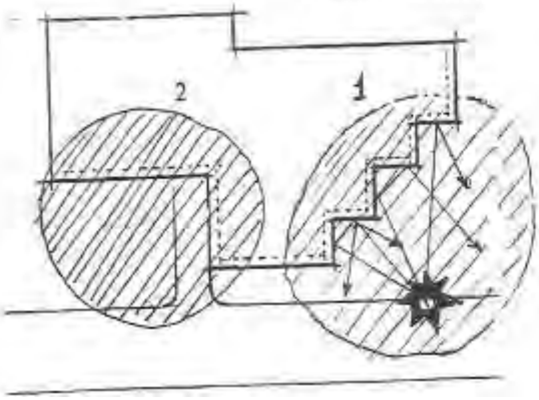


Figure 4.10.c Regions 1 and 2 need special cladding treatments as both regions will suffer from multiple reflections.



This is to avoi... ...ons that weaken the mass. The load will be concen... ...resses created by these multiple reflections. The threat directio... ...ce some of the concepts might be well protected from one direction, whe... opposite direction it will not be provided with the same degree of protection.

2. Positive and negative vertical space arrangements

Figure 4.11.a, b and c show some inward horizontally staggered concepts. When using such concepts one must consider the use of stiffer materials as shown by the given dotted line

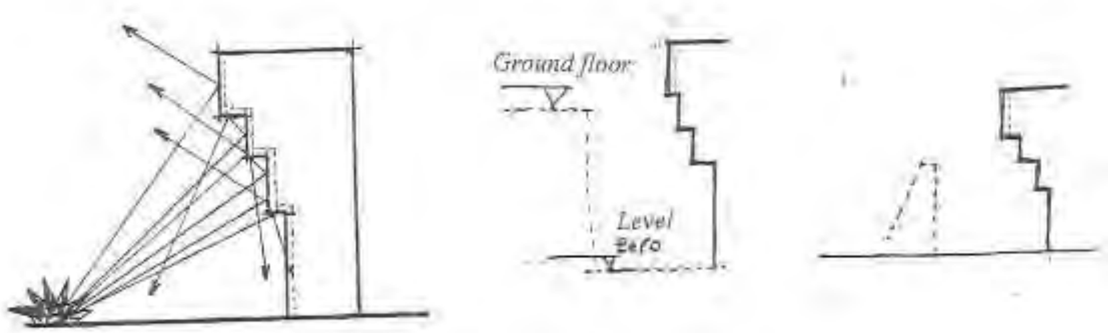


Figure 4.11.a Cross-section treated with a corbel treatment, the upper floor comes outward and the rest of floors are recessed inward as one goes down towards the ground floor level. The dotted lines indicate the need for special cladding treatments.

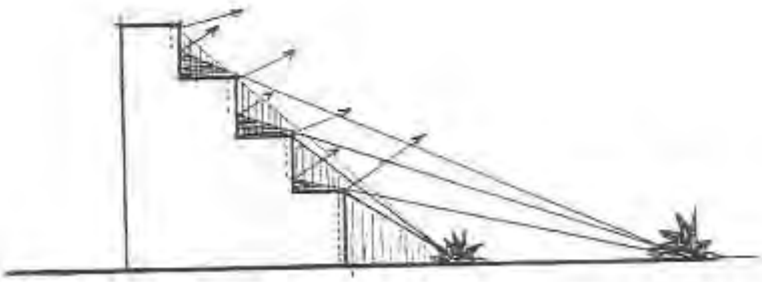


Figure 4.11.b The use of recessed floors as one goes upwards, the concept provides good protection at the upper floors. The dotted line indicate the parts of cladding which need to be treated against blast load.

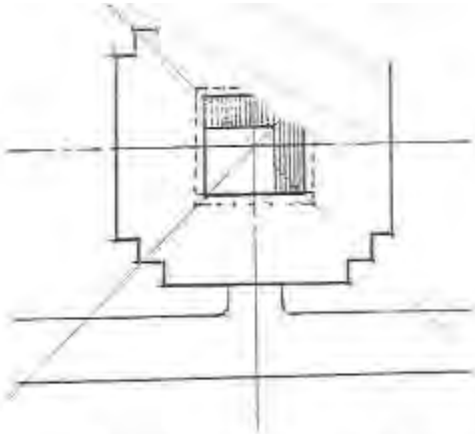


Figure 4.13.a Layout for a building showing the open court. The dotted line is to indicate that the cladding surrounding the court must be protected.

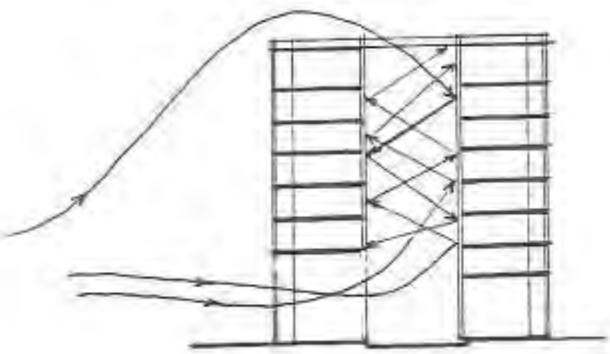


Figure 4.13.b When the top floor is left open a chance for stress waves to penetrate the court opening is provided.

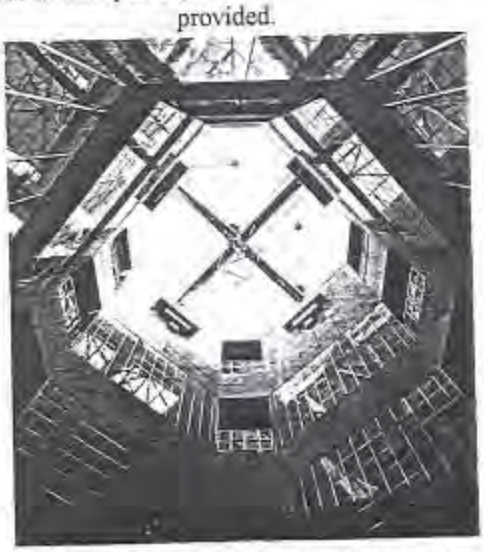


Figure 4.13.c Kisho KuroKawa, Saitoma Museum of Art 1981-82, interior *(Jencks, 1993)*. Using indoor open space it is advised to minimise the openings as shown.

- The court must be closed from the top with either ... able roof or pitched roof, in order to not to let the stress wave came through the top, Figure 4.13.d.
- In plan, sawtooth-like walls are preferred to be used in protection of the inner walls, as shown in Figure 4.17

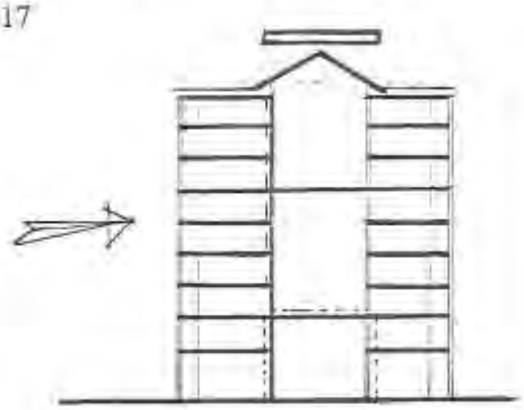


Figure 4.13.d Cross-section showing suggested treatment for the court by closing openings in the direction of the threat, dividing the height into three and closing the top floor with either double roof or pitched roof.

4.5 Enclosure system design and articulation

1. Enclosure system properties

Our recognition of the shape, size, scale, proportion and visual impact of a plane is motivated by its surface properties in addition to its visual context. A clear contrast between the surface colour of a plane and its surrounding field can resolve its shape. Reducing its tonal value can increase or decrease its visual impact. Members of known size within a plane shown in Figure 4.14 can assist our appreciation of its size and scale.

Figure 4.14 Elements of known scale aid our perception of the surface size and scale, *(Ching, 1996)*.

Figure 4.15 The surface texture and colour affect its visual impact, *(Ching, 1996)*.

2. Positive and negative cladding forms with surface geometrical treatments

As *Mays and Smith, 1995* recommended one has to avoid the use of deep profiling, as shown in Figure 4.16, as such features result in the concentration of stress waves. Figure 4.17 and Figure 4.18 show methods of cladding hardening using either a sawtooth like form or a clustered grouping of double height units.

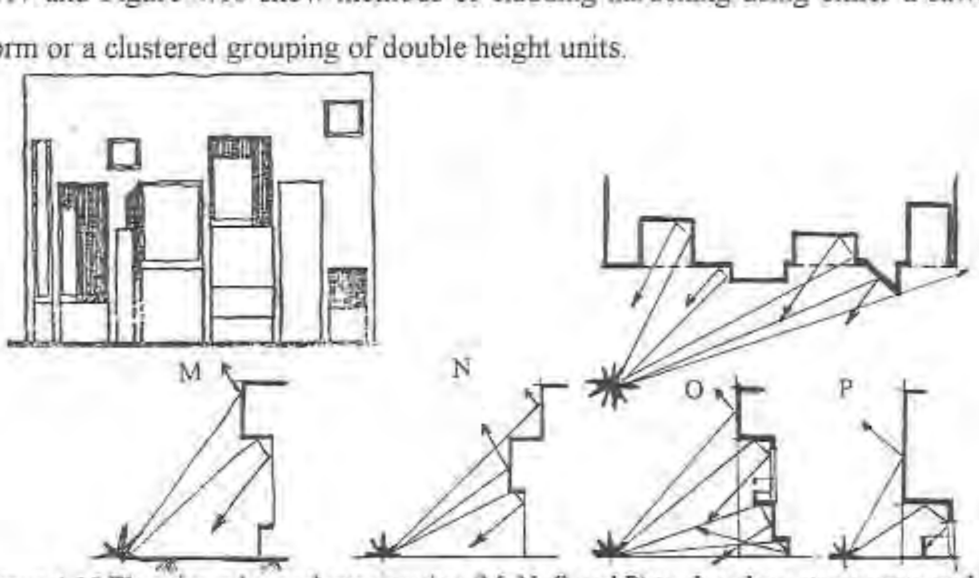


Figure 4.16 Elevation, plan and cross section (M, N, O and P) to show how stress wave could be caused to concentrate by features at the elevation. When using such features it is necessary to harden the cladding.

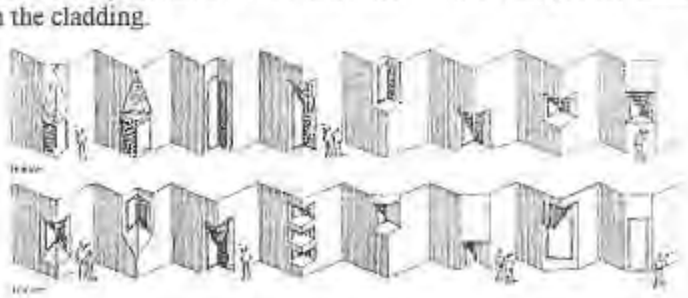


Figure 4.17 Forms of sawtooth-like plane cladding with various window treatments. When constructed with insitu concrete, the wall could be used as a protective wall.

polycarbonate protector w… …ame and suspended from two steel cables

Figure 4.19 shows the consequence… …ding without blast resistant windows. Where Figure 4.20 shows t… …en using the protector.

Figure 4.19 The consequences of a bomb attack on a building without blast resistant windows, *(W. Keenan and G. Meyers, 1993)*.

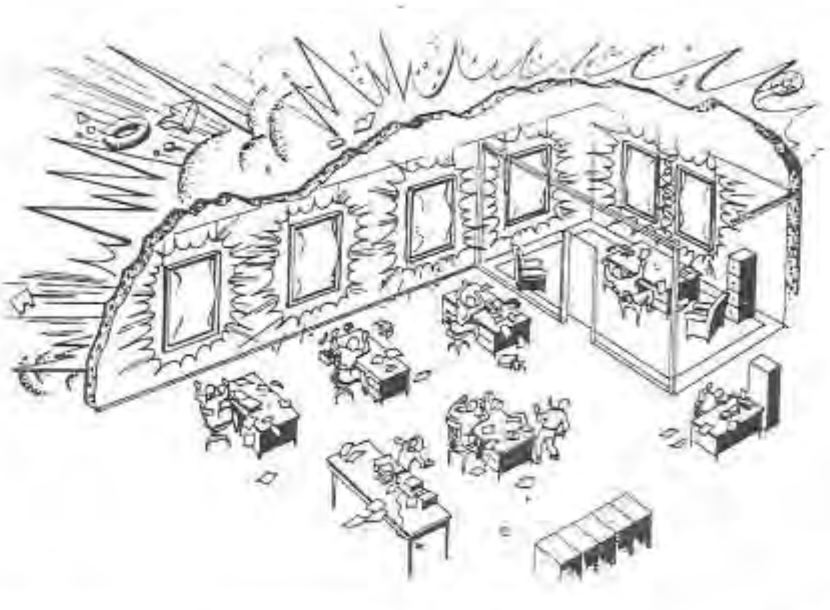


Figure 4.20 The consequence of a bomb attack on building with suspended polycarbonate shields *(W. Keenan, G. Meyers, 1993)*

3.Model geometry

a.Charge level

Figure 4.31 shows 5 different parameters which w. . as variables for the groups of simulations, the Bern or the ditch maximum width is 2m and it is considered to be constant:

1. The increase vs decrease in height of the near-side ground profile face which is ± 3m from the datum level.
2. The increase vs decrease in height of the off-side ground profile face which is ± 3m from the datum level.
3. The distance of explosive threat from the structure which range from 20 to 50m.
4. The Pavement width +1m (the distance from the explosion centre to the first predicted surface will receive the blast wave i.e. the near-side of the ground profile from the blast.
5. The height of the structure which is not more than 25m.

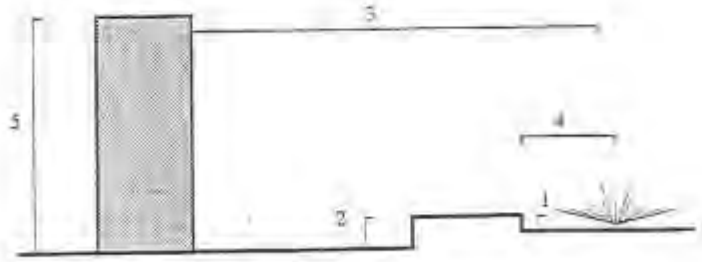


Figure 4.23 The concept different parameters after *Barakat et al, 1997*

Different landscape element levelling treatments are examined in relation to the building datum level. Three main arrangements are examined:

- The explosive threat level (base of vehicle) at datum level. Figure 4.24 illustrates the

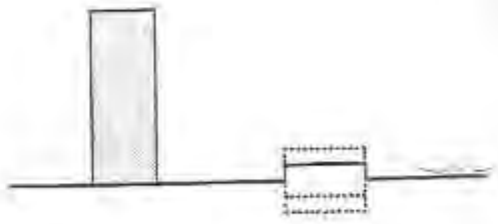


Figure 4.24 The structure zero level and the landscape element at the same level, *Barakat et.al, 1997*

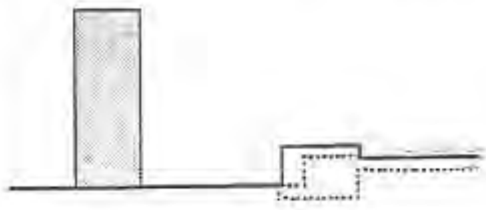


Figure 4.25 The level of the explosive charge is higher than the zero level of the structure, (*Barakat et.al, 1997*).

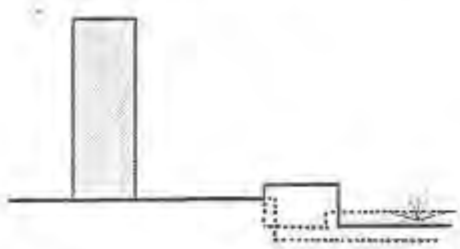


Figure 4.26 The zero level of the structure is above the level of explosion, (*Barakat et.al, 1997*).

The above three cases were applied with respect to two concepts.

b. The landscape element profile

The outline of the landscape element can have two criteria

- A bank form- at 1, 2 and 3 m above datum.
- A ditch form- at 1, 2 and 3 m below datum.

The width of the bank or ditch is kept constant at 2 m.

c. The distance from the explosive charge centre to the ditch or bank

Distance of 3,4 and 5 m were simulated +1 m (centre of parked car) above ground level.

Four distances were considered (20, 30, 40 and 50 m) and applied to each ground profile.

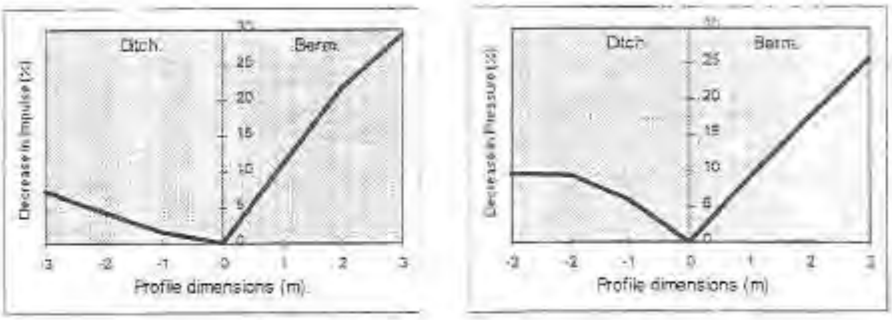
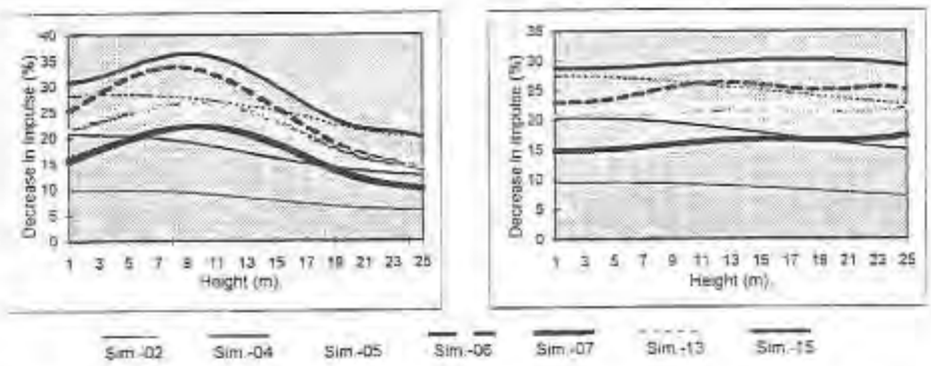


Figure 4.27 The decrease in impulse and pressure values (%) vs profile levels (m), after *Barakat et. al, 1997*

Figure 4.27 shows how typical values of impulse and pressure vary depending on the berm or ditch level.

4. Results

Figures 4.28 and 4.29 represent the change in impulse and at 20 and 30m distance between the explosive threat and the structure. The values of the impulse reduced in some cases by 35% whereas, the values of the pressure was also reduced in some occasions to 40%. Table 4.2 gives the different ground profiles for the simulations shown



Structure at 20m from the explosion

Structure at 30m from the explosion

Figure 4.28 The change in Impulse vs structure Height (m)(*Barakat et.al, 1997*).

The paper examined the pressure and impulse delivered to buildings of different heights up to 25 meters. Figure 4.29 shows the variation in pressure for two charge ranges.

caused by loading, and stresses induced by volumetric changes due to temperature and shrinkage. The different forms of reinforcement are:

- Fibres.
- Wire mesh.
- Plain and deformed bars.
- Post and pre-tension cables.

The different types of fibres used in reinforcement are steel, glass, or plastics. They are ideal for thin facing panels. Fibre reinforcement increases flexure, tensile, and impact strengths. Concrete is classified by either the type of placement (cast-in-place) or the type of reinforcement (conventional, pre-stressed, or fibre). Cast-in-place could be used as an exterior cladding but the majority of concrete cladding is made of precast.

When varying the shape of the mould together with the colour and texture of the exposed surface one can get a wide range of aesthetic expressions. The tensile strength of concrete is low compared to its compressive strength; therefore the design emphasis for concrete components is to minimise tensile stresses.

Flexure, diagonal tension or differential strains are expected to be carried by steel reinforcing bars. Cracking is one of the consequences of tensile stresses which can lead to a decrease in durability. FRC (fibre reinforced concrete) is a technique that provides the concrete with greater tensile stress and ductility. FRC is a mixture of portland cement (with or without aggregates of various sizes) in addition to discrete fibres such as steel, glass, organic polymers, ceramics, and other organic material.

Glass fibres are in a chopped strand form. Kevlar chopped strand is an aromatic polyamide which has both high tensile strength and high module of elasticity. Fibre can be first introduced into the concrete mix in a number of forms. First, by standard concrete mixers in which 3% of the volume of fibres can be added. "Balling up" of the fibres is however a problem. Secondly, by the spray - up method which is often used to produce thin sheets. In this method 10% to 15% of the volume of fibre can be introduced. The concrete mix of FRC is:

- Higher cement content.
- Pozzolan to increase the paste content without increasing the cement.

- Higher air content is needed.
- The maximum aggregate size is 10 mm in addition to a ratio of fine to coarse aggregate used.

Glass -Fibre- Reinforced - Concrete (GFRC), is a material with good mass appearance and durability. Its advantages are:

- Reduction in weight.
- Adds flexure, tensile and impact strength.
- Provides the designer with a variety of shapes and finishes.

Stucco and exterior plastic are field mixed and applied to materials composed of cement, aggregate, and water. The cement may be Portland, masonry, or gypsum, and the aggregate is usually sand. There are many types of admixtures to be used with stucco and plaster materials.

These include plasticzers, air entrainment materials, accelerators, water repellents, bonding agents, and colouring pigments. Wide variations in both colour and texture can be achieved with the placement of the finishing coat.

2. Masonry materials

In the past, buildings were built with massive load-bearing walls. These walls were different from the thin veneer walls typically used for cladding today. Masonry is now used as a type of veneer, which, if not properly designed, the following aspects must be considered:

- Volumetric changes which occur in the masonry unit are accommodated by the back up support system, and the primary structural system.
- Water prevention must be properly detailed to stop water infiltration through the cladding.
- The relative performance in bending of the exterior masonry and the back-up support system needs to be analysed.

Steel studs should be used as a support framing for masonry veneers. Cavity walls are suitable for resisting rain penetration, concrete units in single- width or double - width are used, Figure 5.1.

Sandwich panels are manufactured by facing both sides of a rigid insulation or non-solid core, such a paper or metal honeycomb, with metal sheets. Most metal cladding is installed as a curtain wall, either in an exposed grid frame or back fastened to an interior grid frame system.

5. Glass materials

Glass is usually installed into the exterior cladding of tall buildings in two forms: in a curtain - wall type frame, or as window frames in other types of exterior cladding systems and materials. The type of glass will define its framing, and seals and gaskets must be properly selected and specified by the designer. The designer and the manufacturer's engineer are responsible for specifying the criteria for the structure, water penetration, air leakage, heating, light and sound transmission performance, fire resistance, and provision for thermal movements.

Glass used for exterior cladding is manufactured with a variety of additives and combination of layers, which improve the material's characteristics, its strength, appearance, and heat and visual performance. The common properties of glass are transparency hardness, brittleness and chemical inertness. Glass can be coloured by means of additives. When tempered by heat its strength increases; it is patterned by etching or sand blasting. When varying the chemical make up or when combining sheets of glass and adding different films or coatings one can get additional strength , insulation, heat-absorption and glare-reduction. Double or multiple pane glazing is now commonly manufactured by combining the required number of glass sheets with an air space between them. Primary and secondary sealant are used to prevent moisture accumulation between the sheets.

6. Plastic materials

Plastic is a product of synthetic origin which is capable of being shaped by flow in some stages of its manufacture making it different from rubber, wood, leather, or metal. Plastics used in cladding fall into the general categories of sheet plastics and polymer modifiers. Sheet plastics are used as alternative materials for glass and metal siding. Polycarbonate, whether clear or patterned and textured, is used to create dramatic

Pre-cast concrete panels support framing are integrated to the exterior material in the form of reinforcement. Glass and metal curtain walls support framing are integrated to the design in form of mullions and grids which frame the panels. The designer needs to design the attachment of the mullion and grids frame system to the structural frame.

There are two basic types of support framing, the first is the grid-type frame used for metal and glass cladding but also for other materials, such as stone and pre-fabricated panels of masonry, metal plate, synthetic plaster, and GRC. The grid frame is usually aluminium extrusion or structural steel shapes attached to the structural frame of the building. The second is the back-up wall with an enclosure wall onto which the exterior material is attached. There are two types of back-up walls: masonry and sheathed stud walls.

The back-up wall can be used as a portion of the structural frame, in for example, pre-cast concrete back-up walls. The masonry back-up wall is built by laid -in-place concrete blocks with the exterior material attachments built into the wall as the wall is constructed. Shelf angles are attached to the structural slab to carry the weight of the exterior material. Steel stud back-up wall framing was an improvement for the brick over wood-stud construction. Steel studs are lightweight, easy to install, and capable of being prefabricated and panellised.

Stud deflection must be considered as a panel is allowed to deflect when using steel studs as a back up wall for brick masonry. The maximum allowable deflection is 10 mm for a (6.00-7.20) m wall panel. The different factors affect the support frame design are:

Factors affecting support frame design

Live loads

- Seismic loads
- Loads due to thermal and moisture extends
- Loads exerted from structural factors, depression, deflection, deformation

Dead loads

- Frame
- Panels
- Finishes

important problems which appears, due to the differences in temperature between the outside and the inside the building. Cold interior surface temperatures on the window are the normal cause of condensation in these assemblies. The use of insulation requires the use of an air and a vapour barrier to prevent the passage of moisture vapour to the location of the dew-point temperature.

4. Joints

The primary purposes of joints are either for ease of construction, such as between panellised systems, or for providing compensation for movement. Joints and sealant for each cladding material and their effect on the panel geometry are different.

5. Internal drainage

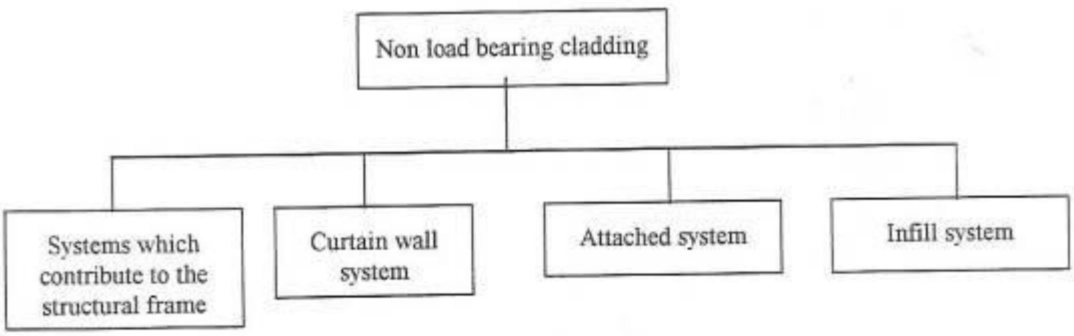
Rain, in combination with high winds, can cause water to penetrate through the wall by trying to find its way to and collecting at the joints. Using an internal drainage system can provide the cladding with a second line of defence. The internal drainage system is designed on the premise that the wall will leak, but when it occurs, the water can be collected, prevented from reaching the interior finishes, and channelled to the exterior of the wall. Gutters and drains are devices for flashing and collection of water behind the joints. The internal drainage is used as a back-up system to the sealant.

5.4 Cladding systems

Cladding systems can be classified according to their structural functions or according to how they are attached to the structural frame. The two main cladding systems are:

- Non load bearing cladding which does not share in the structure system of the building.
- Load bearing claddings which share in the building structure system.

Non load bearing cladding will be discussed in this section together with its different construction methods and its materials. The systems shown as follows:



A stud frame system consists of pre-manufactured studs assembled in a form of a frame work. This frame is supposed to withstand both dead and live loads. Figure 5.4 shows a typical example stud frame system.

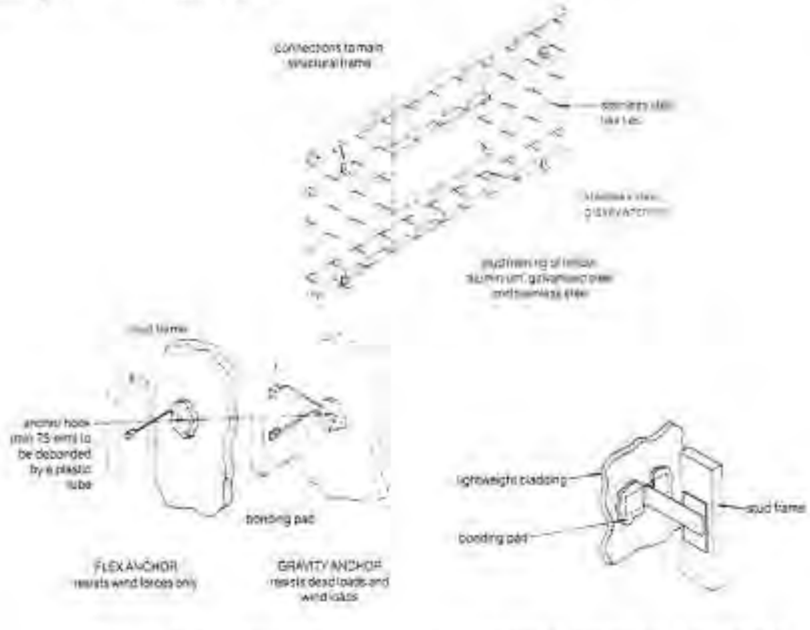


Figure 5.4.a Stud frame for window panel 6 m long x 3 m height.
b. Flex and gravity anchors.
c. Heavy hity gravity anchors. *(Taylor, 1992).*

Flex and gravity anchors shown in the figure are used to transfer the dead load of the outer skin and wind load on the panel. Figure 5.5 shows stud frame system connected to typical concrete frame building. The system consists of a lightweight skin.

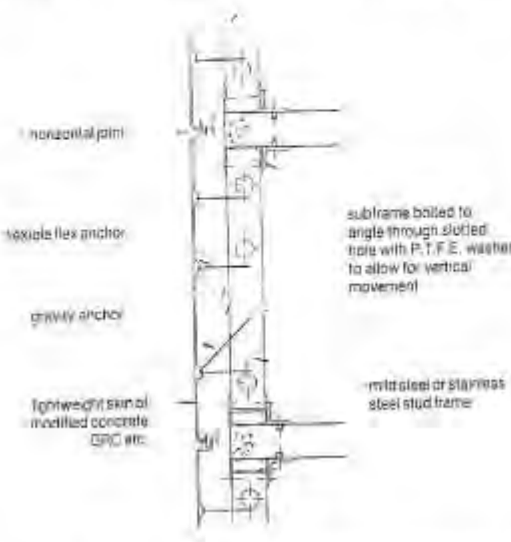


Figure 5.5 Typical stud fixing to concrete frame structure. *(Taylor, 1992).*

Metal or glass curtain-wall joints and connections are designed according to the behaviour of the cladding system and the material and structural frame characteristics.

The joints of curtain-wall systems utilising natural stone, pre-cast concrete, or combination of materials are designed in detail to prevent problems caused by differential movement, *(Allen, 1990)*. Curtain walls can be divided as shown below:

Curtain walls

Constructed in place

- With masonry back-up wall or back-up of light steel studs.
- With steel sub frame.

Pre-fabricated

- Stick system.
- Unit system.
- unit and mullion system
- Column cover and spandrel system.

a. Constructed in place

1) Masonry backup wall

Figures 5.9 shows an example for the use of a masonry curtain wall with back-up wall gypsum board for interior finishes installed to studs of either steel or wood.

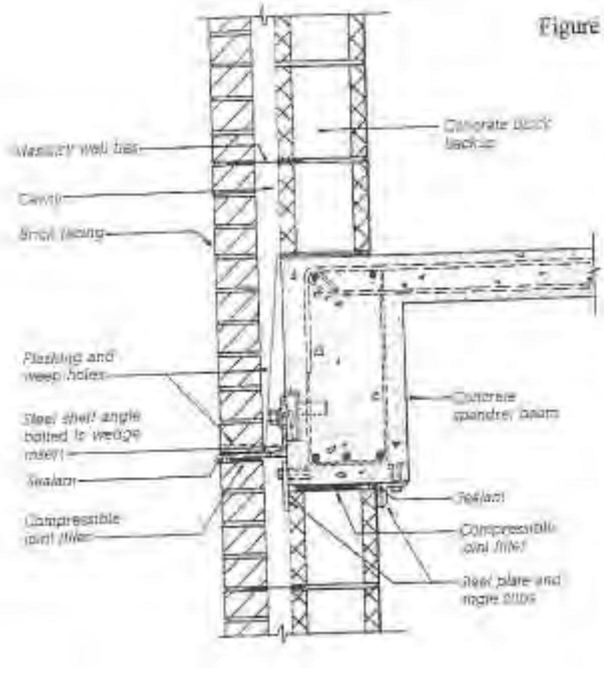


Figure 5.8 A simple brick curtain wall supported by a reinforced concrete frame. It is crucial that the horizontal brick joint below the shelf angle be filled with sealant rather than mortar, and that it be sufficiently thick to allow for any expected expansions and contractions in the masonry and the frame. This detail, used on thousands of buildings, effectively employs the rain screen principle. The cavity between the brick and the backup wall acts as a pressure equalisation chamber to neutralise air pressure differentials between the two sides of the masonry. Some building codes require that in seismic zones the wall ties be secured directly to wire joint reinforcing in the brick facing. The concrete masonry backup wall must be carefully engineered to resist both wind and seismic loads. This usually involves both vertical and horizontal joint reinforcing as well as the steel plate and angle clips that restrain the top of the backup wall from moving, *(Allen, 1990)*.

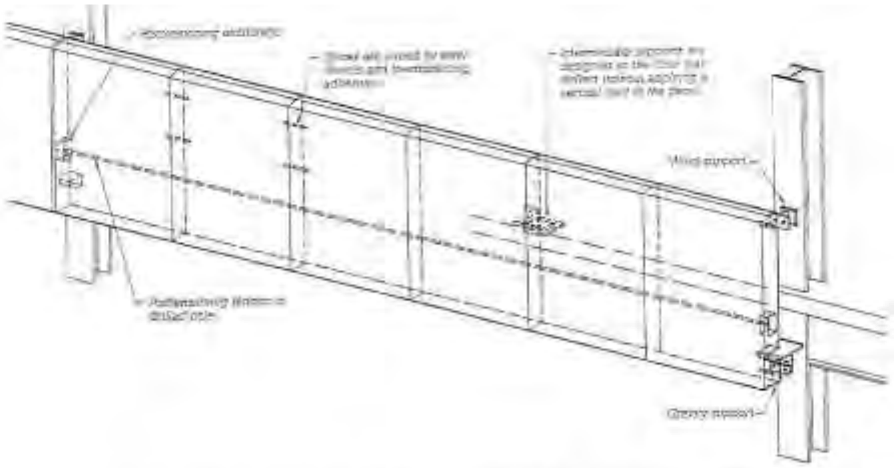


Figure 5.11 Thicker blocks of Indiana limestone may be post tensioned together to make spandrel panels that span from column to column but require little steel. The post tensioning tendon is inserted through matching holes drilled through the individual stones prior to assembly, *(Allen, 1990)*.

b. Prefabricated curtain walls

Allen, 1990 named four types of prefabricated curtain walls:

- Stick system.
- Unit system.
- Unit and mullion system.
- Column cover and spandrel system.

1) Stick system

The components of this system are: longitudinal mullions with rectangular panels from glass or any material that could be adjoined in place. The advantages of this system is the low cost of transportation and the ability to be adjusted by simple instruments. Figure 5.12 shows an example of stick systems.

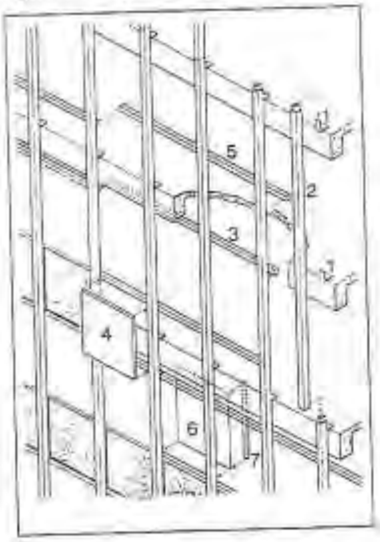


Figure 5.12 Stick system-Schematic of typical version
1: Anchors. 2: Mullion. 3: Horizontal rail(gutter section at widow head). 4: Spandrel panel (may be installed from inside building). 5: Horizontal rail (window sill section). 6: Vision glass (installed from inside building). 7: Interior mullion trim). Other variations: Mullion and rail sections may be longer or shorter than shown. Vision glass may be set directly in recesses in framing members, may be set with applied stops, may be set in sub-frame, or may include operable sash, *(Allen, 1990)*.

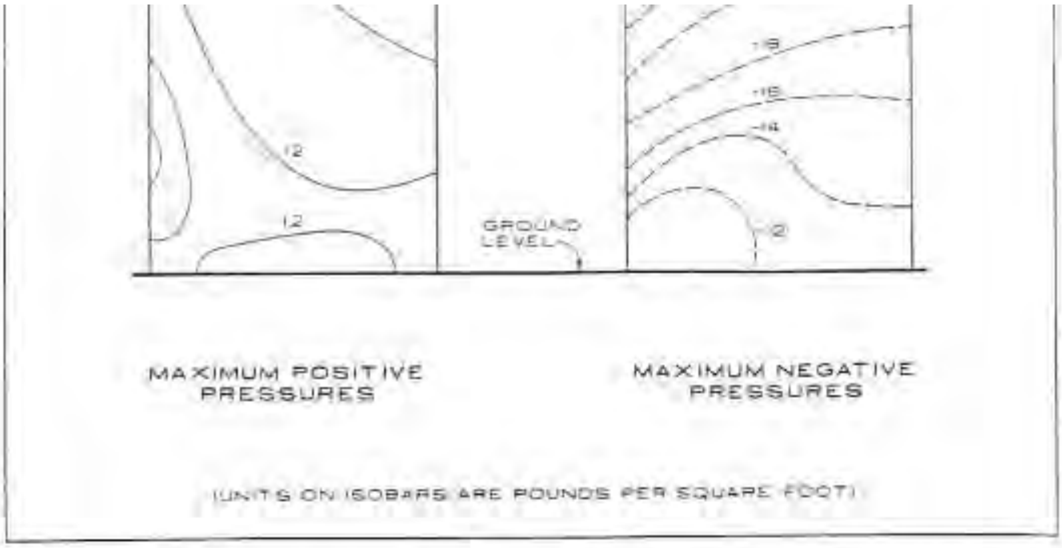


Figure 5.16 An example of expected positive and negative wind pressure on the cladding of a tall building, shown here in elevation, as predicted by wind tunnel testing. The building in this case is 64 stories tall and triangular in plan. Notice the high negative pressures (suctions) that can occur on the upper regions of the facade. The wind pressures on a building are dependent on many factors, including the shape of the building, its orientation, topography, wind direction, and surrounding buildings, so each building must be modelled and tested individually to determine the pressures it is expected to undergo, *(Allen, 1990)*.

'he designer must also be familiar with erection procedures of the cladding systems and ıe integration of different materials that need more than one source of supply. There re many types of erection procedures which utilise different methods of insulation for art of the cladding system. The selection of the procedure for the erection of the ladding must take into account the materials to be used, the type of building and site, vork-force availability, contractor experience, and cost. "Panelization" and "piece by ıiece" are two cladding manufacture procedures which affect erection. The panellised ›rocedure is a specific lift method and attaches a panel to the building. Installation and nterior finishes may or may not be included on the panel.

The joint sealant and the cleaning of the exterior surface are the only operations required o complete the exterior enclosure of the building. The architectural precast concrete wall panel is an example of panellised cladding procedure.

The design of the cladding panel is heavily influenced by the decision to use the panellised procedure. The panel must be constructed or manufactured with the connections for attaching it to the building and means by which it can be lifted and handled during manufacture, shipment, and erection without interfering with the attachment procedure.

The design of the panel must be such that it can resist handling stresses which usually act differently from those which occur while the panel is in place, because the attachment connectors are usually in a different location from the lifting point, and it is the dead-load weight of the panel and not the wind forces that creates the stresses. The panellised procedure is influenced by the site and the methods used to construct the building. A crane limited by the panel type and weight and the building height, is used to erect the panels.

The two common means of attaching panels to the structure are bolted and welded connections. The piece-by-piece procedure for erecting cladding is completely opposite from the panellised procedure in that each part of each component is placed on the building individually. Two examples of this procedure are curtain walls and the conventional masonry wall. The piece-by-piece procedure is more labour intensive at the job site than the panellised procedure, and the quality control is usually not as high as it

b. The threat measured with respect to an explosive charge weight at specific distance, these values are proposed according to risk assessment.

c. Economical design must be considered.

d. The strength of materials will increase due to the high rate of strain permitting some plastic deformation.

When analysing the threat, one can predict the expected size and location of the explosion to protect against. The positive phase duration of the blast wave is then compared with the natural period of response of the cladding panel.

2. The response of cladding

As *Mays and Smith, 1995* said the ductility and natural period of vibration of any cladding panel or element will govern its response to a given explosion. As they also mentioned a tall building has a low natural frequency and therefore a long response time in comparison with the load duration.

Cladding elements, like other building individual elements (columns and beams), will have natural response times which might equate with the load duration. Ductile cladding elements made of steel or reinforced concrete will absorb most of the strain energy; "they can undergo substantial bending without breaking", *Mays and Smith, 1995.*

Brittle materials such as glass, brick, timber and cast-iron will fail abruptly with little prior deformation. Structure members of high flexibility, such as high-mass, long-span beams and floors can absorb a great deal of the energy brought by the blast load. Rigid elements like short-span light weight elements (conventional glazing components) are described as poor energy absorbers and can fail catastrophically.

The response of cladding is a function of material characteristics and the way such materials are going to be used. Reinforced concrete could be used to create bunker - like walls specially with in-situ concrete. Large scale structures have a better response than those of light weight constructions.

3. Cladding design precautions against explosive blast

The type of building being considered in this research is a typical commercial multi-story public building. The cladding ought to meet the fundamental requirement of the safety of g's occupants. The client has to decide upon the threat from which he or she es the building to survive and be subsequently repairable, as this will influence the ng design.

nber of points can be made at this stage in advance of further study:

e lowest level of protection is to enhance resistance of glazing elements.

e amount of glazing within an elevation should be minimised in order to reduce the amage of glazing and the resultant hazard.

ladding should be securely fixed to the structure with easily accessible fixings. This will allow inspection work to take place rapidly after the event and also allow easy replacement of the damaged parts.

The cladding system should provide for the easy removal and insulation of individual panels. This will eliminate the need to remove the whole cladding system in order to replace only one damaged panel.

hese aspects warrant further examination and will be covered by considering the fferent cladding material options (in Chapter 6) and, more specifically, the performance f glass cladding systems (in Chapters 7 and 8). The system related aspects such as ixings and the ability to remove individual panels for maintenance and repair will be carried forward as areas for further study.

5.7 The Secure Failure Concept for Cladding

1. Scope

As was stated previously, protection cannot provide a guarantee of safety. This is evidently reflected in the case of exceeding the designated threat level with which the cladding was assessed (low threat level). Accordingly, substantial casualties would be caused due to the disintegration of cladding. Eventually, a new system has to be introduced for designing the cladding for permitting a "secure failure". This is the name given to the concept. Undoubtedly, this system necessitates separate research work. Nevertheless, it is worth while to pinpointing here the main criteria controlling this system as follows.

causing any damage to the surrounding panels. Figure 5.18 shows a hypothetical example of a panel bay with a configuration for the expected movements that result from the preceding effects. These movements must be accommodated in both the early stage of design and manufacturing and after erection, within the dimensional tolerances shown in the figure.

c. Design of fixings to accommodate movement

Figure 5.19, 5.20, 5.21 and Figure 5.22 show applications of movement accommodation.

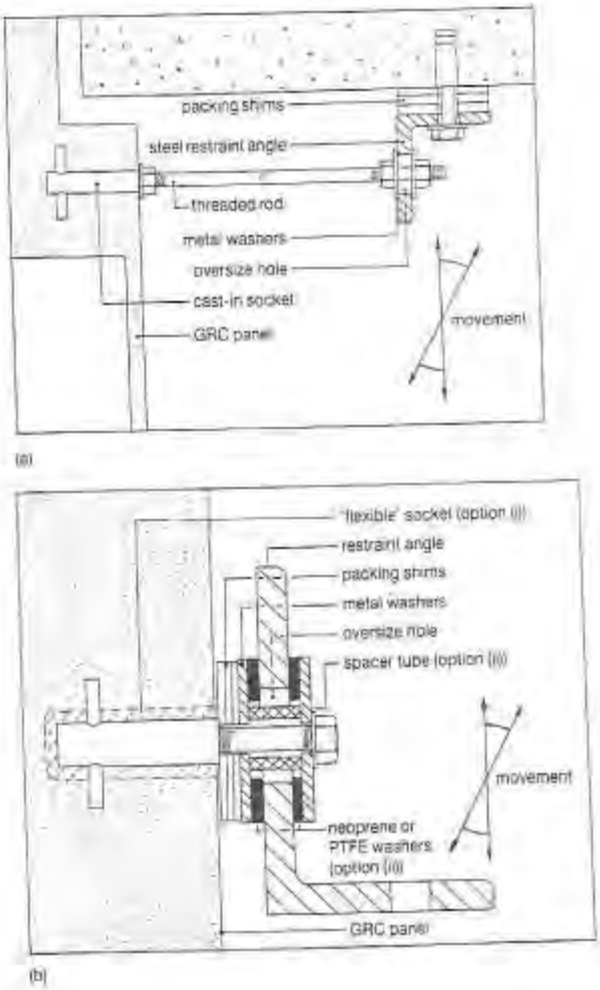


Figure 5.19.a Flexible rod type fixing detail.
b. Bracket fixing alternatives; (I) flexible socket, (II) sliding bracket, *(Taylor, 1992)*.

Two methods are suggested in order to specify a secure failure system in precast concrete. The first idea is to consider the improvement in the method of fixing precast concrete panels. The second idea is to use additional layers of reinforcement placed above the likely axes of failure.

Figure 5.24 shows concrete cladding panels that have been exposed to explosive blast. The explosive blast results in the damage of fixings and illustrates an area needing improvement.

Figure 5.24 indicates that attention needs to be taken of the precast concrete method of fixings.

Figure 5.25 shows how precast concrete panels are traditionally fixed. In this method, the panels are dependent on each other according to the way they are fixed. When a panel at the mid-highet is damaged, the complete height run above the damaged panel must be removed in order to replace the damaged panel.

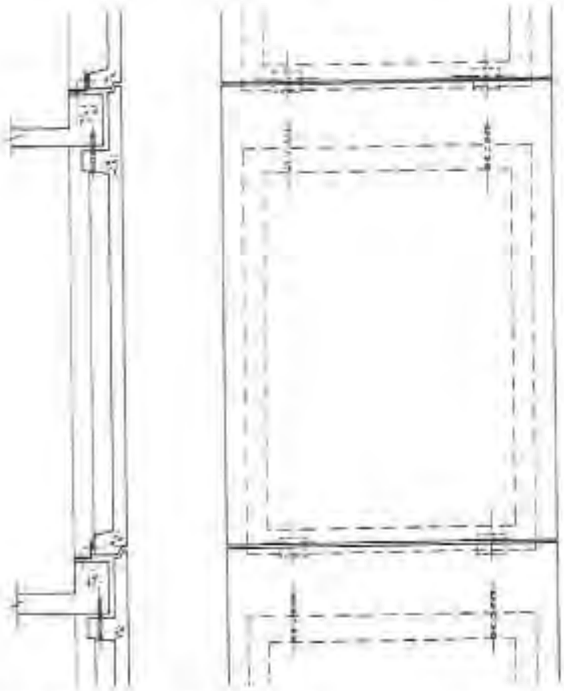


Figure 5.25 Typical loadbearing and restraint points, *(Taylor, 1992)*.

A New method for fixing panels

The proposed improved method incorporates a two dimensional grid of steel members. This construction provides an outer frame for the panel to be installedwithin. The method of construction also makes panels independent of each other. The steel grid may be constructed using steel box sections and angles, as shown in Figure 5.26. The steel element dimensions are illustrative and would need to be designed to counter a specific threat.

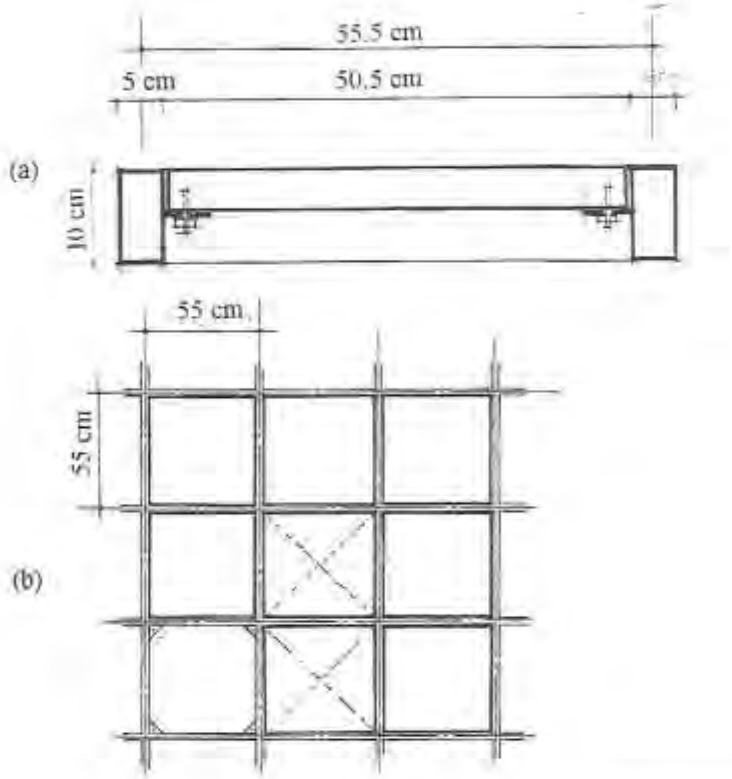
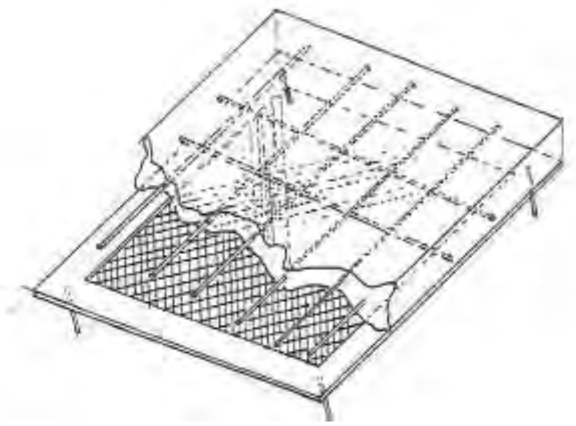


Figure 5.26.a Steel angle or steel box cross sections are used to form the two dimensional grid.
b. Layout for the grid design together with the panels.



(j) The use of wired mesh as a second layer of reinforcement

The additional layer suggested is a layer of wired mesh made of mild steel wires of 1 to 2 mm diameter arranged in a grid of 12.5 mm in both X and Y directions, as shown in Figure 5.30. Figure 5.31 shows that the mesh width is about 300-500 mm. The mesh centre to be placed above and parallel to the panel axis of failure. The two concepts of failure are shown in Figure 5.31.a and Figure 5.31.b, one and two way systems respectively.

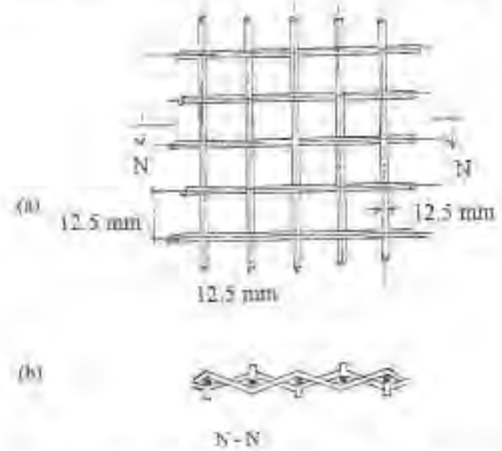


Figure 5.30 a Wired mesh concept.
b Detailed design of wire mesh.

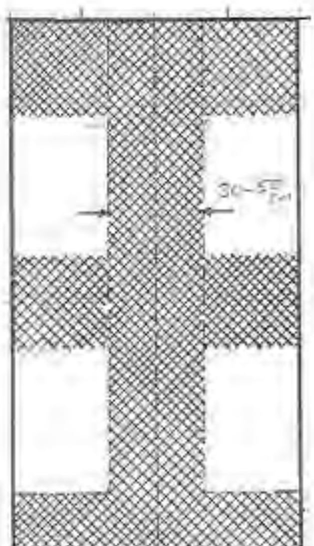


Figure 5.31.a The wired mesh placed in a panel of one way concept of failure, this concept is the specific location of fixings and it cannot be generalised.

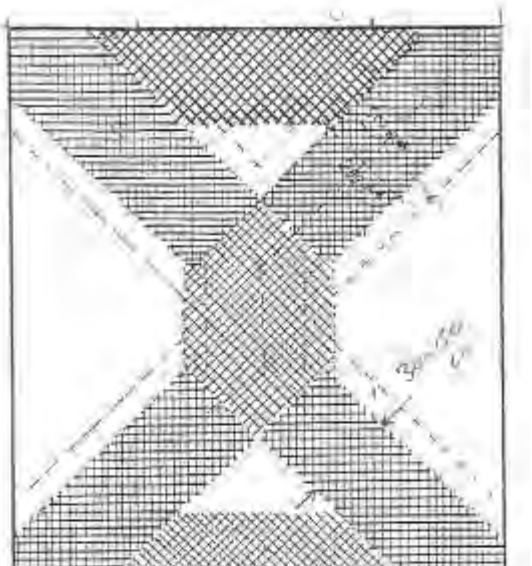


Figure 5.31.b The wired mesh placed in a panel of two concept of failure.

The second concept suggests the use of steel plate of thickness 1 to 2 mm with a width of 300-500 mm. This layer of reinforcement ought to be placed and connected with the main panel reinforcement. The axis of the plate takes the same direction and it must be above and parallel to that of the panel, and the fixing region of failure must be covered with the plate parallel to the lower edge of the panel. The two concepts of failure, one way and two way, are shown in Figure 5.32.a and Figure 5.32.b, Figure 5.32.c shows the details of the plate.

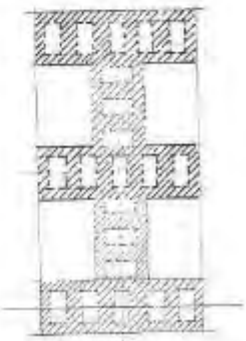


Figure 5.32.a. One way concept of failure.

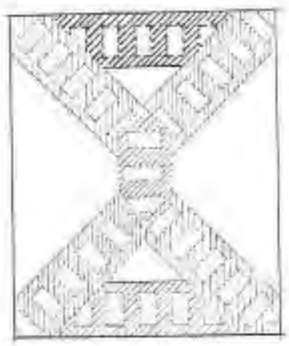


Figure 5.32.b. Two way concept of failure.

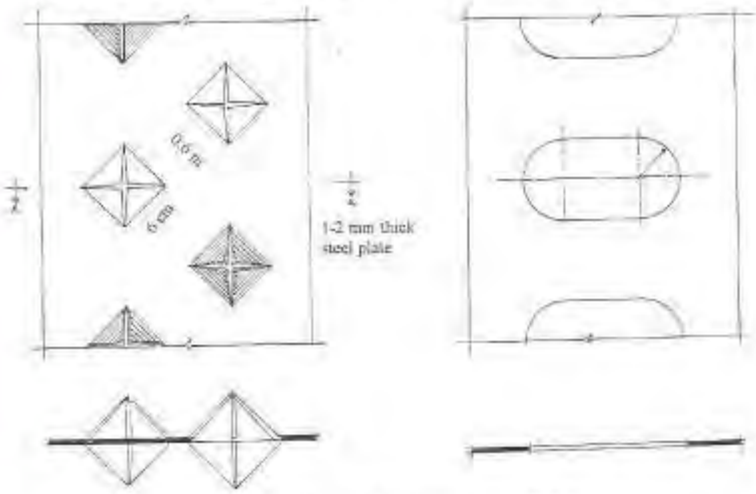


Figure 5.32.c The steel plate suggested detailed design.

Figure 5.32 The use of steel plate as a second layer of reinforcement in order to guarantee the panel secure failure.

The above proposals are speculative at present and are not based upon the work of others. It would be necessary to assess further the practicality and cost of the options proposed. For example, it would be necessary to review the weight of each panel type, as this will affect the dead loading carried into the structure. Also, prototype panels would need to be designed and manufactured and ideally, experimental measurements under blast loading made.

3. Masonry wall secure failure design

a. Masonry wall concept of failure

Figure 5.33 shows possible axes of failure of a brick wall supported on all four sides (after reference to *R. Harris, 1983)*.

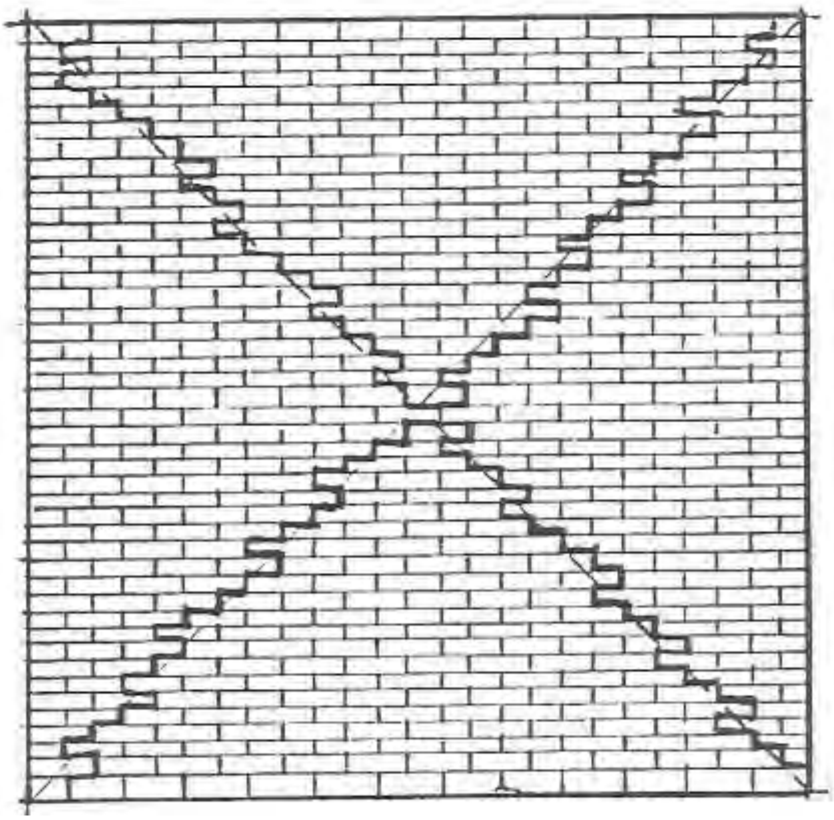


Figure 5.34 Grand Opera House, Belfast: some load-bearing masonry walls are unstable, while others remain relatively intact, an example for masonry wall structure failure, *(Mays and Smith, 1995).*

b. Suggested methods of hardening a masonry wall

Two methods are suggested to harden a brick wall and guarantee its secure failure. The ideology of secure failure is to keep the panel, when caused to fail in its place without flying in the form of fragments.

1) The use of exposed bracing

Figure 5.35 for Menier Chocolate Factory designed by Jules Saulnier and built at Noisiel-sur-Marne in 1872. The building is an early example of the use of a full skeleton frame in iron. The walls are hardened using slender iron members enclosing the brick panels in bold colours to make a contrast with the diagonal bracing of iron, *J. Musgrove, 1989.*

The degree of protection needed defines the need for additional bracing. Also the expected loads to be protected against will define the amount of steel needed within one module of the panel.

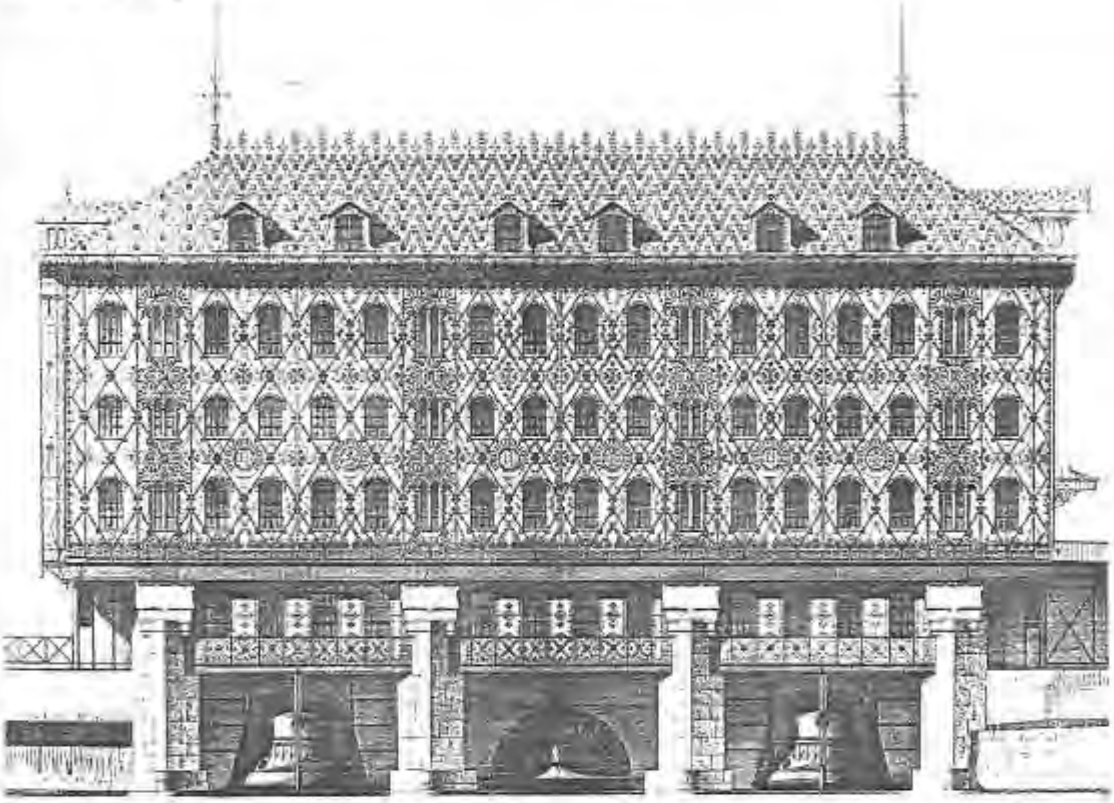


Figure 5.35 Menier Chocolate Factory, Noisiel-sur-Marne, 1872. *J. Musgrove, 1989.*

2) Partial masonry elevation protection

This concept would involve the application of an additional layer in front of the masonry wall, that could be sacrificial in the event of a blast. The protective layer would be suitably reinforced and could be decorative, even to the extent shown in Figures 5.36 to 5.38

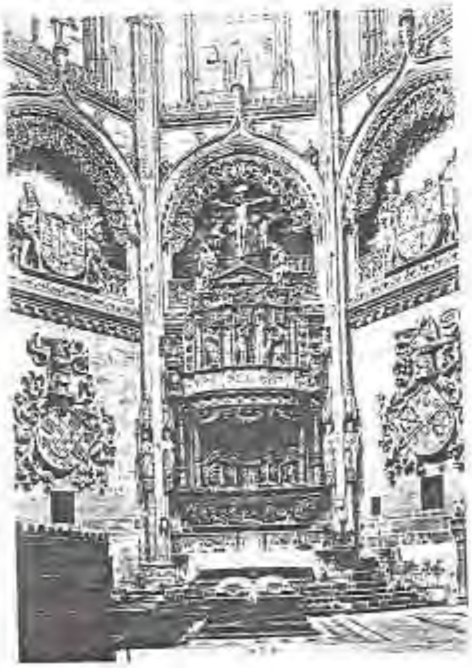


Figure 5.36 La Capilla del Condestable, Burgos Cathedral. *J. Musgrove, 1989.*

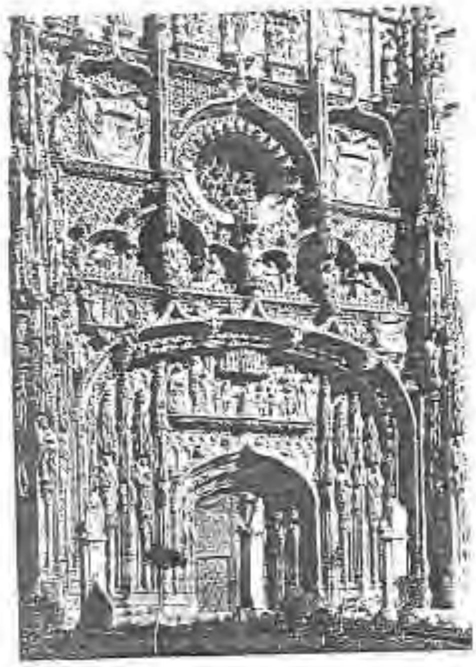


Figure 5.37 S. Pablo, Valladolid: principal doorway, *J. Musgrove, 1989.*

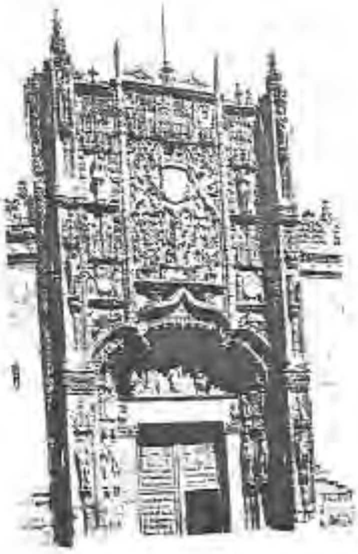


Figure 5.38 College of San Georgio, Valladolid, *J. Musgrove, 1989.*

The following given two treatments could be used to protect and harden a structure and the valuable space in the space behind.

First: *The use of cast steel*

These ornaments could be moulded and cast using metal material such as steel or stainless steel. This method provides a high degree of protection but would be very expensive.

Second: *The use of wired mesh*

This method is more economic than the above but the protection it provides is not as good.

These proposals have not been costed but the use of cast elements would only be justified for the most prestigious projicts.

4. Glass secure failure design

Glass secure failure could be achieved by several methods:

a. The application of transparent polyester anti-shatter film (ASF) to the glass inner surface.

b. The use of blast-resistant glass.

c. The installation of blast-resistant secondary glazing inside the (existing) exterior glazing.

d. The use of small size glass units as advised by *El.Kadi et al, 1997.*

Blast-resistant glass is made of either laminated annealed glass or laminated toughened glass. In case of double glazing, the laminated layer is the inner pane. Having used both panes, laminated glass will provide the glass with better performance. Frames needs to be designed together with its fixings to install the glazing.

The main goal behind glass protection is to minimise the amount of fragments created in order prevent personnel from being injured, and to protect the delicate equipment inside the buildings such as computer systems from being damaged; also, to prevent glass shards from penetrating the air-conditioning ducts. Blast resistant glazing reduces these shards by up to 90%, *(Mays and Smith, 1992).*

Figure 5.39 shows how laminated glass remains attached to the structure after the damage takes place.

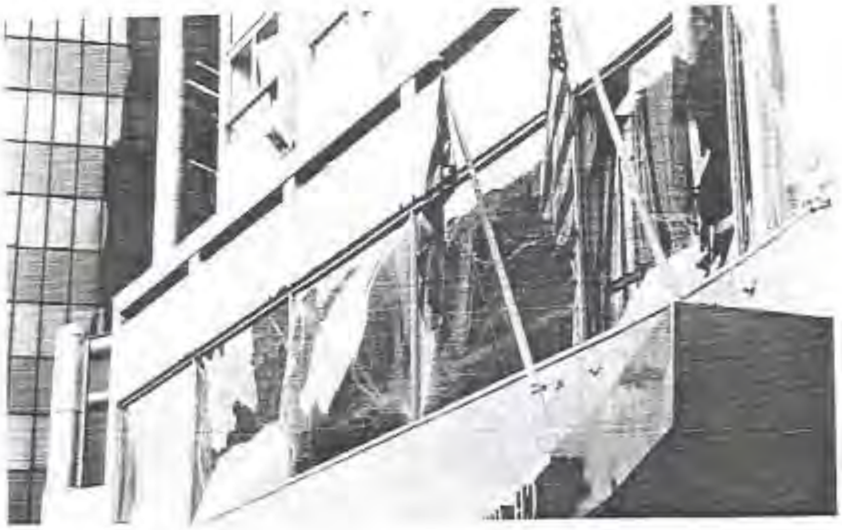


Figure 5.39 Europe Hotel, Belfast, laminated glazing panels are retained in robust frames desite large deflections, *Mays and Smith, 1995.*

5.8 Summary

This chapter reviews the various types of cladding and cladding support systems in use and the factors considered when selecting them. After this, the effect of blast on cladding and support systems is introduced. The concept of "secure failure" is discussed in more detail. The following Chapters study the aspects of blast resistance in a more thorough way.

1. Dynamic versus static yield

Cladding material is like any other material; resistance values are prescribed as a long-term or near static situation, where blast wave loading is considered as short-term. When applied to short-term loading (dynamic), cladding materials show resistance to deformation that is appreciably higher than that for long term loads. Reinforced concrete resistance to dynamic loads is up to 135-140% of the long-term value, and for structural steel the figure is about 155-160%. Figure 6.2 shows the ratio of dynamic to static yield resistance plotted, against the time to reach yield, *(Kinney and Graham, 1985)*.

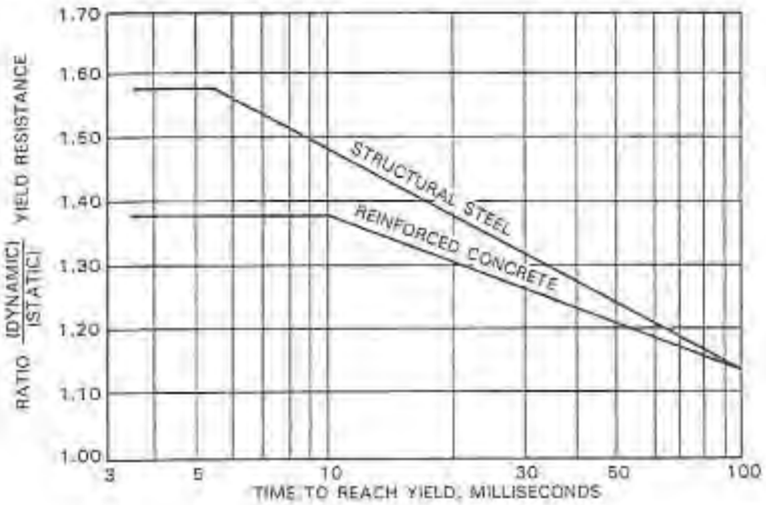


Figure 6.2 Short - term yield resistance (dynamic value) exceeds the long - term (static value), *(Kinney and Graham, 1985)*.

2. Ductile versus Brittle materials - The ductility ratio

A ductility ratio can be defined as the ratio of total elastic plus plastic deformation, that an element can survive without complete failure,to the elastic deformation alone.

$$Ductility\ ratio = \frac{total\ (acceptable)\ deformation}{elastic\ deformation}$$

Cladding materials are classified as either ductile or brittle. Ductile materials are examples of materials that show high ductility ratios without fracture. Brittle materials have a ductility ratio of one.

Ductile cladding materials are: steel, aluminium and reinforced concrete. These materials can suffer plastic deformation, and although permanent damage occurs, it does not represent total destruction.

On the other hand brittle materials are those materials which show fracture or total failure at low ductility ratios i.e. glass, GRP (glass reinforced polyester), brick or stone masonry bearing walls and unreinforced concrete.

3. Equivalent static loads

This chapter intends to study the resistance necessary to prevent a specified level of damage. In other words each cladding material has a specific resistance which enables it to resist a specific level of damage. The way a resistance behaves dynamically is important when designing for protection against blast. However this could be modelled by an equivalent static resistance. In such cases the expected dynamic load is converted to an equivalent static load.

It is important to note that the imposed dynamic load may be idealised to take the shape of a triangular pulse, when plotted with pressure-time co-ordinates and the resistance-displacement relation resembles the formalised curve (as in Figure 6.1). It is possible to represent a graphical solution for the equivalent static load. Figure 6.3 shows an example of this. The example includes the ratio of pulse time for the triangular pulse to the natural period of the structure, the ductility ratio at maximum permissible deformation and where the ratio of the equivalent static load to the peak value of the triangular pulse.

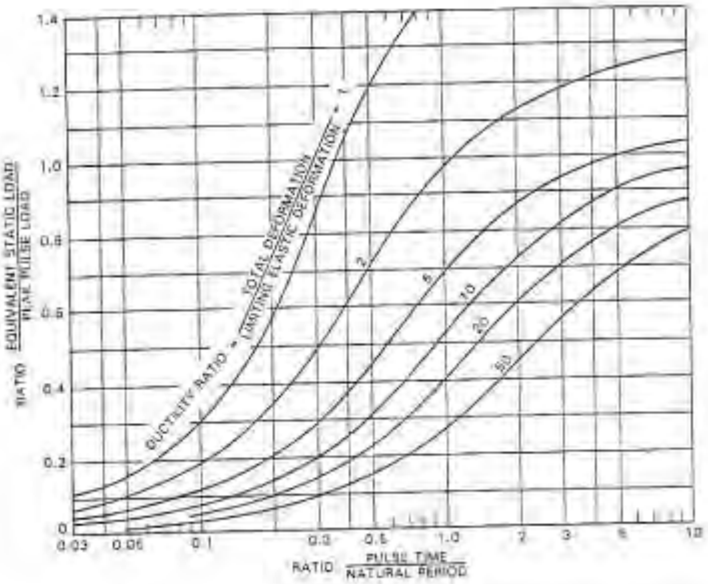


Figure 6.3 The equivalent static load developed by a triangular pressure pulse shown as a function of ductility ratio, pulse time, and natural period. *(Kinney and Graham, 1985)*

4. *The impulse - time criterion for blast damage capability*

The damage potential of an explosive blast depends on:

a. The force that could be exerted on a target.

b. The duration of force being applied.

c. The ability of the target to withstand its effect.

The blast damage capability depends on two aspects, those which are concerned with the blast wave and those concerned with the target. The damaging aspect of a blast wave depends on both its peak overpressure and its duration, that is its impulse per unit area. The second aspect is concerned with the target and the duration that the load is applied to it.

To better understand these aspects consider the following example given by *Kinney and Graham, 1985*: consider a displacement given to a simple pendulum when it is initially at rest. We have to note that the displacement of the pendulum depends upon the impulse received before it appreciably starts to move ie. this impulse is received within a short period of time. Additional impulses given to the pendulum when it has begun its return swing do not contribute to the maximum displacement. Therefore, it could be said that there is a critical time within which an impulse should be received for it to contribute to the maximum displacement. By extension, it can be deduced that there is a critical time within which blast wave impulses must be received by a target for damage to be inflicted. The critical time during which impulses should be delivered to the target in order to inflict damage is estimated as about one quarter of its natural period of vibration.

Figure 6.4 shows the nature of the impulse - time characteristic. Figure 6.4 a shows the impulse when the blast wave duration t_d is less than the critical time for damage t_c. In this case the blast wave impulse is represented by the hatched zone of the figure which contributes to blast damage capability. Figure 6.4b shows the case in which the blast duration t_d is longer than the critical time t_c. Here a segment of the total blast wave impulse contributes to blast damage capability. Figure 6.4 c represents a blast wave with long duration compared with a critical damage time.

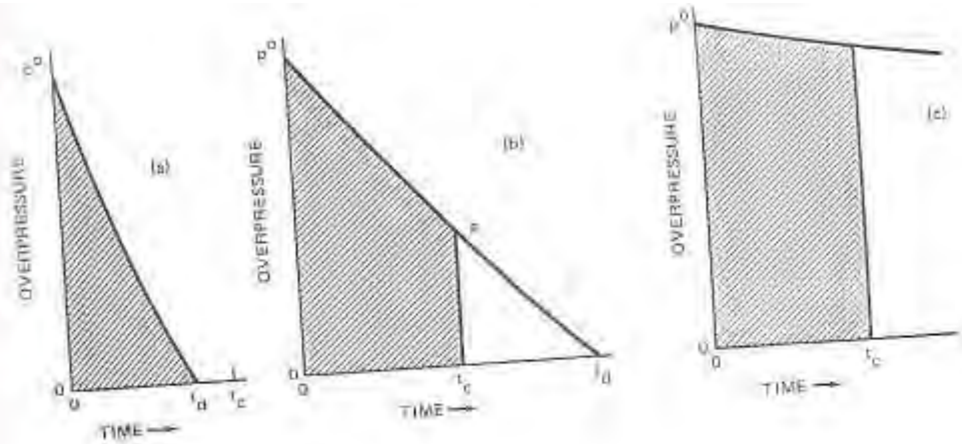


Figure 6.4 Damaging portion of blast impulse (hatched area).
a. Blast duration t_d shorter than critical time t_c.
b. Duration somewhat longer than critical time.
c. Duration markedly longer than critical time.
(Kinney and Graham, 1985).

6.3 Equivalent systems

The technique of (P-I) pressure - impulse curves will be applied to consider the behaviour of specific cladding panels by considering their response to impulsive and quasi-static loading (which can lead to the development of P-I diagrams). There are sequential steps to the solution of particular panels using this method.

One ought to select the suitable mathematical representation of the deformed shape for the cladding panel which satisfies all the necessary boundary conditions related to displacement.

As discussed in chapter 3, if the load is considered as quasi-static the work done on the panel must be calculated and equated with the strain energy generated. In the case of impulsive loading, the response is calculated by equating the kinetic energy acquired with the strain energy produced in the cladding panel.

1. Resistance function for specific structural forms

The resistance-deflection function is represented by a graph of the uniform pressure required to cause deflection at the central point of the element at its transient displacement.

ure 6.5 shows the resistance function of a one way reinforced concrete wall which is pported along two edges only and "fixed" at the supports. Segment OA of the graph ows how the slab elastically deforms.

t point A, the yield lines are produced close to the longitudinal supports (points of adding fixings are points of load concentration). The segment AB represents an elasto-astic phase. The increase in displacement during this phase is estimated from the astic stiffness of a simply supported one-way-spanning slab.

Segment BC shows the ultimate resistance of the element represented by a yield - line system established to form a collapse mechanism. This plastic phase will continue until failure at one of the supports takes place. The dotted line ODC represents a further indication to simplify the resistance deflection function.

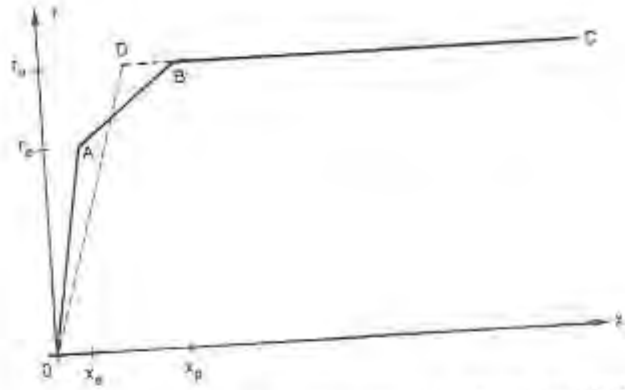


Figure 6.5 Idealised resistance to a one - way- spanning slab, *(Smith and Hetherington, 1994)*.

2. *Evaluating the resistance function*

The method by which the co-ordinates of points A and B in Figure 6.6 are found will be discussed. Point A which indicates the end of the elastic phase for the one-way-spanning slab is reached when the bending moment at the supports reaches its ultimate negative moment capacity.

"If the distance between supports is L and the slab is subjected to a uniform pressure P, the bending moment per meter runs at the supports as given in Figure 6.6 is $P\frac{L^2}{12}$.

Thus the pressure that causes yield r_e is given by. $r_e = \frac{12M_N}{L^2}$ where M_N is the

...ate negative moment capacity per meter runs of the slab at the supports. The ...ction at this stage x_e is found by simple structural analysis to be $5\frac{r_e L^4}{384EI}$ where I ...e second moment of area per metre run of the slab, assuming tension cracking has ...en place, and E is the Young's modulus of concrete", *(Smith and Hetherington, ...94).*

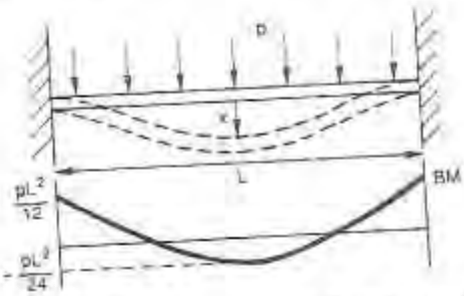


Figure 6.6 Uniformly loaded, one-way-spanning slab. *(Smith and Hetherington, 1994).*

...fter the determination of the co-ordinates of point A, we can determine r_u and x_p, the ...o-ordinates of point B. Here r_u is considered from the solution of the upper-bound ...lasticity, where the work done by the applied pressure is equated with the energy ...issipated in the yield lines where:

$$r_u = \frac{8}{L^2}(M_N + M_P) \quad 6.1$$

Where r_u is the ultimate resistance of the slab

L is the span of the slab

M_N is the ultimate negative moment capacity per metre run of slab

M_P is the ultimate positive moment capacity per metre run of slab.

The increase in deflection at the elasto-plastic phase is obtained by:

$$x_p - x_e = (r_u - r_e)\frac{5L^4}{384EI} \quad 6.2$$

Where x_p is the maximum deflection of the slab

3. Design loads

Because the blast loading is the expected threat in our design but it is unlikely to take place in reality, a factor of safety ought to be considered for the blast loading which could be equal to unity. A factor of 0.33 may be applied to the design values of the ariable imposed and wind loads when acting together with the blast loads, *(BS 5950: Part 1 and BS 8110: Part 1)*

4. Design strengths

When cladding panels are designed, the blast loading threat can be considered as one of the parameters. The strength criteria of the material must be taken into account. When loads are applied at speed, the rate of strain exercised increases and this will influence the mechanical properties of the cladding materials, (see Figure 6.2, earlier).

Yield and ultimate stresses are increased by varying amounts but the modulus of elasticity of both steel and concrete are relatively insensitive to the rate of loading. Also, the elongation of structural steel at faliure is similarly insensitive to the rate of loading, *(Mays and Smith, 1995)*.

5. Dynamic increase factor (DIF)

The enhancement of the design stresses may be done by applying a dynamic increase factor (DIF). Table 6.1 gives typical values of DIF for structural steel and reinforced concrete, *(Mays and Smith, 1995)*.

Table 6.1 Dynamic Increase Factor (DIF) for design of reinforced concrete and structural steel elements, *(Mays and Smith, 1995)*.

Type of stress	Concrete	Reinforcing bars		Structural steel	
	f_{dcu}/f_{cu}	f_{dy}/f_y	f_{du}/f_u	f_{dy}/f_y	f_{du}/f_u
Bending	1.25	1.20	1.05	1.20	1.05
Shear	1.00	1.10	1.00	1.20	1.05
Compression	1.15	1.10	-	1.10	-

Where f_y = yield stress of reinforcement steel

f_{dy} = dynamic yield stress

f_u = peak, or ultimate, strength of reinforcements

f_{du} = dynamic ultimate strength

f_{cu} = compresive strengh of concrete

f_{dcu} = dunamic compresive strength

Deformation or deflection is the main principle to be considered in the design of structural elements to sustain blast loading. When considering this criterion one should control the degree of damage the element should bear.
The damage level which may be allowed in any specific case will depend upon what the designer intends to protect; for instance, is it the structure itself, is it the inhabitant of a building or is it the equipment within the building?
There are two approches which lead to limiting factor deformations being obtained:

a. By specifing the maximum support rotation, θ permitted as shown in Figure 6.7.

b The ductility ratio.

$$\mu = \frac{total \quad deflection}{deflection\ at\ elastic\ \lim it} = \frac{X_m}{X_E}$$

Deformations in reinforced concrete elements are normally measured with respect to support rotations, whereas ductility ratios are usually applied for structural steel.

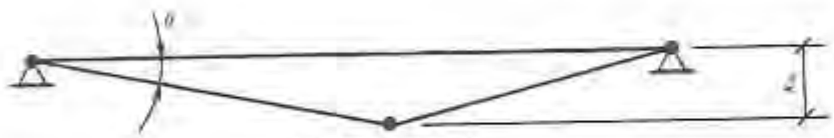


Figure 6.7 Member support rotations, *(Mays and Smith, 1995)*.

Table 6.2 *after Mays and Smith, 1995*, gives deformation limits for two proposed protection levels. Category 1 is considered as the limt to protect people and equipment within the structure. It includes an allowance for blast and fragment protection as well as protection from falling structural elements. Category 2 covers limitations to prevent structural collapse. Category 2 levels mean extensive plastic deformation. If the requirement is avoid repair, the deformations should be kept in the elastic range i.e.

able 6.2 Deformation Limits, *Mays and Smith, 1995*

	Protection category			
	1		2	
	θ	μ	θ	μ
einforced concrete beams and slabs	2°	Not applicable	4°	Not applicable
tructural steel beams and plates	2°	10	12°	20

= Rotation
= Ductility ratio

.4 The manner in which reinforced concrete and structural steelwork react to blast loading

. Structural behaviour of reinforced concrete

When a reinforced concrete element is dynamically loaded , the element deforms until the ime when its strain energy balances the energy the blast load. The element either comes to rest or the concrete breaks up. Figure 6.8 shows the resistance deflection curve and ndicates the flexural performance of a reinforced concrete member.

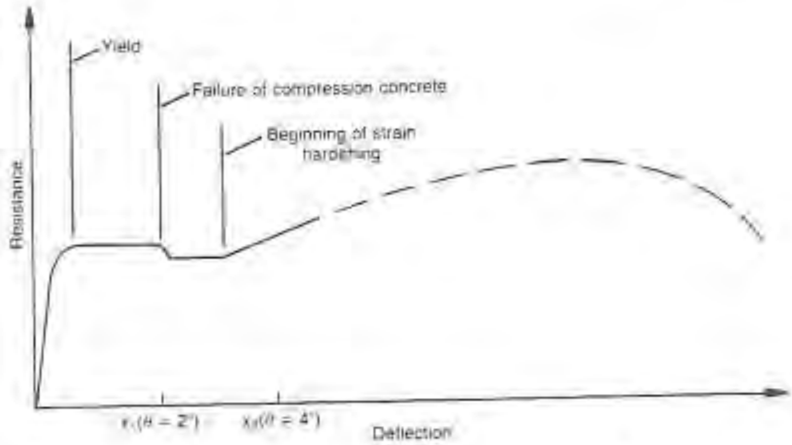


Figure 6.8 Typical resistance - deflection curve for flexural response of concrete elements. *(Mays and Smith, 1995)*

The initial, elastic, phase ends where the tension reinforcement yields. Then various stages of plastic deformation follow, with the member continuing to carry load. At a rotation of about 2°, the compression concrete crushes and spalls and the load is transferred to the compression reinforcing steel. This steel must be tied in using "blast links" so that it can carry the compression load.

e concrete section where the compression concrete has not crushed, i.e. $\theta < 2^{\circ}$, is signated a Type 1 section. The section where crushing has occurred is designated ype 2. This type will cover deflection up to the Protection Category 2 limit at 4" (see ble 6.2). The concrete sections are illustrated in Figure 6.9. For a Type 2 section, the eak moment capacity, M_p, for member of width b is given by:

$$M_p = A_s \frac{f_{ds}}{b} d_c \qquad 6.3$$

Where A_s is the area reinforcement on each side

d_c is the distance between the centroid of the reinforcement on both sides tension and compression.

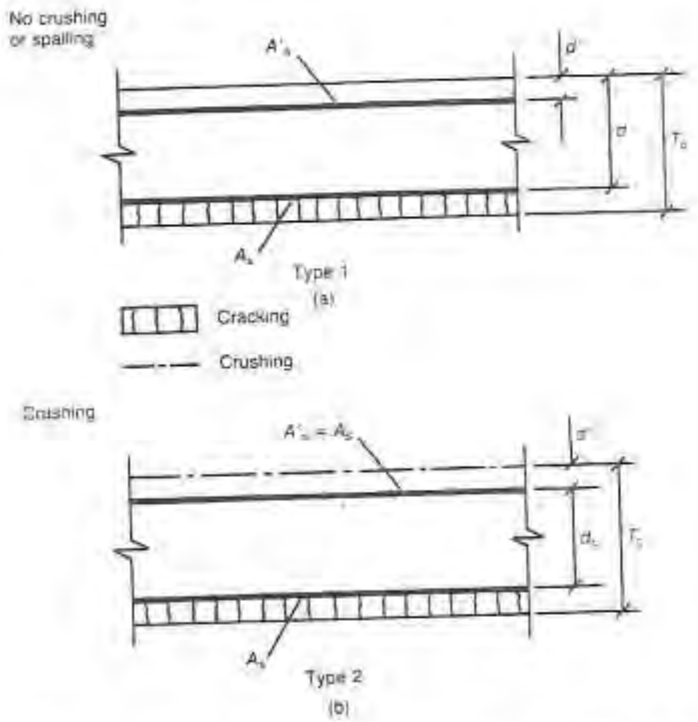


Figure 6.9 Typical reinforced concrete cross sections. *(Mays and Smith, 1995)*

2. *Structural behaviour of steelwork*

Structural steel displays a linear stress-strain relationship up to its yield point, after which it can strain considerably without substantial intensification in stress. This yielding can extend to a ductility ratio of between 10 and 15. After this, strain hardening occurs as shown in Figure 6.10. Eventually, further elongation leads to rupture at a strain of 20 -

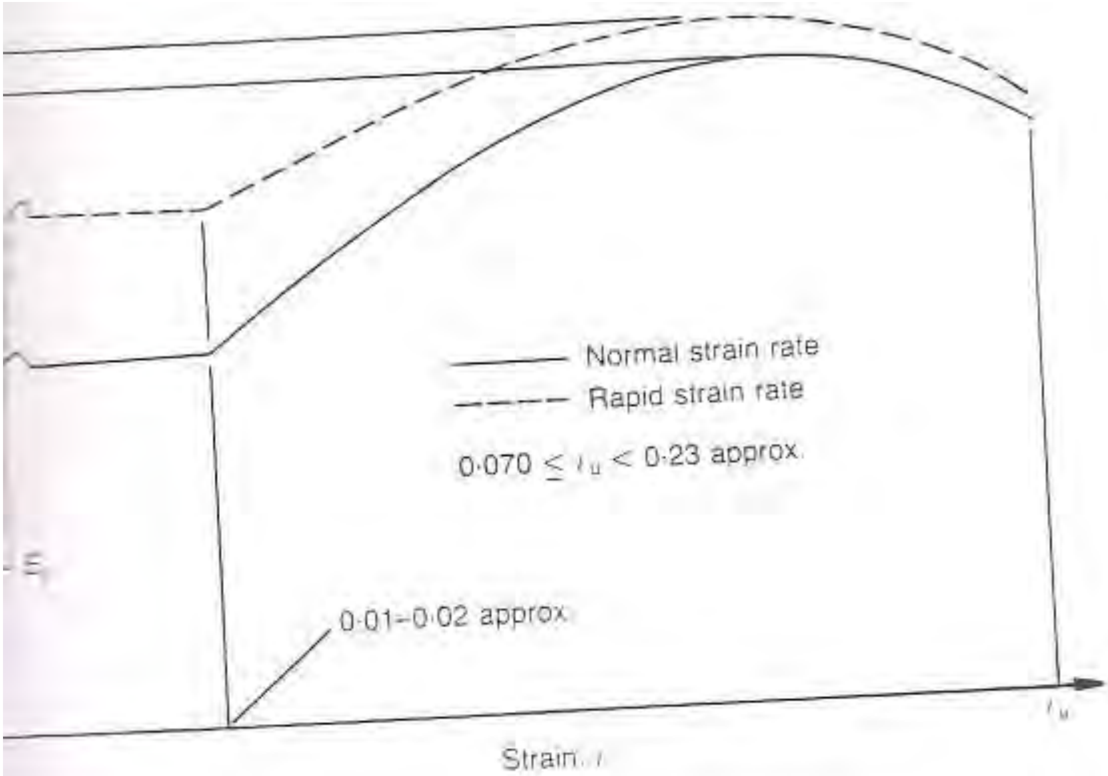


Figure 6.10 Typical stress - strain curves for steel. *(Mays and Smith, 1995).*

, the design plastic moment for steel member with $\mu \leq 3$ is given by:

$$M_p = f_{ds}(z+s)/2 \tag{6.4}$$

ere z and s are the elastic and plastic section modulus respectively.

r $\mu > 3$

$$M_p = f_{ds}S \tag{6.5}$$

Preface to the design of reinforced concrete elements to resist blast loading

M5-1300 recommends methods of design which will be the basis of this section (The otection category classification is after *Mays and Smith, 1995).*

. ***Design stresses.***

able 6.3 shows the applicable dynamic stresses f_{dc} and f_{ds} to be used in the design of e 6.3 Dynamic design stresses for reinforced concrete, *(Mays and Smith, 1995)*.

pe of stress	Protection category	Dynamic design stress design	
		Concrete f_{dc}	Reinforcing bars f_{dr}
ending	1	f_{dcu}	f_{dy}
	2	f_{dcu}	$f_{dy}+\left(f_{du}-f_{dy}\right)/4$
hear	1	f_{dcu}	f_{dy}
	2	f_{dcu}	f_{dy}
ompression	1 and 2	f_{dcu}	f_{dy}

ructural response

re 6.11 shows an idealised resistance-deflection function, (see also Figure 6.5). It esents the structural behaviour of a reinforced concrete element under flexure.

e figure r_u is the ultimate unit dynamic resistance available, having taken account of tatic loads existing at the time of the blast loading.

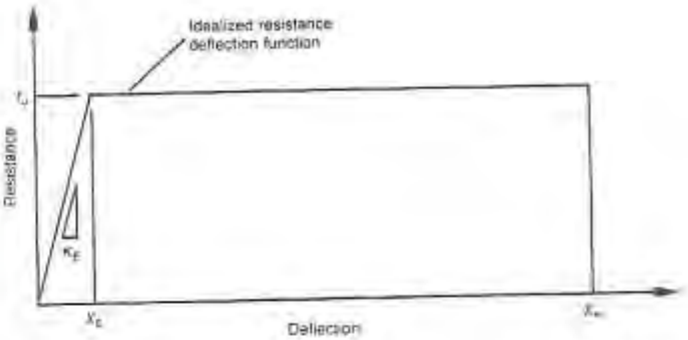


Figure 6.11 Idealised resistance - deflection curve, *(Mays and Smith, 1995)*.

is the deformation at the elastic limit on yield, K_E is the elastic stiffness and X_m is aximum allowable deformation based on the allowable support rotation, θ, given in ble 6.2.

The impulsive load case; flexural design

gure 6.12 shows a blast wave as a triangular pressure-time function with zero rise time. is loading case is given for illustration. The other options are design for the quasi-e case or for the pressure-time regime. In the impulsive regime, the load applied throughout time t_d is short in comparison to the response time t_m of the member (the time for the member to achieve a deflection X_m), such that $t_m / t_d \geq 3$, (see chapter 3). The applied loading is uniformly distributed and is represented by the specific impulse, i.

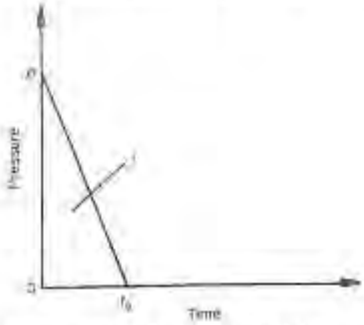


Figure 6.12 Idealisation of blast load, *(Mays and Smith, 1995)*.

Design approach

The kinetic energy conveyed by the impulsive load is to be resisted by the strain energy produced by the member, when the deflection becomes X_m. The design provides the structural strength and ductility to achieve this.

Basic equation

The kinetic energy of the blast load is equated to the strain energy derived from the idealised resistance deflection function. From earlier (equation 3.29) the kinetic energy is given by:

$$KE = \frac{i^2}{2K_{LM}m} \qquad 6.6$$

where i is the specific impulse

K_{LM} is a factor, called the "load-mass factor", to convert the RC member response to that of a SDOF system (on which the KE term was based). K_{LM} factors are given in *US ACE Manual EM 1110-345-415, 1957* and elsewhere.

m is the unit mass

The strain energy is given by the are under the resistance-deflection curve, thus:

$$SE = \frac{r_u X_E}{2} + r_u(X_m - X_E) \qquad 6.7$$

...ne the resistance - deflection function in terms of r_u, X_m, K_E, and X_E

...he ultimate resistance, $r_u = f(M_p, L)$, which depends on the supporting conditions ... the span, L. Standard solutions are given in reference texts, such as *TM5-1300* and ...s and *Smith, 1995*. The ultimate moment capacity, M_p is given by:

$$M_p = \frac{A_s f_{ds}}{b} d_c \tag{6.8}$$

Where A_s is the area of tension reinforcement within a width b

f_{ds} is the dynamic design stress for reinforcement of steel section

d_c is the distance between the centroid of the compression and tension reinforcement

b is the width of flexural member

Also

$$\rho_s = \frac{A_s}{bd_c} \tag{6.9}$$

m, the unit mass, equals ρd_c where ρ is the density of concrete.

Stage 3

Assumptions have to be made about the make-up of the concrete section. It is normal to solve for d_c using an assumed value of ρ_s, the steel ratio, (see equation 6.9). d_c becomes an unknown value in the equation for r_u and K_E and m. These terms are used to solve the basic equation (6.7) to derive d_c and so confirm the concrete section.

Stage 4

Finally, check that the impulsive case was the correct one to use by estimating the time to maximum deflection, t_m, (approximately equal to i/r_u). Then check the ratio t_m/t_d.

In the impulsive case $t_m/t_d > 3$ (see Figure 3.25). If the impulsive case is not correct and quasi-static or pressure-time conditions apply, an amended design procedure is required, as covered in standard texts.

.6 Establishment of the damage tolerance for different cladding panels

As forecast at the beginning of this chapter, the aim is to compare the damage tolerance of a number of generic cladding types, glass, GRP, steel, RC, and aluminium. Sections .2 to 6.5 have dealt with supporting matters.

Various assumptions are needed to carry out the comparison, as follows:

. Panel size	1.2 x 0.6 m
Support	simple support at each end of the long span i.e. span 1.2 m
Panel thickness	see table 6.4 and below
Failure criteria	see tables 6.2 and 6.4 and below

The thickness of the steel, aluminium and glass reinforced polyester (GRP) panels have been fixed by selecting a typical thickness for steel cladding sheet (2mm) and then making the other panels of similar mass. The glass panel does not follow the same criterion but is taken as a typical thickness for such glazing. Similarly, the RC panel thickness has been selected as a practical solution, bearing in mind the need to provide a sensible design that copes with handling loads and provides adequate cover to the reinforcing steel. While the thicknesses selections may seem some what arbitrary the aim is to have typical thicknesses rather than exact equivalents. The failure criteria given in Table 6.4 have been selected using the protection categories given in Table 6.2 plus, in some cases, a deflection limit appropriate to such cladding panels, where the ductility ratio or elastic limits would lead to excessive deflections.

Table 6.4 Panels thickness and maximum displacement

Panel material	Thickness, mm	Max. deflection mm
Glass	8	Brittle failure
GRP	8	Brittle failure but < 100 mm
Steel	2	μ = 10 or 100 mm
RC	60	$\theta = 2^{\circ}$
Aluminium	5.6	μ = 10 or 100 mm

hough it is hard to achieve an accurate comparison, that based upon reasonable umptions of thickness and damage criteria (maximum deflections) can be done. lculations to identify failure for each panel type for both the impulsive and the quasi-tic cases are shown at Appendix A. The failure values of specific impulse and essure are reproduced in Table 6.5. A pressure-impulse diagram, to illustrate the mage tolerance of each panel type, is shown at Figure 6.14.

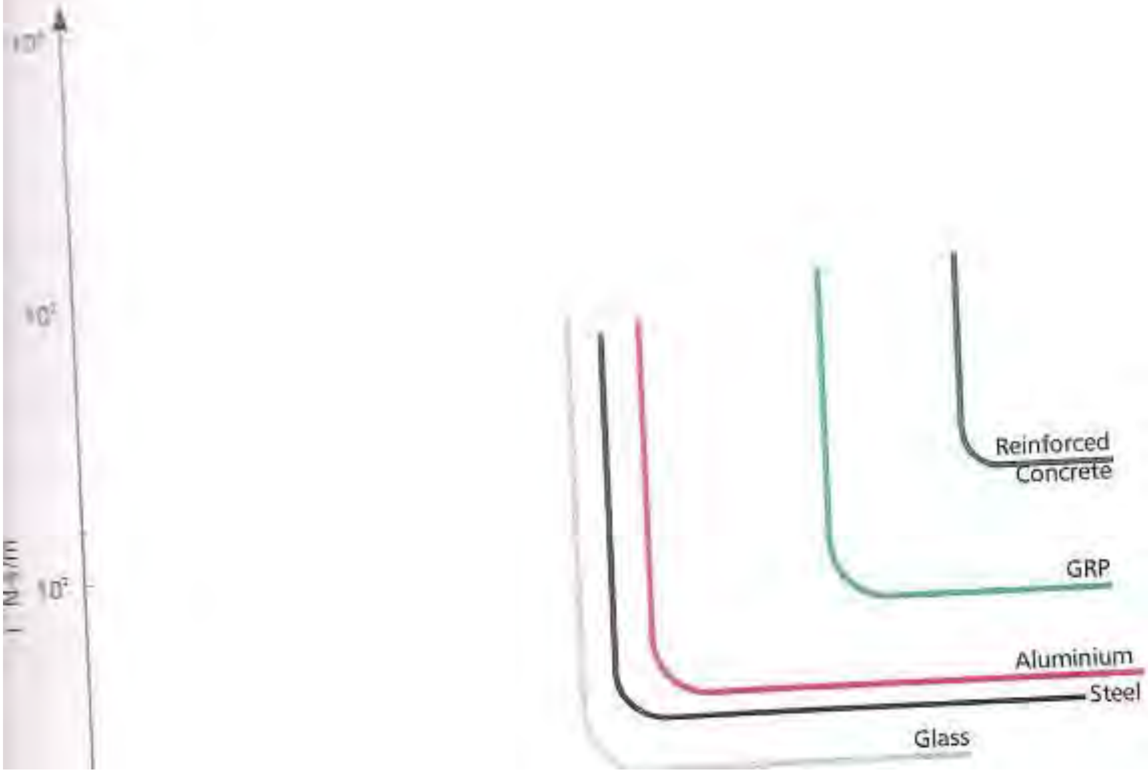


Table 6-5 Panels' Pressure and Impulse at the Damage Limit (after Appendix A).

Material	Impulse $N\text{-}s/m^2$	Pressure N/m^2
Glass 8.00 mm	34	2316
GRP 8mm	89.497	5925.4
Steel 2.00mm	44.956	541.504
Aluminium 5.6mm	52.57	751.397
Precast reinforced concrete 60mm	431.6	32748.538

6.7 *Cladding damage Tolerance Concluding Remarks*

The Iso-damage curves plotted on Figure 6.14 to compare the resistance of different cladding materials, like GRP, glass, aluminium, steel and reinforced concrete. A similar material mass has been taken as a point for comparison between the materials other than the glass and the concrete panels. Due to this and because of the density variation, different thickness have been obtained. After reference to Figure 6.14 and Table 6.4 one could say that the panel thickness is not an indication of its stiffness. GRP, indicates a good performance because of its ability to deflect elastically with low stiffness. In reality it would have to be profiled to increase its stiffness for normal handling. Also it could not deflect to its elastic limit because it would act like a membrane and tear at the fixings.

The cladding panel dimensions, aspect ratio and fixing arrangements will have a considerable bearing upon performance under blast loads but the comparisons made here have standardised these parameters to allow a comparison of the panel materials. The results illustrated in Figure 6.14 are not unexpected but depend to some extent on the selection of failure criteria. A brittle material like GRP appears to perform much better than the ductile materials (e.g. Steel and Aluminium) due to the low stiffness already mentioned. This effect would have been more marked if a deflection limit had not also been applied. The comparison of the steel and aluminium panels is interesting too because the aluminium panel reaches the deflection limit at only half the ductility limit of 10 assumed. Perhaps a lower ductility coefficient should be specified for aluminium structural elements.

comparison purposes, very simple rectangular sections have been assumed. In practice, more complex panel constructions are likely. The profiling of GRP panels to improve stiffness has been mentioned above but double skin constructions, using thin metal or plastic materials are common. The two skins usually sandwich insulation material. Also, it is important to note that some of the data used to obtain the pressure and the impulse are taken from references like Ashby and Jones, 1987. Some of these items, like the yield strength, are given within a range i.e. the aluminium yield strength is given by 25 to 600 MPa, the value used here is 50MPa. Changing this value using bigger or smaller values of yield strength highly influences the value of the stiffness K_e and hence the resistance r_u.

Finally, the analysis confirms that the most damage tolerant cladding material is reinforced concrete and least tolerant is glass. Without the logarithmic plotting of the iso-damage curves on the PI diagram, the differences would have appeared even more marked. It is important to note that, for the reinforced concrete panel type, even more attention would need to be paid to the strength of the fixings transferring the heavier loads into the structural frame.

Chapter 7

The influence of architectural design and features on the response of glazing to explosive blast

Chapter 7

The Influence of Architectural Design and Features on the Glazing Overall Resistance

7.1 Preface

Chapter 4 shows how cladding as a mass skin integrates with the space as its wrapping element. The rules that govern the surface form design are demonstrated in the same chapter.

Chapter 5 illustrates in detail the different types of cladding and their various methods of construction. Heavy and light cladding material classifications are demonstrated in terms of constituents which affect its behaviour at the place of explosion. Such constituents are panel geometry, fixing and joints.

Chapter 3 and chapter 6 present the blast wave mechanism and the response of the different cladding material against blast loading. It is evident that glass provides very low resistance to blast loading and it was therefore decided to concentrate the research effort on improving the performance of glazed cladding. The following main reasons led to this decision:

a) Transparency of glazing provides the interior space with light. The technological development and the recent innovations of the material and its system of construction has enhanced the material utilisation properties from a means of light transmission to a means of visual impact with a substantial amount of aesthetic qualities.

b) Many civic and administrative buildings are glass fronted buildings whose construction is characterised by ease of installation and ease of maintenance.

c) Glass has weak resistance to a blast loading attack. Nevertheless, it is considered that measures can be taken to improve the resistance of glazed panels. The rest of this thesis will establish the potential for achieving this.

This chapter illustrates several examples of glass fronted buildings and discusses how one might improve the behaviour of glass when subject to blast loading by reducing the pane size.

Reducing the pane size will increase the mullion area per square metre. The mullions must be ordered and treated. This is intended to improve and enhance the aesthetic qualities of the structure. Glazing is considered to be a prime material for use as external cladding for buildings and structures.

It has numerous properties from the architectural point of view, which reside in its aesthetic appearance, ease of installation, ease of maintenance, its high transparent property and its power of reflection of light when compared to other cladding materials.

The transmission of light provided by glazing has an aesthetic, as well as a functional purpose. Glass, through the works of Gropius, Mies van der Drohe, Taut and others represents clarity in both the literal and symbolic meanings of the word, *(Button and Pye, 1993)*

Large glass fronted buildings built in the "high technology" style, "high tech" as many architectural critics and historians have classified it, have therefore become popular in the last few decades of this century.

Due to the above mentioned characteristics many of the commercial and administrative establishments use glass as a powerful cladding tool. Some buildings have good natural protection against explosive blast but others do not and, when attacked, the building itself can be a major cause of casualties to people both within and outside the building.

This is particularly true of large glass fronted buildings such as that shown on the left in Figure 7.1 Consequently, due to the recent escalation of the casualties of terrorism and due to the fact that the administration and commercial establishments are often the target of terrorist missions, a technique is needed for the secure use of glass to alleviate the consequences of the explosives and their hazardous side effects on the establishments.

This section discusses examples of glass facades from around the world. But this extensive use of glass in the event of explosive blast results in injuries (due to flying splinters) several hundred yards away from the explosion. When exposed to blast, due to its fragile nature, glass causes injuries to both occupants and people on the street. It is important to realise that glass is like the flowers, although they look beautiful they are surrounded with thorns.

Figure 7.1 Typical glass fronted and brick buildings.

7.2 The reasons behind the use of glass facades

The following scheme analyses the reasons why glass is increasingly used in civic centres.

Glass facades increased use

Construction

-The ease and swiftness of construction.

-Simplicity of replacement and maintenance of its damaged constituents

Aesthetic

-Means of contrast with ancient sites, a glass curtain creates a mirror which reflects the sites old spirit.

-Glass could be used to create a visual impact, or monumentality.

Economical

-The construction rapidity.

- Glass curtain walls are cheap in comparison with several other materials and they need only long term replacement of the gaskets.

Environmental

-Glass curtain walling provides the structure with weather tightness with good sealing against water penetration.

-Double glazing provides heat and sound insulation.

Glazing manufacturing advantages

Glass like other building materials has distinct properties of elasticity, hardness, and durability. These properties are governed by the material size and proportions which are limited not only by their structural properties, but also by their manufacture process.

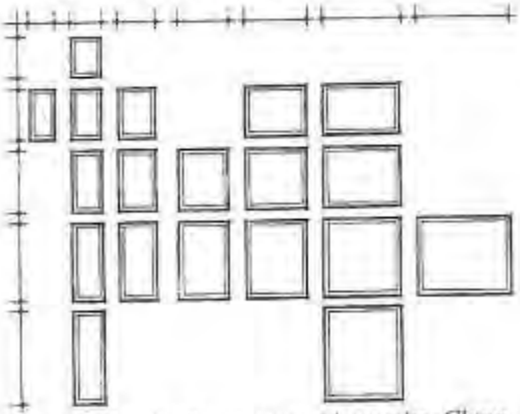


Figure 7.2 Standard casement widow units, *Ching, 1996.*

Unlike structural elements glass size not related to structural task it performs. Glass size is limited by the size and scale of space it helps to enclose and it is also limited by the factory-output size.

2. How glass introduces in architecture

The following figures are to show how architects have introduced glass in architecture. The aesthetic quality of glass is the main reason behind the increased use of the material. Protection from explosive blast was not taken into account for any of these examples.

a. The National Museum of Science

The glass was used to clad a spherical shape. Its reflection qualities were used to integrate with the outdoor green space and not to interrupt it.

Figure 7.3 The national Museum of Science, Paris, France, the glass used in spherical concept. The reflective quality of glass integrated with the out door space, *(Rice, 1995).*

b. An office building, London

The glass has been used to clad a conical form building, in which richness and prestige are qualities the owners need to represent.

Figure 7.4 A typical office building, London, a conical form building cladded with glass curtain.

c. A typical business park

Glass is used also to clad both permanent and temporary spaces in rapid construction. The example in Figure 7.5 is in a typical business park.

Figure 7.5 A typical building used in the park, the parking area is very close to the weak valrnable structure.

...n entrance hall, Outlet Village, Swindon

...ure 7.6 shows the entrance hall of Outlet Village, Swindon. The entrance has been ...lt to integrate with the old structures which were previously used as a railway sheds.

Figure 7.6 The entrance hall, Outlet Village, Swindon. The glass curtain wall used in the construction of a new building to integrate with old existing structures.

e. Saitam Museum of Modern Art

The Saitam Museum of Modern Art designed by Kisho Korokawa is one of the structures in which glass is used to express the modern technology in its form.

Figure 7.7 Saitam Museum of Modern Art, *(Jencks, 1993)*.

Figure 7.8 A typical example of curtain walling in London.

7.3 The architectural approach

Light transmission is the most important criteria which has made glass very important in architecture. In the past when it was only used in the form of windows, its size was limited because of the wall loadbearing function. In addition, the normal pane size was only .75 m by .5 m until the 1830's when it became 1.0 m by 1.3 m. Then, structures using cast iron and wrought iron, and later, steel and reinforced concrete helped the designers to achieve larger window openings.

A glass skin expressed by glass and metal, instead of traditional openings in skeletal structures, has been developed. Walter Gropius in his Fagus factory in Germany, Figure 7.9.a introduced the first example of the use of glass in the complete height of the elevation. In his design of the Bauhaus in Dessau in 1925, he used a flat continuous surface of glass separated from the building structure as an early form of curtain wall. The first real curtain wall structure was by Willis Polk, Figure 7.9.b in his design for the Hallidee Building in San Francisco in 1918 *(David Button and Brian Pye, 1993).*

Figure 7.9.a Fagus factory in Germany by Gropius. *(Button and Pye, 1993).*

Figure 7.9.b Hallidee building in San Francisco by Polk. *(Button and Pye, 1993).*

Burno Taut, in his comment about his design of the Glasshouse Pavilion for the Werkbund Exhibition in Cologne in 1914 *(Button and Pye, 1993)* described the Pavilion as having no other purpose than to be beautiful.

Glass was intended to have an aesthetic and symbolic role. Architects have used the aesthetic qualities of glass in different forms, such as in Foster Associates' Willis Faber Dumas building in Ipswich, U.K in which the mullions are completely hidden and the panes are suspended from the upper floor and joined together by metal plates. The glass used is toughened, reflecting glass.

In the above examples, we see designers using glass for its visual impact. The threat of blast was completely absent from their minds. As architecture is the mirror of any culture, in the past few decades, when there was no threat of blast or terrorism, architects competed to create aesthetic forms belonging to different trends.

Now, with the advent of terrorism, which might ebb and flow, architects must consider the need to keep people safe outside or inside their buildings from the catastrophic failure of glass.

As the complete protection of civic centres would be excessively costly and could be wasted where a blast has not taken place, a treatment could be given to glass curtain wall structures and windows which provides them some improved stiffness.

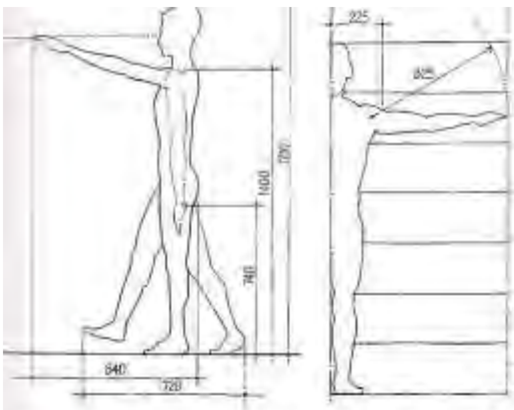


Figure 7.10.c Anthropometric proportions refers to the measurment of the size and proportions of the human body, *(Ching, 1996)*.

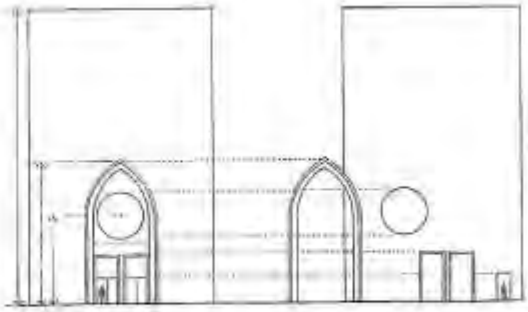


Figure 7.10.d Mechanical scale, the size or proportion of an element relative to a standard visual scale, i.e. other known size elements, *(Ching, 1996)*.

The classical order uses the column diameter as a basic unit of dimension. The visual scale is perceived by relating the size of each element to the other parts or to the whole of a composition, as in Figure 7.10.

The size and proportion of windows in a building facade are visually related to one another as well as the spaces between them. Axis, symmetry, hierarchy, rhythm, datum, harmony and contrast are the ordering principals for a visually impacting composition.

There are two types of mullions according to the type of cross section and this defines its behaviour and appearance :

- Cast cross-section; in which the mullion material is moulded to give solid cross-sections. It could be made of wood, copper, aluminium or stainless steel.
- Hollow cross-section; in which the mullion is manufactured in box like form. It is made of aluminium, PVC or steel.

7.5 The proposed concept

A new treatment for the facade that is proposed is the remodulation of the mullion grid of the glass panels using the ratios 1:1, 1:2, 1:3 up to 1:8 and taking into account a proposed maximum spacing of 0.5 m between mullions in both directions vertical and

7.4 Proportions

While scale alludes to the size of something compared to a reference standard or to the size of something else, proportion refers to the proper or harmonious relation of one part to another or to the whole *(Francis Ching, 1996)*.

All building materials, such as glass, have rational proportions which are dictated by their inherent strengths and weaknesses. Windows are sized and proportioned not only according to their structural properties and function, but also by the process through which they are manufactured.

The theories of proportions are to create a sense of order and harmony among the elements in a visual construction. Any proportioning system ought to establish a consistent set of visual relationships between the parts of the wall, as well as between the parts and the whole.

A proportioned system creates a visual order that can be sensed and recognised through a series of repetitive experiences. They can visually unify the multiplicity of elements of an elevation that belong to the same family of proportions.

The Golden section, classical order and Renaissance theories are old theories which have helped in the development of modular and anthropometry as new methods of proportions, *(Ching, 1996)*. Figure 7.10 gives examples of these systems.

The golden section is the ratio between two sections of a line, in which the lesser of the two is to the greater as the greater is to the sum of both. Algebraically it is the equation of two ratios:

$$\frac{a}{b} = \frac{b}{a+b}$$

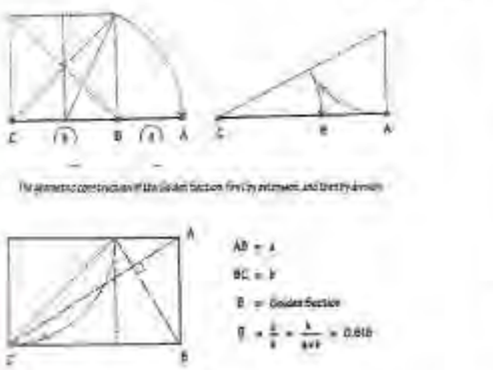


Figure 7.10.a The geometric construction of the Golden section, *(Ching, 1996)*.

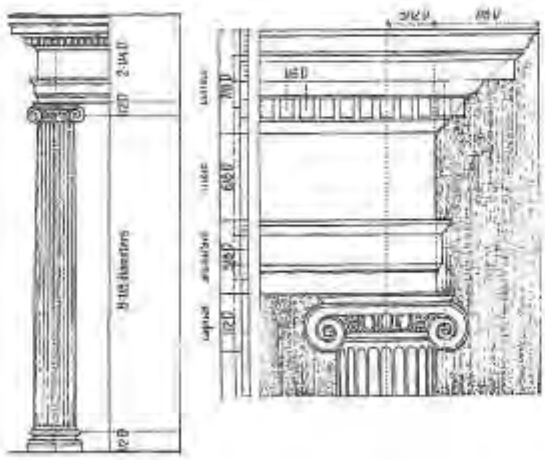


Figure 7.10.b The classical order of proportions, *(Ching, 1996)*.

Some new concepts are presented in Figure 7.15. The area of mullions compared to the area of glass is much higher than usual. The mullions should be ordered in a concept that enhances the aesthetics of the building. The suggested treatments take two main forms. The first arrangement has a spacing of 20 cm between horizontal mullions(see Figure 7.15.a, c, and Figure 7.16), making the following ratios with the vertical mullions: 1:1 to 1:5, and also 1:15 in one case. Some patterns are made by diagonal mullions or circles. The second concept has mullions spaced at 40 cm between the horizontal mullions, as in Figures 7.15.b and d.

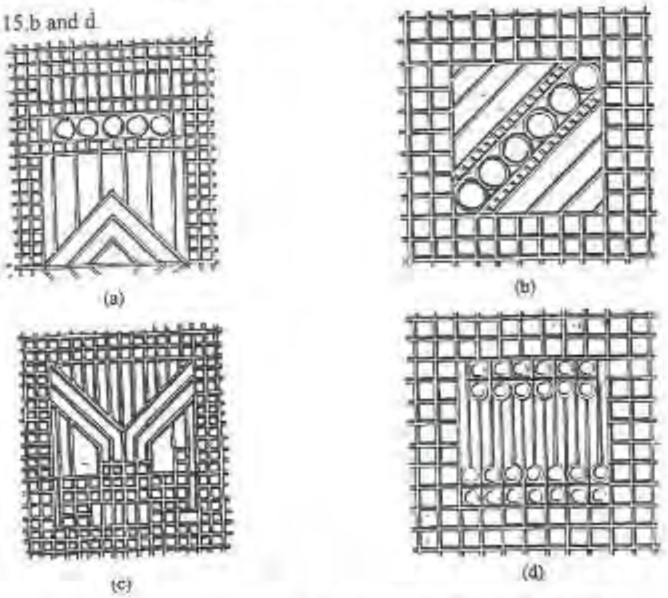


Figure 7.15 Suggested treatments for glazing panels, *(El.Kadi et - al, 1997).*

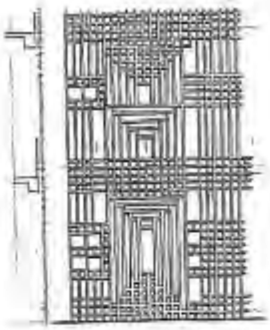


Figure 7.16 Suggested treatment for glass curtain walls used to clad two storey height structure, *(El.Kadi et - al, 1997).*

7.6 Practical application of reducing the glass pane size

Gothic and Renaissance and other ancient styles introduced glass as a means of light transmission, its size was limited by both the pane size and the wall structural function. Figure 7.17 to 7.19 show some historical introduce of small glass pane size in buildings.

Figure 7.17 Westminster Abbey, London. The rose window introduced by the Gothic style is an example for small pane size glass grouped in an aesthietical order.

Figure 7.18 Entrance view for Westminster Abby, London. The arched window above the entrance uses a module of ordered stone mullions in an attractive grid.

ure 7.19 The main entrance view for Oxford Museum, Oxford. The window as a unit divided into four groups of windows each divided into four equal sized panes, the window repetition formed the rhythm of the elevation aesthitical order. A bay window as shows on the left hand side used to break this repetition.

It is important to note that although architects were limited by the use of specific material such as stone with its rules of construction, they succeed to introduce the material in an attractive concepts with special visual impacts.

The use of steel on the industrial age as a material of construction and as a structural element provide the architecture with new forms of large span structures such as shown in Figure 7.20 and Figure 7.21

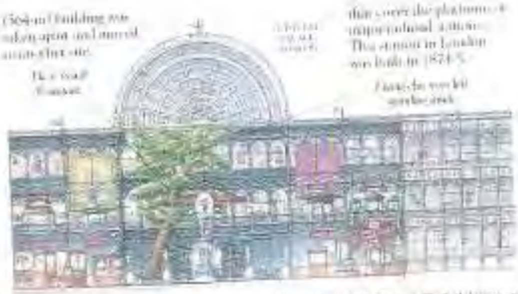


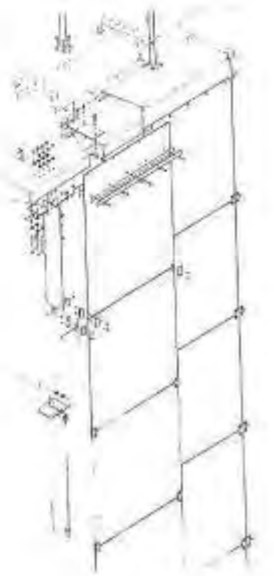
Figure 7.20 The Crystal Palace, London. The building built for the Great Exhibition of 1851, Paxton's Crystal Palace constructed with prefabricated cast iron parts and sheets of glass. It was built as an early form of temporary structure in order to be rebuilt for several times at different places

Figure 7.21 Liverpool St. Station, London, the tensile strength of the steel is greater than that of iron, steel used for large train sheds. the station built in 1874-5. *(P. Wilkinson, 1995)*

Glass was introduced as one of the curtain wall forms as result of continuous improvement of the methods of construction. The principal advantage of the curtain wall is that because it bears no vertical load, it can be thin and light weight regardless of the height of the building. The name "Curtain Wall" derives form the idea that the wall is thin and hang like a curtain on the structure frame.

Figure 7.22.a The reflective glass gives the building a special visual impact. *(Tzonis, 1992)*

gure 7.22.b Detail of fixing the glass curtains to the structure.



Many metal and glass systems consists of strong vertical mullions fixed to the structure with factory assembled units affixed. Such a system is illustrated in Figure 7.22.a and 7.22.b.

Buildings, such as the one shown in Figure 7.23 are clearly very vulnerable to explosive blast loads. However, the general protection of such buildings is not a viable solution. Apart from aesthetic considerations, the cost of a fully protective scheme would be prohibitive.

Also, building owners are unlikely to see the risk to their particular buildings as being particularly high and certainly not sufficient to justify an ugly building and excessive cost.

One option which is examined in the research is to make the glazing more resistant to damage from air blast load.

2 *Typical examples of reduce pane size buildings*

The following figures show examples of using small size glass panels; reducing the pane size used to give a visual impact, Figure 7.26 and Figure 7.27 show how small size can be used to give a especial aesthetical qualities

Figure 7.26 D.G Bank, Hanover, Germany by Bahlo-Kohnke-Stosberg. A small panel size repeatedly used in the elevation, *(Button and Pye, 1993)*.

Figure 7.27 First of August store, New York City, designed by George Ranalli. The shop front fronted with small size panes of glass together with light it gives a good visual impact, *(Jencks, 1993)*.

.. The advantages of the use of reduced area of glass

Mays and Smith, 1995 advised minimising the area of glass within an elevation in order to minimise the losses. The following examples shown in Figure 7.28 could be used as a guide to how to group glass areas while having aesthetic quality.

Figure 7.28 A typical building, Aldgate, London. The buildings have grouped areas of curtain walls together with solid material. This treatment increases the elevation's natural resistance.

Another example shown in Figure 7.29 for the use of grouped area of glass together with solid materials. These examples could be applied instead of using complete glass front as this increases the risk of the elevation complete failure.

Figure 7.29 A typical office building, Hammersmith, London. In the elevation there is a balance between the grouped areas of glass and the solid surfaces.

The following example shows the use of glass with brick and plastics in an attractive concept

Figure 7.30 A typical office building, Swindon. In the elevation glass used with brick and plastic, reducing the area of glass increases the elevation resistance.

thetical balance of increased mullion eare per square metre

important to note that on needs to be taken in ullion arrangement. Figure s an example for the use of sed area of vertical ural elements within the on. The area of the ural elements per square of the elevation when sed this does not affect the on aesthetecal apperance. Figure 7.32 mullions are d in two directional al patterns, the concept a good visual impact.

Figure 7.31 A typical building, London. The steel structure exposed to have a visual expression.

ure 7.32 A typical car park, Swindon. The diagonal mullion used to give an aesthetical order.

5. *Positive and negative design guides*

As discussed, there must be a balance between the use of glass and another solid material in order to increase the elevation overall stiffness. Figure 7.33 shows the use of stripped curtain wall together with brick. Voids resulted from the subtraction of a space from the building mass ought to be avoided because it forms a space of multiple reflection and concentrates the stress waves.

Figure 7.33 A typical office building, Swindon, when in an elevation glass used with brick this will increase the elevation resistance.

Figure 7.34 An office building with subtracted space, this space cladding must be hardened.

Figure 7.34 shows a mass with subtracted space although in the elevation solid material used together with the glass, the subtracted space will cause the stress wave to concentrate and multiply reflect at this space. The glass cladding this inner space must be hardened against these increased stress waves. Finally, reduced size panels when ordered gives a good results of visual impacts such that shown by Figure 7.35.

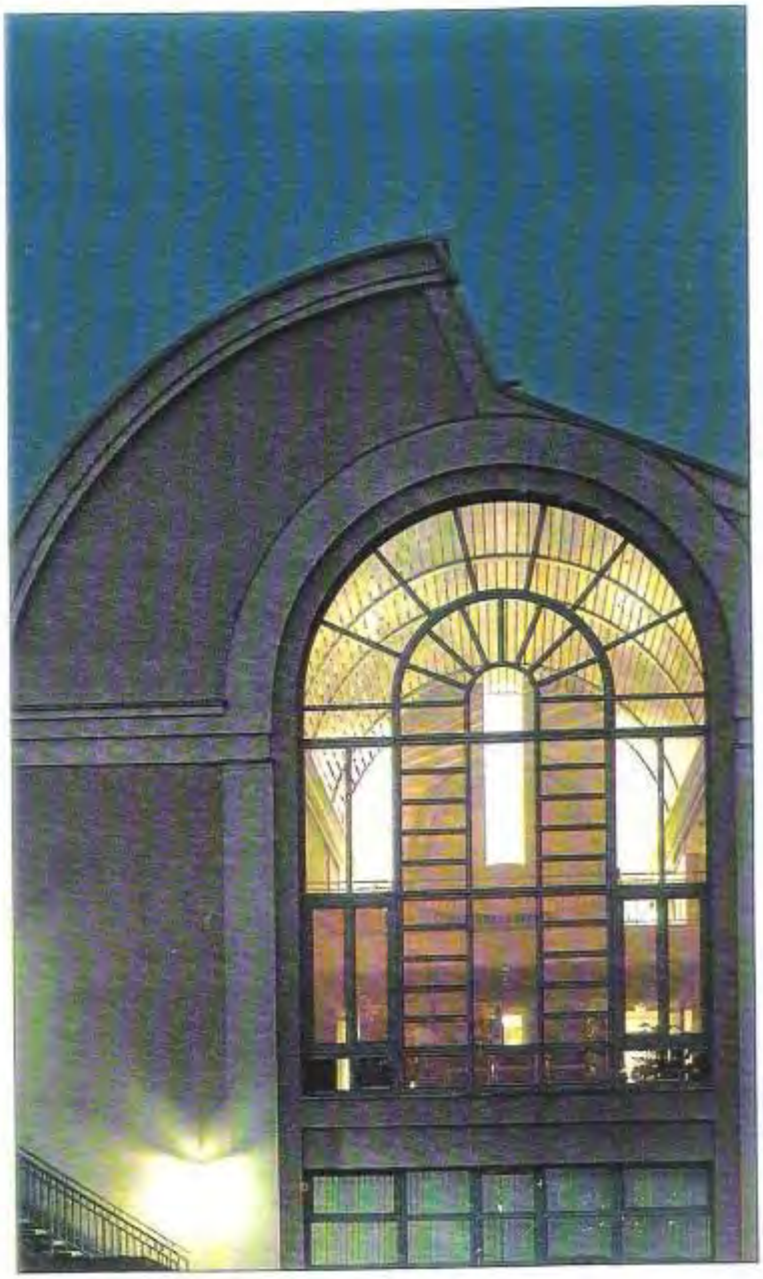


Figure 7.35 An arched window, this example shows how reduced panes of glass when ordered it good visual impacts, *(Pye, 1993)*.

5. *Manufacturers' advice concerning reduced pane size curtain walls*

a. *Cladding Designer's Advice*

Dr. Stephen Ledbetter from The Centre for Widow and Cladding Technology, University of Bath was consulted about his suggestions of how practically the suggested patterns given in Figure 7.15 could be specified.

He gave the following precautions to be considered in the design which can be summarised as follow:

1. The minimum practical size for toughened glass is 200 x 1000 mm. Annealed glass is usually 4, 6 or 10 mm thick and toughened glass comes in the same range of thickness.
2. There are usually proprietary systems of mullion design. Grids are typically 800 mm up to 1350 mm wide and 2.5 to 3.6 m high. With site fixing of glazing there are possible problems with leakage, so that factory production is best. One can have more slender framing members. Also, it is possible to bond glass to aluminium using a silicon adhesive and using double sided adhesive tape as shown in Figure 7..
3. The strength of glazing depends on surface flaws.
4. Dr. Ledbetter mentioned, also, BS 8200: 1985, *Design of non-loadbearing external vertical enclosures of buildings,* which require all buildings to have double glazing for energy efficiency. A typical structure is 6mm toughened on the outside and 7.3 mm laminated on the inside. He said that Plikington Glass in Swindon were developing a sandwich cnstruction with laminated glass.
5. There was a proposed European standard/DIN standard with four protection levels:

Level	Peak Pressure (bar)	Positive Impulse (bar ms)
1	0.5	3.7
2	1.0	9.0
3	1.5	15.0
4	2.0	22.0

top of the glass, with the aluminium bonded to the glass using a silicone adhesive. One would rebate short stubs to hold at the periphery with a gasket.

- Conventional bars were 50 mm. Aluminium would be better than steel or PVC and could be coloured. Also PVC is not suitable for windows or curtain wall expected to be exposed to explosive blast as it melts when exposed to the higher temprature released by the explosion.
- Cast sections were unsuitable. The silicon adhesive might works he said. He suggested double-sided adhesive tape to hold the matrix in position and to use silicon adhesive to complete the joint, Figure 7.36

(1) Plastic Tape

(2) Silicone adhesive

(3) Glass

(4) PVC Solid strap to maintain constant distance between the glass

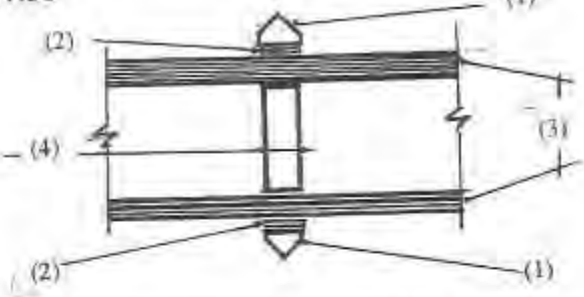


Figure 7.36 Glass bonding method using silicon adhisive,an inermediat PVC is used to maintan the distance between the glass

Figure 7.37 to Figure 7.41 show some details of curtain wall joints and how they could fixed to the structure. Figure 7.42 is an application for the manufacturers advice.

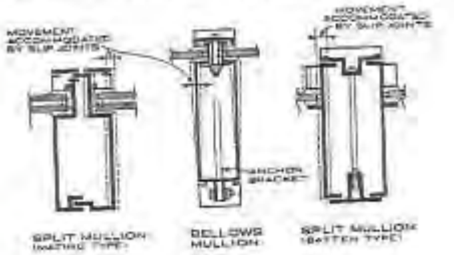


Figure 7.37 Aluminum mullion designes allowing for horizontal expansion and contraction. Such mullions are installed in place of regular mullions at intervals determined by expected thermal and structural movement in the wall, *Allen, 1990*

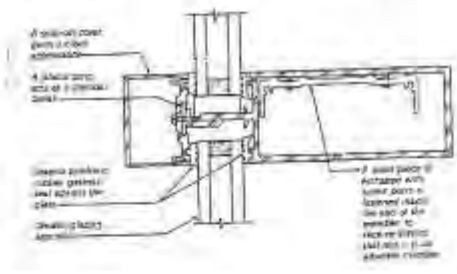


Figure 7.38 An aluminum curtain wall mullion summarizes some of the extrusion features presented here, including a snap-on exterior cover with ratcheting action, extruded channels to receive synthetic rubber glazing gaskets, and a connecting clip that employs screw ports. *Allen, 1990*

7 Normal cladding for office buildings were:

- Heavyweight: precast concrete, load-bearing brick, rain-screen over-cladding (European idea) with blockwork and metal panels.

- Lightweight: glass in aluminium frames, single skin metal and double skin metal, and double skin metals and composite resin-based panels can be frame supported. Opaque glass can be used as a panel in double-skin construction. Metal panels in double skin construction can be 0.8 mm thick and have infill foam (rock wool or mineral wool).

b. Cladding Manufacturers Advices

At a meeting with Mr. Tony Wilson, Technical manager for Granges Building Systems Ltd, Tewkesbury the following notes are descussed when he was asked about his advice on how praciticaly the propsed concepts by *El-Kadi et al, 1997* could be applied. Mr. Tony Wilson advised that:

- The minimum distance between any two welded mullions should not be less than 40.00 cm.
- If the distance between mullions is less than 40.00 cm, in this case the cross jointing of two mullions must be conducted by applying a mechanical joint to avoid weakening the mullions material due to close welding.
- It is recommended that mullions shall be structurally spaced at 1.00 m apart with continuous mullions connecting the two adjacent horizontal structural floor elements as shown by Figure 7.36.
- Blast resistant glass was usually laminated. It was very important to retain the glass in a rebate. He had been involved in trials at West Freugh. Cosiderations were how to hold the glazing and how the frame would accept load. There were patents on fixing systems.
- Concerning Maj El-Kadi's idea for small panes, the welding of aluminium joints would be a problem and would be extremely expensive. He suggested applying a matrix on

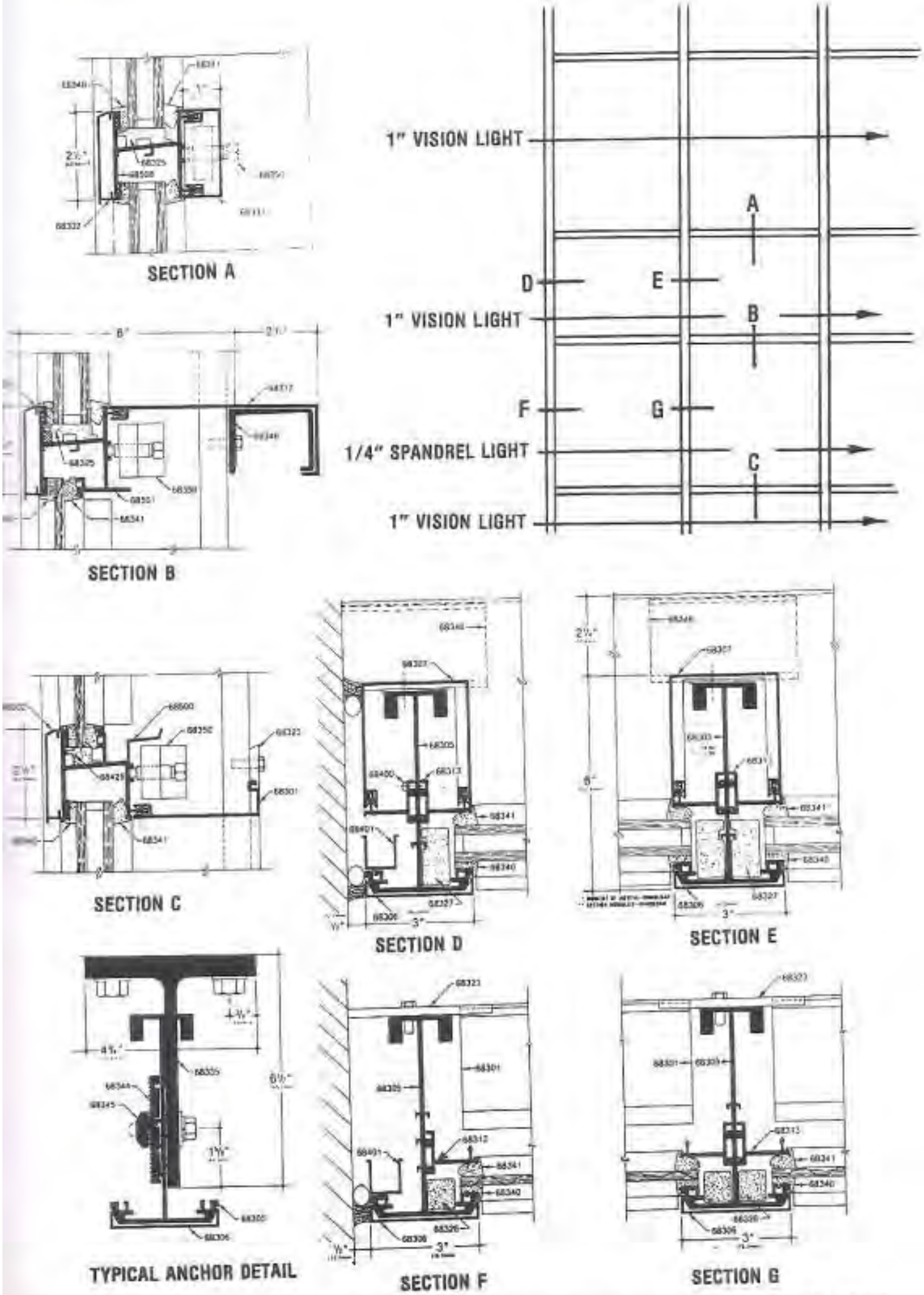


Figure 7.39 Curtain wall different sections keyed to the elevation view at the upper right, *Allen,1990*

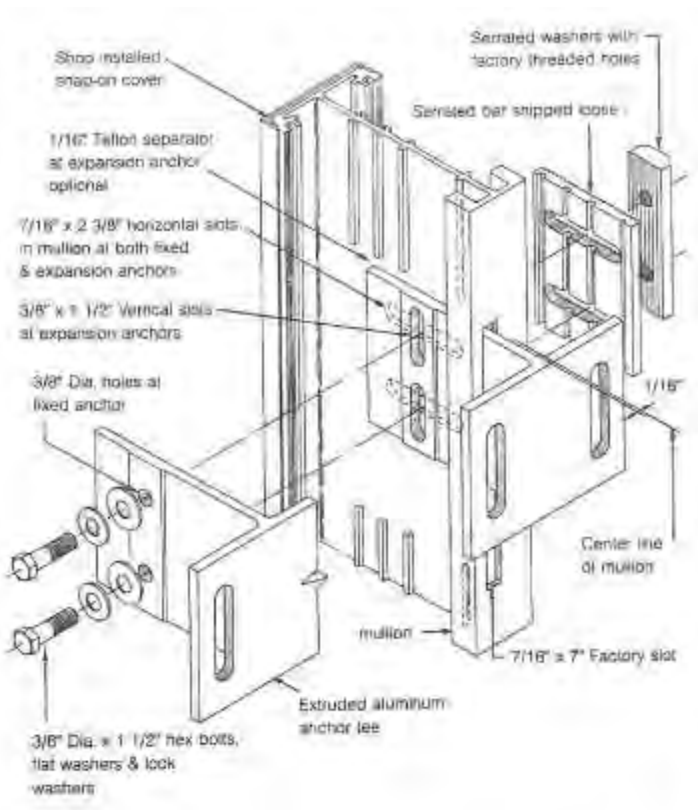


Figure 7.40 Vertical mullions are attached to the edges of the floors of the building using this detail. Slotted holes allow for adjustment of the curtain wall, *Allen, 1990*

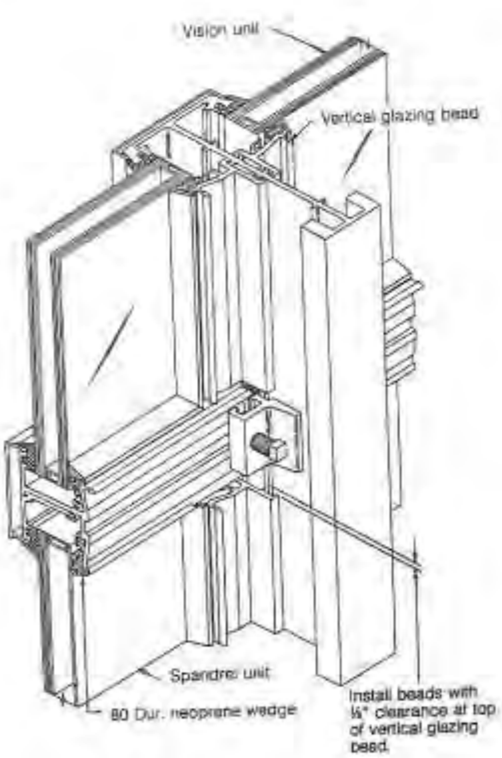


Figure 7.41 A three-dimensional view of the assembly of the horizontal and vertical mullions, *Allen, 1990*

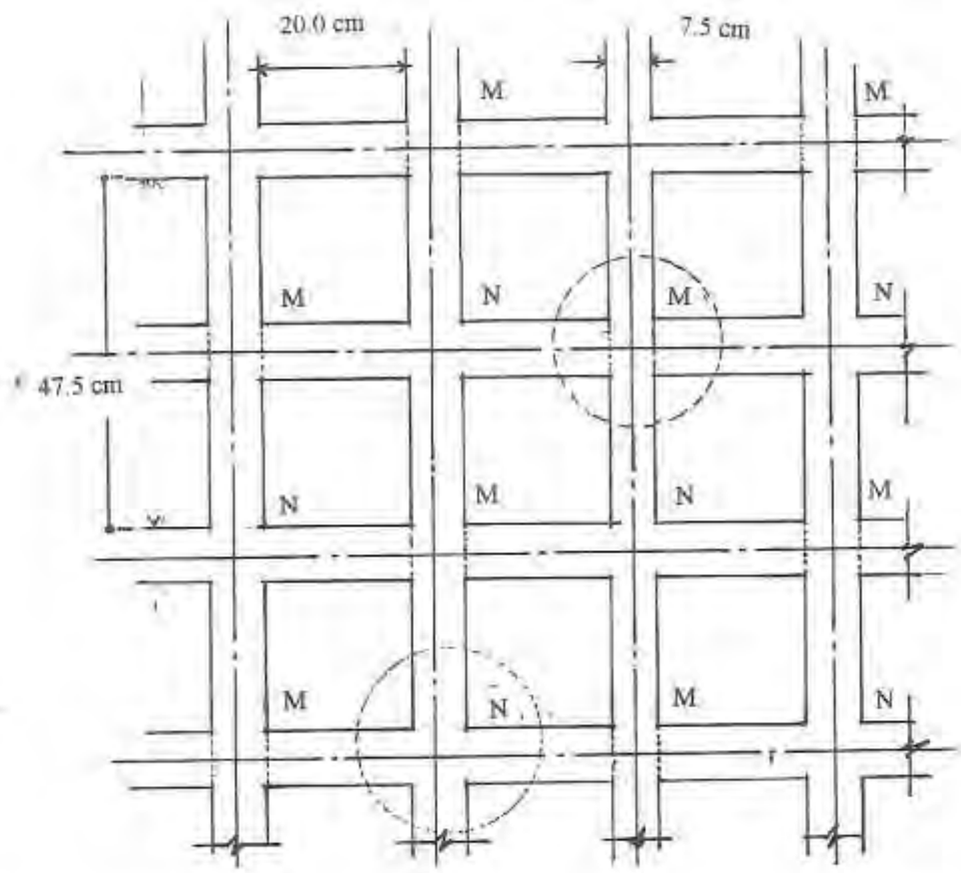


Figure 7.42 Manufacturer's precautions the distance between two close welded mullions N must not be less than 40.00 cm, mechanical joints M applied for mullions spaced less than 40.00 cm.

7.8 Conclusion

Finally, modern architecture often features large and extensive glazed areas as cladding or curtain walling. Such buildings have been shown to be extremely vulnerable to terrorist attacks which use lorry or car bombs. The idea developed in this chapter is the use of glazed panels that comprise small sized panes in patterned form.

Such modifications could be employed in new buildings or even as a retrofit for existing buildings. The aesthetic quality of buildings can be maintained while providing improved performance and security under terrorist attack. A few preliminary designs for small pane panels have been presented.

Chapter 8

Response of Glazing to Explosive Blast

8.1 Preface

Glass should ideally be designed in order to be unbroken when exposed to blast or to remain in place when it is broken. Figure 8.1 shows an explosion caused by a mined car.

Figure 8.1 Urban explosion, *Button and Pye, 1993.*

Using laminated glass helps to improve the resistance of glass. A highly plastic interlayer (Poly Vinyl Butyral) will absorb the explosive energy in deformation and provide resistance, *Saflex, Security Glazing Design Guide, 1983.*

Figure 8.2 The use of laminated glass kept the glass in place when broken at the Docklands, *(Times, 1995).*

Figures 8.2 and 8.3 show how a structure's facad can be damaged when exposed to explosive blast. In addition to the building losses and how much it costs for the building to be restored as discussed in Chapter 2, the personal injuries to both the occupants and people on the street are important to avoid, as people's safety is the most important aspect.

Figure 8.3 Docklands damaged structures. *(Daily Telegraph, 10.2.1996).*

3. *Nature of Blast Loading*

The blast wave generated by a high explosive is characterised (as discussed in Chapter 3) by a rapid rise to a peak over pressure followed by an exponential decay back to ambient atmospheric pressure, this is known as the positive phase duration. There is a subsequent negative phase of pressure below ambient pressure.

An explosion does not alert its target, even using any advanced warning system of impending destruction, for explosive blast travels faster than the speed of sound. The primary mechanism of an explosive blast is a forceful blow from the instantaneous pressure rise in its shock front. This is followed by the crushing effect of blast over-pressure (pressure above atmospheric) and a blast wind of super-hurricane velocity.

It is important to note that, although the negative impulse of the blast wave appears to be less than the positive impulse, the natural period of glazing system is such that windows often fail outwards during the negative phase.

4. *Experimental strategy*

As stated by *M. Davies, 1997* "in order to have confidence in the design of protective structures, it is necessary to carry out validation trials". It was also discussed in the same reference that due to the "complicated response" structures have when exposed to explosive blast loading, the only way to have "sure way of evaluating performance" is to examine the structure with full scale testing.

"Such testing is however prohibitively expensive, particularly if repeat testing is to be conducted, and alternative methods and techniques have been used to reduce the amount of full scale testing necessary", *(A. Williams at. al, 1997).*

In order to prove that "small is better" the smaller the area of glass the stiffer it is, the following investigations were under taken to evaluate the hypothesis:

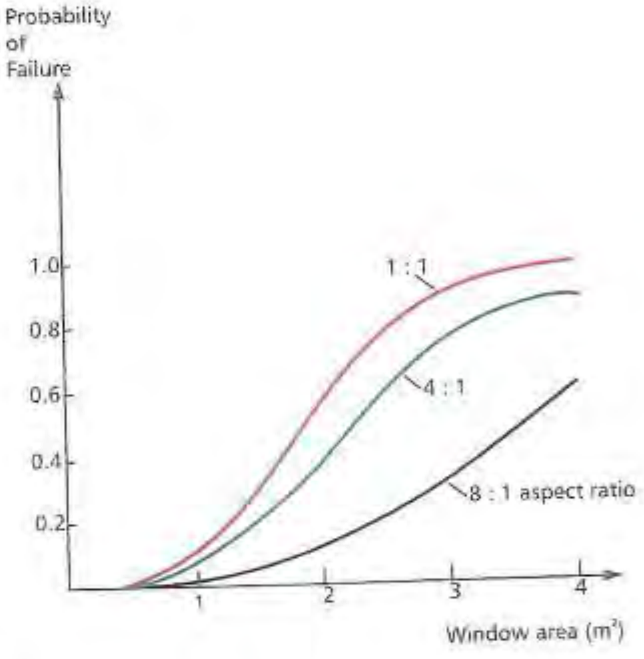
- Dynamic analysis.
- Finite element simulations.
- Experimentalwork at model scale.

8.2 Dynamic analysis

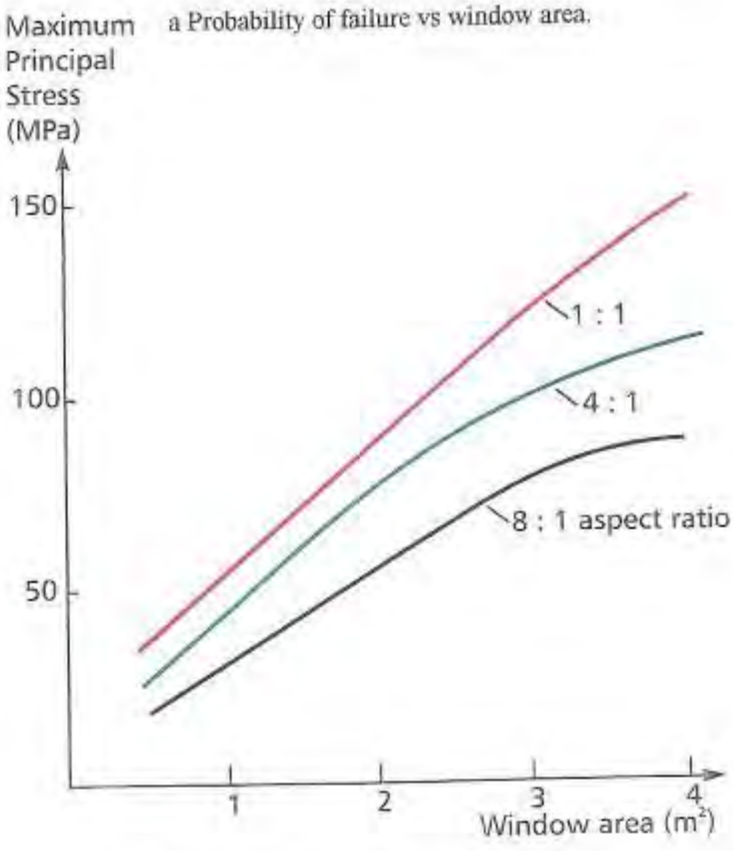
It is not straightforward to analyse the response of a glazing system to the blast loading caused by an explosion. Complications arise from the shape and material behaviour of the glass, the frame supporting the glazing and loading details, such as angle of incidence and the effects of reflections. The analysis can be made easier by making a number of simplifying assumptions and this analysis was published in a paper by *El-Kadi et. al* presented at the 8th International Symposium on Interaction of the Effects of Munitions with Structures, Washington, 1997.

The analysis studies the effect of varying the unsupported area of glazing. In particular, it is assumed that the glazing is made of annealed glass, as it is the weakest type of glass and if it is proved that this type could be hardened, laminated and toughened glass will have an additional resistance. Annealed glass, a homogeneous material, that is rectangular in shape and simply supported around its periphery is considered.

The blast loading is represented by a pressure that is uniform over the area of glazing and decreases linearly with time, *Meyers and others (Meyers and Becker, 1987 and Meyers, 1994)* have developed design procedures for security glazing. These procedures reduce the analysis of a rectangular plate subject to time varying uniform loading to that of an equivalent single degree of freedom system. The resistance function of the glazing is represented by a segmented linear function.



a Probability of failure vs window area.



b. Maximum principal stress vs window area.

Figure 8.4 Response of 8 mm thick annealed glass to blast loading from 500 kg of TNT at a range of 200m *(El-Kadi at el, 1997)*.

A time integration method is used to determine the response of the glazing to blast loading. The advantages of reducing the unsupported area of glazing and of using high length to breadth aspect ratios can be clearly demonstrated using these techniques.

Figure 8.4 illustrates the response of 8 mm thick annealed glass to blast loading from 500 kg of TNT at a range of 200m. The two graphs show the variation with area of the maximum principal stress in the glass and its probability of failure *El-Kadi et.al 1997*. Aspect ratios between 1:1 and 4:1 cover the majority of normal glazing systems.

The reduction in stress and probability of failure as the aspect ratio increases extends as the ratio is further increased. Note, as shown in Figure 8.4.a, the reduction for aspect ratios of 8:1.

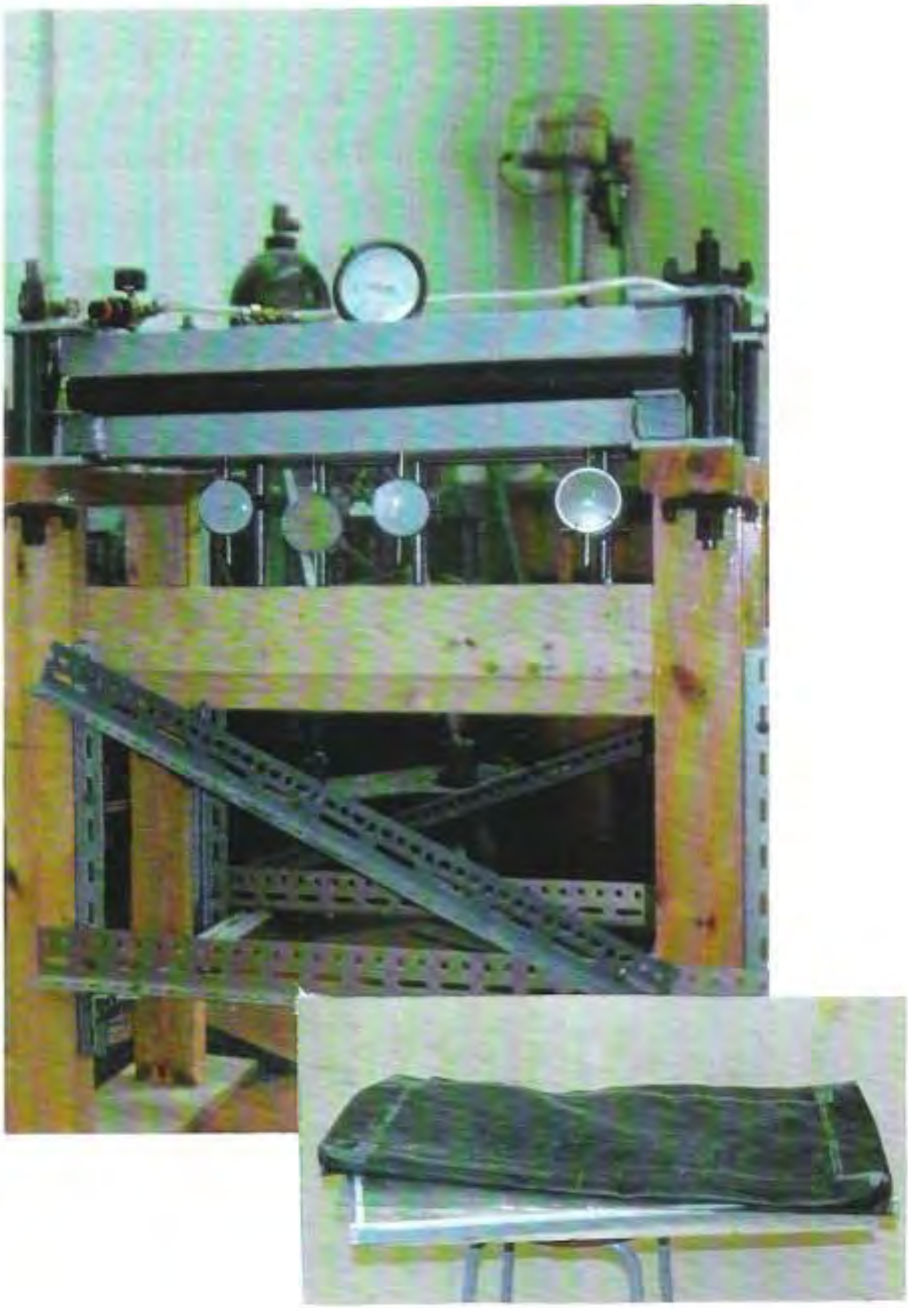
The increased resistance to blast loading of a smaller window can be illustrated in another way. An 8 mm thick, one meter square annealed glass window would be shattered by the blast loading from a 500 kg TNT charge within a range of about 100 m (95% probability of failure).

If the unsupported area were to be reduced to 0.25 m^2 by dividing the window into four smaller panels, the critical range for a comparable probability of failure is reduced to about 50 m. Alternatively, at a range of 100 m the window might withstand the blast from more than 3500 kg TNT. These calculations are based on the simplistic assumptions described above and may not accurately predict a complex real situation.

The principles of their comparison should, however, still be true. By dividing the window into four, the range from the explosion for which some windows might survive is reduced by a factor of two or the size of explosion that can be resisted is increased by a factor of seven. These calculations demonstrate the advantages of small areas of glazing.

8.3 Experimental work

Experiments were conducted in this research with the aim of studying the influence of intermediate mullions on the strength of glazing subjected to a uniformly distributed static loading and to examine the strength of glass with respect to its unsupported area. For substantiating this idea, it was prudent to undertake convenient testing procedures taking into account the difficulty of testing glass samples of different dimensions using explosive charges because of the difficulty of achieving blast conditions at full scale in a range environment. Accordingly, it was decided to perform experimental testing making use of the available equipment for static testing.



One- There is likely to be a considerable difference between the material behaviour when it is subjected to either dynamic loading or static loading.

Two- There is a substantial difference between the material behaviour when it is tested with is natural (full scale) dimensions and the dimensions used in an experimental programme (scale effect).

1. Applying a pressure from an air bag directly located on the surface of the glass

The experimental program was commenced with testing the resistance of glass samples by subjecting them to pressure from an air bag directly located on the glass surface. The loading platform has composed of two wooden surfaces sandwiching the air bag. The air bag was filled with compressed air. The deflection of the glass was recorded against increasing loading. Figure 8.5 illustrates the testing frame and the air bag together with the strain gauges for measuring the deflection of the glass.

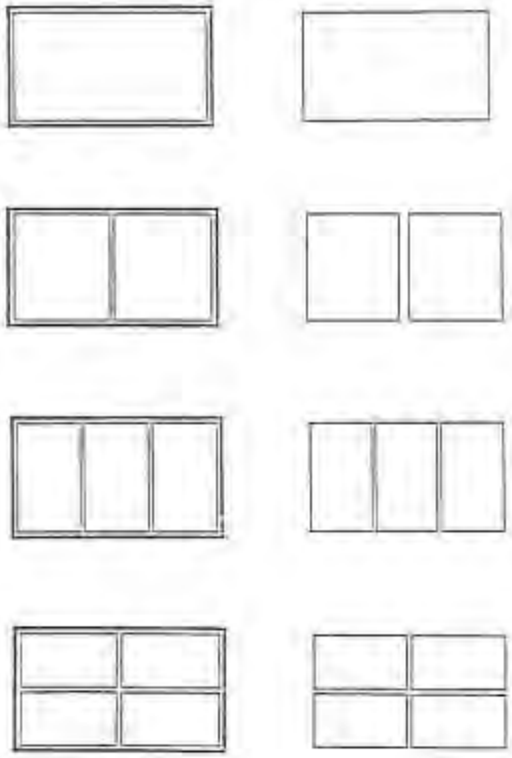


Figure 8.6 Multi-Panel Glazing Configuration with Supporting Mullions, (delineating the panel layout on the left and the glass pane sizes on the right), *(El-Kadi et-al, 1997)*

Different sizes of the same glass were examined by applying the same loading conditions. The glass samples tested were 4.00 mm thick and of dimensions 71 × 43 cm. Panes were also subdivided into two equal portions of dimensions 35.5 × 43 cm each and other test samples were further subdivided to four subdivisions of dimensions 35.5 × 21.5 cm. Figure 8.6 shows the different pane sizes.

The results of the initial testing revealed suspected shortcomings in the loading procedure. There appeared to be a non-uniform distribution of the load.
Results of the pressure applied through an air bag on the surface of different glass pane sizes have been excluded for the following reasons:

a. The pressure records taken refer to the air pressure and it is not clear that this is the pressure on the surface of the glass.

b. There is a doubt that the glass failure might occur due to the uneven pressure due to the deflection of the glass when increasing the air pressure. Hence the pressure is not equally distributed.

2. Modified Procedure

Initially, the loading frame was placed in an Instron 1255 compression testing machine and the air bag was used as before to apply the load to the glass. The arrangement still failed to produce consistent results.

As a second development of the testing procedure, the air bag was inflated to 0.50 bar and then the load applied using the ram at speed rate 5 kN/sec. However, some discrepancies in the pressure-deflection readings were found which were attributed to the deflecting of the hollow metal mullions (18.0 mm by 18.0 mm and 1.00 mm thickness).

Moreover, separation of welding of the intermediate mullions with the periphery frame was noticed due to the concentration of stresses on the mullions and the consequent non-uniformity of the load application. Finally, the force applied by the loading head of the Instron 1255 servo-hydraulic machine was spread by steel and wooden plates, an air bag and sand. The sand was used to ensure that the loading was applied only to the glass and not to the surrounding frame.

Therefore, the sand was layered higher than the frame and was contained by easily compressible soft elastomeric foam (draught extruder foam) to avoid any transfer of load to the supporting frame when compressed.

This is the arrangement detailed in Figure 8.7 and illustrated in Figure 8.8. The air bag was made of an aramid reinforced material that became very rigid when inflated adequately to sustain the applied loading.

It is therefore, possible that, as the glazing deflected under load, the applied lateral pressure became less uniformly distributed.

Because of this, the 4 mm glass originally specified and tested was replaced by 6 mm thick glass that gave reduced maximum panel deflections of about 4 mm.

The following loading procedure was adopted:

1- The glass and the frame were put in the hollow tray supported by wooden posts.

2-Three LVDT's (linear variable displacement transducers) were placed under of the pane to monitor the displacement of glass in each case as follow: one transducer was placed under the centre point and the other two at quarter points of the long span, the centre of the short span direction using the outputs from the LVDT's, load/deflection graphs (scaled in the X direction was the deflection with each 1 cm = 1 mm deflection of glass, and on the Y axis each 1 cm = 2.5 kN.) were plotted using the control panel of the Instron test machine.

3- A layer of sand was applied to the top of the glass to ensure the distribution of the load on the glass, and to minimise any reaction from the mullion the sand was edged by draught extruder.

4- The air bag placed on the top of the sand was inflated with 1 bar pressure.

5- The load was applied at a rate of 5 mm/min.

6- As the load increased the glass deflected up to its breakage point. The experiment was applied to different sizes of panes.

7- Each pane size was tested 3 times to give an average reading.

For the control test, a single 6 mm thick panel of annealed glass was simply supported around the periphery by the aluminium frame to give an unsupported area of 0.69m×0.368 m. This size, which is about a third of a typical curtain walling module, was dictated by the loading system.

A thin clear self-adhesive film was adhered to the upper surface of the glazing to contain the glass when it fractured under load. The load required to fracture the glass was then measured. A last modification was introduced to the testing procedure by neglecting totally the air bag and depending mainly on the sand as a sole medium between the loading ram and the

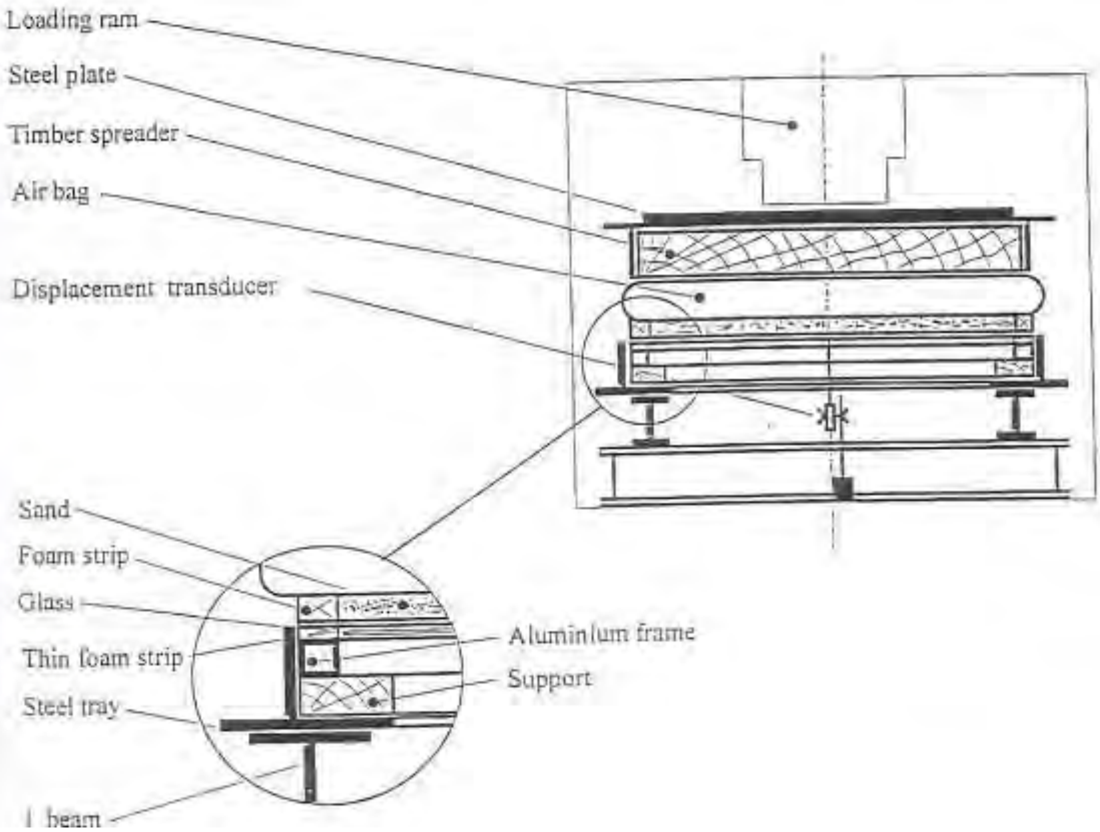


Figure 8.8 A photograph for the loading system showing air bag sandwiched between two frames .

glass with giving much care to the adjustment of the level of sand relative to the frame by the use of draught extruder foam. The last modification seemed to give the best results and was applied on P.V.C samples with similar dimensions as the original glass test pieces but using glass 6.0 mm and 8.0 mm thick.

3. Configuration of panels

The curtain wall panel was represented by a panel of glass of normal annealed glass 6 mm thick. The glass rested on a frame of aluminium with square cross section (18 mm ×18 mm and a 1.2 mm thick wall). A draught excluder strip was introduced in between the glass and the frame to represent the gasket normally used in frames.

The glass panel dimensions were:

1. Full size panel 73 ×42 cm.
2. Two panels of glass 36.5 ×42 each
3. Three panels of glass 42 ×24.3 each.
4. Four panels of glass 36.5 ×21 each

Figure 8.9.a One single pane of glass rested on aluminium frame.

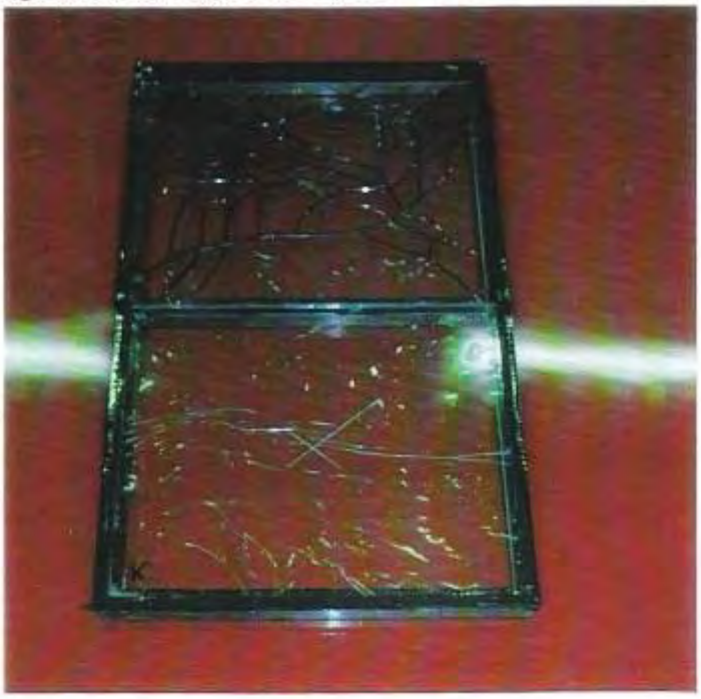


Figure 8.9.b Two single panes of glass rested on aluminium frame.

Figure 8.9.c Three panes of glass rested on aluminium frame.

Mullions were made of aluminium using a 18 mm × 18 mm hollow box section (wall thickness 1.20 mm) .The mullions were welded to the outer aluminium frame to make the panels shown in the Figure 8.9.a, b, c, and d.

These configurations divided the glazing into two, three, and four smaller panes respectively. The glass was cut to size and positioned on the surrounding frame and mullions using thin self-adhesive elastomeric foam.

Figure 8.10 shows a full pane of glass rested on an intermediate mullion. The photograph shows crack parallel to the intermediate mullion. The failure load of the one pane size and the same pane with mullion were very close to each other. The results of this experiement indicated that the intermediate mullion under a single pane did not give improved load capacity.

Figure 8.10 One single continuos pane of glass rested on aluminium frame with intermediate mullion.

Figure 8.11 shows a smaller mullion that was also tried. Again, there was no improvement in the strength of the system. Hence, the breakage load is similar to that of the continuous pane without a mullion. The loads required to fracture the various panels were measured and compared to the failure for the single pane version. Figures 8.12 (a, b, c) show the fractured PVC framed glazing samples after testing to failure.

Figure 8.11 One single pane of glass rested on aluminium frame, thin sheet element of aluminium used to support the glass in the meddle of the pane.

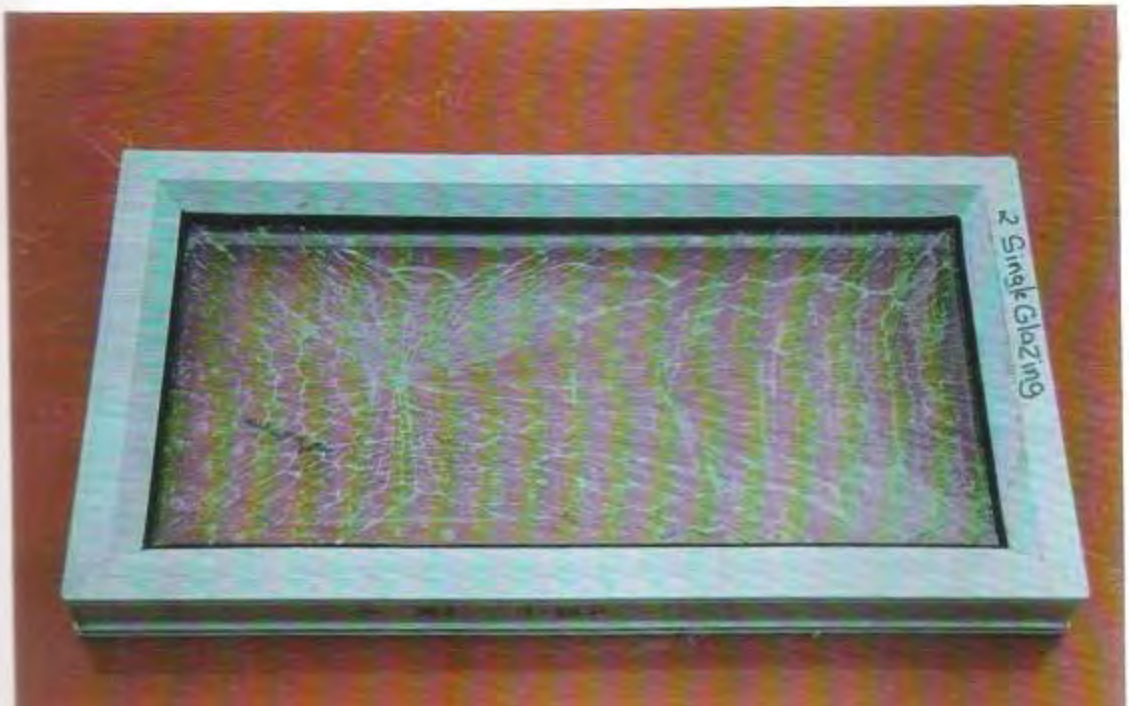


Figure 8.12.a The patterns of fractured glazing samples as revealed from the experimental test.

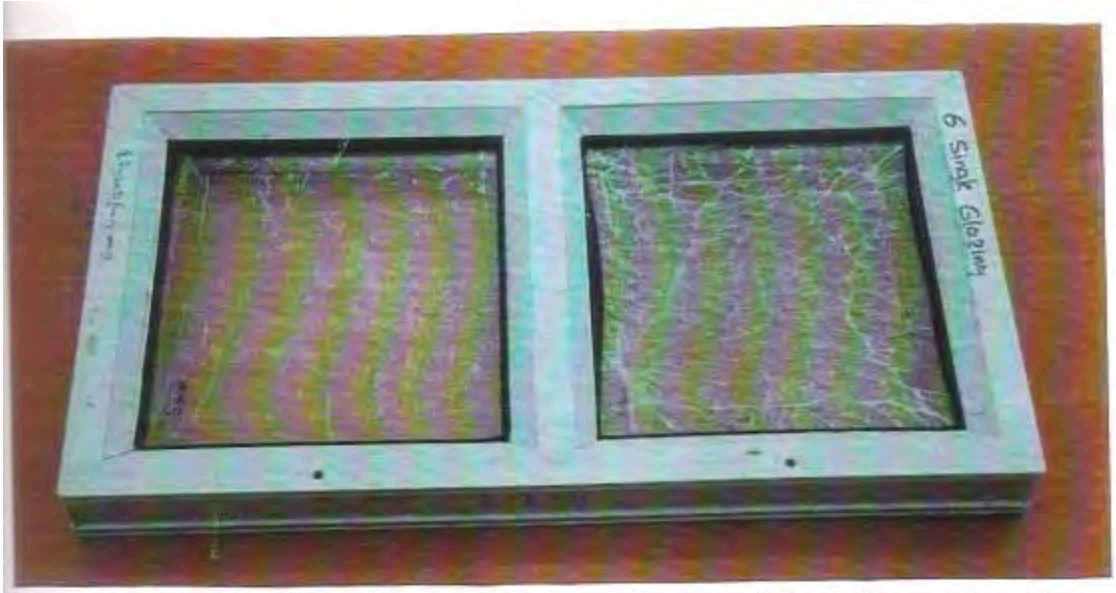


Figure 8.12.b The patterns of fractured glazing samples as revealed from the experimental test.

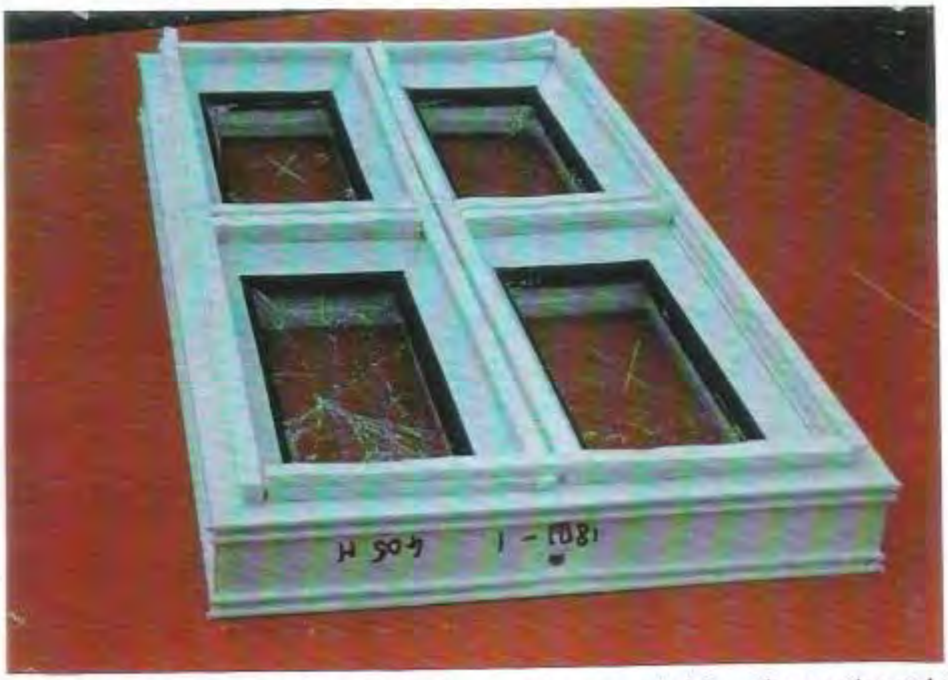


Figure 8.12.c the patterns of fractured glazing samples as revealed from the experimental test.

4. Testing results

Figure 8.13 shows the typical fracture results of 4 mm glass pane rested on aluminium. Four curves are used to represent the four different pane sizes. Table 8.1 gives the maximum loads before glass failure and the deflection reached. The results do not bear out the proposed hypothesis that dividing a glass panel into smaller sized panes increases the loading capacity to any clear extent. There is some increase in failure load and some increase in deflection. However, concern about the effect of deflection on the uniform loading assumed led to the use of thicker glass.

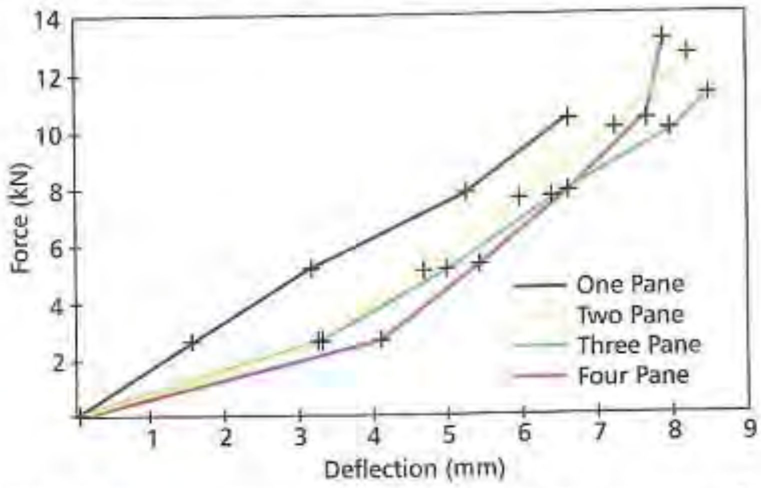


Figure 8.13 Fracture results of 4 mm thickness, glass pane rested on aluminium mullions "effect of pane size"

Table 8.1 Failure Results for 4 mm Glass

Pane Arrangement	Pane size	Failure load	Deflection at failure
One pane	73 × 42cm	10 kN	6.5 mm
Two panes	36.5 × 42	12 kN	8 mm
Three panes	42 × 24.3	11 kN	8.5 mm
Four panes	36.5 × 21	13 kN	7.5 mm

Figure 8.14 shows typical fracture results of 6 mm thickness glass panes rested on aluminium mullion. Four curves represent the four different pane sizes are shown. The failure conditions are listed in Table 8.2.

Table 8.2 Failure Results for 6 mm Glass

Pane Arrangement	Pane size	Failure load	Deflection at failure
One pane	73 × 42cm	11.5 kN	5 mm
Two panes	36.5 × 42	17.5 kN	8 mm
Three panes	42 × 24.3	16 kN	6.3 mm
Four panes	36.5 × 21	17.5 kN	9 mm

Note: The failure loads given are average results for the three tests

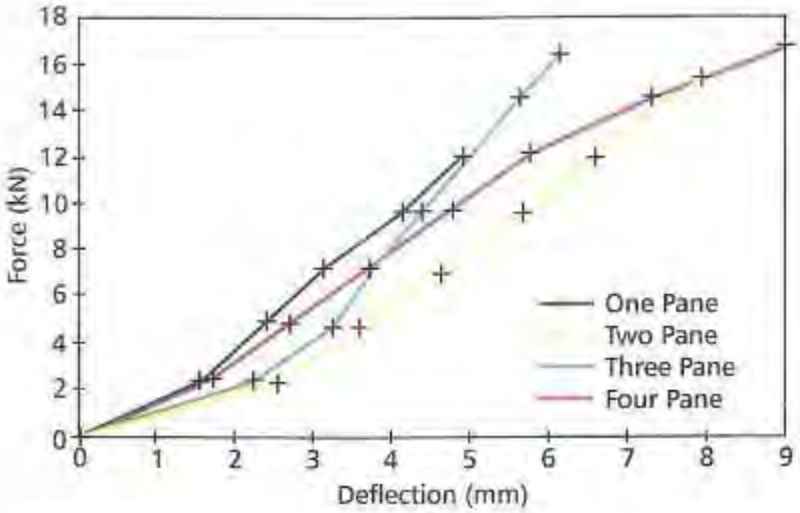


Figure 8.14 Fracture results of 6 mm thickness glass panes rested on aluminium mullions "effect of pane size".

The 6 mm glazing gives more clear cut results, though the sub-division of glass area beyond two panes does not show a progressive improvement. The capacity increase is about 40-50% for the multi-pane panels.

Figure 8.15 shows the effect of the pane size reduction, when it is fully supported around its periphery in a properly designed PVC frame. (It is interesting to note that according to a manufacturer's advice (Glostal) PVC frames are not considered suitable for glazing protection against blast, because they might melt when exposed to the fire, which usually accompanies blast). These frames were used in order to investigate the influence of panel edge support. The failure conditions are listed in Table 8.3.

The two curves shown in Figure 8.15 represent typical results of PVC frames carrying 6 mm glass. The first curve represents one single pane frame. The second represents two single panes supported with an intermediate mullions. Failure results are given in Table 8.3. Improvements in capacity for the 2 pane design are of the order of 33%. Figure 8.16 shows a comparison between the behaviour of glass when it is fully supported around its periphery with PVC frame and its behaviour when it is just rested on an aluminium frame without any further peripheral support.

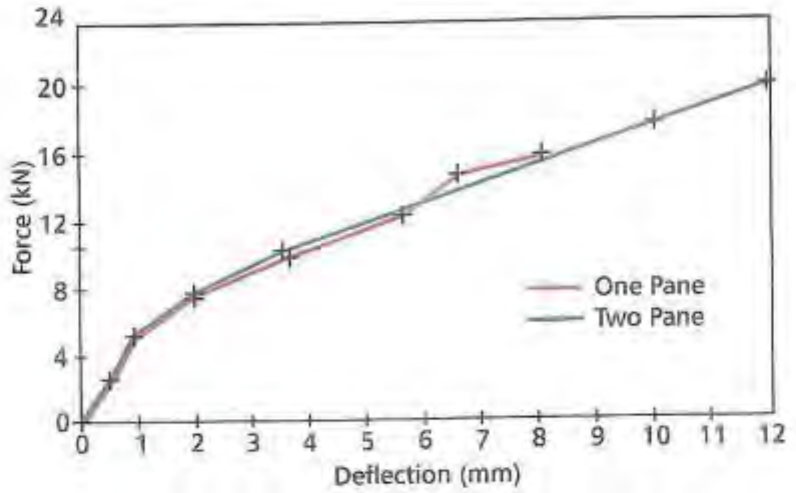


Figure 8.15 Fracture results of 6 mm thickness single glazing glass using PVC frames

Table 8.3 Failure Results for 6 mm Glass using PVC frames

Pane Arrangement	Pane size	Failure load	Deflection at failure
One pane	73 × 42cm	16 kN	8 mm
Two panes	36.5 × 42	20 kN	12 mm

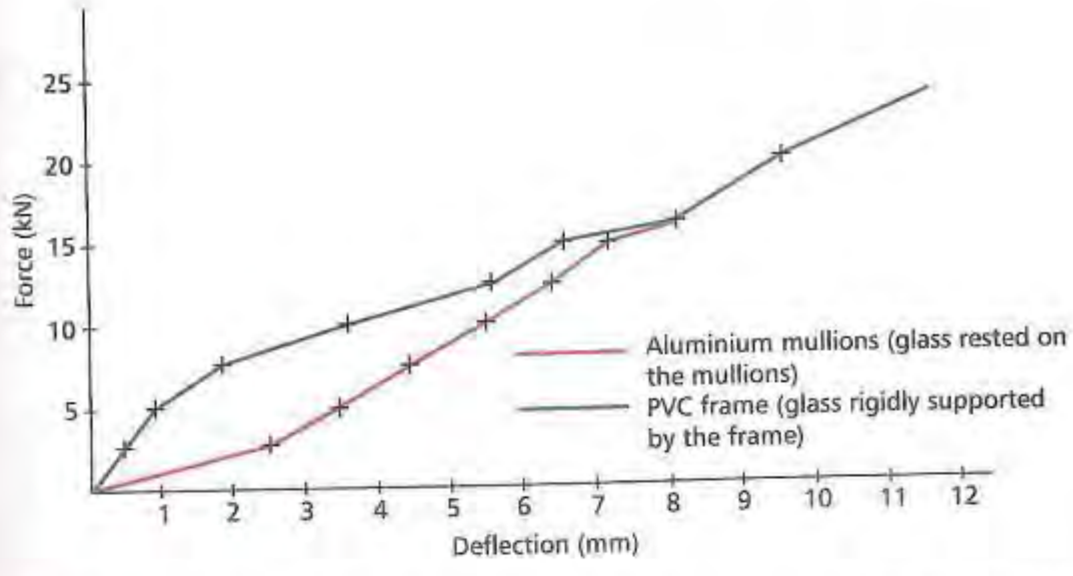


Figure 8.16 Fracture results of 6 mm thickness single glazing glass showing the effect of supporting conditions on the glass stiffness.

uniformly distributed static load. The second analysis considered the areal curtain wall glazing panel, and recorded stiffness variations when it was divided into smaller panels.

Finite Element method is regarded as a powerful tool for solving the most sophisticated problems in structural mechanics. With the tremendous evolution of the electronic computers it becomes feasible to discretize the structure to very fine elements of finite dimensions called 'finite elements'.

The original body or the structure is then considered as an assemblage of these elements connected as a finite number of joints called nodes or nodal points.

The concept of discretization is to formalise the force and displacement methods of analysis using energy theorems of structural mechanics *Zienkiewicz, 1989.*

The properties of the element are formulated and combined to obtain the solution for the entire structure after specifying a certain shape function to approximate the variation of displacements within an element in terms of the displacement of nodes of the element.

Then the principle of virtual strain energy is used to derive the equation of equilibrium of the element and the nodal displacements will be the unknowns in the equations.

The equations of equilibrium for the entire structure or body are then obtained by combining the equilibrium equation of each element such that the continuity of displacement is ensured at each node where the elements are connected.

The necessary boundary conditions are imposed and the equations of equilibrium are solved for the nodal displacements, subsequently, the strains and stresses are assessed using the element properties derived at the early stage which is the relation matrix between strains and displacements and the matrix relating stress and strain of the constituent elements. It is worth noting that it is not intended to go deeply into the numerical finite element calculations. However, the scope of its application is limited to the comparison between the recorded displacement and stresses in the experimental results with those assessed from the finite element analytical model.

2. *Finite element analysis for the experimental samples*

Each of the experimental configurations described in the last section was analysed using the general purpose finite element code, COSMOS/M, *COSMOS/M, software package, by Structural Research and and Analysis Corporation, 1993.*

The glass was modelled as a 0,348m $\times$ 0.148m panel of 6mm thickness using 24 $\times$12 thin shell elements with 4 nodes per element. This represents one quarter of the unsupported area of the glazing. Boundary conditions were used in order to impose the symmetry and simple supports were used around the outer edges of the complete panel. Uniformly distributed pressure loading was applied to the surface of the glass.

Using small displacement linear elastic analysis, values of maximum direct stress and displacement in the glass were predicted for an arbitrary loading. Assuming a maximum principal stress failure criterion of 70 MPa (a typical strength of annealed glass), the loading and displacements associated with failure initiation could be calculated. The predicted failure load was lower than that measured experimentally; though the experimental results are average values. A slightly higher failure load was predicted using non-linear large deflection finite element analysis but the predominant reason for the discrepancy was the assumption of uniform loading. Significantly higher failure loads were predicted when the loading was assumed to inrease (linearly) towards the edges of the glazing. No data was available to quantify this variation. It is, however, reasonable to assume that the non-uniformity of loading was similar for each experimental configuration and, as a consequence, for the relative load carryng performance to be compared with a reasonable accuracy (Table 8.4)

Table 8.4 Load bearing capacity of Multi-pane glazing with mullions

Glazing Configuration	Failure load relative to single panel system	
	Experiment	Finite element
2 panes	1.52	1.41
3 panes	1.39	1.40
4 panes	1.52	1.39

The mullions in the finite element analyses were modelled as beam elements with the same nodal degrees of freedom as the shell elements used for the glass. Common nodes were merged. These beam elements represented the 18mm ×18mm hollow box mullions used in the experiment.

The increase in load bearing capability of the glazing systems containing intermediate mullions is shown in Table 8.4 A reasonable correlation with the experimental results is also demonstrated.

3. Finite element analysis for the curtain wall

Finite element analysis of a curtain wall module was again performed with the commercial general purpose finite element code COSMOS/M. The glass curtain walling was represented as a 1.5m wide, 3m high repeating module. An area of 8mm thick glazing was supported around its periphery by an aluminium frame (Section A in Table 8.5) with section properties typical of those found in a curtain wall elevation.

The glazing was modelled using 32×64 thin shell elements; the frame was rigidly supported at its four corners. With a uniform lateral pressure loading of 10^4 Pa (0.1 bar) applied to the glazing, a linear elastic analysis of the above module gave a maximum deflection of 49.5mm at its centre and a maximum direct stress of 171MPa in the glass at positions half way along each longest edge. Glass would fail at a stress of about half this value i.e. the loading of 0.1 bar could not be sustained. Further analyses were performed with mullions fixed to the glazing. These mullions (which were modelled as beam elements) were assumed to be hollow box beams made of aluminium alloy with the section properties given in Table 8.5 Section A is that used for the frame surrounding the module. With a overall width of 7.5cm and a depth of 15cm, it represents a very heavy mullion. The wall thickness is 5mm across the width of the section and 2mm along its depth. section B, with an overall width of 6cm and a depth of 8cm, represents a more typical section for use as a mullion. Section C, with an overall width of 2cm and a depth of 4cm, represents a small section that could be considered for the multi-mullioned designs proposed. The wall thickness of sections B and C is 2mm. As can be seen from Table 8.4, there is a reduction in the second moment of area (I) of each section and hence its bending stiffness, by a factor of about 10 for each change. Analyses were performed with the area of the glazing supported both horizontally and vertically by 2,4 and 8 mullions respectively along each side.

Table 8.5 Clarifying geometrical properties of the beam element

Section Code	Cross- Sectional Area (cm2)	Moment of Inertia (cm4)
A	13.10	486
B	5.44	51.1
C	2.24	4.5

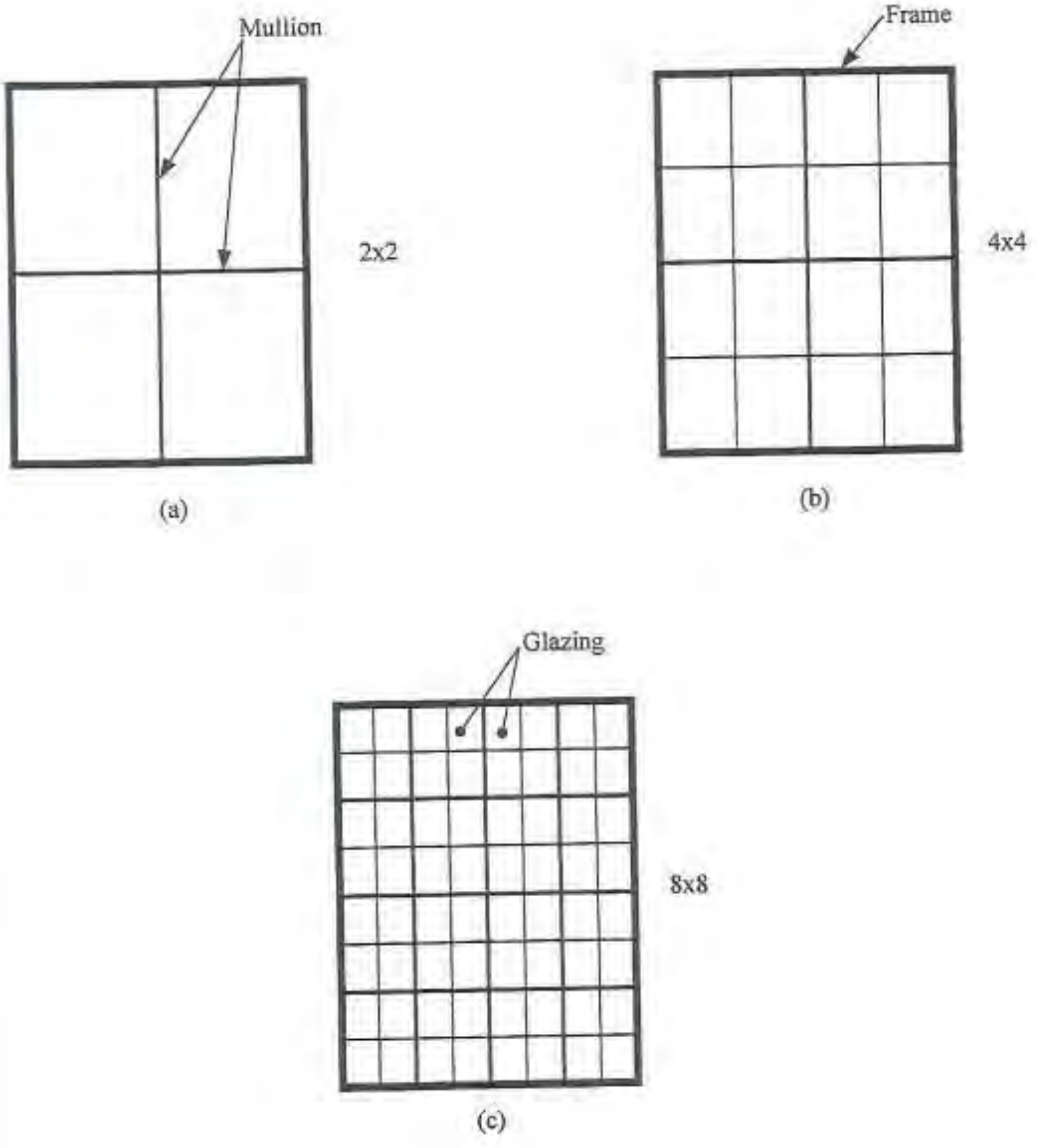


Figure 8.18 Mullion configurations

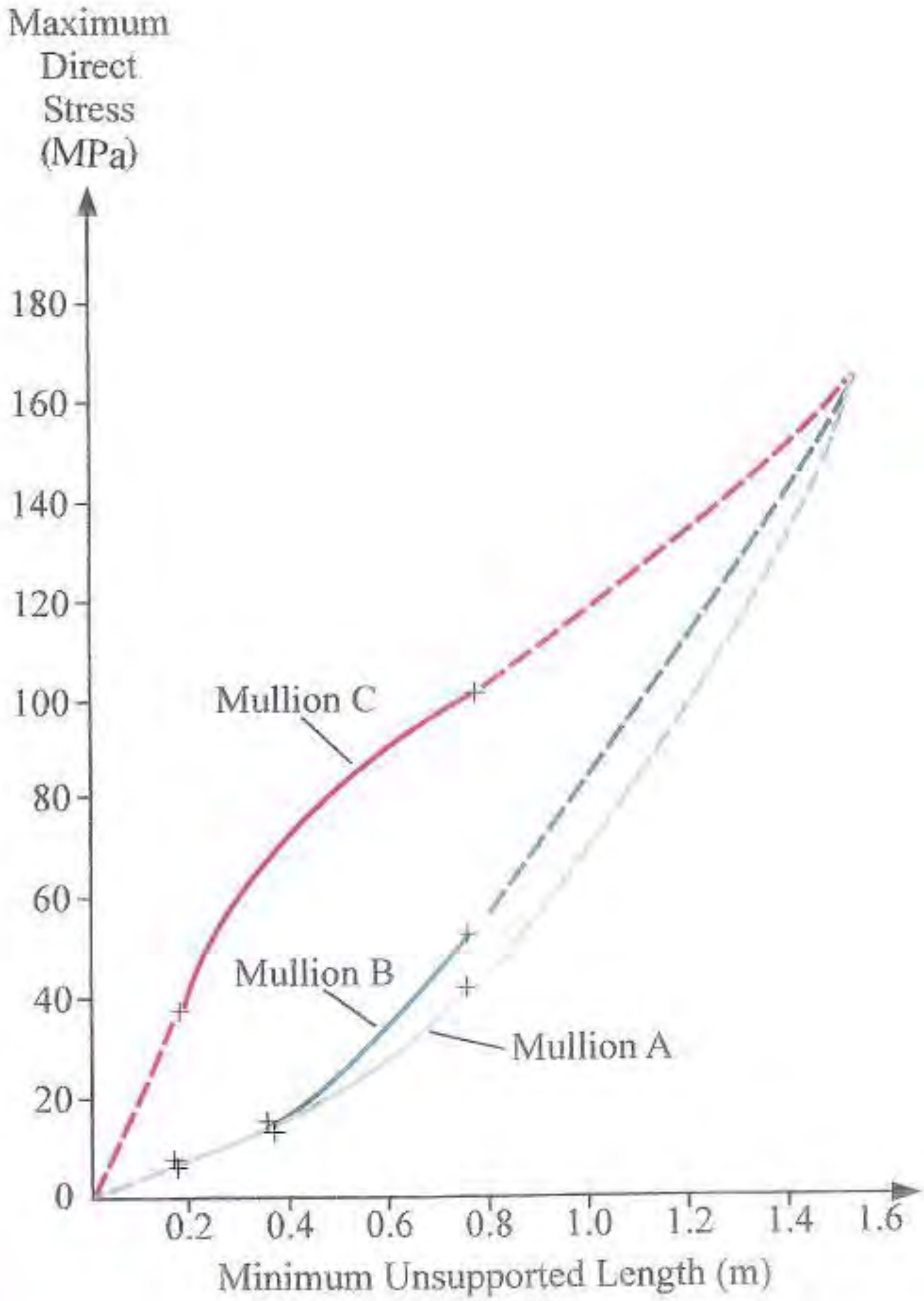
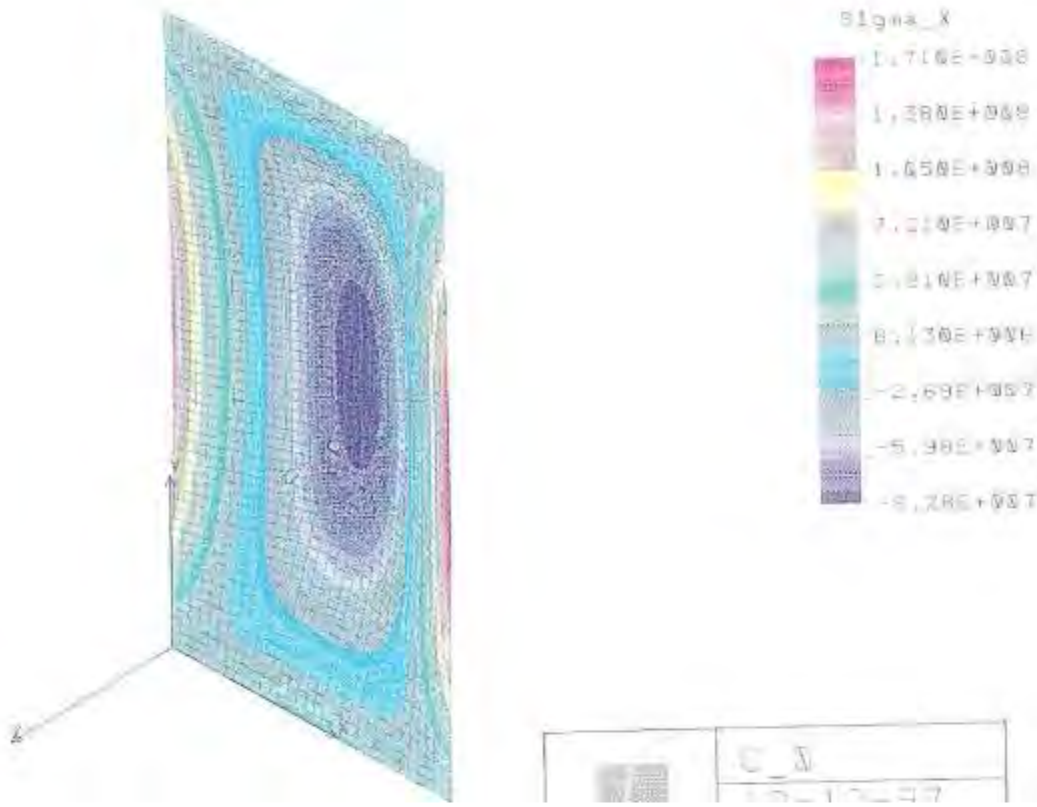


Figure 8.19 Maximum direct stress in curtain wall from F.E Analysis as a function of maximum unsupported length of glazing.

The single unsupported glazing area of 1.5×3m in the curtain wall module with no mullions was, therefore, divided into 4 equal areas of 0.75×1.5m, 16 equal areas of 0.375×0.75m and 64 equal areas of 0.1875×0.375m respectively. These configurations are illustrated in Figure 8.18.



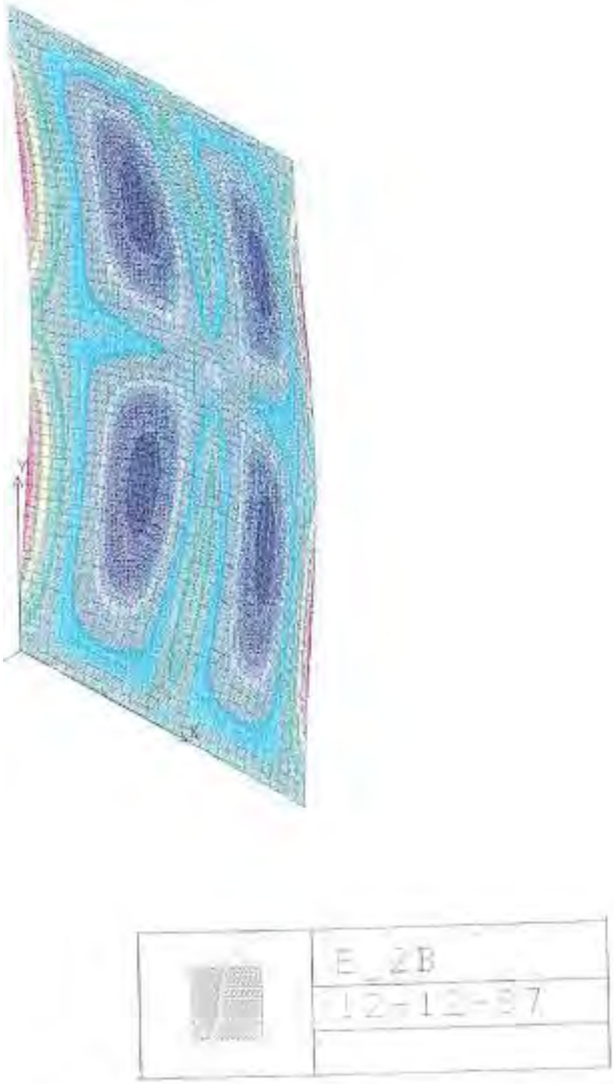


Figure 8.21 Stress contours in glass "Case b" Cosmos/M output

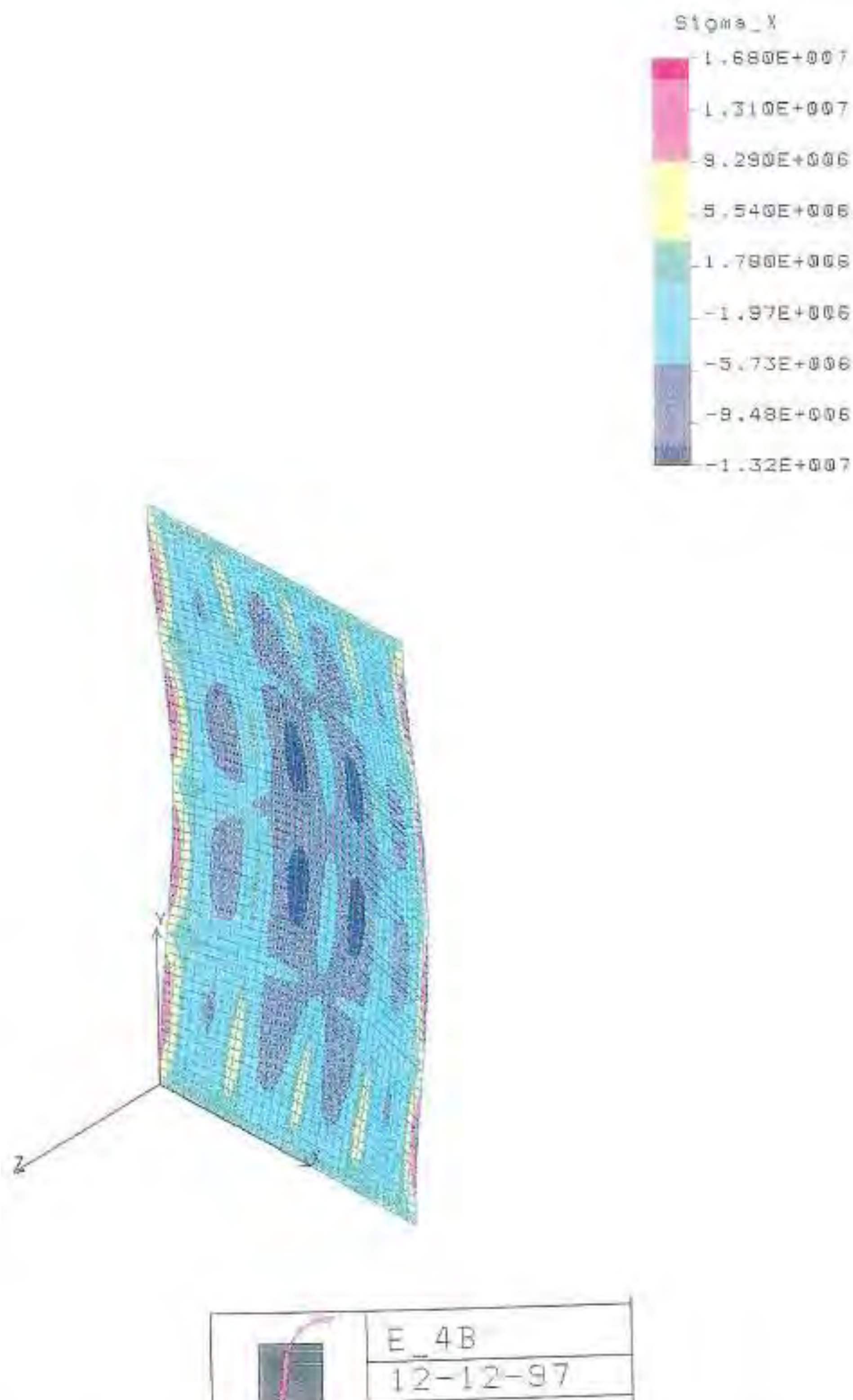


Figure 8.22 Stress contours in glass "Case C" Cosmos/M output

The magnitudes of maximum direct stress for each case are plotted as a function of the minimum unsupported length of glazing in Figure 8.19. Using both the the heavy mullion section A and the typical mullion section B, the stress levels are reduced to well below the failure strength of the glass. The same is true when using the small mullion section C for the 64 pane multi-mullioned system.

Deflections are similarly reduced by using mullions. With mullion sections A and B the maximum deflections are generally less than 10mm and with 8 section C mullions per side the deflection is less than half that of the system with no mullions. In all cases the maximum deflection was at or near to the centre of the the module. The maximum direct stresses occurred along the long (vertical) edges of the glazing. As the mullion stiffness increased, the position of maximum stress moved more towards the corners.

By increasing the glass thickness from 8mm to 12 in the 8 panes adjacent to each long (vertical) edge for the 64 panel module using mullion C, the maximum stress and deflection were reduced by about 10%.

The convergence of the analysis was checked using coarser 16 x 32 element mesh for the glazing and consistent coarser meshes for the frame and mullions. This generally showed differences in stress of less than 1MPa. The modelling does however make a number of simplifying assumptions. Perfect fixity is assumed between the glazing and the frame and mullions. A real curtain might be somewhat weaker but the principles of the compaison would still hold. Figure 8.20, Figure 8.21 and Figure 8.22 present stress contours regarding the stresses generated in cases (a), (b) and (c) previously delineated in Figure 8.18.

The results demonstrate the benefits that accrure from using mullions and smaller areas of unsupported glazing. The increased strength would also be apparent if the systems considered were subject to blast loading instead of the static loading used for the simulations described above. The detailed desgn and specification of the frame, mullions and glazing (and their fixing) would have to address the level of protection required *M.J. Iremonger et al 1997*. The data file of COSMOS/M is given in Appendix B.

8.5 Concluding Remarks

The widespread use of glass in modern buildings can pose a danger to personnel both within and outside the buildings if they are subject to terrorist attack using explosives. It is not practical to harden such buildings on any large scale. An alternative approach, that is addressed in this chapter, is to reduce the the unsupported size of glazing.

Dynamic analysis shows that when the glass is fully supported around its periphery, its resistance to blast loading is significantly increased. However, these analyses take no account of the flexibility of intermediate supports (mullions). To address this deficiency, static loading expriments and finite element analyses have been conducted on glazing panels with mullions to subdivide the unsupported area of glazing.

The experimental work was carried out at model scale and for this reason the realism of the models was compromised and peripheral support was much simplified. An attempt was made to overcome this by having small PVC panels made up by a commercial window supplier. It was hoped that the experiments would give comparative results rather than absolute strength values. In the event, there was no progressive improvement shown. Early on the 4 mm glass was abandoned because of concern that excessive deflection was affecting the uniformity of the loading. 6 mm glass appeared to give better results. The change from a single pane to a two pane design showed a significant sterngth increase. Intnitively, this improvement should have been continued with further sub-divisions to three and four panes but this did not happen. A number of reasons may account for this. The aspect ratio of the glass changed for each panel arrangement. The edge support combinations of fixed peripheral supports and flexible intermediate mullions was different for each arrangement. Finally, there could have been variations in manufacture (eg welded joints) for the intermediate mullions.

In research work illustrated in *M.J. Iremonger et al 1997* , the annealed glass curtain wall panel as used in the experiments was modelled using thin shell elements for the glass and beam elements for the frame of the panel. The finite element was conducted using commercial software package COSMOS/M, nodes of the thin shell element were constrained or tied to those of the beam elements and taking into account that the shell elements are rigidly connected to the beam elements at the panel's perimeter.

The frame was rigidly supported at its four corners. A uniform pressure was applied normal to the plane of the glazing and the finite element process was undertaken via a small displacement associated with failure the predicted load was similar to that measured experimentally. A relatively small increase in the failure load was recorded and a significantly higher failure load was obtained when the load was linearly increased towards the edges of the glazing.

The same programme, COSMOS/M, was conducted on a curtain wall (full scale) 1.5 m wide × 3.0 m high with 8 mm thick glass assumed to be supported by a frame fixed at the perimeter and rigidly supported at its four corners. The glass was modelled to be divided into 32 × 64 shell elements. A uniform pressure of 0.1 bar was applied on the glass and the maximum deflection was obtained. Further analyses were performed with mullions fixed to the glazing. These mullions were also modelled as beam elements and were assumed to be hollow box beams made of aluminium alloy with specific section properties. Three different cross sections used (15, 7.5 and 4) cm depth were used. The 15 cm depth cross section used for the outer edges, the 7.5 used for the inermediate mullions togather with the 4 cm ones to divide the above area by 2, 4 and 8 mullions respectively along each side.

Dividing the panels with mullions casued the stress level to be reduced below the failure strength of the glass. Deflections are similarly reduced when sub-dividing the glass with mullions. However, the maximum deflection was at or near the centre, the maximum direct stresses occurred along the long vertical edges. It was noted that when the mullion stiffness icrease the position of the maximum stress will be moved towards the corners. A10 % decrease in the maximum stress and deflection were obtained when the glass thickniss increased to 12 mm.

Under blast loading, this improvement could mean that the area within which glazing would be broken could be halved. It has also been shown that the use of mullions to reinforce large unsupported areas of glazing can make a significant improvement in the load bearing capacity of the glazing and hence in their resistance to blast loading. Such modifications could be employed in new buildings or even as a retrofit for existing buildings. Architectural styles using small patterned glass panels can be introduced as shown previously (*El Kadi et al. 1997*). The aesthetic quality of buildings can be therefore maintained while providing improved performance and security under terrorist attack.

This gives scope for architects to enhance protection at low cost while providing the functional and aesthetic qualities that is the hallmark of their profession.

Discussion, Conclusions and Recommendations for further studies

9.1 Discussion

It is considered that the topic of this research is a very important subject in today's world. This is due to the wide prevalence of terrorist missions all over the world. Terrorists now aim to attack main public buildings, the infrastructure and commercial and financial buildings; thus provoking the media to publicise their motives and demands, which the terrorists have failed to express through the legitimate channels of expression.

The research presents a definition of the problem under investigation, which is mainly terrorism, and convenient methods of protecting civil structures against the explosive blast waves, accompanied sometimes with high temperatures up to 4000^0C. The different approaches to solve such problems have been elucidated. These are highly dependent on structural and architectural design and detailing as well as the proper choice of cladding material. This chapter summarises the research:

Three case histories of events that have taken place in the UK have been discussed. The case histories presented are London's financial district bombing, the South Quay Docklands bomb and the Manchester bomb. Each case includes the place and time of explosion, the estimated amount of the explosives, the blast result and the pertinent details of damage. The following was revealed from the presentation of the case histories:

- Building cladding, designed to withstand normal wind pressure conditions, has weak resistance against blast loading.

- Explosions can happen anywhere and in a few seconds civic buildings become worthless. Billions of pounds have been spent in repairing or replacing damaged cladding and glazing.
- Most of the injuries are to people walking on the street who are hurt by the splinters of glass. Special care must be taken in the future to enhance the cladding's resistance, specially the glazing, with the aim of reducing losses to minimum levels.
- Predicting structural response is highly sophisticated. The distance from the explosion, the geometry of the building, the structural form and the cladding materials are all relevant in the analysis of structure response.
- A tall slender building will react better than a short massive one; buildings with atriums perform badly and any apparent difference in performance between concrete and steel framed buildings is usually due to the quality of design and detail.

The research includes a description and analysis of the blast wave characteristics and the corresponding structural response. Among the main subjects discussed in the research are; blast wave, blast wave front parameters, the calculation of the positive and negative phases, blast wave scaling laws, blast wave interaction and blast wave external loading on the structures. The reflection surface and multiple deflections were also studied. The structural response to blast loading was clarified with special attention to the structure vibration duration in relation to the blast wave duration. The Single Degree of Freedom System, is used to simulate the forced vibration of undamped structures in order to obtain their resistance. The blast load positive phase duration and the structure natural period of vibration are the parameters used to define the limits of response. From the study the following conclusions are drawn:

- The public buildings threat criteria covered in this thesis considers the far field or larger area of damage. It is very unlikely that cladding systems can be designed to withstand near field blast effects. This comes from the economical considerations and aesthetical aspects which must be maintained and due to the certain fact that no one can be sure whether an explosive blast is going to take place or not. The maximum weight of explosive charges considered is 250 kg acting at a distance of not less than 50 m.

- The maximum displacement the structure or an element receives when exposed to blast loading defines the degree of damage expected.
- Structural response of the blast load is highly dependent on the structure's period of vibration and the duration of the blast wave. This defines the loading type whether it is impulsive, quasi-static or dynamic loading.
- The damage tolerance of cladding panels represented by P-I (Pressure- Impulse) curves provides a profound assessment of response to loading once a maximum displacement is defined (i.e. a damage criterion has been specified). Both values of pressure and impulse that fall to the left or below the curve mean that when a panel is exposed to such loads will not fail. Consequently those points located at the right and above the graph mean such loads causes panel damage.

The following was revealed from the study of the structural response:

- Under the action of very short duration loads, the high rate of strain load application enhances the mechanical properties of the different cladding materials when compared with the corresponding properties generated under static loads.
- Although the mechanical properties of the material is enhanced during rapidly applied loads, the design dynamic stress values are limited by the deformation or the damage limits imposed. Such limits are expressed via the ductility ratio μ for structural steel elements and the support rotation θ for the reinforced concrete elements.
- The selection of a limited value of μ (ductility ratio) and θ (support rotation) governs the criteria of design and the protection category. It also controls the resulting deformation, whether elastic if $\mu \leq 1$ or plastic if $\mu > 1$ and the subsequent cost of both construction and maintenance.
- When reinforced concrete elements are implemented as cladding materials it is prudent to provide symmetrical reinforcement, especially for the reinforced concrete panels located in protection category 2 i.e. $\theta > 2°$.

Structural steel is better suited to resist blast loading of a quasi-static nature due to its high ductility. While reinforced concrete is still a ductile material, it is more suited to withstand the blast effects of impulsive nature due to its mass.

From the architectural point of view, the thesis discusses the wall cladding as a reflecting mirror of both the technological development and the architectural texture of the environment. The wall cladding is considered as the architect's prime tool, which enables him to express and create a certain impression. In order to preserve this impression he can either fully integrate with the general architectural tissue or make a contrast with it.

The architectural aspects have been presented with special attention to form order principles. The impact of proportions and scale upon the form with regard to cladding panels manufacture and structural proportions as well as the proportion techniques have also been discussed.

The research considered the description of cladding classification and curtain walling systems. The cladding materials could be classified into cementitious materials, masonry materials, metals, glass and plastics. The cladding components include support framing, interior finish, insulation, joints and internal drainage. However, cladding systems include attached systems, infill systems and curtain wall systems. The cladding systems and materials are concerned mainly with the methods of fixing, joints and panel geometry. Precast concrete, cast in-situ concrete, brick masonry, glass, Glass Reinforced Polyester (GRP) and metal cladding have been discussed in the current research. The main factors governing the selection of cladding systems and materials include the environmental criteria, structural criteria, aesthetic criteria, erection criteria and maintenance criteria.

The blast effects on the architectural concepts have been demonstrated and it has been found that the subtracted forms are the most susceptible to damage. This is attributed to the multiple reflection that takes place and its subsequent blast wave magnification inside the subtracted space.

It was highlighted that a space distribution varies according to its value (content), so that the substantial spaces are located at the heart of the mass and the other lower value spaces are distributed in a descending order according to their value.

In the three dimensions, it is recommended that a pyramidal concept order (Floors are horizontally recessed towards the inward direction) is preferred as this concept provides the mass with self protected shaded areas of cladding which will not be reached by the blast load stress waves. Alternatively, the inverted pyramid concept (corbel) floor organisation escalates the subsequent reflection. Also, it was confirmed that the cylindrical and spherical masses are considered to be masses of paramount resistance against blast loading in comparison with the cubical masses.

However, buildings clad with either glass curtains or metal sheets possess weak resistance against blast loading. It is advisable for architects and the cladding designers to increase the building cladding's own resistance against blast loading by the use of solid surfaces with grouped minimum areas of glass and not by using complete glass curtain wall fronts.

Cladding materials are formed into specific systems and such systems need specific components. Cladding material is either heavy like concrete or bricks, or light like glass, and metals, whereas the GRP is a material which is not heavy but it is not light weight cladding. Heavy materials together with the GRP show better behaviour than light materials in their response to explosive blast.

The resistance of heavyweight cladding materials could be enhanced more than light weight materials. In other words, heavy materials could be treated to improve their behaviour when exposed to pressure loads, i.e. the use of double layer reinforcement improves the reinforced concrete panels' own resistance to blast loads. When bricks are tied in two directions to a back up wall their behaviour to blast loading will be improved.

Improving the method of fixing reinforced concrete panels by framing them with two directional mullions will improve the panel behaviour to blast load. Smashed panels could be easily replaced without the necessity to remove all the undamaged panels above or below the damaged ones.

An important point was discussed in the dissertation related to the criteria of failure of cladding material. It was recommended that the forms of failure of cladding materials when exposed to loads beyond its limited resistance needed to be designed. In other words the cladding panel design must adopt the secure failure design criteria.

The damage tolerance of various cladding panels like GRP, glass, aluminium, steel, and reinforced concrete, was investigated using iso-damage curves. The panels were given typical dimensions and thicknesses. Due to the ability of GRP to deflect elastically with low stiffness it might be considered to have good performance. However, the deflection would be excessive and the assumption was made that the GRP panel should only have a limited deflection. When comparing both steel and aluminium graphs one will find that the aluminium reaches the deflection limit at only half the estimated ductility limit. Simple rectangular sections were selected to in order to easily compare between the panels. In reality the geometry of panels is more complex and they are often either profiled or formed in double layers to be more stiff. It was also noted from the iso damage curves that the most damage tolerant cladding material is the reinforced concrete and the least tolerant is the glass.

The thesis then focuses on the use of glass as a popular cladding material. Modern architecture often features large and extensive glazed areas, either with other cladding materials or as a complete glass curtain wall on its own. Such buildings have been shown to be extremely vulnerable to terrorist attacks as they have recently directed their attacks towards city centres. The benefits terrorist could get from attacking such centres are enormous compared with those they get when attacking military installations.

Glass breakage, due to its fragile nature when exposed to the blast loads produced by terrorist attacks, results in increased losses to both people and installations. People inside the buildings or walking in the streets, even at distances far away from the explosion, could get injured by flying splinters. Also computer systems and air-conditioning ducts could be highly affected by the broken glass.

The idea developed in the current thesis is to enhance the resistance of the glazing panels against blast loading by using small sized panes in patterned form.
It has been demonstrated that smaller glazing panels provide enhanced blast resistance. Such modifications could be employed in new buildings or even as a retrofit for existing buildings.

The aesthetic quality of buildings can be maintained while providing improved performance and security from terrorist attacks. A few preliminary designs patterns for the use of small panes of glass have been presented.

It is important to note that the behaviour of material when exposed to dynamic loads is different from its behaviour when exposed to static loads (TM5-1300). Also, as was said by A. Williams et. al 1997, "The complicated response of structures to dynamic loading means that full scale testing often offers the only sure way of evaluating performance. Such testing is, however, prohibitively expensive".

An experimental program has been conducted using staticaly applied loads. Several glass samples of different thickness and size have been examined to prove the hypothesis that smaller panes of glass have a better performance under applied loads than larger panes. Moreover, the effect of double glazing was examined and it was shown that as expected the double glazing panels are much stronger than the single glazed ones. In addition, the effect of pane size and its support conditions were examined and it was revealed that smaller panes, fully supported, demonstrated greater strength.

The experimental programme testing results are summarised as follows:

- The experimental work was carried out at a model scale and in this way the panels were simplified and the pane support conditions as well. It was hoped that the results obtained would give useful comparisons rather than absolute strength values. The 4 mm glass panels tested gave inconsistent results and there was much doubt about whether or not the load remained uniformly distributed as the glass deflected. The 6 mm glass showed better consistency. Although dividing the pane into two resulted in strengthening the glass, further sub-divisions did not show greater improvement. The reasons for this are not immediately clear but may include the varying aspect ratios of the glass panes for each arrangement, as well as the different support conditions along the pane edges ie at a fixed support or at an intermediate mullion. Added to these reasons is the uncertainty about the uniformity of loading.

- It was observed that the fixation of the glazing to the mullions, as in PVC frames, enhances its response to a uniformly distributed static load. An enhancement of about 67.0% was recorded in one test but when it was repeated under the same experimental conditions only about 30% higher resistance was obtained.
- According to Mr. Tony Wilson of Granges Building Systems LTD, (Manufacturer representative and curtain walling design specialist) PVC mullions are not recommended to be used in cladding curtain walls of protective buildings or installations expected to be exposed to explosive blast because it melts when exposed to the high temperatures accompanied an explosive blast loading.
- Generated stiffness of glazing (break load / displacement) of 6.0 mm glass thickness is substantially higher than the corresponding 4.0 mm thick glass. As might be expected, increasing the glass thickness directly increases its breaking load accompanied with an obvious reduction in deflection.
- The glass tested in the experimental programme is normal annealed glass which is the weakest type of glass. If we proved our hypothesis using such glass, then using stronger glass types i.e. toughened or laminated glass will increase the overall resistance.

Finite element method (FE) was used to model the structural behaviour of the thin plate "glazing" when subjected to a uniform loading. The finite element analysis used the software package COSMOS/M . The static equally distributed load condition was selected. This method is appropriate for the case of quasi-static loading (i.e. the time duration of blast loading is large relative to the element's natural period of vibration) and, if the hypothesis is proved using static applied load, when exposed to dynamic load conditions, the glass should have a better performance.

For an impulsive loading condition or dynamic loading condition, an alternative numerical analysis has to take into account the dynamic characteristics of the loading and the corresponding structural response.

It was substantiated from the numerical analysis that the effect of supporting mullions is perceived to be a powerful reinforcement tool for large unsupported areas of glazing. This results in significant enhancement to the load bearing capacity of the glazing, and consequently to its resistance against blast loading.

The following results were envisaged from the parametric study conducted by the finite element analysis:

- Deflections are reduced by using mullions. In all cases the maximum deflection was at or near to the centre of the module.
- The maximum direct stress or the maximum flexural moment originates along the vertical or the long edge of the glazing.
- As the mullion stiffness increases, the position of maximum stress moves closer to the corner.
- Increasing the glass thickness in the vicinity of the long frame edge reduces the deflection and the maximum stress.

In practical terms, the experimental program and the numerical analysis using the FE has undoubtedly substantiated the conception of curtain walling systems that can be created using panels divided up into smaller glazed areas.

9.2 Conclusions

As a result of the discussions and the analysis that had been implemented in the research, the following conclusions can be drawn:

Terrorists always aim to attack important public buildings, the infrastructure, and commercial and financial buildings, thus arousing the media to publicise their motives and demands, which the terrorists could not achieve through the legitimate channels of expression.

From the analysis of the case histories, the following can be suggested:

- The maximum explosive charge a vehicle could carry is approximately 250 kg
- The building cladding designed to withstand the normal wind load conditions have weak resistance to explosive blast loading.
- Most of the injuries are caused to people walking on the street, sometimes half a mile away from the explosion which causes the glass to scatter in different sized splinters.
- Billions of pounds have been spent on buildings to be restored after their exposure to terrorist attack. Glass cladding, increases the amount of losses as its splinters cause damage to the air-conditioning and computer systems.
- The building distance from an explosion, and its geometry in addition to the cladding material and type all have an important influence on the structural response against blast loading. This reflects how complicated it is to predict the structural response and its cladding under the blast load conditions.

From the study of the blast wave mechanism and its effect on the cladding, the following were discussed:

- The structural response of the blast load is highly dependant on the structure's period of vibration (T) and the duration of the blast wave. This defines the loading type, whether it is impulsive, quasi-static or dynamic loading.
- P-I curves provide the structural response to loading once a maximum displacement is defined and define the damage tolerance across the range of effects..
- For reinforced concrete elements it is vital to provide symmetrical reinforcement to cope with stress reversals..
- Brittle materials lead to more destructive effect and more casualties than ductile ones.

In respect to the architectural view, the following are concluded from the discussions of the current research:

- The claddings design highly influences the building aesthetics as it influences our perception of the structure by its own design parameters such as its colour, texture and shape.
- As was said by Mays and Smith, 1995 the bigger the glass size within the cladding area. the more destruction one could expect when exposed to blast loading. It has been demonstrated that using smaller glazing panels provide enhanced blast resistance. Such modifications could be employed in new buildings or even as a retrofit for existing buildings.
- The aesthetic quality of buildings can be maintained while providing improved performance and security under terrorist attack.

From the experimental tests performed, the following main points were concluded :

- Reducing the pane size increases its resistance to the uniformly distributed statically applied load.
- There is some doubt about the way in which the load is uniformly distributed on the glass. The relation between the pressure in the air bag and the thickness of the layer of sand, as well as the speed of the load application needs review. Another loading method, perhaps using a water-filled bag, could be more reliable.
- The glass supporting conditions influences its response to applied loads. This is another area of uncertainty, where the relative stiffnesses of the intermediate members and the manufacturing process (welding) affect the results.
- A double glazing system substantially increases the resistance of glass when compared to a corresponding single glazing system. This is obviously due to the increased stiffness of a double glazed unit.

It was substantiated from the numerical analysis conducted in the research that supporting mullions are a powerful reinforcement for large unsupported areas of glazing. Consequently, enhancement to the load bearing capacity of the glazing and high resistance against blast loading is significant.

9.3 Recommendations for Further Studies

It is recommended that any further research in this field should include the analysis of the following:

1. Improved experimental techniques at model scale e.g. the use of a water bag to uniformly distribute the applied load above the glass.
2. Model scaling, especially the mullions, needs further consideration.
3. More samples need to be tested in order to have a wide range of results that gives a good average.
4. Smaller aspect ratios of 1:5 to 1:8 should be modelled.
5. The extension of experimental testing at full scale needs to be carried out for small sized panes because it offers the only sure way of evaluating the performance of glass when it is reduced in size.
6. Secure failure cladding design for the different cladding materials needs to be taken further.
7. Other cladding materials like aluminium and steel sheet cladding could be studied, with concentration on how to improve their behaviour when exposed to explosive blast (Research into blast resistant GRP panels is well advanced under the auspices of the UK Department of Trade and Industry, CIRIA and PERA).

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1. **El-kadi A.**, **Iremonger M.**, and **Kennedy L.** *The influence of Architectural Designs and Features on the Response of Glazing to Explosive Blast*, Proceeding of the 8 th International Symposium on Interaction of the Effects of Munitions with Structures, Mc Clean, Virginia, pp 601. April 1997.

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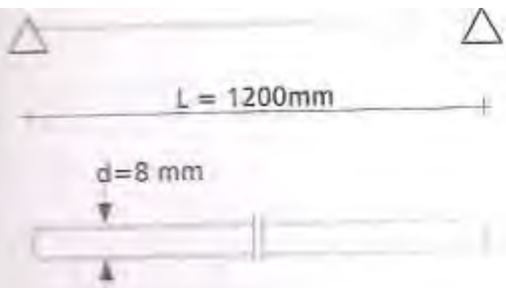
Appendix A

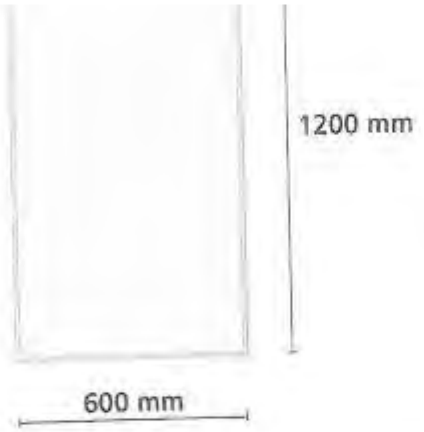
Numerical Analysis to Obtain the Pressure and Impulse of Different Cladding Panels

The numirecal analysis enclosed in this appendix intended to obtain the values of Pressure and Impulse each cladding material of (Glass, Glass Rienforced Polyester GRP, Steel Sheets, Rienforced concrete and Aluminuim Sheets) can Sustain. The results of this numerical analysis are represented by Figure 6.14 Chapter 6

Contents

1. Numerical analysis of Glass pressure and impulse values.
2. Numerical analysis of Glass Reinforced Polyster GRP Pressure and Impulse values.
3. Numerical analysis of Steel Sheets Pressure and Impulse values.
4. Numerical analysis of Precast Rienforced concrete Pressure and Impulse values.
5. Numerical analysis of Aluminium sheets Pressure and Impulse values.





a. *Impulse*

$$r_u = \frac{8M_p}{L^2}$$ *(From table 13.3 page 105, Mays and Smith, 1995).*

Where r_u is the ultimate resistance

M_p is the ultimate positive moment capacity

L is the span length.

M_p is given by:

$$M_p = F_{dy} \times Z$$ *(TM5-1300, Vol. 5, 1986).*

Where F_{dy} is the dynamic failure stress of glass

Z is the elastic section modules

F_{dy} is given by

$$F_{dy} = DIF \times \text{ static failure strength of glass}$$

Where $DIF = 1.0$ *(TM5-1300, Vol. 5, 1986).*

Static failure strength of glass = 50Mpa *(Ashby & Jones, 1986).*

$$= (50 \times 10^3) KN/m^2$$

$$F_{dy} = 1.0 \times (50 \times 10^3) = (50 \times 10^3) KN/m^2 = (50 \times 10^6) N/m^2$$

Z the elastic section modules is given by:

$$Z = \frac{bd^2}{6}$$

Where b is the panel of glass shortest span

d is the panel depth

$$Z = \frac{0.6 \times (8 \times 10^{-3})^2}{6}$$

$$= (6.4 \times 10^{-6}) m^3$$

Hence after getting F_{dy} and Z one can get M_y as follow:

$$M_y = (50 \times 10^6) \times (6.4 \times 10^{-6})$$
$$= 320\ Nm\ for\ 0.6\ m\ panel$$

$$For\ 1\ m\ width \quad M_u = \frac{320}{0.6} = 533\ N-m\ per\ unit\ width$$

Also r_u equals:

$$r_u = \frac{8 \times 533}{(1.2)^2} = 2963 N / m^2$$

When equating the kinetic energy with the strain energy for impulsive loading we get:

$$\frac{r_u x_e}{2} = \frac{i^2}{2K_{LM}(d\rho)}$$

Where r_u is the ultimate resistance
x_e is the equivalent elastic deflection
i is the unit positive impulse
d is the panel (cross-section) depth
K_{LM} is the load mass factor
ρ is the density.

x_e the equivalent elastic deflection is given by:

$$x_e = \frac{r_u}{K_e}$$

Where K_e is the equivalent elastic unit stiffness

$$K_e = \frac{384EI}{5L^4}$$

Where E is the Young's modules of glass
I is the cross section moment of inertia
L is the long span.

E of glass =74Gpa *(Table 15.7, pg. 150 Ashby & Jones, 1986).*

Where I is the moment of inertia of the cross section, which is given by:

$$I = \frac{bd^3}{12}$$

$$= \frac{0.6 \times (8 \times 10^{-3})^3}{12} = \frac{0.1 \times (512 \times 10^{-9})}{2} = 256 \times 10^{-10} = 2.56 \times 10^{-8} m^4$$

$$K_e = \frac{384 \times (74 \times 10^9) \times (2.56 \times 10^{-8})}{5 \times (1.2)^4} = 116938\ N / m^2 / m\ width$$

$$x_e = \frac{r_u}{K_e}$$

$$= \frac{2963}{116938} = 25.3\ mm$$

$$\frac{r_u x_e}{2} = \frac{i^2}{2K_{LM}(d\rho)}$$

$$r_u = 2963 N / m^2\ / m\ width$$

$$x_e = 0.0253 m$$

$$d = 8 \times 10^{-3} m$$

$K_{LM} = 0.78$ *(Table B.1, page 103, Mays and Smith, 1995).*

$\rho = 2.48\ Mgm^{-3}$ *(Table 15.7, page 150, Ashby & Jones. 1986).*

$$= 2.48 \times 10^3 kg/m^3$$

$$\frac{2963 \times 0.0253}{2} = \frac{i^2}{2 \times (8 \times 10^{-3}) \times (2.48 \times 10^3) \times 0.78}$$

$$i = 34.059\ N - s / m^2$$

b. Pressure

When equating the work done by the strain energy of the resistance one can get:

$\frac{r_u^2}{2K_e} = X_m\ p$ *(TM5-1300, Vol. 2, 1986).*

$$\frac{r_u X_e}{2} = X_m P_e$$

$$X_m = X_e\ for\ brittle\ material$$

$$P_e = \frac{r_u}{2} = \frac{2963}{2} = 1482 N / m^2$$

$\frac{P_e}{P} = load\ factor = 0.64$ *(TM5-1300, Vol. 2, 1986).*

$$P = \frac{1482}{0.64} = 2316 N / m^2$$

2 *P-1 of Plastic Sheets, Glass Reinforced Polyester GRP*

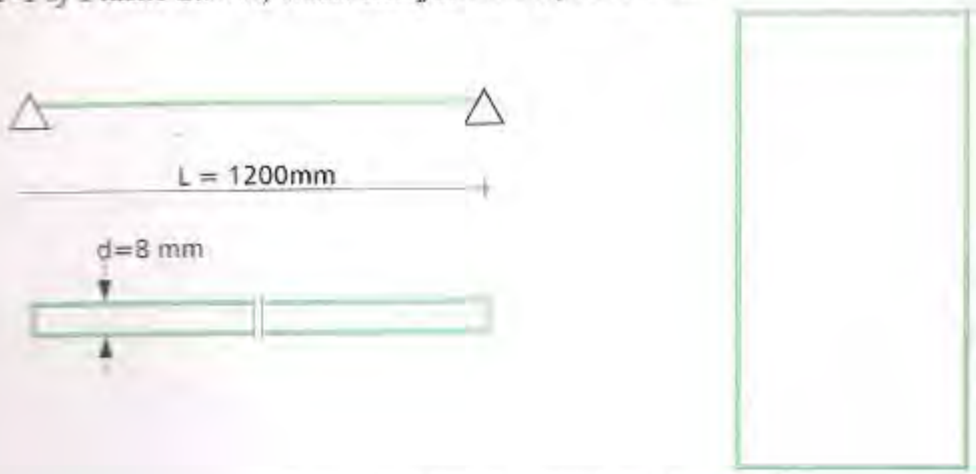


Figure A.2 Panel dimensions (assumed).

a. Impulse

$$r_u = \frac{8M_p}{L^2}$$ *(Table B.3, Mays and Smi*

Where r_u is the ultimate resistance

M_u is the ultimate positive moment capacity

L is the span length

$$M_p = F_{dy} \times Z$$ *(TM5-1300, Vol.*

Where F_{dy} is the dynamic yield stress of GRP

Z is the elastic section modules

F_{dy} is given by:

$F_{dy} = DIF \times$ static strength of GRP *(TM5-1300, Vol. 5, 1986)*

$DIF = 1.0$ (assumed)

Static yield strength of GRP =1240Mpa = (1240×10^6)N/m^2 *(Ashby & Jor*

$$F_{dy} = (1240 \times 10^6) \times 1.0 = (1240 \times 10^6) N/m^2$$

Z the elastic section modules is given by:

$$Z = \frac{bd^2}{6}$$

Where b is the panel of GRP shortest span

d is the panel depth

$$Z = \frac{0.6 \times (8 \times 10^{-3})^2}{6} = (6.4 \times 10^{-6})m^3$$

Hence after getting F_{dy} and Z one get M_p as follow:

$$M_p = (6.4 \times 10^{-6}) \times (1240 \times 10^6) = 7936 N.m$$

$$M_p / m\,width = \frac{7936}{0.60} = 13226.667 N.m$$

Also r_u equals:

$$r_u = \frac{8 \times 13226.667}{(1.2)^2} = 73481.48 N/m^2$$

$$\frac{r_u x_e}{2} = \frac{i_s^2}{2K_{LM}(d\rho)}$$

Where r_u is the ultimate resistance
i_s is the unit positive impulse
d is the panel (cross-section) depth
K_{LM} is the load mass factor
ρ is the GRP density
x_e is the equivalent elastic deflection

$$x_e = \frac{r_u}{K_e}$$

Where K_E is the equivalent elastic unit stiffness

$$K_e = \frac{384EI}{5L^4}$$

Where E is the young's modules of elasticity
I is the section moment of inertia
L is the long span

E of GRP =48 Gpa = (48×10^9)N/m^2

$$I = \frac{bd^3}{12} = \frac{0.6 \times (8 \times 10^{-3})^3}{12} = (2.56 \times 10^{-8})m^4$$

$$K_e = \frac{384 \times (48 \times 10^9) \times (2.56 \times 10^{-8})}{5(1.2)^4 \times 0.6} = \frac{471859.2}{6.221}$$

$$K_e = 75851.851 N/m^2/mwidth$$

$$x_e = \frac{r_u}{K_E} = \frac{73481.481}{75851.851} = 0.969m$$

$$x_e = 969mm$$

Impossibly high value 0.96m but indicative of high strength of GRP which means that failure at fixings would happen before failure of GRP itself. GRP need to be thicker to reduce deflections so that less likely to fail at fixings.

Hence, althought $x_m = \mu x_e = 10x_e$ *x_m as stated above must be limited by 100mm*

$$r_u = x_e K_e$$
$$= 0.1 \times 75851$$
$$= 7585\ N/m^2$$

$K_{LM} = 0.66$ *(Table B.1, Page 103, Mays and Smith, 1995).*

Hence $i^2 = r_u \times x_m \times K_{LM} \times \rho \times d$

$\rho = 2.0 Mgm^{-3}$ *(Ashby & Jones, 1986).*

$$i^2 = 7585 \times 0.1 \times 0.66 \times (2.0 \times 10^3) \times (8 \times 10^{-3})$$
$$= 8009.76$$
$$i = 89.497 N - s/m^2$$

ii. Pressure

When equating the work done by the strain energy of the resistance:

$$\frac{r_u^2}{2K_E} = x_m p$$ *(TM5-1300, Vol. 2, 1986).*

Where p is the peak pressure the panel can sustain

$$\frac{r_u x_e}{2} = x_m p_r$$

$$p_r = \frac{r_u}{2} = \frac{7585}{2} = 3792.5 N/m^2$$

$$\frac{p_r}{p} = load\ factor = 0.64$$

$$p = \frac{3792.5}{0.640} = 5925.4 N/m^2$$

E. P-I of steel sheet

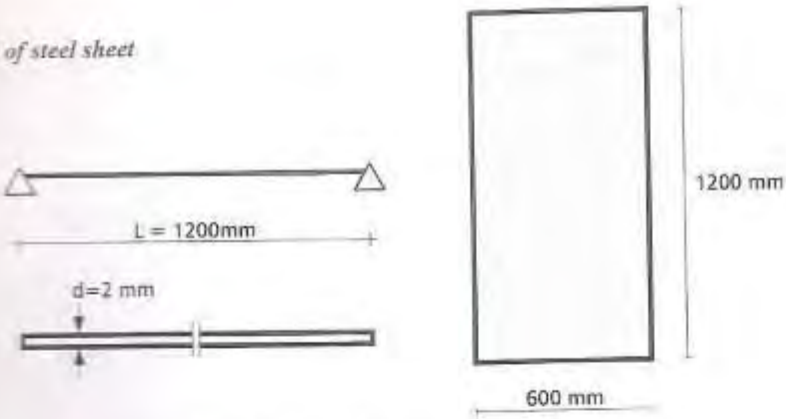


Figure A.3 Panel dimensions (assumed).

a Impulsive

$$r_u = \frac{8M_p}{L^2}$$

Where r_u is the ultimate resistance
M_p is the ultimate positive moment capacity
L is the span length

Where M_p is given by:

$$M_p = \frac{f_{ds}(s+z)}{2} \qquad \textit{(TM5 - 1300, vol. 5, 1986).}$$

Where f_{ds} is the dynamic design stress for (bending, tension and compression)

s is the elastic section modules $= \dfrac{bd^2}{6}$

z is the plastic section modules $= \dfrac{bd^2}{4}$

f_{ds} = DIF $\times \sigma_y \times$ Yield Stress Increase Factor

DIF is the Dynamic Increase Factor
σ_y is the Static Yield Stress

$$\sigma_y = 33000 psi$$
$$= 33000 \times (6.895 \times 10^3)$$
$$= 227.5 \times 10^6 \ N/m^2$$
$$f_{ds} = 1.1 \times 1.21 \times 227.5 \times 10^6$$
$$= 302.8 \times 10^6 \ N/m^2$$

$$s = \frac{bd^2}{6} = \frac{0.6 \times (2 \times 10^{-3})^2}{6} = 0.4 \times 10^{-6} m^3 / m$$

$$z = \frac{bd^2}{4} = \frac{0.6 \times (2 \times 10^{-3})^2}{4} = 0.6 \times 10^{-6} m^3 / m$$

$$M_p = \frac{(302.8 \times 10^6) \times (0.4 \times 10^{-6} + 0.6 \times 10^{-6})}{2}$$

$$= 151.4 N.m^2$$

$$M_p = 151.4 / 0.6 = 252.3 N - m / m$$

$$r_u = \frac{8 \times 252.3}{(1.2)^2}$$

$$= 1401.851 N / m^2$$

$$K_s = \frac{384 EI}{5L^4}$$

Where $E = 210GPa = 210 \times 10^9 N/m^2$

$$I = \frac{bd^3}{12} = \frac{0.60 \times (0.002)^3}{12} = 4 \times 10^{-10} m^4$$

$$K_s = \frac{384 \times (210 \times 10^9) \times (4 \times 10^{-10})}{5(1.20)^4}$$

$$= 3111.111 N / m^2$$

$$= 3111.111 / 0.6 = 5185.185 N / m^2 \ / m \text{ width}$$

$$x_e = \frac{r_u}{K_s}$$

$$= \frac{1401.851}{5185.185}$$

$$= 42 \text{ mm}$$

Hence $x_m = \mu x_e$ and $\mu = 10$ assume that $x_m \leq 100$ mm

$$\frac{i^2}{2K_{DLF} m} = r_u (x_m - \frac{x_e}{2})$$

$$K_{DLF} = 0.66$$

$$m = \rho \times d$$

$$\rho = 7.9 Mgm^{-3} = 7.9 \times 10^3 kgm^{-3}$$

$$m = (7.9 \times 10^3) \times (2 \times 10^{-3}) = 15.8 kg/m^2$$

$$\frac{i^2}{2 \times 0.66 \times 15.12} = \frac{1401.851 \times 0.042}{2} + 1401.851(0.1 - 0.042)$$

$$\frac{i^2}{19.958} = 29.439 + 81.307$$

$$i^2 = 19.958 \times 101.265$$

$$i = 44.956 N\text{-}S/m^2$$

6. *Pressure*

When equating the work done by the strain energy of the resistance

$$r_u (x_m - x_e) + \frac{r_u^2}{2K_e} = x_m P_e$$

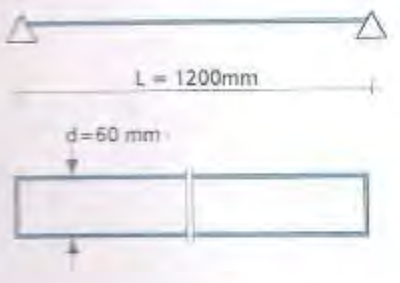
$$1401.85(0.1\text{-}0.042) + \frac{1401.85^2}{2 \times 5185.185} = 0.1Pe$$

$$P_e = 1401.85 \times 0.058 + \frac{1965183.423}{10370.37}$$

$$119.1578 + 189.5 = P_e$$

$P_e = 308.657 N/m^2$
$P = P_e/0.57 = 308.657/0.57$
$P = 541.504 N/m^2$

4. P-I panel of precast reinforced concrete



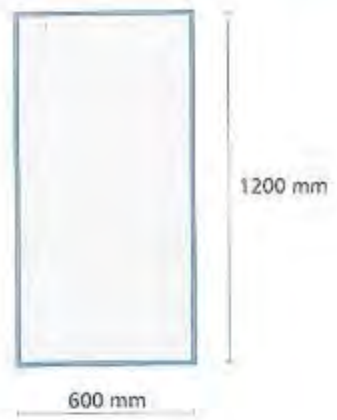


Figure A.4 Panel dimensions (assumed).

a. Impulse

$$r_u = \frac{8M_p}{L^2}$$

where r_u is the ultimate resistance
M_p is the ultimate positive moment capacity
L is the span length

M_p is given by

$$M_p = \frac{A_s f_{ds}}{b} d$$

where A_s is the area of tension reinforcement within a width b
f_{ds} is the dynamic design stress for reinforcement
b is the width of the panel
d is the distance from extreme compression fibre to centroid of tension reinforcement

$f_{ds} = 560 \times 10^6 \ N/m^2$ *(Table 5.1 and 5.3 Mays and Smith, 1995).*

$A_s = 0.35 \times 1000 \times 60/100 = 210 \ mm^2/m$

$$M_p = \frac{(210 \times 10^{-6}) \times (560 \times 10^6)}{0.6} \times 0.06$$

$$= 11760 N/m$$

$$r_u = \frac{8 \times 11760}{(1.2)^2} = 65333.333 \ N/m^2$$

$$K_1 = \frac{384EI}{5L^4}$$

Take

$$E_s = 200kN/mm^2 = 200 \times 10^9 N/m^2$$
$$E_c = 28kN/mm^2 = 28 \times 10^9 N/m^2$$

$$n = \frac{E_s}{E_c} = \frac{200}{28} = 7.14$$

From Figure 5-8 *Mays and Smith, 1995*

$$I = 0.0245bd_c^3$$
$$= 0.0245 \times 0.6 \times (6 \times 10^{-2})^3$$
$$= 3.175 \times 10^{-6} m^4$$
$$K_E = \frac{384 \times (28 \times 10^9) \times (3.175 \times 10^{-6})}{5(1.2)^4 \text{ x } 0.6}$$
$$= 54.876 \times 10^5 N/m^2/m$$

$$\frac{i^2}{2K_{LM}m} = r_u\left(x_m - \frac{x_E}{2}\right)$$

$$m = \rho_c d_c$$
$$\rho_c = 2400 Kg/m^3$$
$K_{LM} = 0.66$ *Table B.1, Mays and Smith, 1995*
$$m = 2400 \times (6 \times 10^{-2})$$
$$= 144 Kg/m^2$$
$$x_E = \frac{r_u}{K_E} = \frac{65333.333}{5487654.321} = 12mm$$

$2^\circ rotation(type1)$

$$x_m = 600 \tan 2 = 21mm$$
$$\frac{i^2}{2K_{LM}m} = r_u\left(X_m - \frac{X_E}{2}\right)$$
$$\frac{i^2}{2 \times 0.66 \times 144} = 65333.333\left(0.021 - \frac{0.012}{2}\right)$$
$$i^2 = 186278.399$$
$$i = 431.6 Nse/m^2$$

b. Pressure

When equating the work done by the strain energy of the resistance

$$P_s x_m = \frac{r_s x_s}{2}$$

$$Factor = \frac{0.64 + 0.5}{2} = 0.57$$

$$P_s \times 0.021 = \frac{65333.333 \times 0.012}{2}$$

$$P_s = 18666,667 N/m^2$$

$$P = \frac{18666.667}{0.57}$$

$$P = 32748.538 N/m^2$$

P-1 of Aluminium sheet

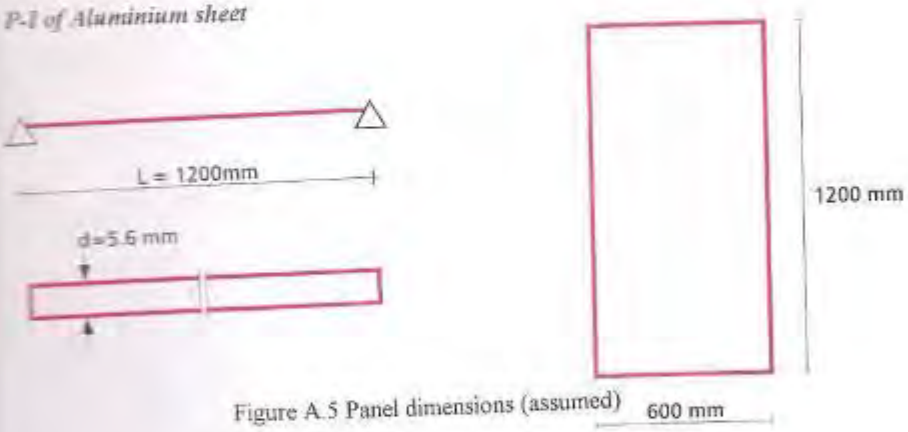


Figure A.5 Panel dimensions (assumed)

$$r_u = \frac{8M_p}{L^2}$$

where r_u is the ultimate resistance

M_p is the ultimate positive moment capacity

L is the span length

where M_p is given by

$$M_p = f_{ds}\frac{(s+z)}{2}$$ *(TM5-1300, Vol.5, 1986).*

where f_{ds} is the dynamic design stress for (bending, tension, and compression)

s is the elastic section modules = $\frac{bd^2}{6}$

z is the plastic section modules = $\frac{bd^2}{4}$

$$s = \frac{bd^2}{6} = \frac{0.6(5.6\times10^{-3})^2}{6} = 3.136\times10^{-6} \quad m^3/m$$

$$z = \frac{bd^2}{4} = \frac{0.6(5.6\times10^{-3})^2}{4} = 4.704\times10^{-6} \quad m^3/m$$

(TM5-1300, Vol.5, 1986).

$$f_{ds} = DIF \times a \times \sigma_y$$

Where DIF is the dynamic increase factor and a is the average yield strength increase factor. Data is unavailable for aluminium; therefore assume both equal 1.0

$$\sigma_y = 50\times10^6\ N/m^2$$ (Ashby&Jones, 1986)

$$f_{ds} = 1\times1\times50\times10^6$$

$$M_r = (50 \times 10^6) \times \left(\frac{3.136 \times 10^{-9} + 4.704 \times 10^{-9}}{2} \right)$$

$= 196/0.6 = 326.667$ N-m/m width

$$r_u = \frac{8M_r}{L^2} = \frac{8 \times 326.667}{(1.2)^2} = 1814.815 N/m^2$$

$$K_e = \frac{384EI}{5L^4}$$

$E = 71 \times 10^9 \quad N/m^2$ (Ashby&Jones,1986)

$$I = \frac{bd^3}{12} = \frac{0.6(5.6 \times 10^{-3})^3}{12}$$

$$= 8.781 \times 10^{-9} \quad m^4$$

$$K_e = \frac{384 \times 71 \times 10^9 \times 8.781 \times 10^{-9}}{5 \times (1.2)^4 \times 0.6}$$

$$= 38484.629 \qquad N/m^2/mwidth$$

$$x_e = \frac{r_u}{K_e}$$

$$= \frac{1814.815}{38484.629}$$

$$= 47.2 \quad mm$$

$x_m = \mu x_e$

$\mu = 10$ but $x_m \leq 100\ mm$

$$\frac{i^2}{2K_{LM}\, m} = \frac{r_u x_e}{2} + r_u(x_m - x_e)$$

$\rho = 2.7 Mgm^{-3}$ (*Ashby& Jones*,1986)

$m = \rho d = (2.7 \times 10^3) \times (5.6 \times 10^{-3}) = 15.12 kg/m^2$

$K_{LM} = 0.66$ *(Mays & Smith, 1995)*

$$\frac{i^2}{2 \times 0.66 \times 15.12} = \frac{1814.815 \times .047}{2} + 1814.815(0.1 - 0.0472)$$

$i^2 = 19.958(42.648 + 95.822) = 2763.584 Ns/m^2$

$i = 52.57 N\text{-}s/m^2$

8. Pressure

When equating the work done by the strain energy of the resistance

$$\frac{P_e}{P} = 0.57$$

When P is the peak pressure that the panel can sustain

$$P_e x_m = \frac{r_u x_e}{2}$$

$$P_e \times 0.1 = \frac{1814.815 \times 0.0472}{2}$$

$$P = \frac{428.296}{0.57}$$

$$P = 751.397 N / m^2$$

Appendix B

Basics and Application for the Use of Finite Element Analysis Package "COSMOS/M"

This introduction is made after reference to *R. Odi, 1997* the term finite element was first introduced by Clough in 1960, it means the deviation of a continuum body into finite regions in order to analyse specific problems. Finite element fields of application is very wide, coverage of all aspects of the methods is beyond the scope of this research which seek to use element package in relation to solid structures. Zienkiewicz, 1987 cover more extensively the FEM and its application in various fields together with solids and structures

1 Linear elasticity theory: Basic Principals

The main hypothesis of any problem seek to define a relation between the stresses and deformations of a solid body. The behaviour of any deformed object is governed by the relationship between its force and displacement condition. Three states must be met at all points within the object:

a. Equilibrium
b. Compatibility
c. Constitutive equations

The first state is considered to be investigated in terms of the state of mechanical equilibrium that must be exist within the body and its boundary. Both internal forces (i.e. forces exerted by the body particle on each other) and external forces (i.e. forces applied by a source outside the body). The second state consider that any deformation ought to be physical (legitimate). The third state discuses the problem with a group of fully defined set of constitutive equations which correspond the internal forces of the object to local deformation going to take place.

2 Displacement Finite Element method

"The main feature of the finite element method as application of the Raleigh-Ritz method is that the trial functions required are generated automatically when the continuum is replaced by a system of discrete elements. These same interpolation functions are given which characterise it. These same

interpolation functions are then used as basis functions of the Raleigh-Ritz expansion equation as follows

$$X = \hat{X}\,(a_1, a_2, \ldots, a_n)$$

which is an approximate expression for X. This approximate expression is a combination of known functions (e.g. polynomials) and unknown coefficients a_i. These coefficients are replaced by the nodes (points where elements connect with each other) displacements, δ,

The finite element procedure follows a well defined number of steps, no matter how complex the problem is. These are: element definition (nodes), element interpolation (which will be such as to fulfil the displacement conditions at known element points-e.g. nodes), calculation of element energy (strain and potential), assembly and energy minimisation (which give a set of simultaneous equations). When these sets of simultaneous linear equations are solved, the finite element solution is complete" *R. Odi, 1997.*

COSMOS/M

As stated in the manual book of the program by Structural Research and Analysis Corporation May 1993, COSMOS/M is a complete, modular, self-contained finite element system which contain modules to solve linear and non-linear static and dynamic structural problems. COSMOS/M system consists of pre- and postprocessor, various analysis modules, interfaces, translators and utilities. The program is completely modular allowing the user to acquire and load only the modules that are needed.

GEOSTAR is the basic pre-and postprocessor of the COSMOS/M finite element system. It is an interactive full three-dimensional CAD-like graphic geometric modular, mesh generator and FEA pre-and postprocessor. The user can create the model, geometry, mesh it, provide all analysis related information, perform the desired type of analysis. In order to linearly static analysis an object STAR uses the linear theory of structures, based on the assumption of small displacements, to calculate structural deformations. STAR calls the STRESS submodule to calculate stresses. Bonding of separate parts of the same model even if nodes and element types don't match. Solid-solid, solid-shell and shell-shell bonds can be specified. The following are data file of the programe used examin a glass curtain wall stiffness when reducing it size as mentioned in Chapter 8.