STUDY ON BEHAVIOUR OF CONCRETE FINS AGAINST BLAST PRESSURE

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Thesis submitted in partial fulfilment of the requirements for the degree Master of Engineering

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Declaration

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Abstract

Designing structures against blast loading is becoming more and more important as the number of terrorists attacks are increasing day by day. It is necessary to protect the structures against a credible blast load to ensure the safety of the occupants. In this context, blast resisting facades are incorporated in buildings to avoid the blast pressure waves entering into the building as the highest damage is done by the pressure waves when compared with the fragments moved by an explosion. Pressure waves could damage the axially loaded elements and it may lead to progressive collapse of the structure. This study investigated the behaviour of concrete fins and they were categorized depending on the failure mode which is based on the occupancy levels such as immediate occupancy, life safety and collapse prevention. Concrete fins were analysed using Sap2000 software by taking into account the material nonlinearity and loading nonlinearity. Weight of blasting materials, standoff distance, fin spacing, fin size and reinforcement ratios were varied to create different analysis cases. When the standoff distance was 50m, all the analysis cases were in immediate occupancy level, and it was found that standoff distance of 25m is as the manageable distance in blast. However, standoff distance of 10m resulted many analysis cases exceeded the collapse prevention limit. It was identified that standoff distance of 25m as the manageable limit with respect to the safety and the cost.

Specially dedicated to my beloved family and to whom were behind me always….

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LIST OF ABBREVIATIONS

Abbreviation	Description
f'c	Concrete cylinder strength
f_{cu}	Concrete cube strength
f_{dy}	Dynamic yield strength of steel
f_{du}	Dynamic tensile strength of steel
f_{dcu}	Dynamic cube strength of concrete
fu	Tensile strength of steel
fy	Yield strength of steel
P(t)	Pressure at time t
P(r)	Reflected pressure
Pr^-	Maximum pressure at negative phase
Pso	Peak incident pressure
R	Standoff distance
t_{of}	Duration of positive phase
t_{rf}^-	Duration of a blast
W	Weight of blast material
Z	Scaled distance

1. INTRODUCTION

1.1 BACKGROUND

Development of the technology has brought the mankind to a different era and it has developed more sophisticated things that make life easier. However, the negative fraction of this development has significant effect on our life. One such negative action by the human being is the terrorism. Media is more commonly used today to popularize the terrorist groups and to give the publicity to their attacks. In addition to that, media reveals the detailed information and strength of terrorist groups, which are most of the time overestimated or underestimated.

Terrorist attacks are increasing globally, and it appears that there is no ending. Therefore, it is necessary to protect structures from terrorist attacks. However, the protection is not an absolute measure to issue and there should be different levels of defensive systems, which also minimize the cost of system. Protection can never offer a guarantee of safety. High level of protection will also increase the cost of the construction and could be a waste of resources. However, it is very important to take measure to protect most vulnerable structures against blasts. It is learnt that the structures, which will be constructed in the future could be designed for at least a credible blast load. There are many examples in the past that proves the above.

A blast is an unusual event and it is very difficult to predict the threat and scale of such event. Generally, the magnitude of the blast is predicted from the vehicle that can reach to the susceptible structure. Hence, estimating the weight of the blast material or the blast pressure is a quite difficult task.

Attack to the Central Bank of Sri Lanka on 31st of January 1996 is believed to be the largest bomb blast occurred within the country. About 85 people died and more than 1500 people were injured. After observing the images of the damaged structure, it can be understood that designers had not predicted such event at the design stage.

Fig- 1.1 Central Bank Building after the bomb blast

The building was severely damaged although it did not collapse. Main causes of damage are due to the blast pressure and subsequent fire occurred inside the building. As per the Fig- 1.1, it can be seen that the entrance of the building has heavily damaged. This could be due to lack of defensive system that bears the blast pressure.

Present development of the world is to protect the structures against blast loading, where it is required. Different types of defensive systems have been introduced to avoid the collapse of the structures due to the failure of load carrying elements and to avoid the fragments entering into the building. One such defensive technique is enhancing the load resisting capacity of facade. Most commonly, facades are constructed from brick, concrete and glass. Different techniques that discuss in the latter part of this report are used to increase the load carrying capacity of facades constructed from above materials.

Failure of the Murrah Federal Building at Oklahoma City shown in Fig- 1.2 is one such example for lack of defensive facade system. Structure has not collapsed completely though front part has completely destroyed. When there is no defensive system to protect the axially loaded elements, it could lead to progressive collapse.

Fig- 1.2 Damages to Murrah Federal Building, Oklahoma City

There are different levels of defensive systems that do not allow the attacker to reach closer to the structure. If access is controlled, the blast pressure could be much less and cost of construction of the defensive façade system will reduce significantly.

Fig- 1.3 US Embassy Attack, Nairobi, Kenya

Attack on US embassy, Nairobi, Kenya has resulted in loss of 224 lives and more than 4000 people injured. Such damage could be due the lack of solid defensive system to protect the building. Despite the structure shown in Fig- 1.3 has not collapsed, facade has damaged completely. If there were different levels of protections to protect the structure, damage could be very less.

In this background, it is observed that more researches are required in this field to know appropriate preventive measures that will minimize loss of life and other damages. Many researches have been carried out studying the effect of blast on buildings. However, very few studies can be found relevant to concrete facade systems.

There are different types of facades constructed considering the defensive system and the aesthetic view of the structure. Although the structure has the capacity to bear the effects of blast, intention is not to construct a bunker type building. It should have an attractive view and the protection to the people who work inside the building. Due to these facts, innovative techniques of developing facades are studied. Different techniques are used to enhance the load resisting capacity of structural elements. Brick walls are strengthened by introducing cross wall, increasing the width of the wall and adding steel plates where it fails. Glass walls are modified by adding fins while windows are modified by laminating membrane. Laminated glasses are used commonly for blast resistance windows. They have very high blast resisting capacity and do not allow moving the broken glass fragments like normal glass. Fig- 1.4 indicates the properly designed glass panel and its deformation due to the blast pressure.

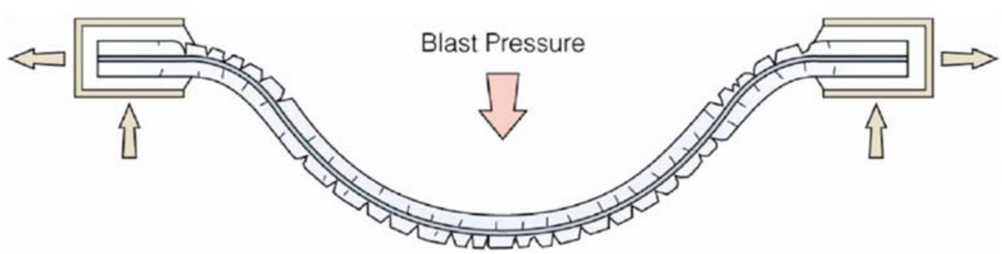

Fig 1.4 Deformed Glass Panel under Blast Loading

With the present trend of increasing terrorist attacks all over the world, many have thought of having solid defensive systems to minimize the risk on lives despite the fact that of increasing the cost of construction. Facades act as defensive element and it does not allow the blast waves entering into the building. As a result, load carrying elements such as concrete columns and walls will not damage, and risk of arising progressive collapse is minimal. Concrete facades can resist very high blast loadings. As the concrete is weak in tension, it is required to provide large sections or increase the area of reinforcement to accommodate a higher blast pressure. As an alternative load carrying capacity of concrete facades can be increased by providing concrete fins at a determined spacing according to the loading on structure. In this research, load carrying capacity of the concrete fins, according to their spacing and area of reinforcement, are studied and failure criteria will be defined by specifying whether each element is on the state of immediate occupancy, life safety and collapse prevention.

1.2 REASEARCH OBJECTIVES

The purpose of this study is to identify the failure criteria depending on loads and section properties of concrete fins. Failure loads will be specified considering the performance of the structure. Main objectives are as follows:

01. Identify failure loads of concrete fins depending on the properties of concrete fins
02. Identify the failure loads of concrete fins by varying the size, area of reinforcement and spacing of fins.
03. Identify the behaviour of structure for performance based design.

1.3 SCOPE OF WORK

The scope of this study was limited to finding the behaviour of concrete fins under the blast loading, and the behaviour of the facade is not considered for the analysis although it was considered at the beginning of the study. Concrete fins were modelled as line elements with SAP2000 V14 software, and failure criteria were defined considering the guidelines given by the FEMA 356. Simplified hinge model illustrated in Chapter 03 is defined as specified in FEMA 356 and it is used to model the plastic rotations of joints.

Blast loads were calculated from the charts given in UFC 03-340-02 published on 05 of December 2008. Influence of varying the fin size, spacing and area of reinforcements were studied in this research.

1.4 OUTLINE OF THE REPORT

Chapter 01 provides introduction about the study, brief overview about the bomb attacks on buildings and subsequent damages to the lives and structures. Detailed description about types of blasts and methods of calculating blast loading on structures, details about the behaviour of materials, façade analysis techniques and past findings are elaborated in Chapter 02. Chapter 03 describes about the research methodology. Calculation of loads, analysis techniques, calculation of material properties and details about material models are discussed in this chapter. While Chapter 04 states about the analysis results, Chapter 05 provides the discussion. Chapter 06 describes about the conclusion and recommendations for future works.

2.0 LITERATURE REVIEW

2.1 INTRODUCTION

This chapter contains some of the relevant literature gathered and studied during the study. Brief explanation has been given about relevant documents, and the method of streamlining of the research objectives has been discussed in this chapter.

This chapter outlines blast loading phenomenon, types of explosions, blast wave characteristics, parameters relevant to blast loads, methods of calculating blast loadings, material models, analysis techniques, failure criteria and types of facades.

2.2 EXTERNAL LAYOUT PLANNING

Taking preventive measures is the most commonly practiced approach to protect the buildings, as it is the cheapest method to have a secured structure against blast loads. External layout planning and access control as shown in the Fig- 2.1 are used to prevent the attacker reaching near to the building.

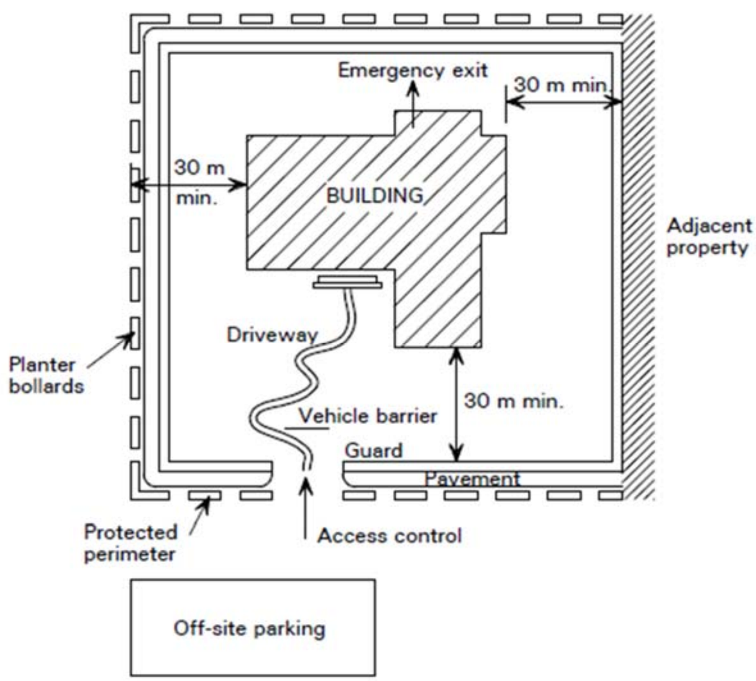

Fig- 2.1: External Layout Planning (Yandzio and Gough, 1999)

Different levels of defensive layers such as perimeter protection, landscaping, road blocks and many others are introduced for blocking the access to the building. Landscaping is done to control the vehicle access and visibility of the building. Perimeter protection is also used to block the visibility of the structure and to avoid unessential vehicles accessing the premises. These techniques are the most common methods implemented all over the world to increase the standoff distance and reduce the blast effect on buildings.

2.2 BLAST PHENOMENON

A blast is a sudden release of energy due to the reaction of explosive materials. Blast effects of an explosion are in the form of a shock wave composed of a high intensity of shock front which expands outward from the surface of the explosive into the adjoining air. As wave expands, it decays in strength, lengthens in duration and decrease in velocity. This phenomenon is caused by spherical divergence as well as by the fact that the chemical reaction is completed, except for some afterburning associated with the hot explosion products mixing with the surrounding atmosphere.

2.2.1 TYPES OF EXPLOSIONS

Mainly there are two types of explosions. Namely they are unconfined explosions and confined explosions. Unconfined explosions can be subdivided into three categories, and it is done according to the elevation that the blast occurs. Free air burst, air burst and surface burst are the three categories identified as unconfined explosions, while explosions occur at a higher elevation are called free air blast. Spherical blast waves can be observed in free air blasts. Air blast occurs above the ground level and blast waves are amplified with the reflection of the ground. When wave travels, Mach stems are formed due to the interaction of initial wave and the reflected wave. Surface burst occurs close to the ground surface or above the ground surface. In these types of blasts, initial wave is reflected and amplified by ground surface, and merging of reflected wave and incidental wave occurs at the point of blast. Surface burst is the most common type of attack that can be observed in terrorist attacks.

Blast occurs inside a building is called as confined burst. Depending on the degree of confinement, they are categorized into three types. They are fully vented, partially vented and fully confined.

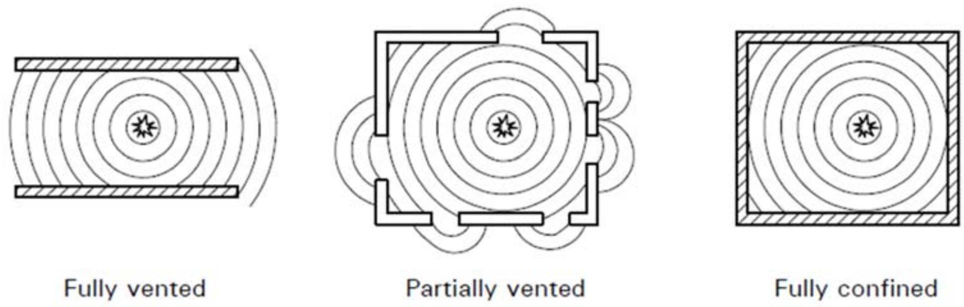

Figure 2.2 Categories of Confined Burst (Yandzio and Gough, 1999)

Fully vented blast occurs when one or more surfaces of a building is open, and the blast waves travel to the atmosphere and does not affect to the confinement significantly. When the area of openings is limited, burst waves are partially vented, and considerable increase in pressure inside the building can be observed compared with fully vented bursts. A fully confined blast occurs in building or barrier which are fully closed or nearly closed.

2.2.2 BLAST WAVE CHARACTERISTICS

Propagation of burst waves follows a spherical shape. It travels three dimensionally and reaches every corner as sound does. Blast produces very high air pressure. Positive pressure generated from the blast reduces with the time at a point, and then it becomes a negative pressure. Variation of blast pressure at a point away from the location where blast is occurred is illustrated in Figure 2.3.

A blast wave has two phases, namely they are positive phase and negative phase, and negative phase is less significant compared to the positive phase due to its magnitude. Sudden increase of incident pressure at a point away from the blast reduces with time and it reaches to zero pressure as illustrated in Figure 2.3. After it reaches zero, wave moves into the negative phase producing a negative pressure.

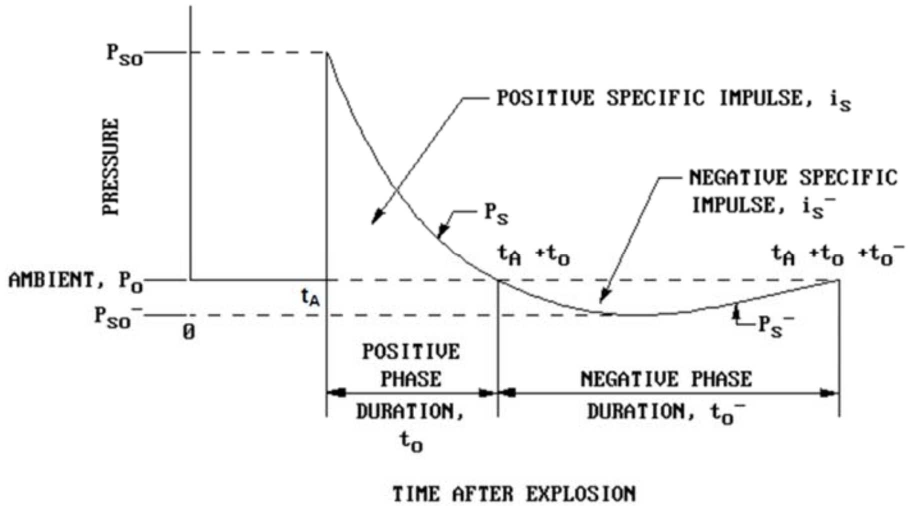

Figure 2.3: Variation of Incidental Pressure with Time (USA, Department of Defence, 2008)

Variation of the blast pressure can be represented mathematically by Friedlander equation.

$$P(t) = P_{so}(1-t/t_0)\ \exp(-bt/t_0)$$

Where $P(t)$ is pressure at time t

P_{so} is peak incident pressure

t_0 is duration of positive phase

b is waveform parameter

t is the time blast pressure is considered

Blast load can be calculated from this equation if the P_{so}, t_0 and b are known. Finding these parameters is the most difficult task in blast load calculations. However, if the guidelines given in the UFC 3-340-02, 2008 are followed, load evaluation can be done for different distances to blasting location from the structure and different weights of blasting materials.

When a blast wave hit on a solid surface or hard object, it is reflected creating a reflected wave. Reflected wave increases the wave pressure significantly and generate amplified wave.

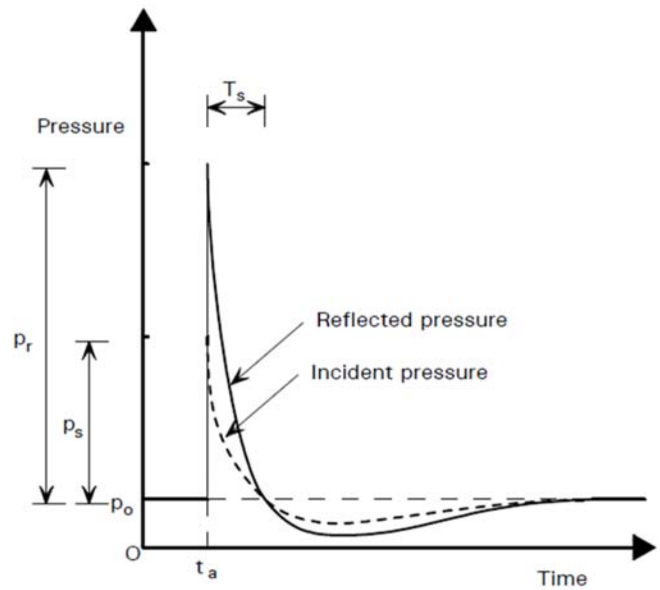

Figure 2.4 : Comparisons of Incidental Pressure and Reflected Pressure (Yandzio and Gough, 1999)

Figure 2.4 indicates the variation of incidental pressure and reflected pressure at the time t_a. Peak reflected pressure (Pr) relevant to the incidental pressure (Ps) shows a significant increase due to the merging of incidental pressure wave with reflected wave.

Reflected blast pressure varies with the angle of incident. When angle of incident is zero, the maximum reflected pressure will be felt. If the reflector is parallel to explosive waves, minimum reflected pressure or incidental pressure will be felt.

Figure 2.5 illustrates the variation of reflected pressure coefficient (reflected pressure/incidental pressure) with the angle of incidence (UFC 3 -340-02, 2008).

$$Pr = C_r P_{so}$$

Reflected pressure can be determined at point away from the blast by using the Figure 2.5 if the incidental pressure and the angle of incidence are known.

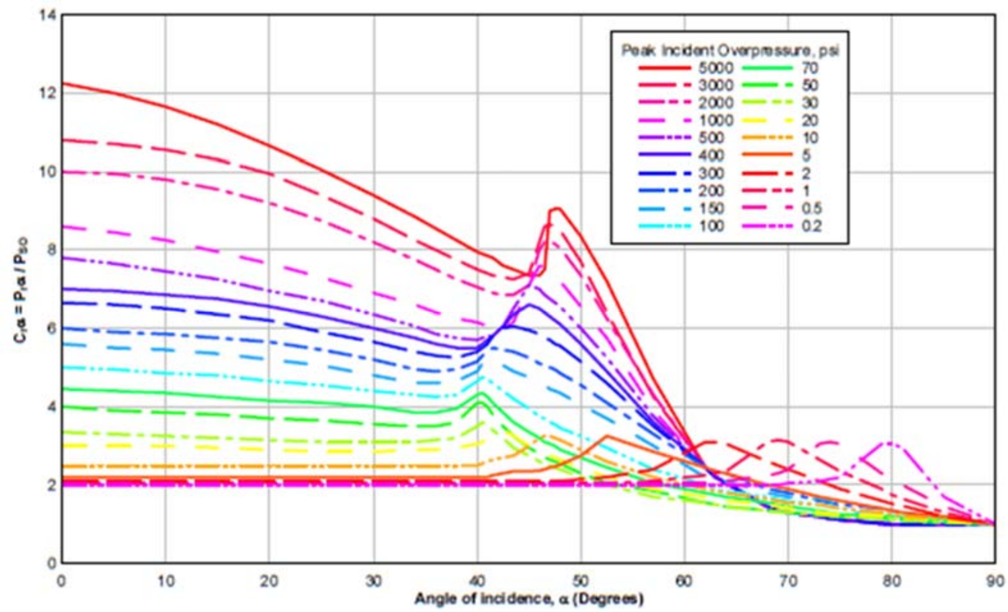

Figure 2.5: Variation of Reflected Pressure Coefficient with Angle of Incidence (UFC 3 – 340 – 02, 2008)

Blast pressure is felt all over the building. Although the incidence occurs in front of the building, it felt each side of the building by different magnitudes. Variation of the blast pressure in each face of the building can be evaluated according to the guidelines given in the UFC 3 – 340 -02, 2008.

Figure 2.6 indicates the application of blast pressure on the façade of a building. It can be seen that when a blast occurs near the building, significant variation in the pressure over the area of the façade. Blast pressure on façade reduces when the distance between the structure and the blasting location getting longer

In addition, the manner of acting the drag forces on a building due to the variation of the pressure can be observed in Figure 2.6. Drag forces can be calculated as from the guidelines given in the UFC 3 – 340 – 2, 2008. Further, it clearly shows the variation of the pressure over the face of the façade. However, when the Mach reflection occurs, effect of pressure on façade becomes uniform over the width and height of the façade. However, there should be considerable standoff distance for occurrence to this phenomenon. This phenomenal will be discussed in Chapter 2.3.3.

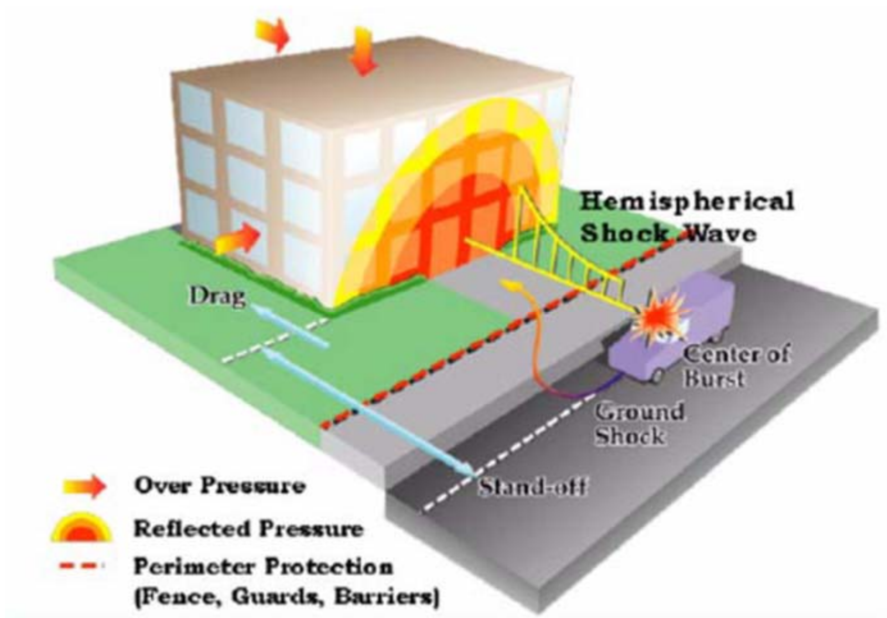

Figure 2.6: Wave Actions on a Building

2.3.3 MACH REFLECTION

Mach reflection is meeting of incident wave with the reflected wave. The wave front created is called the Mach stem. Heights of the Mach stem increases when it moves away from the blast creating a planer wave in vertical direction. Figure 2.7 (a) and Figure 2.7 (b) indicate the propagation of Mach stems when the incidental wave moves away.

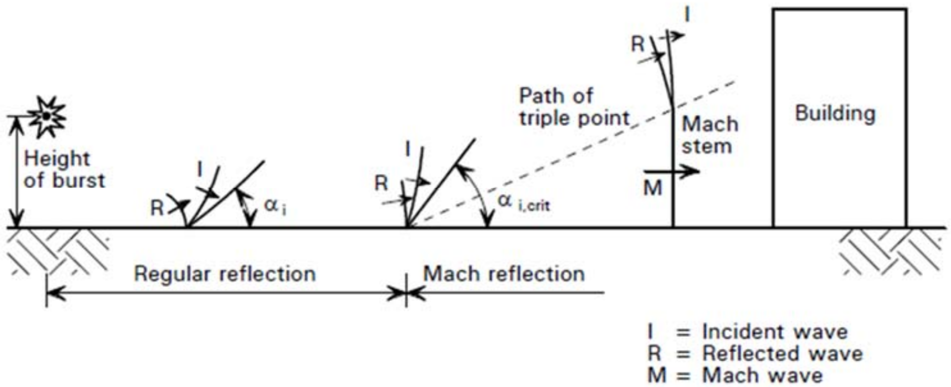

Figure 2.7 (a) Formation of Mach Stem (Yandzio and Gough, 1999)

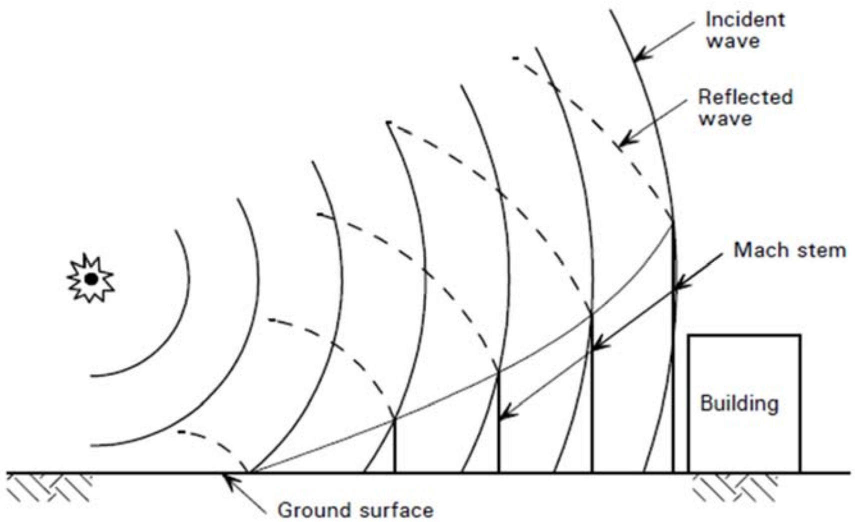

Figure 2.7 (b) Wave Pattern of Mach Stem (Yandzio and Gough, 1999)

Formation of Mach stems can increase reflected blast pressure significantly, and it may generate very high blast overpressure than the pressure values found assuming regular reflection. The point where the incident wave, reflected wave and Mach stem is met, defined as the Triple Point. Reflection coefficient (Cr) for Mach reflection can be applied to the blast load calculations to be taken into account the increase of loads.

In addition, formation of Mach stems makes uniform pressure over the façade of the building. The triple points form at an angle around 40^0 from the ground as per the book Protection of Buildings against Explosions by The Steel Construction Institute. Application of uniform blast pressure over the surface of the façade depends on the distance to the blasting location from the structure.

2.3.4 BLAST LOADING

Blast load calculations can be done with different methods such as use of experimental data, use of empirical formulas or charts developed based on the experiments. There are key parameters to be known for calculating the blast loading on a structure. They are weight of the blast materials and distance from the structures to location where the blast occurs. Combination of these two parameters is defined as scaled distance.

2.3.4.1 SCALED DISTANCE

Magnitude of the incidental pressure and blast wave characteristics depend on the term scaled distance which is calculated by taking into account the weight of the blast material (W) and Standoff distance (R), which is the distance from the structure to where the blast occurs. Relationship between the standoff distance and weight of blast material presented by Hopkinson (1915) and Cranz (1926) is used most commonly to evaluate the blast loadings.

$$Z = R / W^{1/3}$$

Where, Z is the scaled distance ($m/kg^{1/3}$ or $ft/lb^{1/3}$)

R is the standoff distance (m or ft)

W is the weight of blast material (kg or lb)

There are many forms of explosive materials used to detonate the bombs. Each type of explosive materials has its own characteristics. Due to the good repeatability of experimental results, most of the researches have used trinitrotoluene (TNT) as the explosive materials. Therefore weight of the blast materials is expressed as a TNT equivalent value.

2.3.4.2 METHODS OF PREDICTING BLAST LOADING

Most challenging part of the analysis of a structure for blast loading is the estimation of the blast loading accurately. Due to the high uncertainty in the weight of the blast material, accurate evaluation of the weight is a very difficult task. Basic method of predicting the blast load is the Friedlander equation. Incidental pressure need to be found for calculating the variation of the blast loadings. Numbers of studies have been carried out throughout the history to find out the incidental pressure for the scaled distance Z.

Brode (1955) has proposed two equations to find incidental pressure based on its magnitude.

$P_{so} = 6.3/Z^3 + 1$, bar ($P_{so} > 10$ bar) Eq : 01

$P_{so} = 0.975/Z + 1.455/Z^2 + 5.85/Z^3 - 0.019$, bar (0.1 bar < P_{so}< 10 bar) Eq : 02

Newmark and Hansen (1961) have proposed a different equation to evaluate the incidental pressure. In this equation, pressure is in bars, and explosion is at ground level.

$P_{so} = 6784\ W/R^3 + 93(W/R^3)^{0.5}$ Eq : 03

Another equation for finding the peak incidental pressure was suggested by Mills (1987). It is in the form of scaled distance. In this equation, he has considered equivalent charge weight of TNT in kilograms.

$P_{so} = 1772/Z^3 + 114/Z^2 + 108/Z$ Eq : 04

There are many equations of similar nature proposed by different researches for evaluating the blast loadings. Mainly, equations proposed by Henrych (1979), Kinney and Grahm (1985), Held (1983), Sadovskiy (2004) and Bajic (2007) are recognized as worthy findings in this area. However, according to the finding of Manmohan, Vasant Anil, and Steffen (2012), when Z<1 $m/kg^{1/3}$ there is wide variations in peak positive overpressure. Figure 2.8 indicates the variation of blast pressure with the scaled distance. According to their findings, it is very essential to take special care when the scaled distance is smaller.

Due to the variation of blast pressure parameters, it is very important to use a reliable source to evaluate the blast pressure parameters. Commonly used guideline in blast related studies is UFC 3 – 340 – 2, 2008, which is published by Department of Defence, United States of America.

In this study, blast pressure parameters were evaluated using UFC 3 – 340 – 2, which is based on empirical relationships. Guideline states the variation of the blast parameters with the scaled distance (Z) calculated in ft/lb$^{1/3}$.

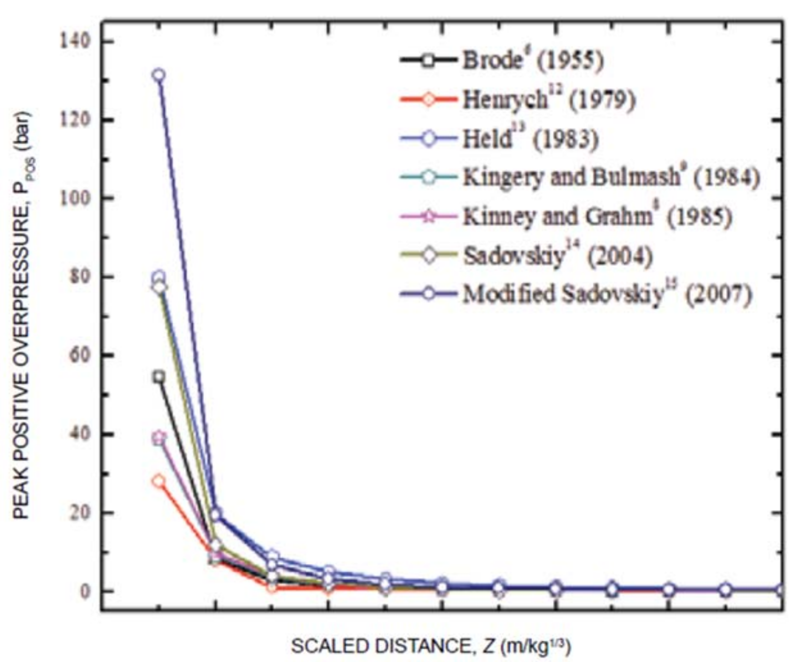

Figure 2.8: Variation of Blast Pressure with Scaled Distance (Manmohan et al., 2012)

When a blast occurs near the surface of the ground, its wave formation is hemispherical. UFC guideline provides charts to evaluate the pressure parameters in positive phase and negative phase separately. Figure 2.9 indicates the chart that can be used to calculate the positive phase parameters, and negative phase parameters can be found from figure 2.10.

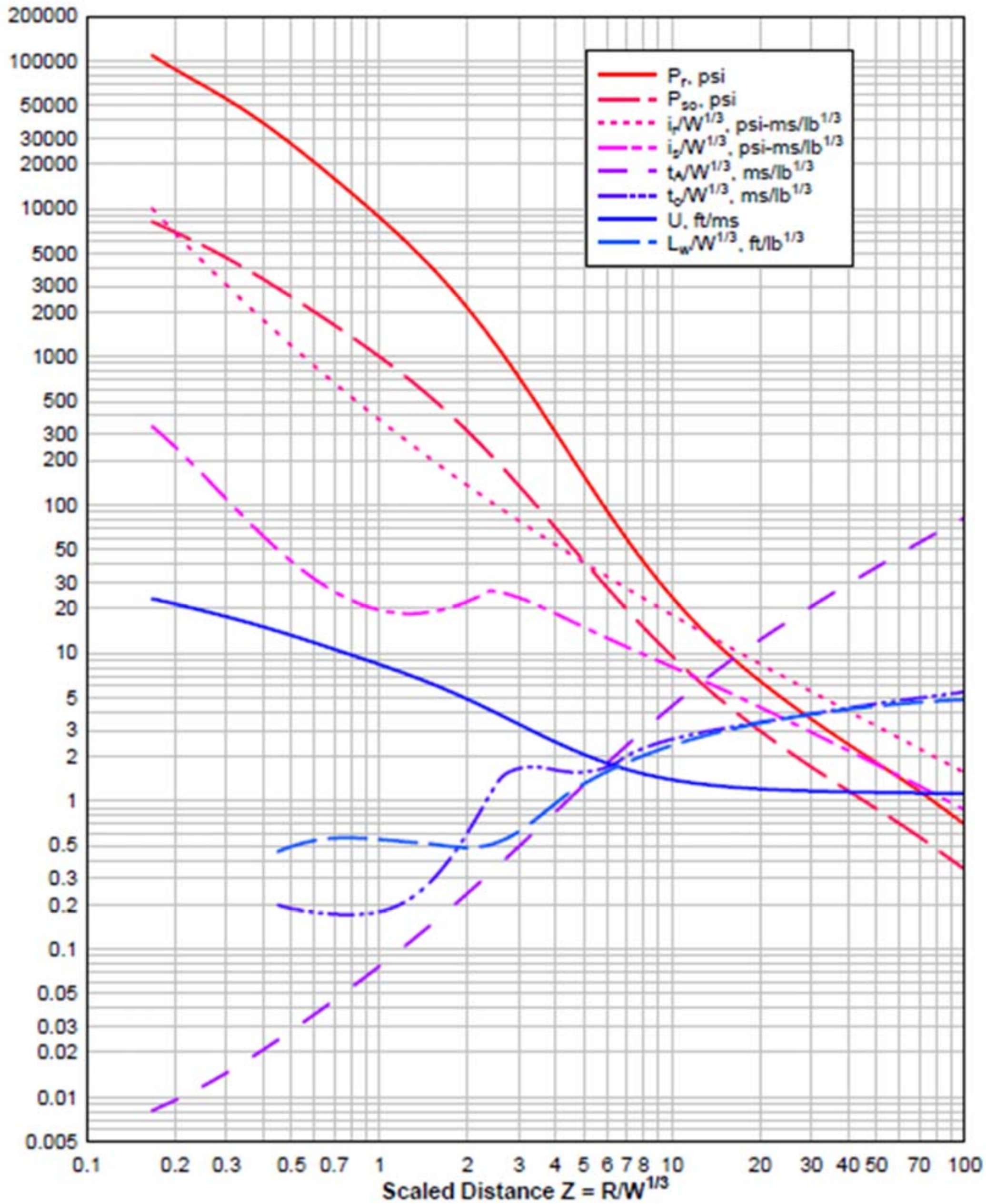

Figure 2.9 : Positive Shock Wave Parameters for a Hemispherical TNT Explosions on the Surface at the Sea Level (Department of Defence, USA, 2008)

Peak reflected pressure and the incidental pressure can be evaluated directly for a given scaled distance from Figure 2.9. Other parameters given in the chart can be used to calculate the time intervals of each phase of simplified pressure profiles.

Maximum negative pressure for a given scaled distance can be found from the Figure 2.9 and the time intervals relevant to negative phase can be calculated from the other parameters given in the chart. Both charts are in log scale and there is a chance of deviation of values when extracted from the charts. Therefore, special attention should be paid when these charts are being used. Small variation of the values found from the chart can have considerable difference in pressure.

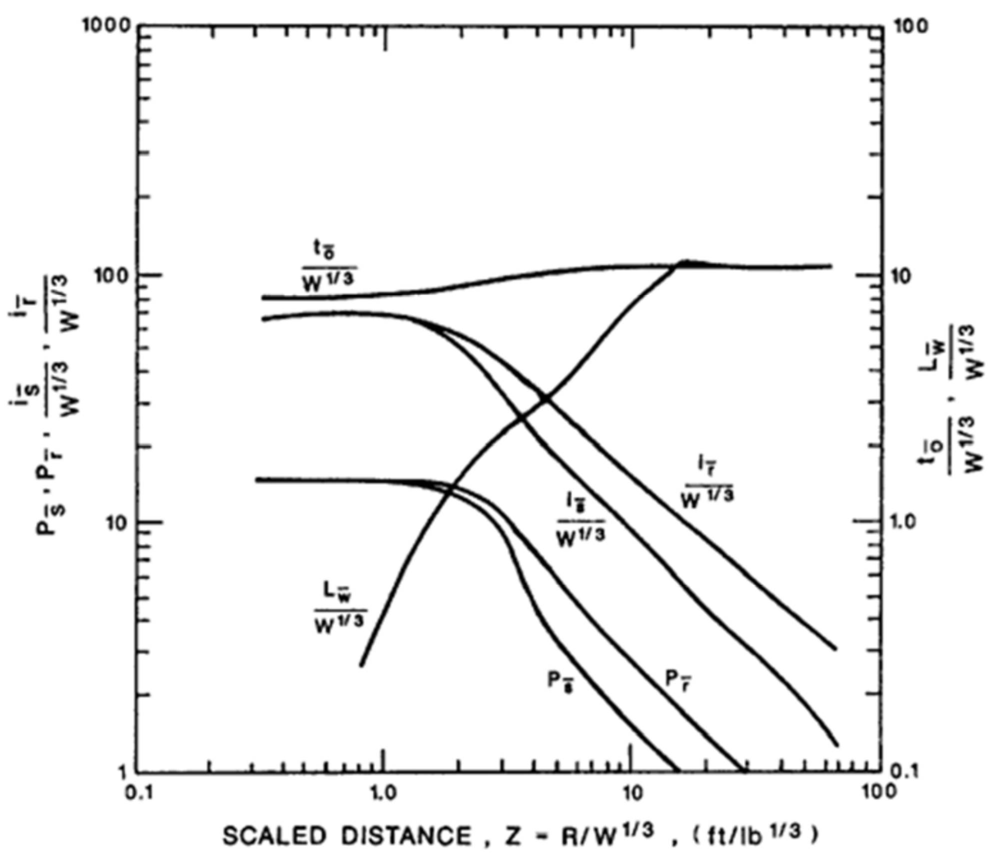

Figure 2.10: Negative Phase Shock Wave Parameters for a Hemispherical TNT Explosion on the Surface at the Sea Level (Department of Defence, USA, 2008)

2.4 MATERIAL BEHAVIOUR

Linear material properties are used for most of the analysis. However, at research level, most of the analysis use the nonlinear range and get the benefits of it. Since the blasting is not a usual event, normally structures are not designed to stay in the linear zone due to the requirement of very large section sizes. As a result of using of larger sections to protect the structures, cost of construction could increase significantly.

2.4.1 ENHANCEMENT OF MATERIAL PROPERTIES

Contribution of the material properties to the analysis differs with the rate of the loading. Different techniques have been proposed by the researches to take into these changes. The book Blast Effects on Buildings by G C Mays and P D Smith (1995) provides guide to enhance the material properties depending on its characteristics. In addition, guideline Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects (2003) by the U.S. General Services Administration (GSA) also has specified factors that can be applied to enhance the material properties. Table 2.1 indicates the material enhancement factors extracted from the book Blast Effects on Buildings. These factors are considered in the present study.

Table 2.1 Material Enhancement Factors (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	—

* Minimum specified f_y for grade 50 steel or less may be enhanced by the average strength increase factor of 1·10.

2.4.2 MATERIAL MODELS

Material non-linearity is a vital factor to be considered in analysis of a structure. In general, it is assumed that the stress is proportionate to the strain. However, beyond some region (for steel – after yielding) this is not valid and it is not directly proportionate. Similarly, concrete also acts like steel. At the initial stage, stress is proportionate to the strain, and when it cracks, the deviation of the stress strain curve can be observed. There are different stress and strain relationships proposed by authors and researches. Material model proposed by Kent and Park (1971) which is used in this study, is most widely used by researches. In addition, there are other similar forms of curves obtained by modifying Kent and Park model or with their own studies. Figure 2.11 indicates the stress and strain relationship found by Kent and Park.

Concrete can be confined by reinforcement and its capacity can increase during the failure. As indicated in the Figure 2.11, confined concrete can retain higher strain compared to the unconfined concrete. The transverse links that are provided act as a confinement link when loads are applied.

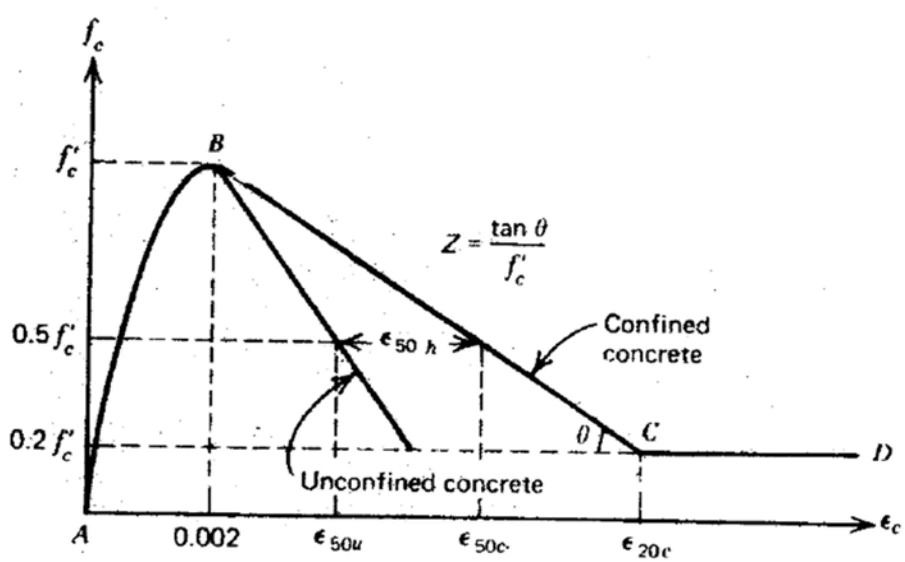

Figure 2.11: Stress-Strain Relationships for Confined and Unconfined Concrete (Kent and Park 1971)

2.4.3 PERFORMANCE BASED DESIGN

In performance based design, performance of the structure is evaluated with sophisticated calculations or structural analysis. Depending on the manner it behaves, performance of it can be monitored. Monitoring can be done with respect to the deformation of the structure or its elements.

The deformation of the structure is evaluated in terms of a drift for monitoring the behaviour of structural and non-structural elements. Limitation of the drift has been given in the Federal Emergency Management Agency (FEMA) 273 for structural vertical elements (concrete walls and columns), and it has also given the limitations for other elements.

Guidelines given in FEMA 356 are used in the study to define the performance levels. Plastic rotation angle is considered to define the performance levels and performance level for flexural elements such as column and beams are given separately in FEMA 356. Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) limits as indicated in the Fig 2.12 can be used to identify the structural performance.

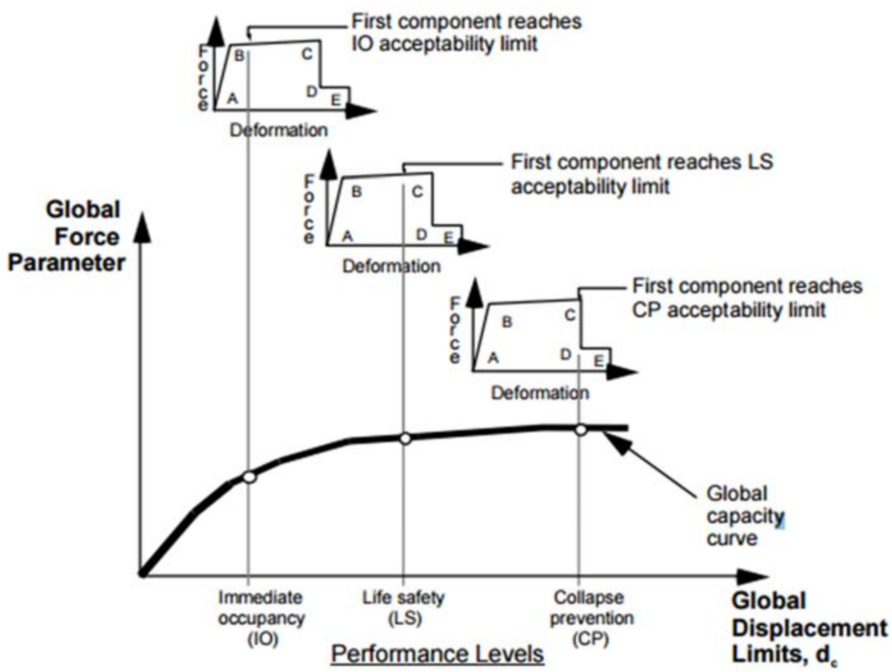

Figure 2.12: Performance Levels (FEMA 356, 2000)

Figure 2.12 shows the performance levels of a structure with respect to global displacement. This is done based on the damage level to the structure. As indicated in the Figure 2.12, when the elements reach the each occupancy level, it says the structure is in that particular occupancy level. If the owner of the building needs to construct a building, which stays in life safety limit for a given blast loading, design engineer shall design a structure that has the capacity to stay in the life safety level.

When an element is considered for performance based design, its plastic rotation is considered. With respect to the hinge rotation, each occupancy level can be defined. Sap2000 software provides comprehensive guideline to define each performance level. Point B in the Figure 2.13 indicates the yielding and no deformation are considered up to the point B. Beyond the point B the rotation of the hinge is considered in the analysis to define the occupancy levels.

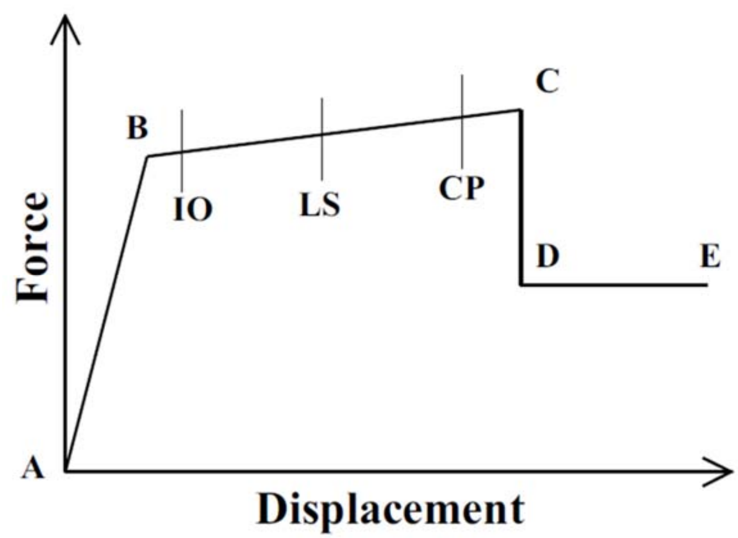

Figure 2.13: Occupancy Levels (Computers and Structures INC, 2009)

2.5 FACADE TYPES AND ITS DEVELOPMENT

Brick, concrete and glass are used most commonly as a material to construct facades. Brick and glass are not ductile materials compared with the concrete. However, these materials are more commonly used to protect the building by enhancing their capacities. Concrete is most commonly used to protect the structures from blast loads as a concrete structural element has higher failure load compared with brick in flexure.

When higher pressure loads are applied on concrete facades, it can fail in flexure due to its incapability of carrying tensile stresses. Reinforcements are provided to carry the tensile stress in the concrete while compressive stress is carried by the concrete. However, when blast occurs near the structure, facades have to carry very high impact loads. In modern construction, different techniques are used to improve the capacity of concrete facades.

Capacity of the glass walls are increased by adding fins to it. Similar arrangement is used for concrete and brick walls also. In addition, different types of membranes are also applied in the tension face of the structural elements to improve the capacity in tension.

Most commonly used construction material in blast resistant buildings is concrete due to its availability and constructability. Since concrete is weak in tension, different techniques are being adapted to enhance its capacity. One of such methods is providing concrete fins. Depending on the span of the panel, it generates bending moments. If the span can be reduced, flexural capacity of the wall can be improved and failure load will be increased. Concrete fins can be introduced in between the floors for reducing its spans.

Concrete fin acts as a flexural element (beam element) to carry the blast pressure loads transferred from the façade wall. It enhances the capacity of the façade wall and adjusting the spacing of fins, economical construction can be done.

2.5.1 BLAST EFFECTS ON FACADES

Limited numbers of guidelines are available to design blast resisting facades. Ngo et al. (2007) have done study on Blast Loading and Blast Effects on Structures. They have studied the blast pressure loading on facades according to the TM 5 – 1300 which was superseded by UFC 3 – 340 – 2, 2008. Figure 2.14 indicates the variation of blast pressure on façade detonated at a distance of 20m.

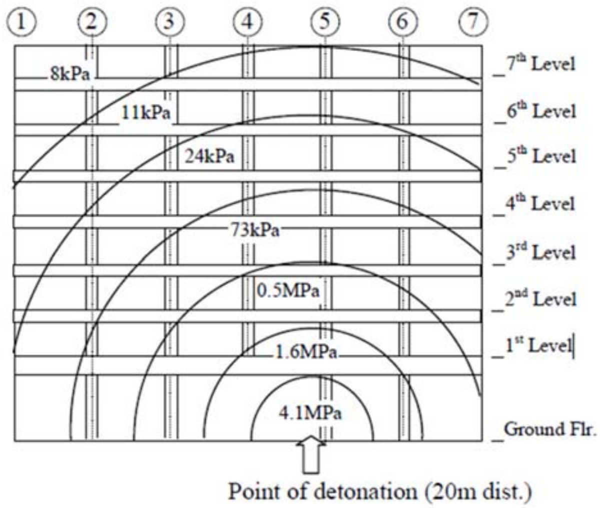

Figure 2.14 Variation of Blast Pressure on Façade (Ngo et al., 2007)

When the blast occurs far away from the structure, blast pressure on façade becomes uniform in part of area of the facade due to Mach reflections. Since, the area where the blast pressure is higher is considered, uniform blast pressure which varies with the time can be considered in this study.

Analysis of concrete wall under the blast loading has been done by Tiwari (2016). Different shapes of concrete walls with and without steel plates and different arrangement of the walls such as “L” shape and “U” shape have studied in this research.

Oswald and Bazan (2014) have studied the performance and blast design for non-load bearing concrete panels. This study has identified that reinforced non-load bearing panels that includes solid concrete panels and insulated sandwich panels with conventional and pre-stressed reinforcement can be designed to resist blast loadings and recommendations are based on the tests of precast panels.

Based on the study limitations in the simplified approach in assessing performance of façade under blast pressure by Lumantarna et al. (2012), it can be identified that factors such as higher mode shape of vibration and the negative phase of a blast pressure have and influence over the performance of the panel. Further, it can be implied from the analysis results that neglecting negative phase in the analysis may lead to un-conservative performance prediction of the dynamic response region. In addition this study highlights the importance of taking into account the negative phase of the blast pressure, especially in the analysis of materials or structural system with limited or no ductility.

2.6 CONCLUSIONS

Blast effects on the structures have been studied extensively in the world due to the increasing number of the terrorist attacks. Landscaping, creating barriers and creating obstructions that increase the standoff distance is done to reduce the blast effects on structures.

Failures of the structures were studied based on the ultimate capacity of the elements or based on the reduction of the stiffness of the elements due to the failure. Occupancy levels such as immediate occupancy, life safety and collapse prevention are considered to define the status of the element depending on the damage level. Rotation of the hinge is considered to define the occupancy level.

3 METHODOLOGY

3.1 BACKGROUND

Analysis of concrete fins for blast loading was done in this study. Different scaled distances were calculated by varying the weight of the blasting material and the distance between the structure and the blasting location. Blast pressure profiles were found for different scaled distance and these loads were applied on fins which were placed at different spacings.

Behaviour of the concrete fins having different arrangement of reinforcement and sizes were studied in this research and they were classified according to the failure criteria considered in this study.

3.2 RESEARCH PROCESS

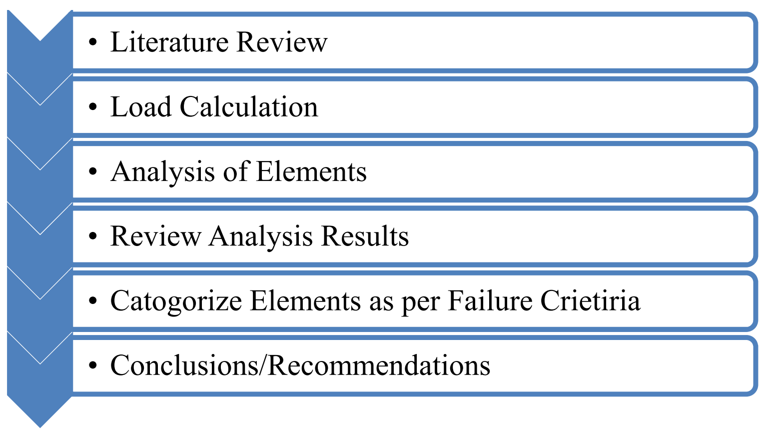

Figure 3.1: Research Process

Fig 3.1 illustrates the research process. A comprehensive study of relevant literature was carried out to get an idea about previous studies. Based on the knowledge gathered from different studies, a platform was created to proceed with this study. After identifying suitable method to calculate the blast loadings on the structure, suitable loading cases were created for different combinations of standoff distances and weights of blasting materials. Reinforcement rations and element sizes were selected as per the existing structures.

Analysis was carried out by considering the material and loading non-linear. After reviewing the analysis results, they were categorized according to the failure criteria.

3.3 MODELLING

Modelling of the concrete fins was done with SAP2000 software. Concrete fins were modelled as frame elements and they were defined with section designer in SAP2000 to get the moment curvature curve accurately. SAP2000 software provides vast range of facilities to designer to analyse structures. Its' capability of considering material non-linearity and non- linear loads in the analysis has given more energy to researches in the twentieth century. In SAP2000, if a section is defined with section designer, moment curvature of that particular section can be generated from program itself. This same curve can be used to define the hinge properties and the failure criteria.

Non-linear behaviour of frame elements and shell elements can be created with the material non-linearity. In addition, capabilities of applying non-linear loads to the model enable doing a nonlinear analysis.

3.3.1 ANALYSIS MODEL

Initially, it was planned to study the behaviour of the concrete fins together with concrete facades. Pressure loads obtained from the UFC 3 340-02, 2008 were applied as a uniformly distributed area load to the model created comprising the concrete façade and fin. Results obtained from the model were not realistic as the effect of the cracking is not considered in the analysis. Due to the incapability of modelling the cracking behaviour of the façade with Sap2000 software, modelling of concrete fin with façade was not continued. Since the prime objective of the research is to find out the behaviour of the concrete fins, they were modelled with concrete frame elements. Fins having different dimensions and reinforcement ratios were analysed for different loadings obtained from UFC 3 340-02, 2008 in the present study.

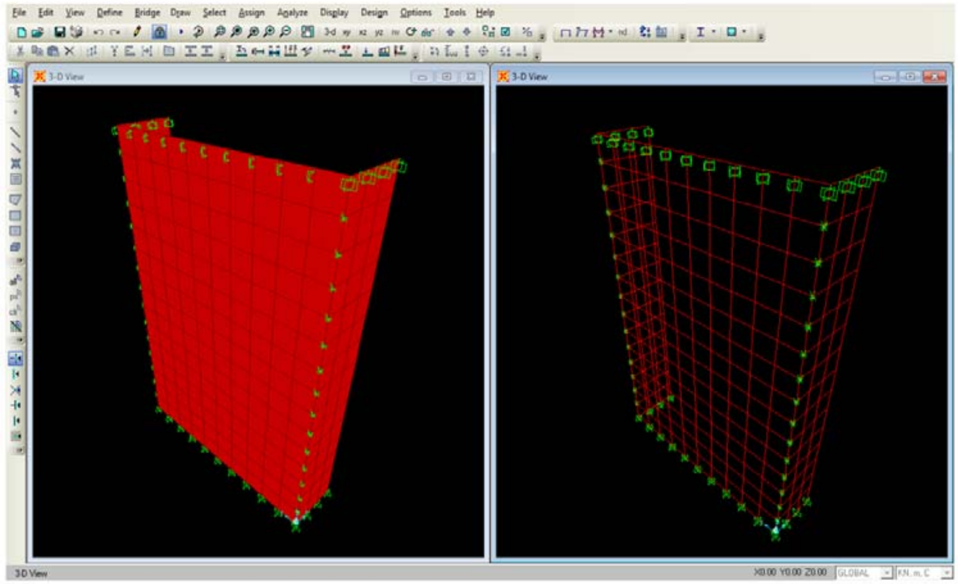

Figure 3.2 Modelling of Concrete Fins with Façade

Figure 3.2 indicates the façade and fins model which was used to for initial analysis with SAP2000 software. Shell elements were modelled with nonlinear shell elements to encounter the nonlinear behaviour. Nonlinear behaviour of the concrete and reinforcement were considered in the nonlinear shell elements. Since the effect of the Mach reflections were considered, uniform pressure loading which varies with the time was applied normal to the face of the façade in the analysis. Results obtained from the analysis did not show a pattern that can be interpreted with general norms.

In this background, it was decided to model the concrete fins alone. When the fin was modelled with frame elements, different sizes of concrete fins, different fin spacing and different reinforcement ratios were considered. Floor to floor height of 3.7m was considered in this study.

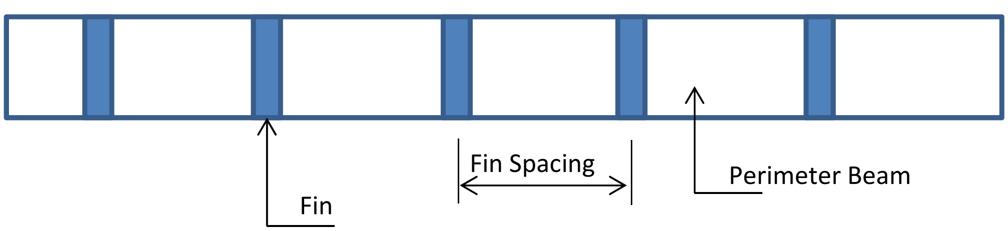

Figure 3.3: Arrangement of the fins

Fins are constructed between the perimeter beams as indicated in the Figure 3.3. In general, when buildings are designed for blast loads, perimeter beams are constructed with comparatively larger sections to avoid the progressive collapse. Therefore, fins can be rested between the perimeter beams.

Due to the rigidity of the beams, both ends of the fins are considered as fixed ends in this study and hinge were assigned for the both ends of the fins near the beams as indicated in the figure 3.4.

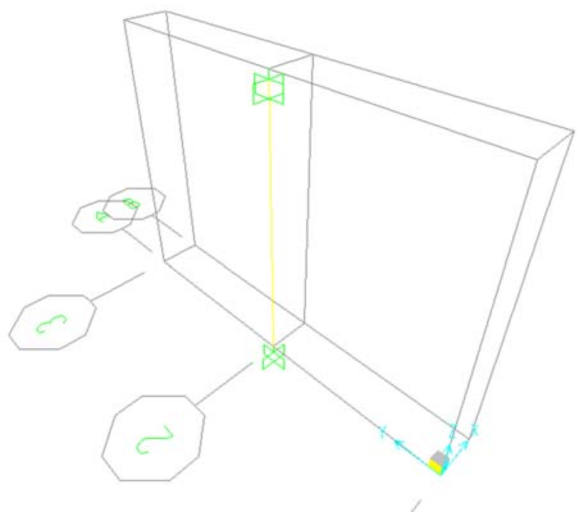

Figure 3.4: Analysis Model with Frame Elements

Analysis model used to analyse concrete fins is indicated in the Figure 3.4. Fins were analysed as concrete frame elements for different load cases as discussed in the Section 3.3.2 and Section 3.3.3.

Based on the literature, variation of the pressure can be considered as uniform over the façade at a given time due to the phenomenon Mach reflection. Mach reflections are formed at an angle of 40^0 approximately. The panel which felt the maximum pressure is considered in the study. Façade at ground floor having the angle of incident is zero is considered in the study as it has the shortest standoff distance.

Different sections having different reinforcement ratios were analysed to study the ability of them to stand against blast pressure with respect to the failure criteria defined in the Section 3.7. Figure 3.5 indicates the dimensions and reinforcement arrangement of the concrete fin having dimensions of 150x500mm. Different reinforcement arrangements such as 8T10, 10T10, 8T12 and 10T12 are considered for this section.

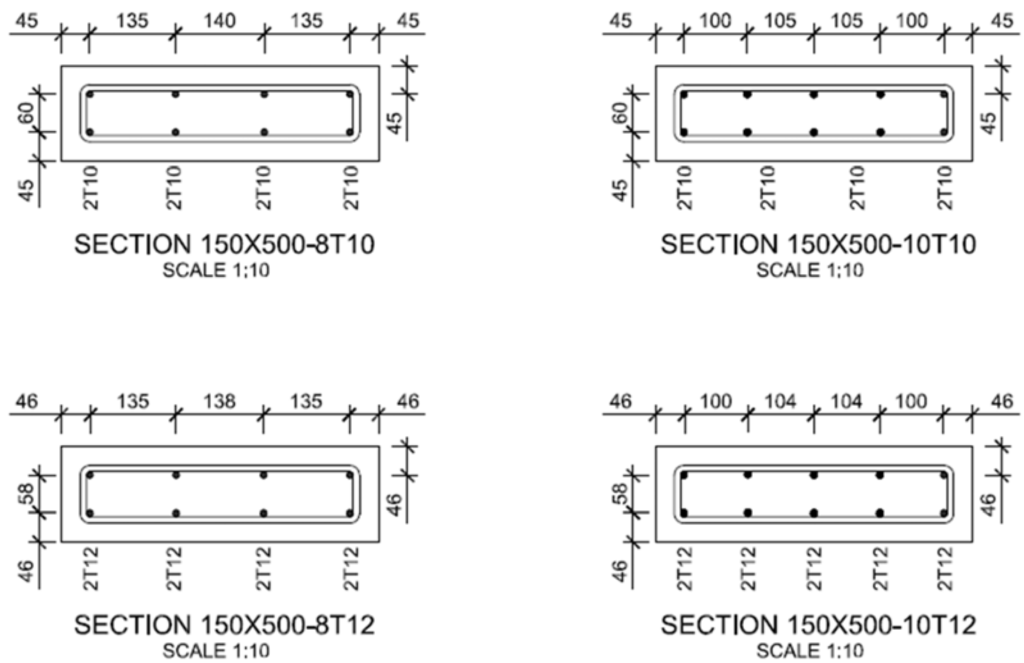

Figure 3.5: Details of Analysis Section 150x500

Figure 3.6 indicates the concrete fin having dimensions 150x600mm. It was required to increase the stiffness of the fins to check whether they can stand for very higher blast loads with the increase of blast pressure significantly as indicated in the Table 3.2 when the standoff distance is 10m.

Reinforcement arrangements such as 10T10, 12T10, 10T12 and 12T12 were considered for fins having dimensions 150x600mm. Further, reinforcement ratio and section dimensions was increased as indicated in Figure 3.7 to check the structural capacity of the fins when they are placed very closed to blast.

Sixteen numbers of reinforcement bars having diameter of 12mm were considered for the section 150x800mm. Main purpose of using this section was to check whether the increasing of stiffness of the fins would enhance the structural capacity.

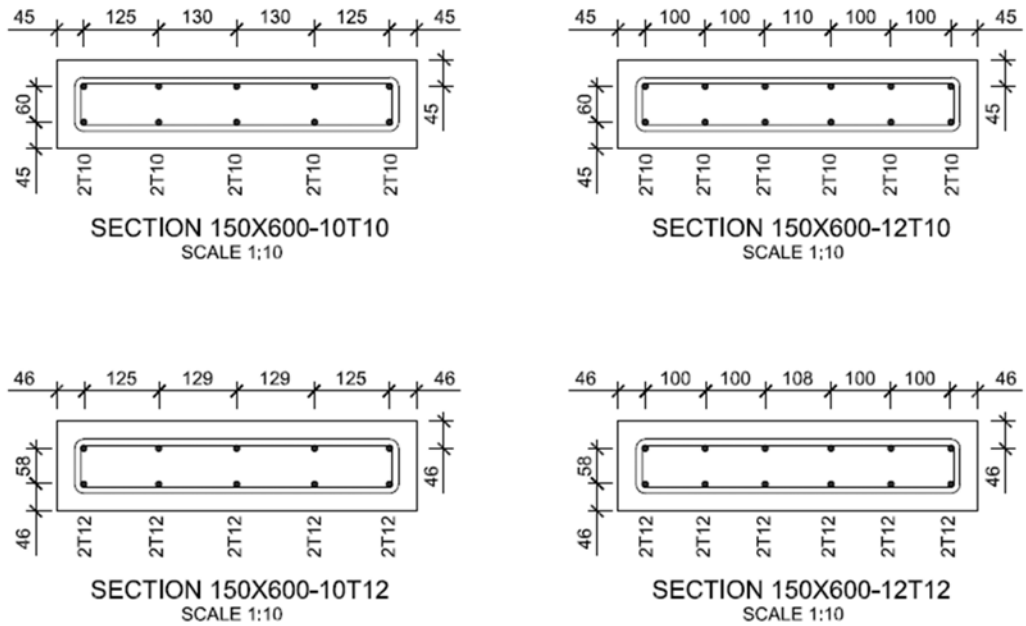

Figure 3.6: Details of Analysis Section 150x600

Effect of the increase of the section dimensions and reinforcement rations were also calculated in the study as comparisons of each analysis cases.

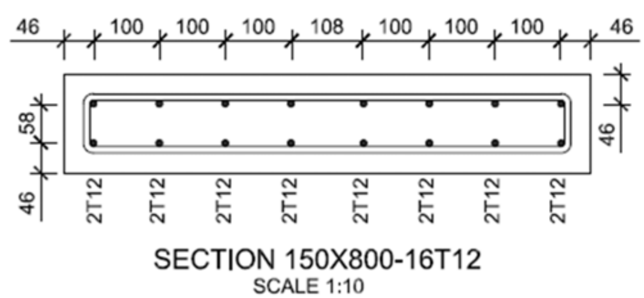

Figure 3.7: Details of Analysis Section 150x800

3.3.2 ANALYSIS CASES

Each section was analysed for three different fin spacings and reinforcement arrangements as indicated in the Table 3.1. Thus, different analysis cases were created by varying the fin spacing and standoff distance.

Table 3.1: Different Analysis Cases

Spacing	1m	2m	3m
Section	150x500, 150x600, 150x800	150x500, 150x600, 150x800	150x500, 150x600, 150x800
Reinforcement	8T10, 8T12, 10T10, 10T12, 12T10, 12T12, 16T12		

3.3.3 LOAD CASES

As per the load calculations, there are 15 load cases. Table 3.2 indicates the load cases found in the load evaluation.

Table 3.2 Load Cases

Number of Charts	R/m	W/kg	Z /(m/kg$^{1/3}$)	Pr /(kN/m^2)	Pso /(kN/m^2)	Tof /(ms)	To /(ms)	Pr- /(kN/m^2)	trf- /(ms)
1	10	10	5	117	50	6	8	17	31
2		25	3	236	89	5	9	23	41
3		50	3	436	144	5	9	29	49
4		100	2	845	239	5	10	38	58
5		200	2	1695	407	4	12	51	68
6	25	25	9	40	18	11	13	9	47
7		50	7	58	26	13	16	11	58
8		100	5	87	38	14	18	14	69
9		200	4	140	58	14	21	18	84
10		300	4	189	74	14	22	21	94
11		400	3	241	90	14	23	23	102
12	50	100	11	28	13	20	23	8	75
13		200	9	40	18	23	27	9	93
14		300	7	49	23	24	29	11	107
15		400	7	58	26	25	31	12	115

The values given in the Table 3.2 indicate the variation of the blast pressure with time at different standoff distances (R) and for different weights of blast materials (W). Tables 3.3 to 3.5 indicate the different analysis cases created by combining the parameters considered in this study.

Combinations of notations to represent the load cases were used throughout the thesis for clarity and simplicity. Load cases were represented by R10W10.2m. In the notation, R10 indicates the standoff distances as 10, W10 indicates the weight of blast material and 2m indicates the fin spacing.

Table 3.3 Calculated Loads for standoff distance 10m and weight of explosive materials 10, 25, 50, 100 and 200 (kg)

Load Case		Variation of Pressure with Time					
R10W10	R10W10.1m	Time/s	0	0.006	0.008	0.017	0.039
		Load/kN/m^2	117	0	0	-17	0
	R10W10.2m	Time/s	0	0.006	0.008	0.017	0.039
		Load/kN/m^2	234	0	0	-34	0
	R10W10.3m	Time/s	0	0.006	0.008	0.017	0.039
		Load/kN/m^2	351	0	0	-51	0
R10W25	R10W25.1m	Time/s	0	0.005	0.009	0.0193	0.05
		Load/kN/m^2	236	0	0	-23	0
	R10W25.2m	Time/s	0	0.005	0.009	0.0193	0.05
		Load/kN/m^2	472	0	0	-46	0
	R10W25.3m	Time/s	0	0.005	0.009	0.0193	0.05
		Load/kN/m^2	708	0	0	-69	0
R10W50	R10W50.1m	Time/s	0	0.005	0.009	0.0213	0.058
		Load/kN/m^2	436	0	0	-29	0
	R10W50.2m	Time/s	0	0.005	0.009	0.0213	0.058
		Load/kN/m^2	872	0	0	-58	0
	R10W50.3m	Time/s	0	0.005	0.009	0.0213	0.058
		Load/kN/m^2	1308	0	0	-87	0
R10W100	R10W100.1m	Time/s	0	0.005	0.01	0.0245	0.068
		Load/kN/m^2	845	0	0	-38	0
	R10W100.2m	Time/s	0	0.005	0.01	0.0245	0.068
		Load/kN/m^2	1690	0	0	-76	0
	R10W100.3m	Time/s	0	0.005	0.01	0.0245	0.068
		Load/kN/m^2	2535	0	0	-114	0
R10W200	R10W200.1m	Time/s	0	0.004	0.012	0.029	0.08
		Load/kN/m^2	1695	0	0	-51	0
	R10W200.2m	Time/s	0	0.004	0.012	0.029	0.08
		Load/kN/m^2	3390	0	0	-102	0
	R10W200.3m	Time/s	0	0.004	0.012	0.029	0.08
		Load/kN/m^2	5085	0	0	-153	0

Table 3.4 Calculated Loads for standoff distance 25m and weight of explosive materials 25, 50, 100, 200, 300 and 400 (kg)

Load Case		Variation of Pressure with Time					
R25W25	R25W25.1m	Time/s	0	0.011	0.013	0.0248	0.06
		Load/kN/m^2	40	0	0	-9	0
	R25W25.2m	Time/s	0	0.011	0.013	0.0248	0.06
		Load/kN/m^2	80	0	0	-18	0
	R25W25.3m	Time/s	0	0.011	0.013	0.0248	0.06
		Load/kN/m^2	120	0	0	-27	0
R25W50	R25W50.1m	Time/s	0	0.013	0.016	0.0305	0.074
		Load/kN/m^2	58	0	0	-11	0
	R25W50.2m	Time/s	0	0.013	0.016	0.0305	0.074
		Load/kN/m^2	116	0	0	-22	0
	R25W50.3m	Time/s	0	0.013	0.016	0.0305	0.074
		Load/kN/m^2	174	0	0	-33	0
R25W100	R25W100.1m	Time/s	0	0.014	0.018	0.0353	0.087
		Load/kN/m^2	87	0	0	-14	0
	R25W100.2m	Time/s	0	0.014	0.018	0.0353	0.087
		Load/kN/m^2	174	0	0	-28	0
	R25W100.3m	Time/s	0	0.014	0.018	0.0353	0.087
		Load/kN/m^2	261	0	0	-42	0
R25W200	R25W200.1m	Time/s	0	0.014	0.021	0.042	0.105
		Load/kN/m^2	140	0	0	-18	0
	R25W200.2m	Time/s	0	0.014	0.021	0.042	0.105
		Load/kN/m^2	280	0	0	-36	0
	R25W200.3m	Time/s	0	0.014	0.021	0.042	0.105
		Load/kN/m^2	420	0	0	-54	0
R25W300	R25W300.1m	Time/s	0	0.014	0.022	0.0455	0.116
		Load/kN/m^2	189	0	0	-21	0
	R25W300.2m	Time/s	0	0.014	0.022	0.0455	0.116
		Load/kN/m^2	378	0	0	-42	0
	R25W300.3m	Time/s	0	0.014	0.022	0.0455	0.116
		Load/kN/m^2	567	0	0	-63	0
R25W400	R25W400.1m	Time/s	0	0.014	0.023	0.0485	0.125
		Load/kN/m^2	241	0	0	-23	0
	R25W400.2m	Time/s	0	0.014	0.023	0.0485	0.125
		Load/kN/m^2	482	0	0	-46	0
	R25W400.3m	Time/s	0	0.014	0.023	0.0485	0.125
		Load/kN/m^2	723	0	0	-69	0

Table 3.5 Calculated Loads for standoff distance 50m and weight of explosive materials 100, 200, 300 and 400 (kg)

Load Case		Variation of Pressure with Time					
R50W100	R50W100.1m	Time/s	0	0.02	0.023	0.0418	0.098
		Load/kN/m^2	28	0	0	-8	0
	R50W100.2m	Time/s	0	0.02	0.023	0.0418	0.098
		Load/kN/m^2	56	0	0	-16	0
	R50W100.3m	Time/s	0	0.02	0.023	0.0418	0.098
		Load/kN/m^2	84	0	0	-24	0
R50W200	R50W200.1m	Time/s	0	0.023	0.028	0.0503	0.12
		Load/kN/m^2	40	0	0	-9	0
	R50W200.2m	Time/s	0	0.023	0.028	0.0503	0.12
		Load/kN/m^2	80	0	0	-18	0
	R50W200.3m	Time/s	0	0.023	0.028	0.0503	0.12
		Load/kN/m^2	120	0	0	-27	0
R50W300	R50W300.1m	Time/s	0	0.024	0.029	0.0558	0.12
		Load/kN/m^2	49	0	0	-11	0
	R50W300.2m	Time/s	0	0.024	0.029	0.0558	0.12
		Load/kN/m^2	98	0	0	-22	0
	R50W300.3m	Time/s	0	0.024	0.029	0.0558	0.12
		Load/kN/m^2	147	0	0	-33	0
R50W400	R50W400.1m	Time/s	0	0.025	0.031	0.0598	0.146
		Load/kN/m^2	58	0	0	-12	0
	R50W400.2m	Time/s	0	0.025	0.031	0.0598	0.146
		Load/kN/m^2	116	0	0	-24	0
	R50W400.3m	Time/s	0	0.025	0.031	0.0598	0.146
		Load/kN/m^2	174	0	0	-36	0

Fin spacing indicated in the Table 3.1 was considered to find the different pressure loads. Different load cases (15 load cases) as indicated in the Table 3.2 and three different fin spacing were considered to create different load cases. Consideration of the different reinforcement arrangements (8T10, 8T12, 10T10, 10T12, 16T12) also increase the number of analysis cases. Tables 3.3, 3.4 and 3.5 indicate the different types of loading arrangements used for the analysis.

Tables 3.3 to 3.5 indicate different analysis cases created by varying the fin spacing, blast pressure and section properties of the fin. These analysis cases were analyzed with Sap2000 software.

3.4 MATERIAL MODELS

Behaviour of the material beyond the linear range (nonlinear region) was considered in this study. There are different material models developed by many researchers through the numerical studies and testing of materials. Most commonly used material models by researchers were used in this study.

3.4.1 CONCRETE MATERIAL MODEL

Concrete is a material that has higher compression capacity compared to its tensile capacity. Compressive strength of concrete can be increased by confining with reinforcement links.

Concrete can be confined by transverse reinforcement which is closely spaced and it can be in the form of spirals or hoops. Material properties were defined by taking into account the effect of the confinement.

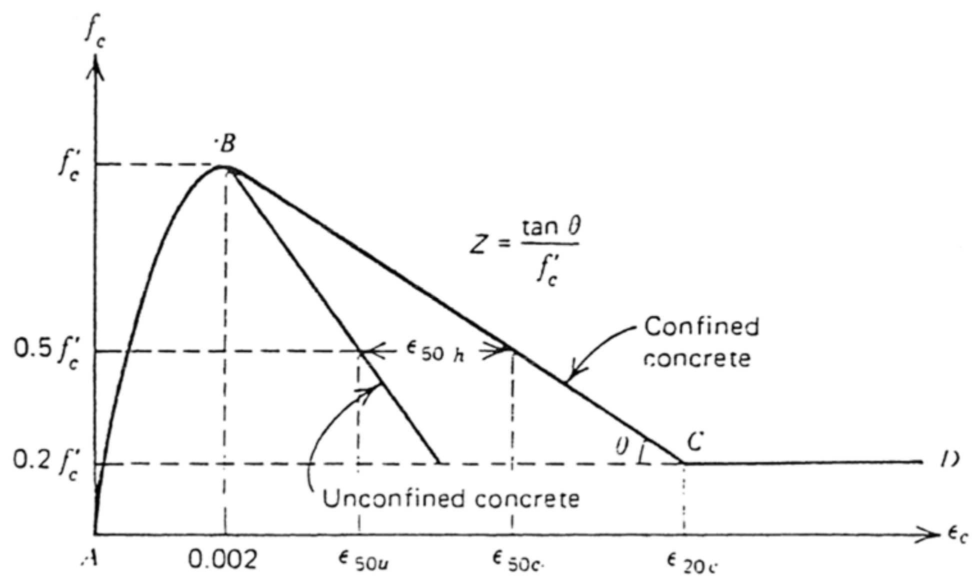

Figure 3.8: Stress-Strain Curve for Concrete Confined by Rectangular Hoops (Kent and Park, 1971)

As indicated in the Figure 3.8, parameter of the material model can be found. Following equation can be used to calculate the different value of the graph.

For region BC: $0.002 \leq \varepsilon_c \leq \varepsilon_{20c}$

$$f_c = f_c'[1 - Z(\varepsilon_c - 0.002)]$$

Where

$$Z \quad = \frac{0.5}{\varepsilon_{50u} + \epsilon_{50h} - 0.002}$$

$$\varepsilon_{50u} = \frac{3 + 0.002 f_c'}{f_c' - 1000}$$

$$\varepsilon_{50h} = \frac{3}{4}\rho_s\sqrt{\frac{b''}{s_h}}$$

f_c' = Concrete cylinder strength

ρ_s = Ratio of volume of transverse reinforcement to volume of concrete core measured to outside of hoops

b" = Width of confined core measured to outside of hoops
S_h = Spacing of hoops
When the concrete cylinder strength and other parameters specified above are known, material model for concrete can be defined.

3.4.2 STEEL MATERIAL MODEL

Stress and strain relationship of the reinforcing steel under tension and compression was considered to create a model. Possibility of buckling under the compression load was not considered for model and it was ignored. Depending on the yield strength and the tensile strength parameters, steel material model can be defined. Figure 3.9 indicates the steel material model used in this study.

Values in the diagram are based on the test results obtained from testing reinforced steel. Default steel material model of the SAP2000 software is modified as per the steel strength parameters obtained from test results.

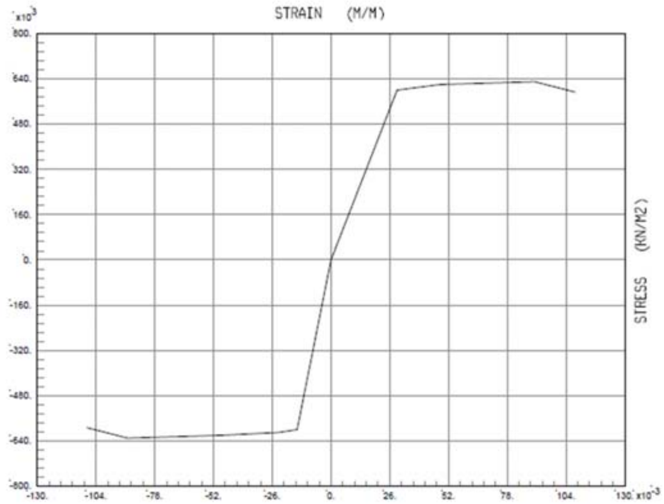

Figure 3.9 Steel Material Model (SAP 2000)

3.4.3 ENHANCEMENT OF MATERIAL PROPERTIES

Mechanical properties of a material changes with the rate of strain. Design strengths of concrete and reinforcement were modified with the increase in the strain rate. Increase of strain is sudden in a blast. Therefore material strengths were modified by dynamic increase factor (DIF) in accordance with the book Blast Effect on Buildings (1995)

Bending, shear and compression stresses of concrete, reinforcing bars and structural steel can be modified with dynamic increase factor. There are different values proposed by the researches to modify the material properties when there are higher rates of changing the strains. However, the values proposed in the book Blast Effect on Buildings was used in this study.

Factors indicated in the Table 2.1 were used in this study to modify the material strengths. Concrete cube strength, yield strength and ultimate tensile strength were modified in this study.

Values obtained for f_{cu}, f_y and f_u were modified from the parameters indicated in the Table 2.1.

Modified stresses for bending can be calculated as follows;

Dynamic yield strength, $f_{dy} \quad = 1.2f_y$

Dynamic tensile strength, f_{du} = $1.05f_u$

Dynamic cube strength, f_{dcu} = $1.25f_{cu}$

3.5 STRUCTURAL ELEMENTS

In the initial study, concrete fins were modelled with the concrete façade. However, method of analysis was changed due to the unavailability of crack modelling for shell element in SAP2000 software. It was decided to model the concrete fins along as a frame element.

Shell elements were modelled with nonlinear shell elements that are available in the SAP2000 software. Frame elements were defined with section designer in the SAP2000 software as the moment curvature curve could be generated from the software itself. Simplified moment curvature diagram was used to define the failure criteria.

3.6 CALCULATION OF LOADS

Variation of the blast pressure on the facade was considered in this study to find out the failure mode and to define the failure criteria for different loadings. Generally, blast loading is given by well experienced persons in this field, considering the type of structure and vulnerability of the structure. However, the pressure loads were calculated for different scale distances in this study.

Different weight of blast materials and standoff distances were considered in this study for obtaining different blast pressures. Blast loadings were evaluated as per the guidelines specified in the UFC 03-340-02, 2008.

UFC 03-340-02, 2008 provides different charts to evaluate positive and negative phase pressure parameters. Once the parameters are known, values of the simplified pressure profile can be calculated. Figure 3.10 indicates the simplified variation of blast pressure with time.

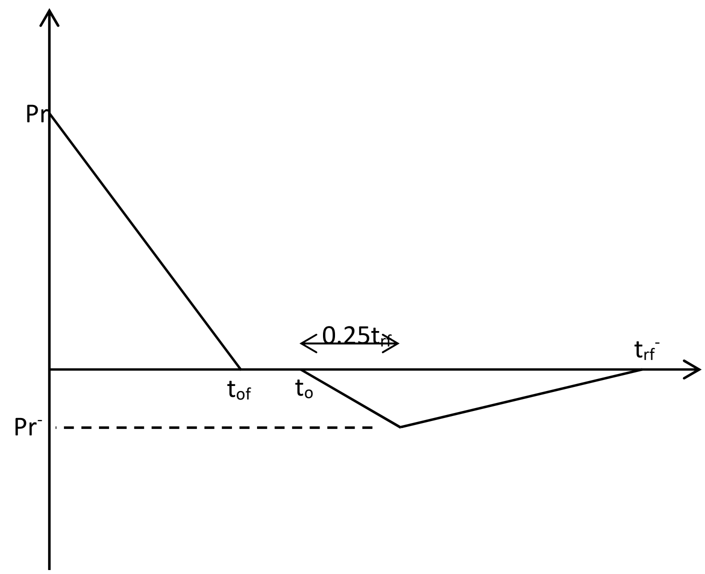

Figure 3.10 Simplified Variation of Blast Pressure (UFC 03-340-02, 2008)

P_r and t_{of} should be known to define the positive phase and with p_r-, t_o and t_{rf}- negative phase can be defined. Figure 2.9 was used to find the positive phase parameters, while Figure 2.10 was used to evaluate negative phase parameters.

Calculation of loads can be done as per the following steps.

Step 01

Calculate the scaled distance, Z

$$Z = \frac{R}{W^{1/3}}$$

Where R is the standoff distance between structure and location of blast and W is the weight of blasting material.

Step 02

Find the value of P_r from Figure 2.9.

Step 03

Find the value of P_{so} from Figure 2.9

Step 04

Find the value of $i_s/W^{1/3}$ from Figure 2.9 and calculate the i_s

Calculate $t_{of} = \frac{2i_s}{P_{so}}$

Step 05

Find P_r^- from figure 2.10

Step 06

Find the value of $t_0/W^{1/3}$ from Figure 2.9 and calculate t_0

Step 07

Find the value of $i_r-/W^{1/3}$ and calculate i_r-

Calculate $t_{rf}- = \frac{2i_r-}{P_r-}$

When the above parameters are calculated, pressure profile indicated in the Figure 3.10 can be obtained for a given scaled distance.

3.7 FAILURE CRITERIA

Failure of an element can be defined with different methods. State of the structure when it deforms, can be defined with respect to the occupancy levels. Generally, occupancy levels are defined considering plastic rotation of the elements.

Immediate occupancy (IO), life safety (LS) and collapse prevention (CP) were considered in this study to define a failure mode of an element. Hinge properties were defined as specified in the FEMA 356. SAP2000 software is capable of defining hinges as specified in the FEMA 356.

Figure 2.13 indicates the force deformation curve of a hinge. Moment curvature diagram of the hinge also has similar pattern as Figure 2.13. After analysis of the structure, status of the hinge (IO or LS and CP) is indicating with relevant color. It is used to categorize the elements capacities considering its physical properties.

Occupancy levels are defined based on the plastic rotation of the hinge. It represents the damage level to the element in the form of occupancy level. If the structure is in the immediate occupancy level, there are no much damages to the structure and it can be occupied even immediately after a blast. Minor cracks which may appear in the structure will not affect to the functions of it.

When the hinge rotation is between immediate occupancy level and life safety limit, there may not be loss of life. However, a considerable damage to the structure can be expected. There could be fallen parts and cracks which are reparable. If the rotation of the hinge is between the life safety limit and collapse prevention limit, there could be loss of life and considerable damage to the structure. Structure is not in the state of reparable even though it has not collapsed. When the hinge rotation exceeds the collapse prevention limit, some of the structural elements may have damaged permanently and it could lead to the failure of whole structure.

3.8 COMPARISON OF COST

Cost of the construction is one of the major factor that has to be considered in the design stages of a project. In general, construction cost would be quite higher when compared with other types of buildings as they are design for very high lateral loads. Having some idea about the final figure would be more useful during the design phase of the project to know whether the Client can bear it.

Comparison of the cost for different arrangements of reinforcements and sections were done in this study. Construction cost of concrete fins only was considered and other related costs were assumed to be zero to simplify the analysis. Cost of concrete, reinforcement and formwork that include the labour, material, over heads and profits were considered for the cost calculations, and rates were obtained comparing few projects.

4 ANALYSIS AND RESULTS

4.1 BACKGROUND

Based on the analysis techniques described in Chapter 03, 64 load cases were analysed. Results obtained from the analysis are discussed in this chapter. Evaluation of the capacity of each fin according to the failure criteria defined in Chapter 03 was done in this chapter, and the failure criterion was used to categorize the element as per the occupancy levels.

4.2 ANALYSIS RESULTS

A failure criterion was defined based on the force deformation curve in Chapter 03 as indicated in Figure 2.13. Occupancy levels (immediate occupancy, life safety and collapse prevention) which are based on the plastic rotation of the hinge were used to identify the state of the hinge. Point A to IO is defined as immediate occupancy (IO) while IO to LS is considered as Life Safety (LS). LS to CP are defined as collapse prevention (CP).

Some of the results obtained from the analysis are elaborated and discussed herein, and summary of the analysis results are tabulated and discussed in Section 4.3.

Table 4.1 indicates the analysis results of analysis case 8T10-R50W400-3m. In this analysis case, fin of size 150x500 mm having eight number of tor steel bars of diameter 10mm was considered. Fin spacing of 3m, 400kg of blast materials and standoff distance as 50m was considered for this load case.

Table 4.1 indicates the stage and status (occupancy level) of the hinge with reference to Figure 2.13. According to the Table 4.1, it can be identified the status of the hinge as Immediate occupancy level as the rotation of the hinge lies between A to IO.

Thus, it can be concluded that the element has not been subjected to much damages due to the blast pressure exerted on the element. Therefore the element can be considered as safe structural element that can be used after the blast. Further, there could be minor damages to the element which falls into the category of minor damages that can even be repaired after occupying the building. If all elements are in this state, structure can be immediately occupied and it can function as usual.

Table 4.1: Analysis Results of 150x500x8T10-R50W400-3m

Time/(s)	Stage	Status of hinge
0	A to <=B	A to <=IO
0.002	A to <=B	A to <=IO
0.004	A to <=B	A to <=IO
0.006	A to <=B	A to <=IO
0.008	A to <=B	A to <=IO
0.01	B to <=C	A to <=IO
0.012	B to <=C	A to <=IO
0.014	B to <=C	A to <=IO
0.016	B to <=C	A to <=IO
0.018	B to <=C	A to <=IO
0.02	B to <=C	A to <=IO
0.022	B to <=C	A to <=IO
0.024	B to <=C	A to <=IO
0.026	B to <=C	A to <=IO
0.028	B to <=C	A to <=IO
0.03	B to <=C	A to <=IO
0.032	B to <=C	A to <=IO
0.034	B to <=C	A to <=IO
0.036	B to <=C	A to <=IO
0.038	B to <=C	A to <=IO
0.04	B to <=C	A to <=IO
0.042	B to <=C	A to <=IO
0.044	B to <=C	A to <=IO
0.046	B to <=C	A to <=IO
0.048	B to <=C	A to <=IO
0.05	B to <=C	A to <=IO
0.052	B to <=C	A to <=IO
0.054	B to <=C	A to <=IO
0.056	B to <=C	A to <=IO
0.058	B to <=C	A to <=IO
0.06	B to <=C	A to <=IO
0.062	B to <=C	A to <=IO
0.064	B to <=C	A to <=IO
0.066	B to <=C	A to <=IO
0.068	B to <=C	A to <=IO
0.07	B to <=C	A to <=IO
0.072	B to <=C	A to <=IO
0.074	B to <=C	A to <=IO
0.076	B to <=C	A to <=IO
0.078	B to <=C	A to <=IO
0.08	B to <=C	A to <=IO
0.082	B to <=C	A to <=IO
0.084	B to <=C	A to <=IO
0.086	B to <=C	A to <=IO
0.088	B to <=C	A to <=IO
0.09	B to <=C	A to <=IO
0.092	B to <=C	A to <=IO
0.094	B to <=C	A to <=IO
0.096	B to <=C	A to <=IO
0.098	B to <=C	A to <=IO
0.1	B to <=C	A to <=IO
0.102	B to <=C	A to <=IO
0.104	B to <=C	A to <=IO
0.106	B to <=C	A to <=IO
0.108	B to <=C	A to <=IO
0.11	B to <=C	A to <=IO
0.112	B to <=C	A to <=IO
0.114	B to <=C	A to <=IO
0.116	B to <=C	A to <=IO
0.118	B to <=C	A to <=IO
0.12	B to <=C	A to <=IO
0.122	B to <=C	A to <=IO
0.124	B to <=C	A to <=IO
0.126	B to <=C	A to <=IO
0.128	B to <=C	A to <=IO
0.13	B to <=C	A to <=IO
0.132	B to <=C	A to <=IO
0.134	B to <=C	A to <=IO
0.136	B to <=C	A to <=IO
0.138	B to <=C	A to <=IO
0.14	B to <=C	A to <=IO
0.142	B to <=C	A to <=IO
0.144	B to <=C	A to <=IO
0.146	B to <=C	A to <=IO

Table 4.2 indicates the results obtained for analysis case 150x500mm-8T10-R25W300-2m. In this case, 150x500mm fins having eight numbers of 10mm

diameter tor steel bars is analyzed placing fins 2m apart for blast load generated from 300kg of blasting materials at a distance of 25m to the structure.

Table 4.2: Analysis Results of 150x500x8T10-R25W300-2m

Time/(s)	Stage	Status of Hinge
0	A to <=B	A to <=IO
0.002	A to <=B	A to <=IO
0.004	B to <=C	A to <=IO
0.006	B to <=C	A to <=IO
0.008	B to <=C	A to <=IO
0.01	B to <=C	IO to <=LS
0.012	B to <=C	IO to <=LS
0.014	B to <=C	IO to <=LS
0.016	B to <=C	IO to <=LS
0.018	B to <=C	IO to <=LS
0.02	B to <=C	IO to <=LS
0.022	B to <=C	IO to <=LS
0.024	B to <=C	IO to <=LS
0.026	B to <=C	IO to <=LS
0.028	B to <=C	IO to <=LS
0.03	B to <=C	IO to <=LS
0.032	B to <=C	IO to <=LS
0.034	B to <=C	IO to <=LS
0.036	B to <=C	IO to <=LS
0.038	B to <=C	IO to <=LS
0.04	B to <=C	IO to <=LS
0.042	B to <=C	IO to <=LS
0.044	B to <=C	IO to <=LS
0.046	B to <=C	IO to <=LS
0.048	B to <=C	IO to <=LS
0.05	B to <=C	IO to <=LS
0.052	B to <=C	IO to <=LS
0.054	B to <=C	IO to <=LS
0.056	B to <=C	IO to <=LS
0.058	B to <=C	IO to <=LS

Time/(s)	Stage	Status of Hinge
0.06	B to <=C	IO to <=LS
0.062	B to <=C	IO to <=LS
0.064	B to <=C	IO to <=LS
0.066	B to <=C	IO to <=LS
0.068	B to <=C	IO to <=LS
0.07	B to <=C	IO to <=LS
0.072	B to <=C	IO to <=LS
0.074	B to <=C	IO to <=LS
0.076	B to <=C	IO to <=LS
0.078	B to <=C	IO to <=LS
0.08	B to <=C	IO to <=LS
0.082	B to <=C	IO to <=LS
0.084	B to <=C	IO to <=LS
0.086	B to <=C	IO to <=LS
0.088	B to <=C	IO to <=LS
0.09	B to <=C	IO to <=LS
0.092	B to <=C	IO to <=LS
0.094	B to <=C	IO to <=LS
0.096	B to <=C	IO to <=LS
0.098	B to <=C	IO to <=LS
0.1	B to <=C	IO to <=LS
0.102	B to <=C	IO to <=LS
0.104	B to <=C	IO to <=LS
0.106	B to <=C	IO to <=LS
0.108	B to <=C	IO to <=LS
0.11	B to <=C	IO to <=LS
0.112	B to <=C	IO to <=LS
0.114	B to <=C	IO to <=LS
0.116	B to <=C	IO to <=LS

According to the hinge results, status of the hinge is at life safety limit as the final rotation of the hinge is between the limits of IO and LS. In this occupancy level, structure might have subjected to considerable level of damage. There could be some cracks in the structure which are reparable. A structure falls into the stage of life safety can be occupied after repairing.

An analysis result for 150x500mm fin of having 8 numbers of tor steel bars is shown in Table 4.3. Fin is analyzed for a blast pressure generated by blasting 25kg of blasting material at a distance of 10m.

Table 4.3: Analysis Results of 150x500x8T10-R10W25-2m

Time/(s)	Stage	Status of Hinge
0	A to <=B	A to <=IO
0.0025	B to <=C	A to <=IO
0.005	B to <=C	LS to <=CP
0.0075	B to <=C	LS to <=CP
0.01	B to <=C	LS to <=CP
0.0125	B to <=C	LS to <=CP
0.015	B to <=C	LS to <=CP
0.0175	D to <=E	>CP
0.02	D to <=E	>CP
0.0225	D to <=E	>CP
0.025	D to <=E	>CP
0.0275	D to <=E	>CP
0.03	D to <=E	>CP
0.0325	D to <=E	>CP
0.035	D to <=E	>CP
0.0375	D to <=E	>CP
0.04	D to <=E	>CP
0.0425	D to <=E	>CP
0.045	D to <=E	>CP
0.0475	D to <=E	>CP
0.05	D to <=E	>CP

When the rotation of the hinge is in between the LS and CP, deaths and significant damage to the structure can be expected. However, collapse of the structure is not expected in this range. Widely opened cracks, which need to be repaired before occupying the structure, can be observed in this occupancy level. In addition, when the rotation of the hinge exceeds the Collapse Prevention (CP) limit, structure has severely damaged or collapse of part of the structure or whole structure can be expected. Further, when the structure is in this level, it is not repairable.

According to the Table 4.3, it can be identified that the hinge rotation has exceeded the collapse prevention (CP) limit. Therefore the element is not repairable.

Variation of the bending moment with the time can be used to identify the bending capacity of the structural elements. In addition, behavior of an element can also be predicted with the plot of the moment vs time curve.

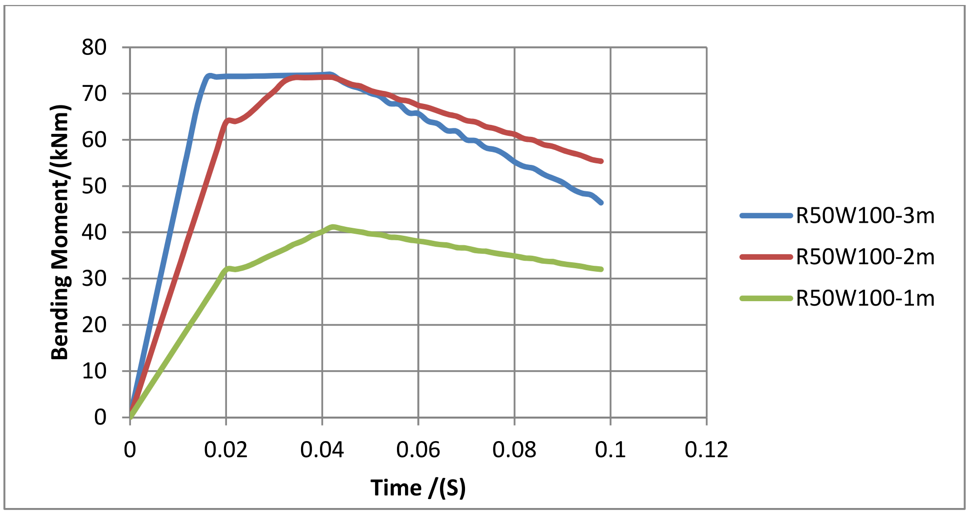

Figure 4.1: Variation of Bending Moment with Time for R50W100

Bending moment diagrams of 150x500mm concrete fin having eight numbers of 10mm tor steel bars is indicated in the Figure 4.1. It shows three bending moment diagrams at the hinge for fin spacing 1m, 2m and 3m. Fin is in the immediate occupancy level for three analysis cases. Maximum bending moment can be observed when they are placed at a spacing of 3m. However, maximum bending capacity, 80kNm, has not reached at any of the three cases.

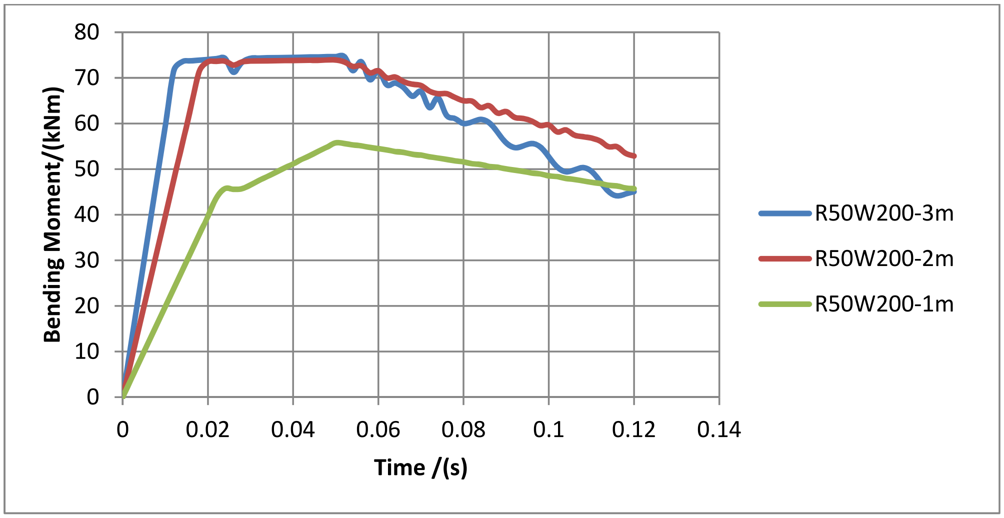

Figure 4.2: Variation of Bending Moment with Time for R50W200

Figure 4.2 indicates the variation of the bending moment for fin spacing 1m, 2m and 3m of 150x500mm having 8 numbers of 10mm tor steel bars. Fin has reached to a maximum bending moment when they are placed at a spacing of 3m and 2m. However, fin has not reached to its bending capacity for all three load cases. Further, it can be observed from the analysis results that the fin is in the state of immediate occupancy level for this load case.

Different rates can be observed when developing the bending moment due to the variation of the rate of loading. Since, the loads are applied in different time frames, time taken to reach the maximum bending moment are not similar in each case. After the fin reaching to its maximum bending moment, it gradually reduces with the time as the blast pressure loading reduces with the time.

Comparison of the analysis results for load cases; R50W100-3m, R50W100-2m, R50W100-1m, R50W200-3m, R50W200-2m and R50W200-1m is done in Figure 4.3.

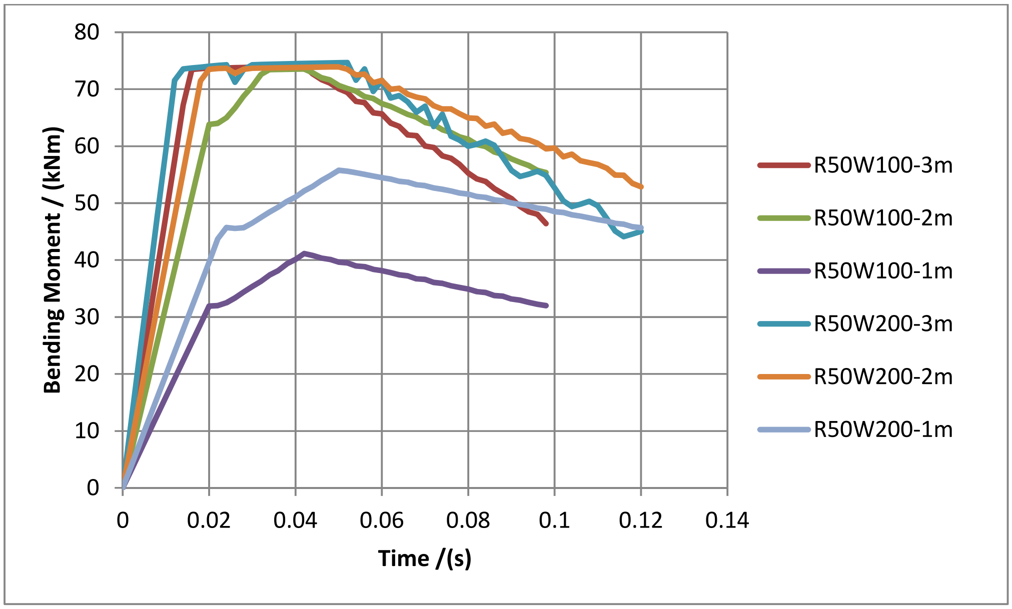

Figure 4.3: Comparisons of Analysis Results of (150x500, 8T10)

Figure 4.3 indicates the comparison of analysis results of fin 150x500 having eight numbers of tor steel bars. According to the analysis results, it can be concluded that the fin has not reached to its bending capacity. Further, due to the variation of rate of the loading, each curve does not follow a similar path and deviation has occurred as per the rate of loading.

Above discussion on analysis results is done for few load cases to elaborate the outputs. Summary of the analysis results are tabulated and discussed in Section 4.3 of which state the status of the hinges (immediate occupancy, life safety and collapse prevention) according to the failure criteria.

4.3 SUMMAY OF RESULTS

Summary of analysis results are tabulated and discussed in this section. Table 4.5 indicates the summary of analysis results for the fin of 150x500mm having eight numbers of tor steel bars. According to the spacing of the fins, state of the hinge is shown in the Table 4.5. It can be observed that most of fins are in the immediate occupancy level when blast occurs at distances of 25m and 50m. However, some of them have reached to the Life Safety limit and collapse prevention level. Further, when a blasting occurs at a distance of 10m from the fin most of the fins have passed the Collapse Prevention Limit.

Table 4.4 indicates analysis results for the fin 150x500 having eight numbers of tor steel bars. According to the tabulated results, some of the fins that are in life safety limit for 8T10 (Table 4.5) has moved to the immediate occupancy level with 8T12 while some of the fins were in collapse prevention limit have reached to life safety limit. Similarly, fins had passed the collapse prevention limit have moved to collapse prevention line. Thus, it can be concluded that there is an improvement of occupancy levels with the increase of the percentage of reinforcements.

Table 4.4 to Table 4.12 indicate the summary of analysis results for 150x500, 150x600 and 150x800 concrete fins of having 8T12, 8T10, 10T10, 10T12, 12T10, 12T12 and 16T12 reinforcement. 1m, 2m and 3m fin spacing were considered for above analysis cases. According to the summary of results, it can be concluded that increase of the reinforcement ratio and size of the fins have considerable effect on its collapse.

Table 4.4 Stage of Hinge of 150x500 8T12

Fin	Load Case	Fin Spacing	State of the Hinge
150x500 8T12	R10W10.2m	2	IO
	R10W10.3m	3	LS
	R10W25.1m	1	IO
	R10W25.2m	2	CP
	R10W25.3m	3	>CP
	R10W50.1m	1	LS
	R10W50.2m	2	>CP
	R10W50.3m	3	>CP
	R10W100.1m	1	>CP
	R10W100.2m	2	>CP
	R10W100.3m	3	>CP
	R10W200.1m	1	>CP
	R10W200.2m	2	>CP
	R10W200.3m	3	>CP
150x500 8T12	R25W100.3m	3	LS
	R25W200.2m	2	LS
	R25W200.3m	3	LS
	R25W300.2m	2	LS
	R25W300.3m	3	>CP
	R25W400.1m	1	LS
	R25W400.2m	2	CP
	R25W400.3m	3	>CP

Table 4.5 Stage of Hinge of 150x500 8T10

Fin	Load Case	Fin Spacing	State of the Hinge
150x500 8T10	R10W10.1m	1	IO
	R10W10.2m	2	LS
	R10W10.3m	3	LS
	R10W25.1m	1	LS
	R10W25.2m	2	>CP
	R10W25.3m	3	>CP
	R10W50.1m	1	CP
	R10W50.2m	2	>CP
	R10W50.3m	3	>CP
	R10W100.1m	1	>CP
	R10W100.2m	2	>CP
	R10W100.3m	3	>CP
	R10W200.1m	1	>CP
	R10W200.2m	2	>CP
	R10W200.3m	3	>CP
150x500 8T10	R25W25.1m	1	IO
	R25W25.2m	2	IO
	R25W25.3m	3	IO
	R25W50.1m	1	IO
	R25W50.2m	2	IO
	R25W50.3m	3	IO
	R25W100.1m	1	IO
	R25W100.2m	2	IO
	R25W100.3m	3	LS
	R25W200.1m	1	IO
	R25W200.2m	2	LS
	R25W200.3m	3	CP
	R25W300.1m	1	IO
	R25W300.2m	2	LS
	R25W300.3m	3	>CP
	R25W400.1m	1	LS
	R25W400.2m	2	CP
	R25W400.3m	3	>CP
150x500 8T10	R50W100.1m	1	IO
	R50W100.2m	2	IO
	R50W100.3m	3	IO
	R50W200.1m	1	IO
	R50W200.2m	2	IO
	R50W200.3m	3	IO
	R50W300.1m	1	IO
	R50W300.2m	2	IO
	R50W300.3m	3	IO
	R50W400.1m	1	IO
	R50W400.2m	2	IO
	R50W400.3m	3	IO

Table 4.6 Stage of Hinge of 150x500 10T10

Fin	Load Case	Fin Spacing	State of the Hinge
150x500 10T10	R10W10.2m	2	IO
	R10W10.3m	3	LS
	R10W25.1m	1	IO
	R10W25.2m	2	>CP
	R10W25.3m	3	>CP
	R10W50.1m	1	CP
	R10W50.2m	2	>CP
	R10W50.3m	3	>CP
	R10W100.1m	1	>CP
	R10W100.2m	2	>CP
	R10W100.3m	3	>CP
	R10W200.1m	1	>CP
	R10W200.2m	2	>CP
	R10W200.3m	3	>CP
150x500 10T10	R25W100.3m	3	LS
	R25W200.2m	2	LS
	R25W200.3m	3	CP
	R25W300.2m	2	LS
	R25W300.3m	3	>CP
	R25W400.1m	1	IO
	R25W400.2m	2	CP
	R25W400.3m	3	>CP

Table 4.7 Stage of Hinge of 150x500 10T12

Fin	Load Case	Fin Spacing	State of the Hinge
150x500 10T12	R10W10.3m	3	LS
	R10W25.2m	2	CP
	R10W25.3m	3	>CP
	R10W50.1m	1	LS
	R10W50.2m	2	>CP
	R10W50.3m	3	>CP
	R10W100.1m	1	>CP
	R10W100.2m	2	>CP
	R10W100.3m	3	>CP
	R10W200.1m	1	>CP
	R10W200.2m	2	>CP
	R10W200.3m	3	>CP
150x500 10T12	R25W100.3m	3	LS
	R25W200.2m	2	LS
	R25W200.3m	3	LS
	R25W300.2m	2	LS
	R25W300.3m	3	>CP
	R25W400.2m	2	CP
	R25W400.3m	3	>CP

Table 4.8 Stage of Hinge of 150x600 10T10

Fin	Load Case	Fin Spacing	State of the Hinge
150x600 10T10	R10W10.3m	3	IO
	R10W25.2m	2	LS
	R10W25.3m	3	CP
	R10W50.1m	1	LS
	R10W50.2m	2	>CP
	R10W50.3m	3	>CP
	R10W100.1m	1	>CP
	R10W100.2m	2	>CP
	R10W100.3m	3	>CP
	R10W200.1m	1	>CP
	R10W200.2m	2	>CP
	R10W200.3m	3	>CP
150x600 10T10	R25W100.3m	3	IO
	R25W200.2m	2	IO
	R25W200.3m	3	LS
	R25W300.2m	2	IO
	R25W300.3m	3	LS
	R25W400.2m	2	LS
	R25W400.3m	3	CP

Table 4.9 Stage of Hinge of 150x600 10T12

Fin	Load Case	Fin Spacing	State of the Hinge
150x600 10T12	R10W25.2m	2	LS
	R10W25.3m	3	LS
	R10W50.1m	1	LS
	R10W50.2m	2	>CP
	R10W50.3m	3	>CP
	R10W100.1m	1	CP
	R10W100.2m	2	>CP
	R10W100.3m	3	>CP
	R10W200.1m	1	>CP
	R10W200.2m	2	>CP
	R10W200.3m	3	>CP
150x600 10T12	R25W200.3m	3	LS
	R25W300.3m	3	LS
	R25W400.2m	2	LS
	R25W400.3m	3	LS

Table 4.10 Stage of Hinge of 150x600 12T10

Fin	Load Case	Fin Spacing	State of the Hinge
150x600 12T10	R10W25.2m	2	LS
	R10W25.3m	3	CP
	R10W50.1m	1	LS
	R10W50.2m	2	>CP
	R10W50.3m	3	>CP
	R10W100.1m	1	CP
	R10W100.2m	2	>CP
	R10W100.3m	3	>CP
	R10W200.1m	1	>CP
	R10W200.2m	2	>CP
	R10W200.3m	3	>CP
150x600 12T10	R25W200.3m	3	LS
	R25W300.3m	3	LS
	R25W400.2m	2	LS
	R25W400.3m	3	CP

Table 4.11 Stage of Hinge of 150x600 12T12

Fin	Load Case	Fin Spacing	State of the Hinge
150x600 12T12	R10W25.2m	2	LS
	R10W25.3m	3	LS
	R10W50.2m	2	CP
	R10W50.3m	3	>CP
	R10W100.1m	1	CP
	R10W100.2m	2	>CP
	R10W100.3m	3	>CP
	R10W200.1m	1	>CP
	R10W200.2m	2	>CP
	R10W200.3m	3	>CP
150x600 12T12	R25W200.3m	3	IO
	R25W300.3m	3	LS
	R25W400.2m	2	LS
	R25W400.3m	3	LS

Table 4.12 Stage of Hinge of 150x800 16T12

Fin	Load Case	Fin Spacing	State of the Hinge
150x800 16T12	R10W25.2m	2	IO
	R10W25.3m	3	IO
	R10W50.2m	2	IO
	R10W50.3m	3	LS
	R10W100.1m	1	IO
	R10W100.2m	2	LS
	R10W100.3m	3	>CP
	R10W200.1m	1	LS
	R10W200.2m	2	>CP
	R10W200.3m	3	>CP
150x800 16T12	R25W300.3m	3	IO
	R25W400.2m	2	IO
	R25W400.3m	3	IO

Summary of all the analysis results that are tabulated in Table 4.4 to Table 4.12 can be represented graphically as indicated in Figure 4.4, Figure 4.5 and Table 4.13. When it is required to select an element based on the occupancy level, it can be done with Figure 4.4, Figure 4.5 and Table 13. After identify the occupancy level to be reached by the element in an event of blasting, arrangement of the reinforcement and size of fin can be selected depending on the blast pressure. Blast pressure is represented with a load cases similar to R10W100.2. This notation represents the distance between the structure and blasting location as 10m, weight of TNT equivalent value of blasting material as 100kg and spacing of the fins as 2m. More importantly, simplified charts indicated in Figure 4.4, Figure 4.5 and Table 13 can be understood even by a person who does not have the knowledge in designs. He can get an idea about the element categories that fall into each load case.

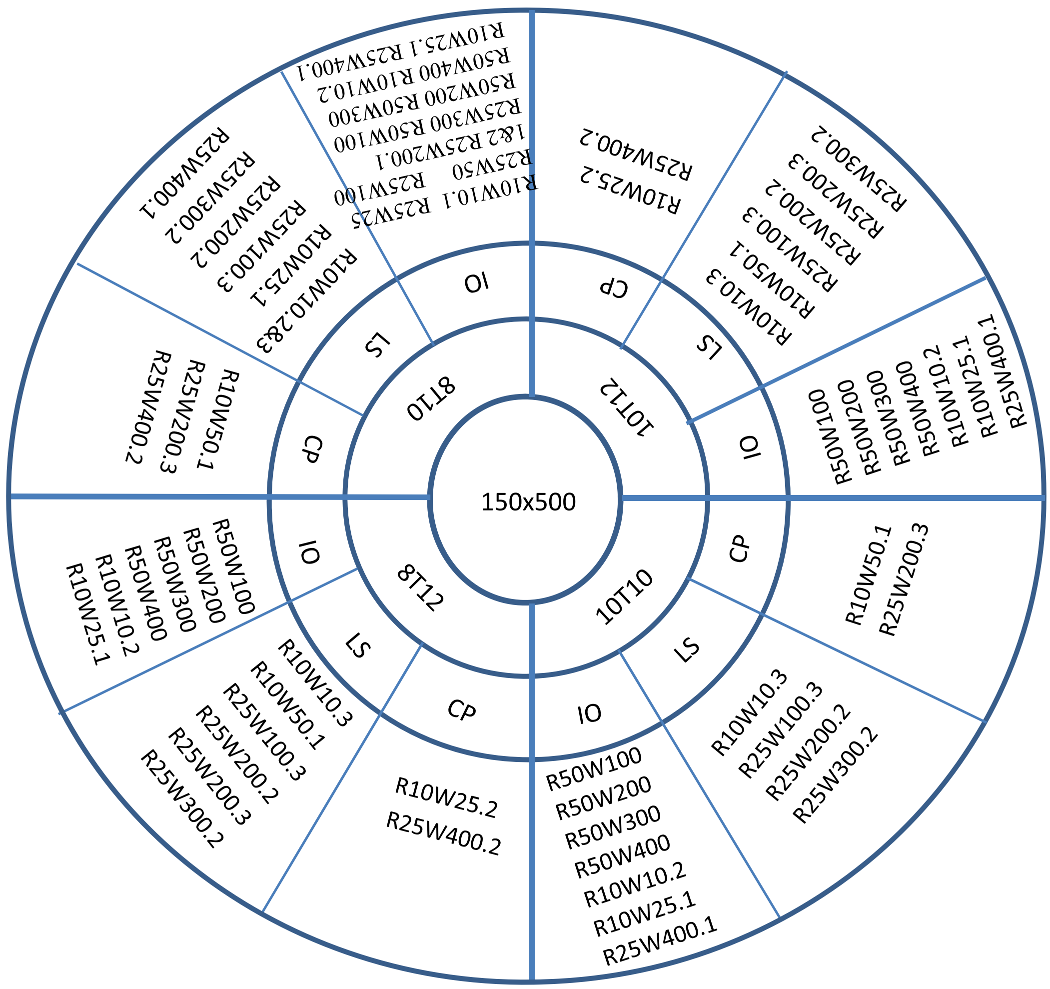

Figure 4.4: Graphical Representations of Analysis Results of 150x500 Fin

In addition, cost factor was also considered when selecting the size of fins and percentage of reinforcements. Effect of the construction cost on the design of element against blast loading is briefly discussed in Section 4.4

Figure 4.5 indicates the summary of analysis results of 150x600mm size fin for different combination of loadings. Since all the fins were in immediate occupancy level when a blast occurs 50m away from the structure, that load case is not considered for the analysis. According to the analysis results, considerable amount of cases have exceeded the collapse prevention limit when the blast occurs at a distance of 10m from the structure. Therefore increase of the percentage of reinforcement is done to study the behaviour of the fins. There was an improvement of the occupancy level with the increase of the reinforcement; however, it was not that significant. In this background, fin size was also increased in addition to increase of reinforcement ratio.

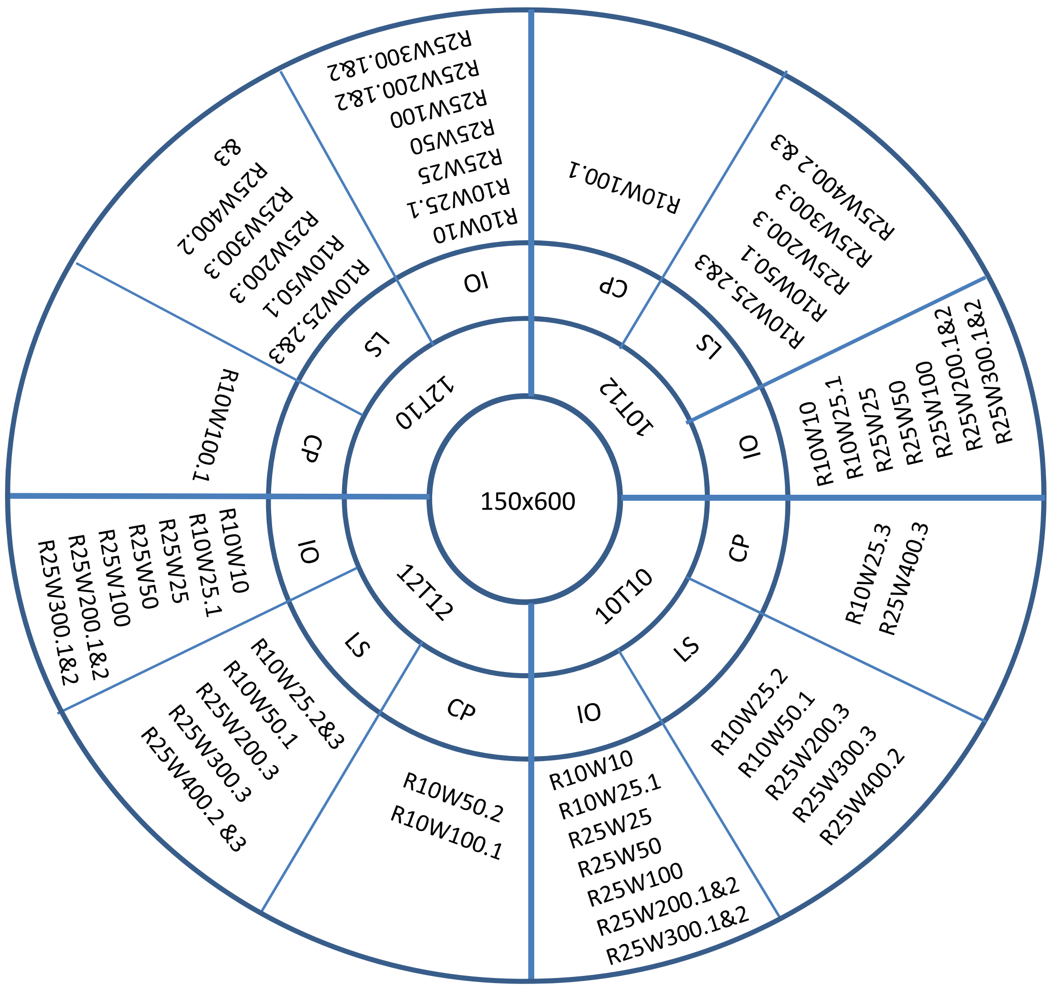

Figure 4.5: Graphical Representations of Analysis Results of 150x600 Fin

There was a significant improvement in the occupancy level with the increase of amount of reinforcement and fin size as indicated in the Table 4.13. All the load cases were in the life Safety Limit (LS) shifted to Immediate Occupancy Level. In addition, some of the load cases had exceeded in the Collapse prevention limit moved into the Life Safety limit with fin size 150x800mm having 16 numbers of 12mm tor steel bars.

Table 4.13: Analysis Results of 150x800 Fin

150x800	16T12	IO	R10W25.2, R10W25.3, R10W50.2, R10W100.1, R25W300.3, R25W400.2, R25W400.3
		LS	R10W50.3, R10W100.2, R10W200.1
		CP	-

4.4 COMPARISON OF COST FOR DIFFERENT OPTIONS

There is an improvement of the occupancy level with increase of section dimensions and the percentage of reinforcements. At the same time, the cost of the construction also will increase. Thus, detail analysis on the proportion of increase of the cost is suitable before finalizing an element size. Cost factor can be used as an indicator when an economical design is done.

Table 4.14 Comparison of Construction Cost of Fins

Element Type		(Cost /m)/Rs	Increase of cost (%) with respect to 150x500 - 8T10
150x500	8T10	3970.60	0
	10T10	4217.00	6
	8T12	4405.80	11
	10T12	4761.00	20
150x600	10T10	4502.00	13
	12T10	4748.40	20
	10T12	5046.00	27
	12T12	5401.20	36
150x800	16T12	7401.60	86

Increase of element sizes will increase the area of the formwork and volume of the concrete. In addition, increase of reinforcement will also increase the cost of construction. With reference to these three factors, basic cost of construction for one metre height of concrete fin is calculated for comparison purpose. Results obtained from the detailed calculation are tabulated in Table 4.14.

According to the calculated results, it can be observed that the increase of the cost of construction for 150x600mm fins of having reinforcements 12T12 is 36% when it is compared with 150x500mm having eight numbers of 10mm bars. When the fin size is increased to 150x800mm with reinforcement of 16T12, percentage increase of the cost of construction is 86%. As per the above results, it can be concluded that there is a significant increase of cost with the increase of the fin sizes and the area of reinforcement. Further, it is evident from the results that enhancing the blast resisting capacity will increase the cost of construction which is more common in these types of structures.

4.5 SUMMARY

Behaviour of concrete fins of having dimensions 150x500mm, 150x600mm and 150x800mm are studied by varying the area of reinforcement (8T10, 8T12, 10T10, 10T12, 12T10, 12T12 and 16T12). Analysis results for different blast pressure loading created by varying the standoff distance, weight of blasting materials and fin spacing are discussed in this chapter.

All the analysis cases for standoff distance of 50m were in the immediate occupancy level, and the load cases with standoff distance of 25m were in a manageable limit with respect to the occupancy level and cost of construction. However, when the standoff distance is 10m, more load cases exceeded the collapse limit. With the increase of the section dimension to 150x800mm, there was an improvement of the occupancy level even though the cost of construction increased by 86% with respect to fin 150x500mm having 8T10.

5 DISCUSSIONS

One of the most commonly identified issue in designing blast resisting structures is the progressive collapse. If the vertical elements that carry the weight of the structures and horizontal loads are damaged, it could lead to a progressive collapse of the structure. Therefore, it is prime important of having due care on this matter. Preventive measures are to be taken to avoid the damaging of the critical elements.

Different techniques are being used worldwide to enhance the blast resisting capacity of structures while paying due care on the above subject. Most commonly used technique is having façade systems in addition to access control by landscaping or having obstructions or any other means or having all the measures. Concrete façade systems, glass facades supported with glass frames or cable nets, masonry facade systems are the most commonly used methods in the world to protect structures against blast loading. A use of membranes for enhancing the strength is required for masonry and glass facades as they are very weak in flexure.

Analysis of concrete fins that carry the loads transferred from the facade was done in this study. Concrete fins are used as structural elements to carry the blast pressure to the main structure without damaging the structural elements that carry the vertical loads. Concrete elements have higher flexural capacity when compared with the other methods. Further, concrete is readily available material in Sri Lanka and would be more economical when compared with the glass façade systems for larger spans.

Studying the behaviour of the concrete fins with respect to the failure criteria defined in Chapter 3 was done in this study. Occupancy levels such as immediate occupancy, life safety limit and collapse prevention limit were considered in categorizing the elements according to their capacity against blast loading. 150x500mm, 150x600mm and 150x800mm size fins were analysed with different percentages of reinforcement ratios. Summary of all the analysis results were graphically represented in Figure 4.4, Figure 4.5 and Table 4.13. All the load cases considered as the blasting occurs at a distance of 50m (R = 50m) from the structure, were in the immediate occupancy level as indicated in the Figure 4.4. However, with the standoff distances 10m and 25m fins behave in life safety limit, collapse prevention limit while some of the load cases have passed the collapse prevention limit. Around 50% of the load cases, where the standoff distance considered as 10m with fin size 150x600 had passed the collapse prevention limit creating a situation where it required to further increase the stiffness of fins by increasing the percentage of reinforcement or size of the fins. When fins of 150x800mm having 16T12 were analyzed, all the load cases were in the Life Safety Limit moved into Immediate Occupancy Level while some of the load cases had passed the Collapse Prevention Limit moved into the Life Safety Limit. Significant improvement of the occupancy level is observed for fin 150x800 (16T12). However, the cost of the construction has increased by 86% compared with the fin 150x500 (8T10).

The load cases considered with standoff distance 25m were identified to be in the manageable zone with a blast. Load cases with smallest section dimensions (150x500mm) and reinforcement (8T10) had reached to all the occupancy levels, while more than 50% of the cases considered in this study were in the immediate occupancy level. With the increase of the reinforcements and the section dimensions, maximum limit reached by load cases were life safety limit as shown in Table 4.11 demonstrating the idea that if a blast occurs at standoff distance of 25m, it can be managed with the maximum section dimensions (150x600mm) and reinforcements (12T12). Thus, distance of 25m can be considered as a case where fins are not collapsing and lives are safe.

Cost of a construction is one of the most important aspect to be considered in these type of construction. Mostly, the cost would be higher due to the consideration of very high lateral loads compared with other lateral loads such as wind or earthquake loads. According to the comparison of cost done in this study, cost of construction of a fin have the dimensions 150x600mm with 12T12 reinforcements is 36% higher than that of a fin 150x500mm (8T10). Fin of 150x800mm having 16T12 has 86% increment of cost compared to 150x500mm (8T10).

6 CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORKS

6.1 CONCLUSIONS

Analysis of concrete fins having dimensions 150x500, 150x600 and 150x800 for different reinforcement arrangement (8T10, 8T12, 10T10, 10T12, 12T10, 12T12 and 16T12) were done considering the effects of the blast pressure when it occurs 10m, 25m and 50m away from the structure.

Following conclusions can be made from this study:

- Further increase of section dimensions or the percentage of reinforcement is required to let the structure behave at least in the collapse prevention limit when the blasting occurs at a 10m standoff distance.
- When the distance between the structure and blasting location is 25m and with the section dimensions and reinforcement rations considered in this study, occupancy level did not exceed the life safety limit indicating that the minimum standoff distance which can be managed without collapsing the structure and loss of a life.
- When a blast occurs 50m away from the structure, none of the load cases did exceed the immediate occupancy level.
- There is a significant increase of cost when increasing the fin size and area of reinforcement.

6.2 RECOMMENDATIONS FOR FUTURE WORKS

Three different fin sizes (150x500, 150x600 and 150x800) were considered in this study with different reinforcement arrangements such as 8T10, 8T12, 10T10, 10T12, 12T10, 12T12 and 16T12.

Following recommendations are proposed as future works to as an extension to this study.

- Consider different reinforcement percentages and section dimensions to study the behaviour of concrete fins
- Analysis of concrete façade together with concrete fins by using suitable software will be more appropriate to incorporate the three dimensional effects.
- Consideration of cost of construction when selecting the element categories would end up with an economical design.

REFERENCES

Brode H. L. (1955). *Numerical solution of spherical blast waves*, Journal of Applied Physics, American Institute of Physics, Vol. 26, Issue 6, pp. 766-775

Bajić Z, Dimitrijević R and Sirovatka R.(2007) *The dependence of shockwave parameters on scaled distance for plastic explosive PPE-01*. Proc. of 2nd symposium TEH, Belgrade, Serbia

Chandra N.,Ganpule S., Kleinschmit N. N., Feng R., Holmberg A. D., Sundaramurthy A., Selvan R. and Alai (2012). *Evolution of Blast Wave Profiles in Simulated Air Blasts: Experiment and Computational Modelling*. Shock Waves 22:403-415

Cranz C (1926), *Textbook of ballistics: Internal Ballistics*, Springer, Berlin, pp. x + 454. 37

Dewey M. *The Shape of the Blast Wave: Studies of the Friedlander Equation*. Retrieved from http://www.blastanalysis.com

Draganic H. and Sigmund V. (2012). *Blast Loading on Structures*. Technical Journal 19, 3, pp. 643-652

Federal Emergency Management Agency (FEMA 356).(2000). *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*. Prepaired by American Society of Civil Engineering for the Federal Emergency Management Agency, Washington, D.C.

Elvira, Mendis P., Lam N. and Ngo T. (2012). *Progressive Collapse Analysis of RC Frame Subjected to Blast Loading*. Australian Journal of Structural Engineering, Vol 7, Issue 01, pp. 47-55

Held M. (1983). Blast waves in free air. *Propellant. Explosieve, Pyrotechnics*, 8(1), pp.1-8.

Henrych J. (1979). *The dynamics of explosion and its use*. Development in civil engineering, Elsevier Science Publishers, Amsterdam, The Netherlands, pp. 558

Hopkinson, B. (1915), *British Ordnance Board Minutes*, 13565, 1915

Indian Standard.(2013). IS 4991: *Criteria for Blast Resistant Design of Structures for Explosions above Ground*. New Delhi, Bureau of Indian Standards

Kent D.C. and Park R. (1971). *Flexural Members with Confined Concrete*, Journal of the Structural Division, ASCE, Vol. 97, ST7, pp. 51-93

Kinney G. F., Graham K.J. (1985). *Explosive Shocks in Air*, 2nd ed., Springer, Berlin, pp. 269

Lam N., Mendis P. and Ngo T. (2004). *Response Spectrum Solution for Blast Loading*. Electronic Journal of Structural Engineering, 4(2004) pp. 28-44

Lumantarna R., Ngo T. and Mendis P. (2012). *Limitations in Simplified Approach in Assessing Performance of Façade under Blast Pressure*. Civil Engineering Dimension, Vol. 14, No. 3, (Special Edition), pp.147-155

Mander J. B., Priestley M. J. N. and Park R. (1988). *Theoretical Stress-Strain Model for Confined Concrete.* Journal of Structural Engineering, Vol. 114, No. 8, pp 1804-1826

Manmohan D.G, Vasant A. M., Anil K. G. and Steffen M. (2012) *An Abridged Review of Blast Wave Parameter.* Defence Science Journal, Vol. 62, No. 5, pp. 300-306

Mays G. C. and Smith P. D. (1995). *Blast Effects on Buildings*. London, Thomas Telford Publications

Mohammed S. and Al- Ansari. (2012). *Building Response to Blast and Earthquake Loading*. International Journal of Civil Engineering and Technology, Volume 3, Issue 2, pp. 327-346

Newmark N.M., Hansen R.J. (1961). *Design of blast resistant structures*, Shock and Vibration Handbook, Vol.3, New York, McGraw-Hill

Ngo T., Mendis P., Guptha A and Ramsay J. (2007). *Blast Loading and Blast Effects on Structures-An Overview*. EJSE Special Issue: Loading on Structures, pp 76-91

Ngo T. and Mendis P. (2008). Modeling *Reinforced Concrete Structure Subjected to Impulsive Loading Using Concrete Lattice Model*. Electronic Journal of Structural Engineering, (2008) 8, pp. 80-89

Nguyen T. P. and Tran M. T. (2011). *Response of Vertical Wall Structure under Blast Loading by Dynamic Analysis*. The Twelfth East Asia-Pacific Conference on Structural Engineering and Construction, Procardia Engineering, 14 (2011), pp. 3308-3316

Mills C. A. (1987). *The design of concrete structures to resist explosions and weapon effects*. Proceedings of the 1st Int. Conference on concrete for hazard protections, Edinburgh, UK, pp. 61-73

Oswald, C. J., and Bazan, M. (2014) *Performance and Blast Design for Non-Load Bearing Precast Concrete Panels*. SEI Structures Congress, Boston, MA

Remennikov A.(2003). *A Review of Method for Predicting Bomb Blast Eeffects on Buildings*. Journal of Battlefield Technology, 6(3) pp. 5-10

Rueda J. E. M. and Elnashai A. S. (1997). *Confined Concrete Model under Cyclic Load*. Materials and Structures, Vol. 30(197), pp. 139-147

Sadovskiy M.A. (2004). *Mechanical effects of air shockwaves from explosions according to experiments*. Symposium No 4, Nauka Press, Moscow

Tiwari A.K., Tiwari A. K. and Dhiman A. (2016). *Analysis of Concrete Wall under Blast Loading*. International Conference on Advances in Emerging Technology (ICAET 2016) Vol. 07, pp. 12-23

Unified Facilities Criteria. (2008). *UFC 03-340-02 Structures to Resist the Effect of Accidental Explosions*. United States of America, Department of Defense

Yandzio E. and Gough M. (1999). *Protection of Buildings against Explosions*, Berkshire, Steel Construction Institute Publication 244

Appendix A

Design Calculations

Reference	Calculation	Output
	Modulus of Elasticity at 28 of Normal Weight Concrete	
BS 8110 Part 2 Cl 7.2	$E_{28} = K_0 + 0.2\ f_{cu,28}$	
	$K_0 = 20\ N/mm^2$	
	$f_{cu,28} = 30\ N/mm^2$	
	$E_{28} = 20 + 0.2x30$	
	$= 26\ N/mm^2$	$E_{28} =$ $26\ N/mm^2$

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Reference	Calculation	Output

Material Strength Enhancements

Enhancements are calculated based on the equations given in the book Blast Effects on Buildings.

Blast Effects on Buildings

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	—

* Minimum specified f_y for grade 50 steel or less may be enhanced by the average strength increase factor of 1·10.

Concrete Strength Enhancement

For Bending

f_{du}/f_{cu} = 1.25

f_{cu} = 30 N/mm^2

f_{du} = 30x1.25

= 37.5 N/mm^2

Output: f_{du} = 37.5 N/mm^2

Steel Strength Enhancement

f_{dy}/f_y = 1.2

f_y = 500 N/mm^2

f_{dy} = 1.2x500

= 550 N/mm^2

Output: f_{dy} = 550 N/mm^2

f_{du}/f_u = 1.05

f_u = 600 N/mm^2

f_{du} = 1.05x600

= 630 N/mm^2

Output: f_{du} = 630 N/mm^2

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Reference	Calculation	Output

Calculation of Basic Cost of Structural Elements

Unit Cost of Construction Work

Concreting works = Rs 19000.00 per m^3

Formwork = Rs 1200.00 per m^2

Reinforcement work = Rs 200.00 per kg

Calculation of Cost of 150x500 fin having 8T10

Calculation of cost for a one meter length is done in this analysis

Volume of the concrete = 0.15x0.5

= 0.075 m^3

Cost for concreting = 0.075x19000

= Rs 1425.00

Area of the formwork = 0.15x1x2 + 0.5x1x2

= 1.3 m^2

Cost for formwork = 1.3x1200

= Rs 1560.00

Weight of the reinforcement = 0.617x8x1

= 4.936 kg

Cost for Reinforcements = 200x4.936

= Rs 985.60

Total Cost for 150x500-8T10 = 1425 + 1560 + 985.50

= **Rs 3970.60**

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Reference	Calculation			Output
	Calculation of Cost of 150x500 fin having 8T12			
	Calculation of cost for a one meter length is done in this analysis			
	Volume of the concrete	=	0.15x0.5	
		=	0.075 m^3	
	Cost for concreting	=	0.075x19000	
		=	Rs 1425.00	
	Area of the formwork	=	0.15x1x2 + 0.5x1x2	
		=	1.3 m^2	
	Cost for formwork	=	1.3x1200	
		=	Rs 1560.00	
	Weight of the reinforcement	=	0.888x8x1	
		=	7.104 kg	
	Cost for Reinforcements	=	Rs 1420.80	
	Total Cost for 150x500-8T10	=	1425 + 1560 + 1420.8	
		=	**Rs 4405.80**	
	Calculation of Cost of 150x500 fin having 10T10			
	Calculation of cost for a one meter length is done in this analysis			
	Volume of the concrete	=	0.15x0.5	
		=	0.075 m^3	
	Cost for concreting	=	0.075x19000	
		=	Rs 1425.00	
	Area of the formwork	=	0.15x1x2 + 0.5x1x2	
		=	1.3 m^2	
	Cost for formwork	=	1.3x1200	
		=	Rs 1560.00	

Reference	Calculation			Output
	Weight of the reinforcement	=	0.0.617x10x1	
		=	6.16 kg	
	Cost for Reinforcements	=	Rs 1232.00	
	Total Cost for 150x500-8T10	=	1425 + 1560 + 1232	
		=	**Rs 4217.00**	
	Calculation of Cost of 150x500 fin having 10T12			
	Calculation of cost for a one meter length is done in this analysis			
	Volume of the concrete	=	0.15x0.5	
		=	0.075 m^3	
	Cost for concreting	=	0.075x19000	
		=	Rs 1425.00	
	Area of the formwork	=	0.15x1x2 + 0.5x1x2	
		=	1.3 m^2	
	Cost for formwork	=	1.3x1200	
		=	Rs 1560.00	
	Weight of the reinforcement	=	0.888x10x1	
		=	8.88 kg	
	Cost for Reinforcements	=	Rs 1776.00	
	Total Cost for 150x500-8T10	=	1425 + 1560 + 1420.8	
		=	**Rs 4761.00**	
	Calculation of Cost of 150x600 fin having 10T10			
	Calculation of cost for a one meter length is done in this analysis			
	Volume of the concrete	=	0.15x0.6	
		=	0.09 m^3	
	Cost for concreting	=	0.09x19000	
		=	Rs 1710.00	

Reference	Calculation			Output
	Area of the formwork	=	0.15x1x2 + 0.6x1x2	
		=	1.5 m^2	
	Cost for formwork	=	1.5x1200	
		=	Rs 1800.00	
	Weight of the reinforcement	=	0.617x10x1	
		=	6.16 kg	
	Cost for Reinforcements	=	Rs 1232.00	
	Total Cost for 150x500-8T10	=	1710 + 1800 + 1232	
		=	**Rs 4742.00**	
	Calculation of Cost of 150x600 fin having 10T12			
	Cost for concreting	=	Rs 1710.00	
	Cost for formwork	=	Rs 1800.00	
	Weight of the reinforcement	=	0.888x10x1	
		=	8.88 kg	
	Cost for Reinforcements	=	8.88x200	
		=	Rs 1776.00	
	Total Cost for 150x500-8T10	=	1710 + 1800 + 1776	
		=	**Rs 5286.00**	
	Calculation of Cost of 150x600 fin having 12T10			
	Cost for concreting	=	Rs 1710.00	
	Cost for formwork	=	Rs 1800.00	
	Weight of the reinforcement	=	0.617x12x1	
		=	7.404 kg	
	Cost for Reinforcements	=	7.404x200	
		=	Rs 1478.40	

Reference	Calculation	Output

Total Cost for 150x500-8T10 = 1710 + 1800 + 1478.40

= **Rs 4988.40**

Calculation of Cost of 150x600 fin having 12T12

Cost for concreting = Rs 1710.00

Cost for formwork = Rs 1800.00

Weight of the reinforcement = 0.888x12x1

= 10.656 kg

Cost for Reinforcements = 10.656x200

= Rs 2131.20

Total Cost for 150x500-8T10 = 1710 + 1800 + 2131.20

= **Rs 5641.20**

Calculation of Cost of 150x800 fin having 16T12

Calculation of cost for a one meter length is done in this analysis

Volume of the concrete = 0.15x0.8

= 0.12 m^3

Cost for concreting = 0.12x19000

= Rs 2280.00

Area of the formwork = 0.15x1x2 + 0.8x1x2

= 1.9 m^2

Cost for formwork = 1.9x1200

= Rs 2280.00

Weight of the reinforcement = 0.888x16x1

= 14.208 kg

Cost for Reinforcements = 14.208x200

= Rs 2841.6

Reference	Calculation			Output
	Total Cost for 150x500-8T10	=	2280 + 2280+ 2841.6	
		=	**Rs 7401.60**	

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