EFFECTS OF AN EXTERNAL EXPLOSION ON A CONCRETE STRUCTURE

Submitted by
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Supervised by
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University of Engineering and Technology,
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<tr>
<td>$Ca$</td>
<td>Sound speed in the air</td>
</tr>
<tr>
<td>$C_f$</td>
<td>The drag coefficient for the front face</td>
</tr>
<tr>
<td>$C_r$</td>
<td>The drag coefficient for the rear face</td>
</tr>
<tr>
<td>$C_t$</td>
<td>Reflective pressure coefficient</td>
</tr>
<tr>
<td>$C_t$</td>
<td>The drag coefficient for the roof</td>
</tr>
<tr>
<td>CFD</td>
<td>Computational Fluid Dynamics</td>
</tr>
<tr>
<td>DLF</td>
<td>Dynamic Load Factor</td>
</tr>
<tr>
<td>DIF</td>
<td>Dynamic Increase Factor</td>
</tr>
<tr>
<td>$H_{TNT}$</td>
<td>Heat of the detonation of TNT</td>
</tr>
<tr>
<td>$\text{H}_{\text{exp}}$</td>
<td>Heat of detonation of explosive</td>
</tr>
<tr>
<td>HDAS system</td>
<td>Shock-hardened data acquisition system</td>
</tr>
<tr>
<td>$i_s$</td>
<td>Positive specific impulse</td>
</tr>
<tr>
<td>I</td>
<td>Impulse</td>
</tr>
<tr>
<td>$I_0$</td>
<td>the positive phase impulse</td>
</tr>
<tr>
<td>$I_r$</td>
<td>the reflected impulse</td>
</tr>
<tr>
<td>K</td>
<td>Stiffness constant</td>
</tr>
<tr>
<td>L</td>
<td>Length of the structure</td>
</tr>
<tr>
<td>$L_w$</td>
<td>Blast wave length</td>
</tr>
<tr>
<td>Ms</td>
<td>The Mach number</td>
</tr>
<tr>
<td>M</td>
<td>The structural mass</td>
</tr>
<tr>
<td>PPA</td>
<td>Peak particle acceleration</td>
</tr>
<tr>
<td>$P_o$</td>
<td>Ambient pressure</td>
</tr>
<tr>
<td>$P_i$</td>
<td>Peak incident pressure</td>
</tr>
<tr>
<td>P-I</td>
<td>Pressure Impulse diagram</td>
</tr>
<tr>
<td>$P_{so}$</td>
<td>Peak side on pressure</td>
</tr>
<tr>
<td>$P_s$</td>
<td>Peak overpressure</td>
</tr>
<tr>
<td>Q</td>
<td>Charge weight</td>
</tr>
<tr>
<td>R.C.</td>
<td>Reinforced Concrete</td>
</tr>
<tr>
<td>$R/Q^{1/3}$</td>
<td>Scaled range</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$R$</td>
<td>Charge distance</td>
</tr>
<tr>
<td>$R_{rr}$</td>
<td>Pipe ruptures reaction</td>
</tr>
<tr>
<td>$R_{sj}$</td>
<td>Jet impingement loading</td>
</tr>
<tr>
<td>SDOF</td>
<td>Single Degree of Freedom</td>
</tr>
<tr>
<td>SSC</td>
<td>Structures, Systems and Components</td>
</tr>
<tr>
<td>$t_a$</td>
<td>Arrival time of ground shock wave</td>
</tr>
<tr>
<td>$t_d$</td>
<td>Ground shock wave duration</td>
</tr>
<tr>
<td>$T_{lag}$</td>
<td>Time lag between ground shock and airblast pressure</td>
</tr>
<tr>
<td>$T_a$</td>
<td>Shock front arrival time</td>
</tr>
<tr>
<td>$T_r$</td>
<td>The rising time from arrival time to peak value</td>
</tr>
<tr>
<td>$T_d$</td>
<td>The decreasing time from peak to the ambient pressure</td>
</tr>
<tr>
<td>$t_s$</td>
<td>Time taken by the blast wave to reach ambient pressure</td>
</tr>
<tr>
<td>$t_{\alpha}$</td>
<td>The theoretical time due to a dimensionless parameter $\alpha$</td>
</tr>
<tr>
<td>$t_o$</td>
<td>the positive phase duration</td>
</tr>
<tr>
<td>$T$</td>
<td>Positive phase duration</td>
</tr>
<tr>
<td>TNT</td>
<td>Trinitrotoluene</td>
</tr>
<tr>
<td>U</td>
<td>Shock Front Velocity</td>
</tr>
<tr>
<td>$W_{TNT}$</td>
<td>Equivalent TNT charge weight</td>
</tr>
<tr>
<td>$W$</td>
<td>The weapon yield</td>
</tr>
<tr>
<td>$y_m$</td>
<td>Maximum dynamic deflection</td>
</tr>
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</table>
ABSTRACT

Analysis for the structural behavior of reactor containments under Impact/Explosive loadings is an emerging field of research. The containment is the most important structure in a nuclear power plant. It is classified as a Seismic Category 1 Structure. Its protection against external aggression such as explosion, aircraft, missiles and fires is essential to keep the masses safe from the hazards of radiation. The present study has, therefore, been directed to study the effect of external explosion on a typical reinforced concrete containment structure.

The general practice is to utilize the air blast pressure values in the structural analysis and design against external explosion. The ground shock parameters are usually neglected during blast resistant analysis and design.

Many empirical relations have been proposed in the past to calculate the airblast pressure. Most of them, however, only predict peak pressure values.

In this thesis, not only the airblast parameters have been studied but also the ground shock parameters have been dealt with. Therefore, the thesis deals with the experimental determination of relationships of following airblast and ground shock parameters against scaled distance on a reactor containment scaled model.

**Airblast Time History Parameters**
(a) Peak pressure ($P_{so}$)
(b) Shock wave front arrival time ($T_a$)
(c) Rising time ($T_r$)
(d) Decreasing time ($T_d$)
(e) Duration of the positive pressure phase ($T$)

**Ground Shock Time History Parameters**
(f) Peak Particle Acceleration (PPA)
(g) Arrival Time ($t_a$)
(h) Shock Wave Duration ($t_d$)
(i) Time lag between ground shock and air blast pressure arrival at structures ($T_{lag}$)
The results have been compared with that of previous researchers and CONWEP. The variation of results is due to curved surface of containment model.

In the second part of the study, full scale typical reactor containment has been modeled against external blast loads varying from 30 t to 160 t of Trinitrotoluene (TNT) at a detonation distance of 50-200 m using the above mentioned relationships. It is concluded that all the failure points lie either within the lowest 10m region or at top of the shell. It is observed that an increase of 5-20 MPa occurs with the simultaneous application of air blast and shock wave on reinforced concrete containment as compared to that of airblast only. It shows that an accurate analysis of structural response and damage of structures to a nearby external explosion requires application of ground shock and air blast pressure time history parameters at the same time.

A comparative study has also been carried out to calculate the critical distance for the various external blast charges. The distances at which 90% of the shell elements have failed may be termed as critical distances. In the present study, the critical distances vary from 110 to 200 m for above blast charges. The 70% of the shell elements are cracked on both faces and may be described as doubly cracked gauss points. These occur at the locations which have been crushed in the plastic range.

The research work and the conclusions drawn may be utilized for evaluation of the effect of an external explosion on the reinforced concrete containments of other reactors.
Chapter 1

Introduction

1.1 General

The role of power generation through Nuclear Reactors is rapidly increasing in the world. For nuclear power plant analysis and design, it is desirable to carry out research work in the area of structural safety of reactor containments against external explosions. Nuclear containment structure in many countries is a pre-stressed concrete containment shell. Research papers are reported in literature on the impact of an aircraft on the concrete nuclear containment shells, but the studies are rare on the effect of explosions on the containment shells. The present study has therefore been directed to study the effect of external explosions on a typical reinforced concrete containment structure. In this regards, calculation of response of concrete structures for blast loading requires three-dimensional structural idealization, true modeling of the material non-linearities and precise modeling of the blast phenomenon. Computer simulation of the blast loading parameters for a specified charge weight for a cylindrical structure like nuclear containment shells is required in precise determination of the response. The determination of parameters for modeling the blast shock interaction is required in the form of equations. The response of the reinforced concrete shell is studied subjected to different amounts of blast charges at varying distances. The structural response encompasses the extent of cracking in the concrete, stress in steel and concrete after failure and deflections.

1.2 Classification of Blast Loads on Structures

1.2.1 Classification on the basis of confinement

Blast loads on structures can be classified into two following main groups on the basis of the confinement of the explosive charge TM 5-1300(1990).

(a) Unconfined explosion, which include free air burst, air burst and surface burst explosion having un reflected and reflected pressure loads respectively.

(b) Confined explosions, the confined explosions include fully vented explosions, partially confined explosions, fully confined explosions.

Unconfined Explosion
Free Air Burst Explosion

The blasts taking place in free air generates an initial yield. The shock wave progresses away from the center of the ignition, interacting the protective structure without intermediate magnification of blast wave.

Air Burst Explosion

The explosion generated at a distance from and above the protective structure so that the ground reflections of the initial wave happen before the advent of the shock wave at the protective structure.

Surface Burst explosion

A surface burst explosion results during the ignition positioned adjacent the ground so that the primary shock is intensified at the point of detonation owing to the ground reflections.

Confined Explosions

Fully Vented Explosion

A fully vented explosion will be generated within or immediately adjacent to a barrier or cubicle type structure with one or more surfaces open to the environment.

Partially Confined Explosion

A partially confined explosion will be generated within a barrier or cubicle type structure with partial openings.

Fully Confined Explosion

Full confinement of an explosion is related with either total or almost total confinement of the explosion by a barrier.

These blast loading categories are shown in Figure 1.1 which provides the six blast loading categories possible. Figure 1.1 also illustrates the five possible pressure loads associated with the blast load categories, the location of the explosive charge which would generate these pressure loads, and the protective structures subjected to these loads.
1.2.2 Classification on the basis of ratio of the blast wave duration to time period of the structure

The ratio of the duration to the time period of the structure provides the response of a structure to a dynamic force. The classification of three load regimes on the basis of this ratio are

(i) Impulsive
(ii) Dynamic
(iii) Quasi-Static
If the period of vibration is long, then the load may be impulsive for the structural member. The intermediate region between impulsive and quasi-static, where the periods are almost equal the loading duration, is termed dynamic. If the time period of the structural member is very short as compared to the load duration, the load may be classified as quasi-static. The factors which define the shape of the pressure time history, such as the rise time, load duration, impulse and peak pressure, also require detailed application in finding out the peak structural response. In effect, if the loading is impulsive much higher dynamic pressures can be applied than if the load were applied over a longer period of time. It is due to the fact that the structure has inadequate time to respond. Table 1.1 provides the role of the loading characteristics.

*Table 1.1: Explosion Pulse Load Characteristics (L. Louca et al. 2002)*

<table>
<thead>
<tr>
<th></th>
<th>Impulsive $td/T &lt; 0.3$</th>
<th>Dynamic $0.3 &lt; td/T &lt; 3.0$</th>
<th>Quasi-static $td/T &gt; 3.0$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Peak load</strong></td>
<td>Preserving the exact peak value is not critical</td>
<td>Preserve peak value-the response is sensitive to increase or decrease the peak load for a smooth pressure pulse</td>
<td>Not important if response is elastic, but is critical when response is plastic</td>
</tr>
<tr>
<td><strong>Duration</strong></td>
<td>Preserving the exact peak value is not critical</td>
<td>Preserve load duration since in this range it is close to the natural period of the structure. Even slight changes may affect response.</td>
<td>Not important if response is elastic, but is critical when response is plastic</td>
</tr>
<tr>
<td><strong>Impulse</strong></td>
<td>Accurate representation of impulse is critical</td>
<td>Accurate representation of the impulse is important</td>
<td>Accurate representation of impulse is not important</td>
</tr>
<tr>
<td><strong>Rise time</strong></td>
<td>Preserving rise time is not important</td>
<td>Preserving rise time is important; ignoring it can significantly affect response</td>
<td>Not important if response is elastic, but is critical when response is plastic</td>
</tr>
</tbody>
</table>
1.3 Blast Basics

Explosives are substances that are changed rapidly and violently to gaseous state, through chemical reaction accompanied by high temperatures, extreme shock and a loud noise.

An explosion from a charge located on or very near the ground surface is referred to as a surface burst. The initial wave is reflected and reinforced by the ground surface to produce a reflected wave. The reflected wave merges with the incident wave at the point of detonation to form a single wave, essentially hemispherical in shape.

Air blast is the foremost damage mechanism. Air blast phenomena occur within milliseconds and the local effects of the blast are often over before the building structure can globally react to the effects of the blast. Also, initial peak pressure intensity (referred to as overpressure) may be several orders of magnitude higher than ambient atmospheric pressure. The overpressure radiates from the point of detonation but decays exponentially with distance from the source and time and eventually becomes negative. In many cases, the effects of the negative phase are ignored because it usually has little effect on the maximum response.

The shock wave reflects off and diffracts around the structure after striking the object. The stress waves are also produced within the object. The true response is dependent upon the structural configuration, the angle of incidence and the strength wave. The blast wave configuration impinging upon the structure is illustrated in Fig. 1.2.

When a blast wave encounters an object, it will both reflect off the object and diffract around it. It will also generate stress waves within the object. The exact behavior depends upon the geometry of the object, the angle of incidence and the power of the wave. An example of a wave striking a rectangular object is shown in Fig. 1.2. Three shock wave configurations are generally considered, one for a large scale blast wave and a large object, one for a large blast wave and small object and one for a small blast wave and large object.

Large Blast Wave, Large Object

- The shock wave completely surrounds the large object, producing very strong crushing forces
- Translational force is generated, but the object is not likely to move owing to its larger size.

Large Blast Wave, Small Object

- The object is surrounded and crushed by the shock wave
- Squashing overpressure is more or less equal over the entire object and the translational force lasts for a brief time
- Translatory force owing to dynamic or drag loading is also generated. It has a longer duration and leads to considerable movement and damage.

Small Blast Wave, Large Object
- Analysis is necessarily carried out on the object elements separately. It is due to the fact that the small wave will not uniformly load the structure.
- Till the final stages of the loading duration of structure, the initial phase experiences different loads.

Figure 1.2: A blast wave striking a rectangular object (TM5-1300 (1990))
The reflection of the blast wave occurs after striking with the structure. The structure moves after receiving the blast impact, and its magnitude depends on the impulsive force. Within the elastic range, no permanent deformation occurs owing to inadequate pressure or scarce duration. The structure transforms into plastic range with the excessive pressure load. The structure may fail with the displacement in the plastic range.
1.4 Explosion Scenarios and Designer Options

There may be many possible scenarios which need evaluation in the design of blast resistant structures. Some threats are not considered, such as aerial attack or nuclear attack because their design is not feasible. Some threats are out of scope of the present work because they do not cause structural damage, such as chemical/biological contaminants. Recently, the most serious threat against a structure has been the “car bomb”, i.e., a large external explosion.

The designer deals with four broad groups of countermeasures: “deterrence,” “keep-out,” “deception,” and hardening.” These terms are listed in order of decreasing effectiveness. “Deterrence” refers to the perceived protection level of the facility. If effective, the facility will not be selected as a target because it presents more obstacles than the intruder is willing to overcome. If deterrence is not effective, then the next level of protection is “keep-out.” This refers to devices such as fences, walls, etc., which prevent the intruder from reaching the target or at least delay him until outside help arrives. If the “keep-out” measures are overcome, then the attacker is confronted by deception. This diverts his attention from the most vulnerable or valuable part of the facility towards a more visible, less important part. If all these countermeasures are overcome, then as a last remedy, the building is “hardened” to protect the occupants and contents against the effects of the explosion.

1.5 Structural Response against Blast

The structural response against blast is associated with stress–wave propagation, along with impact and missile penetration. A comprehensive application of shock wave phenomenon is required during traveling of blast waves through the transmitting medium.

If the explosion initiates from extremely great scaled distance e.g. a small charge weight or a large scaled distance from a structure, then global deformation will result in the structure. It shows that all the structural elements offer some resistance to the shock wave. It is of utmost importance that the expected loading and the resisting elements to absorb shock wave should be incorporated in the dynamic analysis and design for proper visualization of structural response.

If the blast occurs very near to a wall or floor i.e. with a small scaled range, abrupt failure results with spalling on the front and back sides. The wall fragments may move as missiles. These fragments can harm people, damage structure, and consequently resulting structural
collapse if the structural support is excessively disturbed. Both global and local response with excessive cracks occurs at intermediate scaled range, along with near face disintegration and spalling on the back face, at intermediate scaled ranges.

A blast wave generates dilatational waves (tension or compression) at speeds of 2700-3400 m/s in ordinary concrete and 4900-5800 m/s in steel. Reflections and refractions are produced abruptly within the material in milliseconds. High rate straining and major disintegration effects can happen on the basis of material properties. For instance, concrete which is brittle material develops many fractures resulting fragmentation. Yielding and fracture produces in steel on the basis of material properties and geometry under similar conditions.

The response of doors and windows require special design procedures in order to restrict the explosion damaging effects. The labyrinth entrances with blast resistant doors and ventilation blast valves provide workable solutions. It is an established fact that explosions in a partially or fully confined space generate excessive damage as compared to that in free air.

The structural response of an explosion can be calculated with the application of theory and experiments simultaneously. Experimental results are visualized with explosion theoretical aspects in order to define material behavior at high strain rates. The resulting computations should be verified with the experimental outputs up to possible extent.

The true determination of structural response is dependent on the cause and effect relationship. The experiments carried out in the past by the military demonstrate that various blast scenarios are possible with the similar high explosive devices. Resultantly, it is very difficult to design structures against blast waves. Therefore, the development in the blast resistant design of structures requires a program of precision testing and code simulations with proper visualization of structures.

The evaluation of structural response through computer simulations deals with two physical disciplines:

1. Computational Fluid Dynamics (CFD): It is employed for calculation of the airblast effects due to explosion and the resulting pressures on the surface.
2. Computational Solid Mechanics (CSM): It deals with the response of the structures against blast loads.
1.6 Review of Previous Research Works regarding effects of Impulsive Loading on Concrete Structures

R. Zinn et. al. (1981) explored the dynamic response analysis of a Pressurized Water Reactor (PWR) containment structure subjected to a free field pressure wave with 0.3 bar maximum overpressure. Firstly, the effects of different finite element representations were pointed out. The structural responses of a plane strain beam model and of a refined axisymmetric 3-d shell model were presented. It was concluded that the resulting structural load of German PWR-containment types with wall thickness about 1.8 m was very low, whereas induced vibrations in the inner structure could increase to a considerable level in lower frequency ranges. The second aspect which was pointed out is related to the load model. Accordingly, a triangular load time function in connection with a certain circumferential pressure distribution as input load function for buildings with cylindrical and spherical shapes avoid a detailed load evaluation and leads to conservative results. Based on numerical results, the principal conclusions were:

1. The structural responses may be computed with a plane strain beam model in order to get global responses and for global analysis and design. Such a model is convenient for extensive parametric studies, for instance the variation of soil parameters.

2. A 3-dimensional analysis allows the evaluation of shell strains as well as a detailed time-dependent load distribution in the circumferential direction. Some higher modes are better represented by use of a 3-d finite element model. However, these higher modes are not important in practice.

3. If an axisymmetric 3-d model is used for evaluating the structural response, higher harmonics from the 2nd or 3rd upwards are not necessary because their influence on the response is negligible.

B. Barbe et. al. (1981) modeled the cylindrical reactor building to 1:40 scale. In the light of experimental results, acoustic code has been developed. The model was submitted to incident shock waves obtained by T.N.T. explosions. Experimental data was recorded with pressure and displacement transducers and accelerometers. The results show that the geometrical configuration between reactor and fuel building induces local overpressures five times the incident pressure.
Y. Kivity et. al. (1981) assessed the vulnerability of reactor containment structure to the internal air blast resulting from a high explosive charge. The analysis focused on the global response and integrity of the containment structure. A parametric study of the containment response was carried out with explosive charges in the range of 1000 kg TNT. It was found that a typical concrete containment with a thickness of the order of 1m can survive an explosion of a charge in the range of 1 ton TNT.

F. Stangenberg et. al. (1981) studied (i) the explosion hazards due to bursting pressures (ii) mechanism of pressure wave (iii) TNT equivalence with reference to Prototype Plant for Nuclear Process Heat. Besides, the behaviour of reinforced concrete structures was also investigated. The paper indicated that the phase of underpressure overlapped with the phase of increasing strains and resulted in a considerable decrease of total load. This effect became more significant with the increasing degree of plasticity because of increasing response periods. The second important aspect was related to the indoor TNT-explosions: during these tests relatively long pressure load functions with high frequency contents were measured. The indoor blast test results showed that rough simplifications of the very complicated load time history can be done for structural design purposes.

A. Huber et. al. (1983) discussed the propagation of shock waves in the vicinity of reactor buildings. He presented analytical results of shock wave propagation computed by linear wave equation and nonlinear equations of motions. He concluded that the analytical isobars of the reflected front show a backward inclination at the reflection point. He termed the phenomenon as Mach Stem. He pointed out that loadings on buildings and structures due to shock waves as e.g. caused by chemical explosions are influenced by reflection and diffraction.

T.A. Duffey et al. (1982) investigated the response of steel containment vessels to the internal blast loading produced by the detonation of high explosives. He concluded that the two dimensional motion, which occurs after significant wave interactions have taken place in the test vessels, can be simulated with reasonable accuracy by finite element calculations.

Delroy J. Forbes (1999) covered the development of the design blast loads for buildings in petroleum refineries and petrochemical plants as reported by the ASCE Task Committee on Blast Resistant Design. It briefly discussed the overall design process for such buildings and the considerations and methods for predicting the blast loading from vapour cloud explosions.
The main focus was on determining the building component loads once the incident blast overpressure is defined. He emphasized the need for a better understanding of the interaction between the blast wave from a Vapour Cloud Explosion (VCE) and a building to better define the design blast loads. This interaction included the extent of blast wave reflection, its clearing time on the front wall, and the wavelength and propagating velocity.

Yousuke et al (2001) carried out a nonlinear dynamic response analysis on a Pressurized Water Reactor (PWR) 3 Loop type reactor building using a 3-D FEM model in the case of simultaneous horizontal and vertical ground motions. He concluded that

(i) A 3-D building model made for this study can accurately simulate seismic observation records including responses for a vertical ground motion and that this model can be applied to investigations into vertical ground motions.

(ii) It was also confirmed that a nonlinear analytical method used in this study can simulate the tests on RC cylindrical walls which is carried out using a vertical force as a parameter. It is applicable to investigations into vertical ground motions which are affected by a vertical axial force.

P. Mendis (2003) et. al. presented a vulnerability/survivability assessment procedure based on the analysis of a typical tall building in Australia. The structural stability and integrity of the building was assessed by considering the effects of the failure of some perimeter columns, spandrel beams and floor slabs due to blast overpressure or impact. The criterion of the analysis was to check if failure of any primary structural member would cause progressive collapse propagating beyond one story level above or below the affected member vertically, or to the next primary structural member vertically. The overall stability of the structure relied on continuity and ductility of these elements to redistribute forces within the structure. The authors suggested some methods to improve the impact resistance of concrete walls and slabs, as well as the rotation capacity of the beams, columns and joints.

Gintauta (2005) performed structural integrity analysis for Ignalina Nuclear Power Plant (INPP) using the dynamic loading of an aircraft crash impact model. The purpose of the investigation was to determine if global structural failure of the building could occur. Local failure mechanisms, such as perforation and scabbing, were also considered.

In this study, the impact loading from an aircraft crash was treated as a uniform pressure history (corresponding to the airplane impact characteristics) applied to the impact area of the
wall. The impact area includes the frontal areas of the body, the engines and the wings of the airplane. Strain-rate dependent material models for the concrete and reinforcing bars were used. Also, the values for the material properties used in the models had been obtained from tests on specimens taken from the Ignalina NPP building walls. It is noted that these test values were found to be higher than the standard material properties listed in handbooks (e.g., norms). For an airplane crash, the highest damaged area of the building was the area adjacent to center point of the airplane-impacted area. It was observed that the tensile failure surface was reached in two layers of the impacted wall. Concrete cracking began after attaining the limit for tension. The compressive failure surface was not reached in any of the layers of the concrete walls. No indication of concrete failure in compression was observed. The maximum axial stress in the reinforcement bars element was 150 MPa. The static ultimate strength for rebar steel is 850 MPa (experimental data). It should be noted that the interior walls, which were perpendicular to the impacted exterior wall, provided additional support to the exterior wall. The results from local response analyses showed that the 1.0 m reinforced concrete walls were of sufficient thickness to prevent either scabbing from the backside of the impacted wall or perforation of the aircraft engine through the wall.

Wu et al (2005) defined simultaneous ground shock and airblast forces that can be easily applied in structural response analysis. Parametric numerical simulations of surface explosions were conducted. Empirical expressions of airblast pressure time history as a function of surface explosion charge weight, distance to structure, structure height, as well as the ground shock time history spectral density function, envelope function and duration have been presented. The results and the empirical relationships have been correlated and discussed with the experimental data in the present thesis.

A.K. Pandey et al. (2005) demonstrated the effect of an external explosion on the outer reinforced concrete shell of a typical nuclear containment shell. The analysis was made using appropriate non linear material models till the ultimate stages. He performed parametric studies for surface detonations of different amount of blast charges at a distance of 100m from a nuclear containment shell. Also, critical distances were evaluated for different amount of blast charges for nuclear containment shell.

Wu. et al. (2005) demonstrated the characteristics of response and collapse of three structures to airblast load. The three structures include a one-storey masonry infilled RC frame, a two
storey masonry infilled RC frame and an eight storey RC frames filled with masonry wall. The authors developed 3D homogenized model for masonry including equivalent elastic moduli, strength envelope and failure characteristics, and a material level damage model developed for reinforced concrete. The authors conclude that at the same scaled distance, the more severe damage takes place in front of columns and beams of the one-storey and two storey buildings as compared to the front columns and beams of the eight storey building while the masonry walls of the eight storey building suffer more damage in comparison with those of the one-storey and two-storey buildings.

Wu. et. al. (2005) simulated the response and damage of a one-storey and a two-storey building generated with or without a protective wall between the explosion center and building. It was observed that 1m high protective wall was not efficient in protecting both the one-storey and two-storey buildings, although it delayed the collapse of the buildings. A 2.5 m high protective wall between the explosion center and building was very effective to ensure the structural safety. The authors concluded that a protective wall was not an appropriate option to protect structural elements and residents.
1.7 Salient Findings of Previous Researchers

The salient findings of previous research works are as follows.

(i) The impulsive loading from chemical blasts is estimated by the TNT equivalent yield concept. The effect of normal burning velocity of the fuel is also taken into account.

(ii) The shock wave is described by a loading-time history that is generated on the structures as transient loading for dynamic analysis and design.

(iii) The accurate loading data is difficult to record and may be not valid owing to numerous uncertainties present in the interaction process between the shock wave and the structure and the ideal gas consideration in the formulation of available criteria.

(iv) Linear idealization of time history of shock pressures is easy to implement due to abovementioned uncertainties.

(v) The manner a shock wave interacts with the structure is dependent not only upon its intensity but also upon its duration, rise time and configuration of the blast loading curve.

(vi) The load can be distinguished as impulsive, dynamic or quasi-static relative to a particular structural member. If the time period of the member is significantly less with reference to the load duration, the load may be regarded as quasi-static. If the period of vibration is long, then the load may be regarded as impulsive for that member. The intermediate region between impulsive and quasi-static, where the time period is about the same as loading duration, is denoted as dynamic.

(vii) Shock effects may be visualized as impulsive or dynamic loading for most concrete structures.

(viii) Hopkinson (1915) presented his theory for using scale models with the statement: ‘If two structural systems, identically similar except in size, be subjected to blast loading from two explosive charges whose weights are in proportion to the cube of the ratio of the linear dimensions of the two structures, then the behavior of the two structural systems will be identically similar with the distortions scaling as the ratio of the linear dimensions.’
After World War II, the published literature (Christopherson 1945, White 1946) was related to research carried out during the war on the behaviour of shelters against explosions and the invention of the first non-spherical charges to destroy hardened structures. The explosions were localized in nature owing to problems associated with the delivery of large quantities of explosives.

The invention of nuclear weapons transformed the scenario altogether. Two aeroplanes, each with one nuclear bomb, destroyed Hiroshima and Nagasaki in 1945 in two nuclear attacks. Consequently, the design procedures of simple shelter structures changed completely. (Whitney 1945, Newmark 1956).

After 1945, the number of reported experimental and analytical studies of the effects of blast loading on the behaviour of structures has markedly increased. The objective has been, first, to study the nature of the blast wave and factors affecting its characteristics in free air and as it encounters a structure. Secondly, the aim has been to examine and develop means of predicting the response of a structure to blast loads.
1.8 Problem Statement

Important buildings and some critical infrastructures might be targets of terrorist threat and need be designed against such threats. The design and construction of reactor containments to provide life safety in the case of external explosions is receiving significant attention from structural engineers. Existing blast design approaches require structural design along with a buffer zone surrounding the structure. This highly effective approach is only feasible where a keep-out zone is available and affordable. For many urban areas, the closeness to unregulated traffic brings the terrorist threat to or within the perimeter of the structure. For these structures, blast protection is more appropriate option for containing damage in the immediate vicinity of the explosion and the prevention of progressive collapse.

Explosions may occur due to a variety of reasons that need to be identified and for which the probability of occurrence may need to be quantified. ACI Standard 359 (2006) “Code for Concrete Reactor Vessels and Containments” deals with the impulse loads as time dependent loads e.g. the dynamic effects of accidental pressure $P_a$, the effects of pipe rupture reactions $R_r$ and Jet impingement loading $R_j$ etc. These impulse loads lie within the purview of internal explosions. However, the provisions to deal with the effects of external explosion against reactor containment are still under consideration. The research regarding blast resistant design of containment structures is also underway through United States Nuclear Regulatory Commission (USNRC). The Clauses in this regards may be incorporated in ACI code after the approval of its review committee.

The blast loading on a structure caused by a high-explosive detonation is dependent upon several factors:

1. The charge weight
2. The location of the explosion with reference to the structure i.e unconfined or confined explosions.
3. The structural configuration
4. The structure orientation with respect to the explosion and the ground surface

This work explores the scenario of external explosions at various distances against reactor containment. The airblast pressures generated by external explosions have been extensively studied since World War II. Many empirical formulae and curves have been proposed for
prediction of peak overpressure attenuation against scaled distance, pressure time history of shock wave, and shock wave reflection to structures. These formulae calculate shock wave propagation in the air. However, certain parameters, which are necessary for modeling simultaneous ground shock and airblast, such as arrival time of airblast wave to structures, have not been fully investigated. The available empirical relations are usually inconvenient to use since many parameters are given in empirical curves rather than in analytical equations. In general, simplified approximations have to be pursued. For example, airblast pressures are usually assumed as having a triangular or an exponential decay shape with the pressure rising phase being completely neglected. The pressure is also assumed uniformly distributed along the structural height. Such simplifications are expected to introduce errors in estimation of airblast loads on curved structures.

The present practice in designing structures against surface explosive loading usually considers only the airblast pressures generated by the explosion. A surface explosion, in fact, generates both ground shock and airblast pressure on a structure. Although the ground shock reaches the structure foundation before the airblast pressure, both the ground shock and airblast might act on the structure at the same time, depending on the distance between the explosion center and the structure. Even though they do not act on the structure concurrently, ground shock will excite the structure and the structure will not respond to airblast pressure from rest. However, little information is available for simultaneous ground shock and airblast pressure loads generated by surface explosions on structures in the literature.

The scope of this work lies in defining the simultaneous ground shock and airblast forces that can be easily applied in structural response analysis of reactor containments. The required empirical relationships are essential for more accurate estimation of explosive loads in modeling response and damage of containments to external explosions.
1.9 Objectives & Scope

The research works in the field of blast resistant reactor containment are either scarce or not for public release. In accordance with the available published literature in blast resistant design of reactor containments, the objective of this research work is to find relationships of following parameters of overpressure in the free air from external explosions on reactor containment.

(a) Peak pressure \( (P_{so}) \) and scaled distance \( (R/Q^{1/3}) \)
(b) Shock wave front arrival time \( (T_a) \) in terms of distance \( (R) \) and charge weights \( (Q) \)
(c) Rising time from arrival time to the peak value \( (T_r) \)
(d) Decreasing time from peak to the ambient pressure \( (T_d) \)
(e) Duration of the positive pressure phase of the airblast pressure wave \( (T) \)
(f) Relation of the peak reflected pressure \( (P_{ro}) \) to the peak free air pressure \( (P_{so}) \)

Also, the following empirical relationships of ground shock wave from external explosions are to be explored through scaled model reactor containment experiment.

(a) Peak Particle Acceleration (PPA)
(b) Arrival Time \( (t_a) \)
(c) Shock Wave Duration \( (t_d) \)
(g) Time lag between ground shock and air blast pressure arrival at structures \( (T_{lag}) \)

The abovementioned relationships may be utilized in the precise determination of the response of concrete containments against external explosions.
Chapter 2

Fundamentals of Blast Loading

2.1 Introduction

The blast effects of an explosion are in the form of a shock wave composed of a high intensity shock front which expands outward from the surface of the explosive into the adjoining air. As the wave expands, it decays in strength, lengthens in duration and decreases 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.

The one-third portion of the chemical energy available in most high explosives is discharged during the ignition process. The residual two-third portion is discharged slowly as the detonation products combine with air and burn. This afterburning development has slight effect on the initial blast wave because it happens much slower than the original detonation process. On the other hand, the next stages of the blast wave can affect by the afterburning, especially for blasts in confined spaces. As the shock wave spreads out, pressures reduce quickly owing to geometric divergence and the consumption of energy in heating the air. Pressures also decrease rapidly over time and have a very short period of survival, calculated in milliseconds. An explosion can be envisaged as a sphere of extremely compressed air that attains balance after expansion.

2.2 Pressure Time History

At any location away from the explosion, the pressure disturbance has the configuration shown in Figure 2.2. The blast wave front reaches at a given location at time $t_A$, and after the rise to the peak value, $P_{so}$ the incident pressure decreases to the ambient value. The time taken is known as the positive phase duration. This is followed by a negative phase with a duration $t_{o^-}$ that is usually much longer than the positive phase. It is classified as a negative pressure below ambient pressure having peak magnitude of $P_{so^-}$. It occurs along turbulence of the particle flow. The negative phase has less significance in a design than is the positive phase,
and its amplitude $P_s^+$ must be less than ambient atmospheric pressure $p_o$. The incident impulse density related with the blast wave is the integrated area under the pressure time curve. It is described as $i_s$ for the positive phase and $i_s^-$ for the negative phase.

\[
P = P_s^+ \left( 1 - \frac{(t-t_a)}{t_d} \right) e^{-\frac{(t-t_a)}{t_d}}
\]

(2.1)

### 2.3 Effect of Angle of Incidence on Reflected Pressure

The incident blast wave reflects and generates reflected pressure after striking a structural member that is at an angle to the direction of the wave’s travel (Beshara 1994, Louca 2002). The reflected pressure is generally larger than the incident pressure at the same scaled distance from the explosion. The reflected pressure differs with the change in angle of incidence of the shock wave. When the shock wave strikes a structure that is perpendicular to the path it is progressing, the location of impact will face the peak reflected pressure. When
the reflector is parallel to the explosion wave, the minimum reflected pressure or incident pressure will be felt. Besides the angle of incidence, the maximum reflected pressure depends on the maximum incident pressure, which is related with the net explosive weight and distance from the explosion.

Figure 2.2 illustrates representative reflected pressure coefficients against the angle of incidence for four different maximum incident pressures. The reflected pressure coefficient is the ratio of the peak reflected pressure to the peak incident pressure \( \left( C_r = \frac{P_r}{P_i} \right) \). This figure demonstrates that reflected pressures for explosive detonations is about 13 times larger than maximum incident pressures. The reflected pressure coefficients are considerably larger closer to the detonation location.

---

\[ Pr = P_i \times C_r \]

\( C_r = \frac{Pr}{Pi} \)

**Figure 2.2: Reflected pressure coefficient vs. angle of incidence ((FEMA 426)**

*Peak Incident Pressure*

- 5,000 psi
- 500 psi
- 100 psi
- 0.2 psi

0° = Perpendicular to surface

90° = Parallel to surface

\( C_r \) = Coefficient of reflection
2.4 Path of Triple Point

After crossing the point of explosion by an air burst, it moves as an incident wave until it impinges upon some object of density greater than the ambient atmosphere density. A reflected shock wave moves towards the point of explosion after impinging upon the object. The reflected overpressure may be much more than the pressure due to the incident blast wave. As the reflected shock wave velocity is more than that of the incident wave, it signifies that the reflected wave will coincide with the incident wave away from the ignition location. A single vertical wave front is generated called a Mach stem (Biggs 1964, Norris 1959, Rogers 1959, Crawford 1974) which progresses in the horizontal direction along the ground. The junction location is termed as the triple point. As the shock wave moves along the ground, the triple point characterize a path as shown in Figure 2.3. Structural members located below this path will receive a single shock, whereas objects above this path will receive two shocks-the incident and reflected waves. The shock front advances on the ground with a vertical front near the ground below the path, as in the case of a surface burst.

![Figure 2.3: Shock wave reflection phenomena (Rogers 1959)](image)

2.5 Impulse of the blast wave

Impulse is the extent of the energy from an explosion passed on to a building. The impulse comprises the negative and positive phases of the pressure-time waveform.

The area under the pressure against time curve is termed as the impulse:
\[ I = \int P(t)dt \] (2.1)

\( I \) = impulse (psi-ms or MPa-ms)

\( P \) = Pressure (psi or MPa)

\( T \) = time (ms)

Figure 2.4: Typical Impulse Waveform (FEMA 426)

Figure 2.4 demonstrates the variation of impulse and pressure with time from a characteristic explosion. The intensity and delivery of blast loads on a structure are dependent on several factors:

- Explosive characteristics (type of matter, energy output, and amount of explosive)
- Orientation of the detonation with reference to the structure
- Increase in the pressure value through its interface with the ground or structure (reflections)
The structure reacts to the forces which include the reflected pressure and the reflected impulse. The forces differ in time and space over the uncovered surface of the structure. The forces depend on the orientation of the explosion with reference to the structure. Consequently utmost effort should be made to spot the most awful case of blast location during the design of a building for a particular blast event.

The blasts possess the following salient characteristics in the background of other hazard (e.g., earthquakes, winds, or floods),

The magnitude of the force acting on the target structure can be manifold larger than these other perils. It is usual for the maximum incident pressure to be greater than 100 psi on a structure in urban surroundings owing to a vehicle weapon. Major damages and malfunctions are likely due to generation of these pressures.

Blast pressures reduce abruptly with distance from the source. As a result, the damages on the front side of the structure may be considerably more severe than on the converse side. Resultantly, direct air-blast damages may produce more localized damage. In urban surroundings, on the other hand, reflections off adjacent buildings can result excessive damages to the other side.

The event is transient, considered in milliseconds. The phenomenon is different from earthquakes and strong winds occurring in seconds, or persistent wind or flood condition happening in hours. Owing to this, the mass of the building has a strong extenuating effect on the response. It takes time to activate the mass of the building. When the mass is activated, the force disappears, thus extenuating the response. This is the converse of earthquakes, where forces are approximately in the same time domain as the reaction of the building mass, resulting resonance which increases the damage.
2.6 Factors Affecting Blast Loading

The blast loading on a structure caused by a high-explosive detonation is dependent upon several factors:

(1) Quantity of Charge Weight

Large-scale truck bombs possess 10,000 pounds or more of TNT equivalent during design. The quantity varies on the size and capacity of the vehicle used to transport the weapon. Bombs in vehicles normally have 4,000 to 500 pounds of TNT equivalent, correspondingly. A brief case bomb roughly contain 50 pounds, and a pipe bomb is usually in the range of 5 pounds of TNT equivalent.

The designer calculates the extent of damage and the attained degree of protection in case of a still vehicle bomb on the basis of the size of the bomb (TNT equivalent in weight), its distance from the structure, how the structure is put together, and the constituents used for walls, framing, and glazing.

(2) The location of the explosion relative to the structure in question (unconfined or confined).

The significant position of the detonation is dependent on the site, the building layout, and the existing security measures. The vital location for vehicle bombs is the contiguous point that a vehicle can draw near on each side, considering all security procedures are followed. Generally, this is a vehicle located along the barrier nearest to the building, or at the entry control point where scrutiny is pursued. Position for internal blasts is dependent on the region of the structure that is openly reachable e.g. lobbies, corridors, auditoriums, cafeterias, or gymnasiums. Range or stand-off is calculated with reference to the center of gravity of the charge situated in the vehicle or the structural member.

It is extremely hard to characterize correct stand-off distance for a given structural member to survive blast impact. Usually, it is neither possible nor realistic to obtain proper stand-off distance in urban scenarios. The visualization of suitable stand-off distance needs a calculation of the explosive weight of the weapon.

The DoD (UFC 2002) defines minimum stand-off distances on the basis of the required...
intensity of safety. Conventional construction methodologies can be employed, if minimum stand-off distances are ensured. The locations where the minimum stand-off cannot be met, the structure must be strengthened to attain the required protection level.

(3) The geometrical configuration of the structure
The methodology presented in TM5-1300 for the determination of the external loads on structures are valid for rectangular superstructures where the structure will be subjected to a plane wave shock front. The procedures can also be applied to structures of other shapes (cylindrical, arch, spherical, etc.) as well as structures located at and below the ground surface.

The interaction of the incident wave with a structure is a complex phenomenon. It is assumed to visualize the phenomenon, that

(a) the structure is rectangular in shape
(b) the incident pressure is 1.38 MPa or less,
(c) the structure under consideration is in the region of the Mach stem, and
(d) the Mach stem rises over the height of the structure

(4) The structure orientation with respect to the explosion and the ground surface
When the shock wave strikes a structural member that is an angle to the direction of the wave’s travel, it reflects and produces reflected pressure. The reflected pressure is generally larger than the incident pressure at the same scaled distance from the explosion. The reflected pressure is dependent upon the angle of incidence of the blast wave.

2.7 Calculation of Blast Loads
The primary step in the calculation of explosion effects on a building is to determine shock loads on the structural member. In case of an external explosion, it is the blast pressure wave that damages to the structure. As the pressure pulse fluctuates on the basis of stand-off distance, angle of incidence, and reflected pressure over the outer surface of the structure, the blast load forecast should be carried out at numerous threat locations. Nevertheless, the worst scenario is generally adopted for analysis and design.

The blast designers employ advanced methods e.g. Computational Fluid Dynamics (CFD) to analyze complex structures which need precise evaluation of impulsive loads. These advanced programs need special tools and guidance to operate.
For realistic analysis, more simplified procedures may be employed by blast engineers to calculate blast loads. The overpressure suddenly attains its maximum value and decreases to zero in a time termed as the duration time. Numerous techniques can be employed. In order to find out the blast load, tables may be utilized or soft wares may be used, e.g.

ATBLAST (GSA Manual 2001)

CONWEP (TM5-855-1 -1986)

\[ \text{Figure 2.5: Incident Overpressure Measured In Pounds per Square Inch, As a Function of Stand-Off Distance and Net Explosive Weight (Pounds-TNT)(TM5-853 (1994))} \]

\[ 1 \text{ psi} = 0.0068 \text{ MPa} \]

Figure 2.5 illustrates a swift technique for calculating the expected overpressure (in psi) on a structure for a particular explosive weight and stand-off distance. Read the x-axis with the probable explosive weight and the y-axis with an identified stand-off distance from a structure. The extent of damage that the various components of a structure might experience can be calculated by associating the consequential effects of overpressure with other information. The vehicle icons in Figure 2.5 specify the comparative size of the vehicles that might be employed to carry various magnitudes of explosive materials.
2.8 Calculation of Blast Effects

Consequent upon the calculation of blast loads, damage levels may be estimated by explosive analysis and engineering analysis.

Usually, experiment is very costly and an engineering analysis is a better option. The analysis should be time dependent which incorporates non-linear characteristics to precisely capture the response of the blast event.

Structural components e.g. beams, slabs, or walls can be modelled by a SDOF system. The response can be determined by employing the charts developed by Biggs (1964) and military handbooks. SDOF models are appropriate for numerical analysis on PCs and microcomputers. However, the most refined FEM systems (with non-linear material models and options for explicit modeling of reinforcing bars) may be accomplished on mainframes. Table 2.1 describes incident pressures at which damage may happen.

\[
\begin{array}{|c|c|}
\hline
\text{Damage} & \text{Incident Overpressure (psi)} \\
\hline
\text{Typical window glass breakage} & 0.15 – 0.22 \\
\hline
\text{Minor damage to some buildings} & 0.5 – 1.1 \\
\hline
\text{Panels of sheet metal buckled} & 1.1 – 1.8 \\
\hline
\text{Failure of concrete block walls} & 1.8 – 2.9 \\
\hline
\text{Collapse of wood framed buildings} & \text{Over 5.0} \\
\hline
\text{Serious damage to steel framed buildings} & 4 – 7 \\
\hline
\text{Severe damage to reinforced concrete structures} & 6 – 9 \\
\hline
\text{Probable total destruction of most buildings} & 10 – 12 \\
\hline
\end{array}
\]

Table 2.1: Damage Approximations (UFC-2002)
2.9 Effects of blast loading

2.9.1 Extent of damage during explosion

The intensity and degree of damage and injuries during blast cannot be foreseen in all respects. Previous experience shows that the exclusive particulars of the failure cycle for a building considerably influence the intensity of damage. Notwithstanding these ambiguities, it is likely to provide some all-purpose information about the general intensity of damage and injuries to be anticipated during blast on the basis of the size of the explosion, distance from the event, and assumptions about the construction of the building.

Damage owing to the air-blast shock wave may be classified into direct air-blast effects and progressive collapse. Direct air-blast effects are damage produced by the high-intensity pressures of the air-blast contiguous to the detonation and may bring the localized failure of exterior walls, windows, floor systems, columns, and girders.

The air blast shock wave is the principal damage factor during the blast. The pressures it generates on the structural member may be manifold greater than the loads for which the building is planned. The blast wave front also exert pressure in scenarios that the building may not have been designed for, such as in upward direction on the floor system. As regards order of response, the air-blast first strikes the weakest point in the surrounding area of the device closest to the detonation on the exterior surface of the structure. The shock wave thrusts the exterior walls at the lower stories and may result wall failure and window breakage. While the shock wave keeps on progressing, it pierces the structure, pushing both upward and downward on the floors (Figure 2.6).

2.9.2 Floor Failure

Floor failure is frequent in large-scale vehicle-delivered explosive attacks, because floor slabs usually have a large exterior area for the pressure to be applied compared to little thickness. The building is surrounded by the blast wave and direct air-blast damage happens within tens to hundreds of milliseconds from the instant of explosion in case of coincidence of the activities. If progressive collapse is generated, it normally happens within a very short time.
2.9.3 Glass Breakage

Glass is the weakest part of a structure. It fails at little pressures in contrast to other components such as the floors, walls, or columns. Previous events demonstrate that glass breakage may result over a large distance in huge external explosions. High-velocity glass wreckage has been shown to be a key factor in wounds in such events. Falling glass inflicts injuries to people walking on foot during such occurrences within business district areas.

**Figure 2.6: Blast pressure effects on a structure (NFESC-1998)**
2.10 Extent of Protection

The magnitude of explosive material and the consequent explosion prescribe the level of protection needed to avoid a building from destruction or reducing wounds and casualties. Table 2.2 demonstrates how the DoD associates levels of protection with probable damage.
Table 2.2: DoD Minimum Antiterrorism (AT) Standards for New Buildings, (NFESC-1998)

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Table 2.2: DoD Minimum Antiterrorism (AT) Standards for New Buildings, (NFESC-1998)

<table>
<thead>
<tr>
<th>Condition</th>
<th>Load Type</th>
<th>Action</th>
<th>Protection</th>
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The levels of protection in Table 2.2 can approximately be interrelated for conventional construction without any blast strengthening to the incident pressures illustrated in Table 2.3 (Kinnery 1985).

Table 2.3: Correlation of DoD Level of Protection to Incident Pressure

<table>
<thead>
<tr>
<th>Level of Protection</th>
<th>Incident Pressure (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>High</td>
<td>1.1</td>
</tr>
<tr>
<td>Medium</td>
<td>1.8</td>
</tr>
<tr>
<td>Low</td>
<td>2.3</td>
</tr>
</tbody>
</table>
Figure 2.7: Explosives Environments - Blast Range to Effects (FEMA 426)

Figure 2.7 illustrates an example of a range-to-effect chart that points to the distance or stand-off to which a known size bomb will create a specified consequence. This type of chart can be employed to demonstrate the blast response of a structural member or window at diverse levels of protection. It can also be utilized to merge all building response data to evaluate required procedures if the threat weapon-yield changes. For instance, a certain quantity of explosive material is stolen and it is feared that they may be employed against a particular
structure. A structure-specific range-to-effect chart will rapidly establish the required stand-off for the magnitude of explosives, after the explosive material is changed to TNT equivalence.

Research accomplished as branch of the threat evaluation procedure should categorize bomb sizes used in the area or section. Safety consultants have precious data that may be employed to assess the output of probable charge weights. Knowing an explosive weight and a stand-off distance, Figure 2.7 can be employed to forecast damage for a particular structure.

2.11 Research in Reactor Containment Analysis and Design against External Explosions

The fundamentals of blast loading outlined in this chapter are also applicable on reinforced concrete containments for the evaluation against external explosions. The external explosions generate both airblast and ground shock. The calculation of airblast pressure and ground shock time history and its true visualization simultaneously is needed to study the containment design against external explosions.

Computer simulation of the blast loading parameters for a specified amount of blast charge for a cylindrical structure like nuclear containment shells is very important in structural analysis against external explosion. And, therefore, parametric analysis has been carried out in Chapter 5 in order to calculate the critical distances for the external explosion.

ACI Standard 359 (2006) “Code for Concrete Reactor Vessels and Containments” deals with the impulse loads as time dependent loads e.g. accidental pressure $P_a$, the effects of pipe rupture reactions $R_{rr}$ and Jet impingement loading $R_{rj}$ etc. These are classified as internal impulsive loads. The external explosion effects due to terrorist attacks are presently under developmental stage in various research organizations related to reactor analysis and design.

Studies are available in literature on the impact of an aircraft on the outer reinforced concrete nuclear containment shells, but the studies are limited on the effect of explosions on the containment shells. The present research work has, therefore, been directed to study the effect of external explosions on a typical reinforced concrete containment structure. It calculates response of concrete structures for blast loading through three-dimensional modeling of the structure, modeling of the material non-linearities and the blast phenomenon.
After 9/11, countries around the world are evaluating the safety and security of their nuclear facilities against sabotage acts. Each year, thousands of people around the world fall victim to various explosions. The methods used by terrorists are becoming ever more diversified and refined. In these circumstances, the danger of terrorism against nuclear facilities cannot be disregarded.

On the basis of their analysis of the security status of research reactors, the International Atomic Energy Agency (IAEA) scientists warn that the most dangerous consequences may take place during attacks on reactors of medium (1-10 MW) and large (10-250 MW) capacity. There is no regulation requiring any level of protection for reactor containments from terrorists. (Saleem, 2008).

IAEA Information Circular 225 contains no specific recommendations on how to guard against sabotage of research reactors (Bunn, 2007). Government regulations on physical protection of reactors vary a great deal around the world. Differences in understanding the perceived threat, financial and technical resources and national laws are some of the reasons. Most research reactors lack adequate exclusion zones to guard against the potential for truck bombs and perimeter protection. In addition, many research reactors in the West are located on university campuses, where security may be less scrupulous than at commercial reactor sites and they also have no containment structure. (Ferguso, 2007)

Today, the danger of a terrorist attack at a nuclear power plant in the World, either from the air or from the ground is apparently as great as ever. If an attack on a nuclear facility is successfully carried out, society will be faced with medical, psychological, social, political, economic, and organizational challenges. The need for research in containment safety against external explosions in Pakistan cannot be overemphasized. The Pakistan Nuclear Regulatory Authority (PNRA) is authorized to control, regulate and supervise all the matters related to civil sector nuclear safety and radiation protection in Pakistan. It is the leading Agency for ensuring that national preparedness for nuclear and radiological accidents is maintained by the operating organizations or licensees. In the light of the current wave of terrorism, the PNRA has taken the necessary steps with the help of Pakistan Atomic Energy Commission (PAEC) to strengthen the safety and security of its civil nuclear installations. In this regard, the PNRA is pursuing for the physical protection of nuclear and other radioactive material. The PNRA initiated in 2006 a five year National
Nuclear Safety and Security Action Plan (NSAP) to establish a more tough nuclear security regime. It seeks capacity building in Pakistan’s ability to plan for, respond to, and recover from terrorist incidents in association with relevant governmental agencies. The plan has broad area of applications pertinent to containment safety, radiation protection, transport safety, deployment of radiation detection equipment widely, etc.
Chapter 3

Modelling Of External Explosion Effects on Aboveground Structures

3.1 Prediction of Blast load Parameters

3.1.1 Blast Scaling Law

The factors which affect the blast wave during an explosion are dependent both on the explosive energy release and on the nature of the medium through which the blast progresses. These properties may be measured during controlled explosions in experiments. Such test explosions are called reference explosions. Scaling laws can be utilized to determine values for other explosions.

A detailed discussion dealing with the blast scaling methods can be found in references (Tgo 2007, Louca 2002). The most frequently used blast scaling is Hopkinson (1915) or ‘cube root’ scaling law.

The most widely used approach to blast wave scaling is the cube root scaling law proposed independently by Hopkinson (1915) and Cranz (1926). The law states that, similar blast waves are produced at the same scaled distances when two explosive charges of similar geometry and of same explosive but of different sizes are detonated in the same atmosphere. Thus, if charges of weights \( Q_1 \) and \( Q_2 \) are detonated then the same peak pressure is produced at distance of \( R_1 \) and \( R_2 \), respectively. The distances \( R_1 \) and \( R_2 \) are related as given below;

\[
\frac{R_1}{R_2} = \left( \frac{Q_1}{Q_2} \right)^{\frac{1}{3}} \tag{3.1}
\]

The duration of the positive phase of a pressure wave \( T \) against the detonation of charge weights of \( Q_1 \) and \( Q_2 \) is also given by a similar equation.

\[
\frac{T_1}{T_2} = \left( \frac{Q_1}{Q_2} \right)^{\frac{1}{3}} \tag{3.2}
\]

Equation 3.2 is valid when the scaled distance is the same for both charge weights.
3.1.2 Atmospheric considerations

This basic scaling principle can be extended (Mills 1987) so that explosions under different atmospheric conditions could be related. The degree of energy release over a given distance is dependent on the density, $\rho$ of the propagation medium which, itself, is proportional to the atmospheric pressure, $p$ and inversely proportional to the temperature, $T$. If the actual peak overpressure from a blast in a given medium is to be derived from a reference value in a different medium, the actual value must also be divided by the cube root of the density ratio, $f_d$:

$$f_d = \left( \frac{\rho_{\text{actual}}}{\rho_{\text{ref}}} \right)^{1/3} = \left( \frac{P_{\text{actual}}}{P_{\text{ref}}} \right)^{1/3} \left( \frac{T_{\text{actual}}}{T_{\text{ref}}} \right)^{1/2}$$

(3.3)

The time for the shockwave propagation is dependent on the speed of sound within the medium and hence the actual arrival times and durations must be divided by the product of $f_d$ and the speed of sound ratio for the medium, $f_t$:

$$f_t = f_d \frac{a_{\text{actual}}}{a_{\text{ref}}} = \left( \frac{P_{\text{actual}}}{P_{\text{ref}}} \right)^{1/3} \left( \frac{T_{\text{actual}}}{T_{\text{ref}}} \right)^{1/2}$$

(3.4)

Since the specific impulse is a function of the overpressure and the duration the actual value must be divided by both $f_d$ and $f_t$.

3.1.3 Equivalency to TNT

Beshara (1994) has mentioned various blast pressures results of a spherical charge of TNT explosive. The output can be extrapolated to other explosives such as nuclear weapons, by relating the explosive energy of the effective charge weight of those materials to that of an equivalent weight of TNT. The equivalency of material against TNT is dependent on many parameters e.g. the material shape (flat, square), the explosive quantity, explosive confinement, nature of source and the pressure range (HNDM 1977, Doering 1949). With
reference to TNT, the magnitude of the energy output of explosive material can be defined as a function of the heat of detonation.

\[ W_{TNT} = \frac{H_{\text{exp}}}{H_{\text{TNT}}} W_{\text{exp}} \quad (3.5) \]

Where \( W_{TNT} \) = equivalent TNT charge weight, \( W_{\text{exp}} \) = Weight of the explosive in question, \( H_{\text{TNT}} \) = Heat of the detonation of TNT, and \( H_{\text{exp}} \) = Heat of detonation of explosive

The heat of detonation of the more generally used explosives and chemicals are available in References (Tomlinson 1971, Kingery 1966, Tgo 2005). The heat ratio is defined in some references as the TNT equivalent factor.

### 3.2 Features of Overpressure Phase

(i) External impulsive loads develop from the ignition of high explosives such as atomic warheads, or from low explosives such as flammable gases and vapours.

(ii) The airblast pressure on an uncovered surface is dependent on the airblast pressure intensity, and the orientation, geometry and size of the surface with which the shock wave strikes.

(iii) Three constituents of loading are generally relevant; overpressure, reflected pressure and dynamic pressure. The overpressure is merely the airblast pressure-time histories that arise in the free field outside the structural elements.

(iv) Reflected overpressure takes place owing to momentum change when the progressive shock waves encounter a surface in the route of propagation.

(v) Dynamic pressures produce drag and lift on structures that intercept the movement of such air.

Beshara (1994) described the rate of decrease of the overpressure as shown in Figure 3.1 which is quasi-exponential in nature. It is quite difficult to visualize the shock wave numerically. Baker (1973) illustrates the positive phase with various relationships.
Figure 3.1: Blast wave load-time curve representation (Carpenter 1975)

The relationship proposed by Beshara (1994) is as follows. The following expression to describe the positive phase is the most commonly used. In terms of a dimensionless wave form parameter $\alpha$ and time $t$ measured from the instant the shock front arrives, the relation is established as

$$P_s(t) = P_s \left(1 - \frac{t}{t_s}\right)e^{-\alpha t_s}$$  \hspace{1cm} (3.6)

$\alpha$ is a dimensionless parameter & $t$ is the time taken from the moment the blast wave reaches. $P_s$ is the peak overpressure. $t_s$ is the time taken by the blast wave to reach ambient pressure. $t_\alpha$ is the theoretical time due to a dimensionless parameter $\alpha$. The impulse per unit of projected area for a shock wave is expressed as

$$i_s = \int_0^{t_\alpha} P_s(t)dt$$  \hspace{1cm} (3.7)

$$= P_s t_s \left[\frac{1}{\alpha} - \frac{1}{\alpha^2} (e^{-\alpha})\right]$$  \hspace{1cm} (3.8)

The positive phase parameters of the surface burst scenario for hemispherical TNT explosions are illustrated in Figure 3.2. A comparison of these parameters with those of free-air explosions (Figure 3.3) depict that all the results of the surface burst environment are more than those for the free-air environment at a certain location away from the ignition of the same weight of explosive.
Figure 3.2: Positive phase shock wave parameters for a hemispherical TNT explosion on the surface at sea level (TM5-1300 (1990))

Figure 3.3: Positive phase shock wave parameters for a spherical TNT explosion in free air at sea level (TM5-1300 (1990))
Where
Pr = the reflected pressure

Pso = Peak side on pressure

Io = the positive phase impulse

Ir = the reflected impulse

\( t_o \) = the positive phase duration

U = Shock Front Velocity

Lw = Blast wave length

### 3.3 Features of Nuclear Weapons

Numerical relationships to the nuclear burst overpressures as a function of time and range have been produced and employed broadly for many years (Newmark 1961, Newmark 1962, Newmark 1963). A comprehensive discussion of nuclear air blast phenomena is found in References (Baker 1973, Glasstone 1977). During the last four decades, numerous experimental and numerical studies have been pursued to derive the features of blast waves from atomic sources. The experiments carried out in the 1970s (Carpenter 1975) have provided a more appropriate understanding of peak overpressures. At a certain point, where the blast wave traverses, the peak overpressure is dependent on the energy yield of explosion, the scaled distance, the height of burst, and the medium in which the weapon is exploded (Crawford 1974, Agbabian 1985). The maximum value at a given range from a given yield can be calculated with confidence within a factor of two. The value of a certain overpressure can be approximated within a variation of 20%.

The below mentioned relationship provides a precise calculation for the change of peak overpressure, \( P_s \), with distance and explosion yield for a surface nuclear burst (Rogers 1959)

\[
P_s = 15 \left[ \frac{1000}{10^{15}} \right] \left[ \frac{1000}{R} \right]^3 + 11 \left[ \frac{W}{10^{15}} \right]^{1/2} \left[ \frac{1000}{R} \right]^{1/2} \quad \text{(N/cm}^2) \quad (3.9)
\]

\( W \) = the weapon yield (in joules) and

\( R \) = the ground range at the point of interest (in metres)

The weapon yield is denoted in units of one megatonne (1 Mt = 4.184 x 10\(^{15}\)J).

Surface bursts are nearly double in effectiveness as air bursts for shock effects. Resultantly, one half of the actual burst yield, \( W \), in Eq. (3.11) can be employed to find
out free air overpressure owing to an air burst. The impact of reflection of air overpressures on the ground surface needs consideration. The peak overpressure\(^{24}\) initiated in the ostensible meteorological standard atmosphere as determined for a spherical charge with energy release equivalent to one kilotonne (1 kt) for nuclear air burst, is stated as (Kinnery 1985)

\[
\frac{P_s}{P_o} = 3.2(106)Z^{\frac{1}{3}}\left[1 + \left(\frac{R}{87}\right)^{2}\right]^{1/2}\left[1 + \frac{R}{100}\right]
\]  

(3.10)

where \( \frac{P_s}{P_o} \) = the ratio of explosion overpressure to ambient pressure \( P_o \),
scaled distance \( R \) = the distance away from a nuclear explosion with an energy release of 1 kt of TNT. The overpressures may be extrapolated to other yields, \( W \), by multiplying by the explosive yield factor where the weight of reference explosive, \( W \), equals 1 kt.

The instant of arrival, \( t_a \) and the positive phase duration, \( t_s \) for a given overpressure can be determined with the cube root law, it is expedient to associate these parameters to a reference overpressure ratio. Crawford (1974) & Agbabian (1985) provided graphical relationships for a surface burst of 1 Mt. It is recommended that the cube root of the ratio of one half the air burst yield could be employed to utilize surface burst data in case of air bursts. According to Biggs (1964), the time needed for a shock front to reach a certain point \( R \) is stated as

\[
t_a = \frac{1}{C_o} \int_{r_c}^{R} \frac{1}{Ms} \, dr
\]  

(3.11)

\( r_c \) = charge radius, \( C_o \) is the speed of sound in the undisturbed atmosphere. For the related peak overpressure, \( Ms \), the Mach number, is stated as

\[
Ms = \left[1 + \frac{\gamma_h + 1}{2 \gamma_h} \frac{P_s}{P_o}\right]^{1/2}
\]  

(3.12)

where \( \gamma_h \) = the ratio of specific heat of air. The subsequent duration of the overpressure positive phase is expressed as (Baker 1983)
The change of overpressure at a certain point with time is a function of peak overpressure with the initial rate of decay. The rate is fast at higher overpressures. Consequently, a characteristic nuclear explosion provides a smaller blast impulse per unit area with reference to conventional explosion of similar duration and peak overpressure (Kinnery 1985). The time history of overpressure for surface bursts from the initial maximum value can be expressed as the total of exponentially decaying phases specified in the following equation (Newmark 1961)

\[
P_s(t) = P_s(1 - \tau_r)(ae^{-\alpha \tau_r} + be^{\beta \tau_r} + c^{-\gamma \tau_r})
\]

\[
\tau_r = \frac{t}{t_s}
\]

where \( t \) is the time calculated from the instant of shock wave arrival.

The empirical coefficients \( a, b, c, \alpha, \beta, \gamma \) may be obtained from reference. (Newmark 1963) It is pertinent to note that the simplified exponential form for the analytical expression of the overpressure history provided before has been utilized in References (Baker 1983, Kinnery 1985, Biggs 1964, Norris 1959, Newmark 1953, Beshara 1994) as an experimental modification to considered overpressure-time records. The wave form parameter, \( \alpha \) denotes a constant value of unity (Beshara 1994) and as a variable varying with the magnitude of the shock front (Kinney 1985). The time estimate ordinates \( (t_s, t_{s1}) \) required for triangular representation of overpressure-time curves are discussed by Crawford (1974) through graphs for 1 Mt surface burst.
3.4 Characteristics of conventional high explosives

Numerous research works exist for scaled blast parameters for conventional explosions. Tomlinson (1971) & Newmark (1961) provide shock front characteristics for incident and perpendicularly reflected waves for spherical pentolite charges exploded in free air. Kingery (1966) provides data for incident waves for surface bursts of TNT which are usually regarded as the standard waves for this scenario. Details for both air and surface bursts of TNT are discussed by Baker (1973) & Strehlow (1976).

A comparative study of the calculations of blast wave properties have been carried out by Newmark (1972)

When a high explosive ignites at the ground surface, at distances away from the volume detonated by the explosive itself, the maximum blast overpressure is given, in bars (TM5-855-1 (1965)) as

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

(3.16)

in which \( R \) = the distance in metres on ground surface from centre of detonation to the point of interest and \( W \) = the total energy of detonation measured in equivalent weight of metric tons of TNT.

According to Mills (1987) & Lees (1980), peak pressure on a structure in the distant area of an overpressure associated to the proximity factor, \( Z \), is as follows:

\[
P_s = \frac{1772}{Z^3} - \frac{114}{Z^2} + \frac{108}{Z} \text{kN/m}^2
\]

(3.17)

in which

\( W \) = the equivalent charge weight measured in kilograms of TNT. The overpressure-distance equation for conventional air burst is described as (Baker 1983)

\[
\frac{P_a}{P_0} = \frac{808 \left[ 1 + \left( \frac{Z}{4.5} \right)^2 \right]}{\left[ 1 + \left( \frac{Z}{0.048} \right)^2 \right] \left[ 1 + \left( \frac{Z}{0.32} \right)^2 \right] \left[ 1 + \left( \frac{Z}{1.35} \right)^2 \right]^{1/2}}
\]

(3.18)
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The proximity factor, \( Z \) = the real distance scaled to an energy discharge of 1 kt of TNT in the standard atmosphere.

As regards a surface burst, the total positive phase duration of blast overpressure, in millisec, is defined as (Beshara (1994))

\[ t_s = 10 W^{1/3} \quad (3.19) \]

The subsequent value for air burst, in millisec, is derived as (Baker 1983)

\[
\frac{t_s}{W^{1/3}} = \frac{980 \left[ 1 + \left( \frac{Z}{0.54} \right)^{10} \right]}{\left\{ 1 + \left( \frac{Z}{0.02} \right)^3 \left[ 1 + \left( \frac{Z}{0.74} \right)^6 \left[ 1 + \left( \frac{Z}{6.9} \right)^2 \right] \right] \right\}^{1/2}} \quad (3.20)
\]

It is an established fact that the pace of decrease of overpressure with time is much less for conventional blasts compared with nuclear explosions, the overpressure-time curve can be taken as the triangular demonstration in which the interruption on the time axis are defined in millisec, as (TM5-855-1 (1990))

\[
t_s = 3.20 \frac{W^{1/3}}{P_s^{1/8}}, \quad \text{for } P_s < 3.4 \text{ bar} \quad (3.21)
\]

\[
t_s = 6.21 \frac{W^{1/3}}{P_s^{7/8}}, \quad \text{for } P_s \geq 3.4 \text{ bar} \quad (3.22)
\]

\[
t_i = 10.23 \frac{W^{1/3}}{P_s^{1/2}}, \quad \text{for } P_s < 70 \text{ bar} \quad (3.23)
\]

\[
t_i = 20.77 \frac{W^{1/3}}{P_s^{2/3}}, \quad \text{for } P_s \geq 70 \text{ bar} \quad (3.24)
\]

The equation for conventional air burst can be employed to calculate records for surface burst by employing effective charge weight which considers ground reflections. The value of 1.8 has been recommended for conversion factor.

### 3.5 Unconfined Vapour Cloud Blast

Unconfined vapour cloud explosion is very important in the background of industrial and nuclear plants. It is the detonation of a cloud of flammable vapour. It is the consequence of a massive spill of a combustible hydrocarbon into the open environment, followed by
ignition and adequate acceleration of the flame progressing through the cloud to generate a critical shock wave. Excessive harm can occur in structures positioned hundreds of metres away from the cloud centre. There are two dissimilar mechanisms in the explosion (Mills 1987, Lees 1980, Lee 1977, Pfortaser 1977, Koch 1977, Gugan 1978) on the basis of the speed of the flame front, i.e. deflagration and detonation. The combustion velocity is of the order of 10 m/sec in the case of deflagration. In spite of numerous findings available on many dimensions of the unconfined vapour cloud explosions, enough work is still required.

The significant characteristics of vapour cloud explosions which distinguish them from a TNT explosion have been discussed by Lee (1977) & Pfortaser (1977).

The overpressure at the point source of the TNT explosion is more than the overpressure at the explosion center. The blast impact in the cloud can excessively vary on the basis of the combustion mode. It has been recommended (Lee 1977) that the practical upper limit of overpressure is approximately 1 bar at the center and about 0.7 bar at the boundary of the cloud. The profile of the early blast wave is dissimilar from that of concentrated charge explosion. It is understood that it is at a adequate distance from the source, it becomes impossible to differentiate in form from the wave of a TNT explosion.

The explosion source can be of very large size on the basis of fuel release and the delay before detonation. A vapour cloud is like a pancake shape which increases the spatial dimensions. The explosion energy is only a fraction of the total combustion energy of the cloud since a considerable constituent remains unutilized. The duration is usually more than that of the condensed phase explosion. The overpressure of 1 bar at the cloud centre, a duration time of 30 msec has been adopted (Baker 1983, Lee 1977) for the analysis and design of elements.

The available information for calculating the blast damages from chemical explosions are based on the TNT equivalent yield concept (Lee 1977, Pfortaser 1977, Koch 1977, Jungclaus 1977, Strehlow 1979). The other models presented by Lee (1977) are untested.

If $W_f$ (kg) of a certain fuel is discharged into the atmosphere and $\Delta H_f$ is the standard heat of combustion of this fuel in J/kg, then the TNT equivalent yield is given as

$$W_{TNT} = \alpha \Delta H_f W_f \frac{4.198 \times 10^6}{\Delta H_f W_f}$$

(3.25)
Where $\alpha = \text{experimental factor (} 0 < \alpha < 1 \text{)}$ and $4.198 \times 10^6$ is the explosion energy of TNT (J/kg). The experimental factor $\alpha$ is used to incorporate the differences between the two types of explosions.

The past experience (Pfortaser 1977) shows that $\alpha$ differ from a trivial fraction of a percent to values as high as 30%. A value of 0.05 to 0.10 may be employed as a rule. Once $W_{\text{TNT}}$ is found a characteristic explosion distance $R_o$ can be defined by

$$R_0 = \left( \frac{W_{\text{TNT}} \times 4.198 \times 10^6}{P_0} \right)^{1/3} \tag{3.26}$$

At a certain point $R_o$ from the centre of explosion, the energy scaled distance, $\overline{R}$ is defined as

$$\overline{R} = \frac{R}{R_0}$$

A methodical perusal of the influence of normal burning velocity on the shock wave generated by central ignition of a spherical cloud has been accomplished by Strehlow et al. (1979). The normal burning velocity of the fuel divided by the local velocity of sound was taken as a reference Mach number $M_{su}$. The result of the study was standard charts to illustrate the scaled overpressure $P_s$ depending on the energy-scaled distance. These graphical representations are illustrated in Figures 3.4 and Figure 3.5. In Figure 3.4, curves are given for deflagrative explosions with a range of normal burning velocities. The curve tag P is for pentolite, D is for detonation, and S is for open sphere rupture. The least value of $R = 0.01$ is taken as the effective wave width of the idealized spherical cloud. The peak wave overpressure and impulse are then calculated from the scaled values (Louca 2002) as

$$P_s = \overline{P}_s P_0 \tag{3.27}$$

$$i_s = i_s \left( W_{\text{TNT}} \times 4.198 \times 10^6 \right)^{1/3} \frac{P_0^{2/3}}{C_0} \tag{3.28}$$
Figure 3.4: Maximum wave overpressure versus energy-scaled distance for deflagrative explosions (Baker 1973)

Figure 3.5: Energy-scaled impulse versus energy-scaled radius for deflagration explosions (Baker 1983)
Using the triangular illustration of the overpressure-time curve, the evaluation of the overpressure duration time results in

\[ t_s = \frac{2i_s}{P_s} \quad (3.29) \]

### 3.6 Airflow and Reflection Process

The most negative impact of a blast wave is usually described by the maximum overpressure. Generally, depending on the structure geometries, the strong transient winds behind the shock front can be of larger consequence. The drag effects are dependent on the size and shape of the structure, and the maximum value of the dynamic pressure resulted from the wind behind the shock front. The shock front velocity, peak wind velocity and the density of the air behind the shock front are required to compute the peak value of dynamic pressure. The shock front progresses outward from the location of burst with a velocity which is dependent on the peak overpressure just behind the shock front and the ambient conditions of the air ahead of the shock wave. The front velocity, \( U_s \) at the point of interest is considered (Strehlow 1979) as

\[ U_s = C_u M_s \quad (3.30) \]

In expressions of shock velocity, the wind speed is specified (Louca 2002, Baker 1973, Baker 1983) as

\[ u_s = \frac{2}{1 + \gamma} \left( \frac{U_s^2 - C_o^2}{U_s} \right) \quad (3.31) \]

The maximum wind velocity and the air density, \( \rho_s \), behind the shock front, in expressions of overpressure, are articulated (Baker 1973) as

\[ u_s = \frac{C_o P_s}{\gamma \rho_0} \left[ 1 + \left( \frac{\gamma + 1}{2\gamma} \right) \frac{P_s}{P_0} \right]^{-1/2} \quad (3.32) \]

\[ \rho_s = \rho_0 \left[ \frac{(\gamma_h + 1)P_s + 2\gamma_h P_0}{(\gamma_h - 1)P_s + 2\gamma_h P_0} \right] \quad (3.33) \]

The dynamic pressure is related to the square of the wind velocity and the density of the air. The peak dynamic pressure, \( P_d \), is expressed (Newmark 1961, Newmark 1963,

\[ P_d = \frac{P_s^2}{2\gamma_b P_o + (\gamma_b - 1)P_s} \]  \hspace{1cm} (3.34)

Under ideal gas conditions (\(\gamma_b = 1.4\)), Eq. 3.34 reduces to

\[ P_d = \frac{5}{2} \left( \frac{P_s^2}{7P_o + P_s} \right) \]  \hspace{1cm} (3.35)

Eq. 3.35 is valid up to an overpressure of approximately 689 N/m\(^2\). In effect, it is likely that the predicted range at which a given dynamic overpressure owing to a nuclear burst event is computed consistently to within an error of 25%.

The dynamic pressure time arrival is measured (Newmark 1961, Newmark 1963, Newmark 1962, Crawford 1974, Beshara 1994, Strehlow 1976, TM5-855-1(1991)) to be equal as that of the peak overpressure. The dynamic pressure positive phase duration, \(t_q\) is articulated in seconds for 1 Mt surface nuclear burst (Newmark 1962) as

\[ t_q = \left[ \frac{4}{1 + 0.085P_o + 0.0075P_s^2} + \frac{0.077P_s}{1 + 0.00042P_s^2} + \frac{0.02662P_s}{1 + 0.011P_s} \right] W^{1/3} \]  \hspace{1cm} (3.36)

in which \(W\) = the explosion yield in megatonnes. Standards for other nuclear weapon yields can be obtained by the cube root rule. For conventional blasts, the drag pressure duration is defined in millisec (TM5-855-1 (1991)) as

\[ t_q = 20W^{1/3} \]  \hspace{1cm} (3.37)

in which \(W\) = the explosion yield in tonnes of equivalent TNT.

The difference of dynamic pressure with time is similar to the overpressure with an abrupt decrease but a longer period. It is generally expedient to employ an equivalent triangular pulse to characterize the dynamic pressure-time curve for overpressure. The peak dynamic pressure is employed as initial value of the equivalent pulse. The duration time and the drag impulse duration are illustrated graphically in Reference (Crawford 1974, Rogers
1959) for 1 Mt nuclear surface burst. The resultant values for conventional explosions, the duration is articulated in millisec (TM5-855-1 (1991)) as

\[ t' = 9.04 \left( \frac{W}{P_s} \right)^{1/3}, \quad \text{for } P_s < 2 \text{ bar} \]  
(3.38)

\[ t' = 14.35 W^{1/3} P_s^{-1/3}, \quad \text{for } P_s \geq 2 \text{ bar} \]  
(3.39)

### 3.7 Calculation of Reflected overpressure

Reflection is the result of a momentum alteration when the progressing airblast impinges upon surface in the way of propagation. The ratio of reflected overpressure to incident overpressure is termed the reflection factor (TM 5-1300 (1990), Agbabian 1985) which is a function of the peak overpressure in the incident wave and the angle at which the wave interacts with the surfaces. When the blast strikes an object at right angles, or nearly so, the resulting reflection creates a peak reflected overpressure, \( P_r \), given by (Lee 1977)

\[ P_r = 2P_s + (1 + \gamma_h)P_d \]  
(3.40)

Taking into consideration, the ideal gas conditions \( \gamma_h = 1.4 \) and replacement for \( P_d \) from Eq. 3.35, the peak reflected overpressure is defined as

\[ \frac{P_r}{P_s} = 2 + \frac{6P_s}{P_s + 7P_0} \]  
(3.41)

Eq. 3.41 is applicable for ideal gas for the overpressure, \( P_s \) less than 10 bars. The peak reflected overpressures given by Eq 3.40 can attain a value of twice the peak incident overpressure for shocks having less intensity in which the peak dynamic pressure is unimportant, but may attain a value of eight times the peak overpressure for strong shocks in which the peak dynamic pressure is the governing criteria (Tomlinson 1971, Strehlow 1976). This upper limit is possibly significantly inaccurate (Baker 1973, Lee 1977). It assumes that the air acts as a perfect gas even at the high pressures and temperatures obtainable under strong shock conditions. It has been shown (Doering 1949) that this ratio can be much greater, perhaps 20, if real gas effects such as dissociation and ionization of the molecules are incorporated. Brode (1959) has computed this ratio for normal reflection.
of shocks on the basis of air dissociation and ionization. For $P_s$ greater than 10 bars, he proposed the following relationship

$$\frac{P_r}{P_s} = 4 \log_{10} P_s + 1.5 \quad \text{for } P_r/P_s < 14$$

(3.42)

Eq. 3.40 or 3.41 provides only peak pressures with little emphasis on time histories of reflected pressure. Owing to absence of precise prediction procedures, it is recommended (Kingery 1966) that one can approximately calculate the reflected impulse, $i_r$, if the side-on impulse time is available. The relationship between the time histories of side-on overpressure and normally reflected overpressure is considered. This supposition provides

$$\frac{i_r}{i_s} = \frac{P_r}{P_s}$$

(3.43)

The real reflected overpressure-time history is visualized by an equivalent triangular pulse. The actual positive duration is substituted by a fabricated duration articulated as

$$t_{ir} = 2i_r/P_r$$

(3.44)

### 3.8 External Impulsive Loading on Superstructures

The following assumptions are made during the investigation of airblast loading on structures (Newmark 1963, Newmark 1962).

1. The loading is associated with an ideal shock in which the maximum overpressure is attained suddenly.
2. The structure lies in the Mach reflection region where the airblast front is advancing parallel to the ground surface. A structure positioned in the regular reflection region can be assumed by employing higher shock loads in the Mach reflection region to incorporate the incident blast wave reflections on the ground surface.
3. The initial shock loads on solid objects can be differentiated from the behaviour of the objects to the shock pressures.
4. The structures are assumed as rigid entities which produce processes such as shock reflection, diffraction and alteration of air flow behind the shock front. This is permissible owing to variation in elastic properties as well as in density between the air and concrete structural members.
The methodology (Strehlow 1976) for the determination of the external blast loads is restricted to rectangular superstructures where the structures will experience a plane shock front. Nevertheless, the procedures can be applied to structures of other configurations. There are numerous approximations inherent in the evaluation of blast loads as well as in the interaction process between the blast wave and the structure. It is suggested (Baker 1983, Newmark 1961, Newmark 1962, Newmark 1963, Baker 1973, Glasstone 1977, Carpenter 1975, Crawford, 1974, Agbabian 1985) that the shock effects in the incidental and reflected shock waves may be taken as equivalent triangular pulses of identical impulses. Each pulse has a maximum pressure value identical to the actual blast effect and fabricated durations discussed before as functions of the impulse and the peak pressure.

The pressures vary with respect to time in the mode shown in Figure 3.6 in the front face of a superstructure. There is a sudden rise time the peak reflected pressure \( P_r \), pursued by an abrupt decrease as the high reflected pressure produces movement around the structure. The decrease of the reflected pressure to the side-on overpressure plus dynamic pressure occurs in a clearing time, \( t_c \), which is related to the shock velocity \( U_s \) and the shortest distance from the point on the structure where the lessening in pressure happens most slowly. The clearing time is specified by the relation (Crawford 1974, Kinney 1985)

\[
t_c = \frac{3S}{U_s} \quad (3.45)
\]

Where \( S \) is the smaller of the height of the structure or one-half the structure width, which ever is a smaller amount. The pressure comes to zero after the clearing time with the decrease in side-on overpressure and dynamic pressure. In this decay phase, the peak pressure is specified (Rogers 1959, Baker 1983, Kinney 1985, Biggs 1964, Norris 1959, Newmark 1953, Beshara 1994)

\[
P = P_s + C_f P_d \quad (3.46)
\]

\( C_f \) is the drag coefficient for the front face. It usually lies in value from 0.8 to 1.5 and may be taken as 1.2 (TM5-855-1 (1991)). The values of \( t_i \) and \( t_i' \) shown in Figure 3.6 are taken as the fabricated durations of the dynamic pressure and incident overpressure, respectively.

The rise time, \( T_{rr} \) to the peak pressure is minute and can be generally ignored.

The effect on the back side of an aboveground rectangular structure is illustrated in Figure 3.7. No pressure is spread to the back face until the shock front arrived at that face. Employing the same time reference as for the front face, average force starts to build up on
the back face at a time equal to the length of the structure (L) in the direction of the shock propagation divided by the velocity of shock propagation. After a certain duration, the rear face is totally surrounded in the blast, the pressure reaches the peak value equal to the side-on overpressure minus the drag pressure. It serves as a suction on the rear side.

\[ t_b = \frac{3S}{U} \]

\[ T_{rb} = \frac{L + 5S}{U_s} \]
The drag coefficient for the rear surface $C_r$, generally ranges from about 0.5 for low pressures to less than 0.3 for high pressures and may be adopted as 0.4 generally (TM5-855-1 (1991)). The overall transverse force on the structural member as a whole is the difference between the front and rear face loading. It must be ensured that the pressure values at the time ordinates of the loading histories may be subtracted.

After crossing the blast wave over the structure, the roof loading at a certain duration is equal to the incident overpressure minus the negative drag pressure or suction related with the movement of air around the structure. The average roof loading is visualized as a triangular load (TM5-1300 (1990)) with rise time specified by

$$T_{rr} = T_{rf} + \frac{L}{U} \quad (3.48)$$

The maximum value of the pressure is defined (TM5-1300 (1990), Agbabian 1985) as

$$P = P_s - C_d P_d \quad (3.49)$$

and the extent of the equivalent pulse is equivalent to

$$T_{ds} = T_{rr} + T_i \quad (3.50)$$

The drag coefficient for the roof $C_t$ is taken equivalent to that of the rear side of the structure.

The scenarios where the front and rear of a structure are divided by enough distance so that it characterizes a substantial scaled distance, the shock wave at the front and the rear may vary on the basis of their fundamental parameters. Therefore, different loading-time histories are calculated for the front and rear faces.

### 3.9 Dynamic Response of Blast Loaded structures

The dynamic analysis of blast-loaded structures involves many complications e.g., the effect of high strain rates, the non-linear plastic material behavior, the uncertainties of blast load calculations and the time-dependent deformations. Accordingly, numerous suppositions related to the response of structures and the forces have been suggested and generally approved to execute the analysis. The structure is visualized as a single degree of freedom (SDOF) system and the relation between the positive duration of the blast load and the natural period of vibration of the structure is created to analyze comprehensively. This guides to blast load idealization and makes the classification of the blast loading domains quite simple.
3.9.1 Elastic SDOF System

The method demonstrates that a particular structural element such as beam can be modeled as a lumped mass and a spring with either a linear or non-linear response. The modeling depends on the kinematic similarity such that the displacement and velocity of the equivalent system is same as the actual system. An illustration of the idealization is given in the Figure 3.8. The structural mass, \( M \), is under the effect of an external force, \( F(t) \), and the structural resistance, \( R \), is expressed in terms of the vertical displacement, \( y \), and the spring constant, \( K \).

The blast load can also be visualized as a triangular pulse having a peak force \( F_m \) and positive phase duration \( t_d \). The forcing function is expressed as

\[
F(t) = F_m \left(1 - \frac{t}{t_d}\right)
\] (3.51)

The blast impulse is calculated as the area under the force-time curve, and is given by

\[
I = \frac{1}{2} F_m t_d
\] (3.52)

The equation of motion of the un-damped elastic SDOF system for a time spanning from 0 to the positive phase duration, \( t_d \), is stated as (Biggs 1964)

\[
M\ddot{y} + Ky = F_m \left(1 - \frac{t}{t_d}\right)
\] (3.53)
The general solution can be demonstrated as:

**Displacement**

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

(3.54)

**Velocity**

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

(3.55)

\(\omega = \) the natural circular frequency of vibration of the structure

\(T = \) the natural period of vibration of the structure which is stated as Eq. 3.56

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

(3.56)

The peak response is defined by the maximum dynamic deflection \(y_{\text{m}}\) which occurs at time \(t_{\text{m}}\). The peak dynamic deflection \(y_{\text{m}}\) can be solved by setting \(dy/dt\) in Eq 3.55 equal to zero, i.e. when the structural velocity is zero. The dynamic load factor, DLF, is the ratio of the maximum dynamic deflection \(y_{\text{m}}\) to the static deflection \(y_{\text{st}}\) which may occur from the static application of the peak load \(F_m\). It is described as follows:

\[ \text{DLF} = \frac{y_{\text{max}}}{y_{\text{st}}} = \frac{y_{\text{max}}}{F_m} = \psi(\omega t_d) = \psi \left( \frac{t_d}{T} \right) \]  

(3.57)

The structural response to blast loading is considerably affected by the ratio \(t_d/T\) or \(\omega t_d (t_d/T = \omega t_d / 2\pi)\). Three loading regimes are classified

- \(\omega t_d < 0.4\) : impulsive loading regime.
- \(\omega t_d > 0.4\) : quasi-static loading regime.
- \(0.4 < \omega t_d < 40\) : dynamic loading regime.

### 3.9.2 Elasto-Plastic SDOF Systems

Structures exhibits large nonlinear deformation under shock load or high velocity impact. Precise evaluation of dynamic response is only feasible by step-by-step numerical solution with the help of a nonlinear dynamic finite-element software. Nevertheless, the extent of ambiguity in both the loading calculation and the elucidation of the consequent deformation is such that solution of a assumed equivalent ideal elasto-plastic SDOF...
system (Biggs 1964) is generally employed. Explanation is based on the required ductility factor \( \mu = \frac{y_m}{y_e} \). (Figure 3.9).

![Resistance vs. Deflection graph](image)

**Figure 3.9: Simplified Resistance Function of an Elasto-Plastic SDOF System**  
*(Biggs 1964)*

For instance, a uniform simply supported beam has first mode shape \( \varphi(x) = \sin \frac{\pi x}{L} \) and the equivalent mass \( M = \frac{1}{2}mL \), where \( L \) = span of the beam and \( m \) = mass per unit length. The corresponding force related to a uniformly distributed load of intensity \( p \) is \( F = \frac{2}{\pi}pL \). The behaviour of the ideal bilinear elasto-plastic system can be determined in closed form for the triangular load pulse consisting of sudden rise and linear decrease, with maximum value \( F_m \) and duration \( t_d \). The outcome for the maximum displacement is usually displayed in chart form (TM5-1300 (1990)), as a family of curves for selected values of \( R_u/F_m \) displaying the required ductility \( \mu \) as a function of \( t_d/T \). \( R_u \) represents structural resistance of the beam and \( T \) denotes the natural period (Figure 3.10).

![Chart for maximum response](image)

**Figure 3.10: Maximum response of elasto-plastic SDF system to a triangular load**  
*(T. Ngo 2007)*
3.10 Damage Mechanism at High Strain rate

Shock loads normally generate high strain rates in the range of $10^2$ - $10^4$ s$^{-1}$. This high loading rate modifies the dynamic mechanical properties of target structures. Correspondingly, the expected damage mechanisms for different structural members also vary. The strength of concrete and steel reinforcing bars can enlarge appreciably owing to strain rate effects for R.C.C. structures subjected to shock loads. Figure 3.11 demonstrates the estimated ranges of the probable strain rates for various loading conditions. It is observed that ordinary static strain rate lies in the range : $10^{-6}$-$10^{-5}$ s$^{-1}$. The blast pressures generally produce loads related with strain rates in the range : $10^2$-$10^4$ s$^{-1}$

3.10.1 Concrete under High-Strain Rates

The mechanical characteristics of concrete under dynamic loading conditions can vary from that under static loading. While the dynamic stiffness does not differ considerably from the static stiffness, the stresses that exist for a certain period of time under dynamic conditions may be extremely higher than the static compressive strength (Figure 3.12). Strength magnification factors upto 4 in compression and up to 6 in tension for strain rates in the range : $10^2$–$10^3$ /sec have been calculated (CEB-FIP(1990)).

![Figure 3.11: Strain rates associated with different types of loading (TM5-1300)](image)
Figure 3.12: Stress-strain curves of concrete at different strain rates (Grote 2001)

A dynamic increase factor (DIF) is incorporated in the CEB-FIP (1990) model (Figure 3.13) for strain-rate enhancement of concrete for the increase in peak compressive stress ($f'_c$), as follows:

$$\text{DIF} = \left( \frac{\dot{\varepsilon}}{\dot{\varepsilon}_s} \right)^{1.026\alpha} \quad \text{for } \dot{\varepsilon} \leq 30 \text{s}^{-1}$$  \hspace{1cm} (3.58)

$$\text{DIF} = \gamma \left( \frac{\dot{\varepsilon}}{\dot{\varepsilon}_s} \right)^{1/3} \quad \text{for } \dot{\varepsilon} > 30 \text{s}^{-1}$$  \hspace{1cm} (3.59)

where

$\dot{\varepsilon} = \text{strain rate}$

$\dot{\varepsilon}_s = 30 \times 10^{-6} \text{ s}^{-1} \text{ (quasi-static strain rate)}$

$\log \gamma = 6.156 \alpha^{-2}$

$\alpha = 1/(5+9f'_c/f_{co})$

$f_{co} = 10 \text{ MPa} = 1450 \text{ psi}$

### 3.10.2 Reinforcing Steel under High-Strain Rates

The elastic and plastic response to dynamic loading owing to the isotropic characteristics of metallic materials can easily be examined and evaluated. Norris et al. (1959) tested
steel with two different static yield strength of 330 and 278 MPa under tension at strain rates ranging from $10^{-5}$ to 0.1 s$^{-1}$. Strength magnification of 9 - 21% and 10 - 23 % were measured correspondingly for the two steel types, respectively. Dowling and Harding (1967) performed tensile experiments using the tensile version of Split Hopkinton's Pressure Bar (SHPB) on mild steel using strain rates ranging between $10^{-3}$ s$^{-1}$ and 2000 s$^{-1}$. The conclusion was drawn from this test series that materials of body-centered cubic (BCC) structure (such as mild steel) exhibited the greatest strain rate sensitivity. It has been observed that the lower yield strength of mild steel can increase by 100%; the ultimate tensile strength can be augmented by about 50%; and the upper yield strength can significantly increase. In juxtaposition, the ultimate tensile strain decays with increasing strain rate.

\[
\text{Figure 3.13: Dynamic Increase Factor for peak stress of concrete (CEB-FIP-1990)}
\]

Malvar (1983) investigated strength increase of steel reinforcing bars under high strain rates. This was expressed in terms of the dynamic increase factor (DIF), which can be assessed for various steel grades and for yield stresses, $f_y$, ranging from 290 to 710 MPa as expressed by Eq 3.60

\[
\text{DIF} = \left( \frac{\dot{\varepsilon}}{10^{-4}} \right)^\alpha
\]  

(3.60)

Where for yield stress calculation, $\alpha = \alpha_{f_y}$

\[
\alpha_{f_y} = 0.074 - 0.04 \left( \frac{f_y}{414} \right)
\]  

(3.61)

for ultimate stress calculation $\alpha = \alpha_{f_u}$
\[ \alpha_{fu} = 0.019 - 0.009 \left( f_y/414 \right) \] (3.62)

3.11 Failure Modes Associated with Blast Loading

Shock loading effects on structures may create both local and global responses related with various failure modes. The nature of structural response relies mostly on the loading rate, the orientation of the target with respect to the direction of the blast wave propagation and boundary conditions. The normal failure modes related with blast loading can be flexure, direct shear or punching shear. Local responses are illustrated by localized bleaching and spalling. They occur owing to the close-in effects of explosions, while global responses are normally exhibited as flexural failure.

3.11.1 Global response of Structural elements

The global response of structural elements is usually the outcome of out-of-plane loads with quasi-static loading, and is generally associated with global, bending and shear responses. For that reason, the global response of reinforced concrete superstructures owing to blast loading is described as membrane/bending failure.

The second global failure mode is shear failure. Four types of shear failure are possible under the application of both static and dynamic loading: diagonal tension, diagonal compression, punching shear, and direct shear. The first two types are frequent in reinforced concrete elements under static loading while punching shear is related with local shear failure. The well-known illustration of this phenomenon is column punching through a flat slab. These shear response processes have comparatively inconsequential structural effect in case of blast loading and can be ignored.

The shear failure mode is first and foremost related with transient short duration dynamic loads that occur from shock effects, and it relies on the strength of the pressure waves. The related shear force is manifold higher than the shear force related with flexural failure modes. The high shear stresses may occur in direct global shear failure and it may happen instantly before any happening of related considerable strains.

3.11.2 Localized Response of Structural Elements

The close-in effect of blast may result localized shear or flexural failure in the nearby structural members. This is dependent on the distance between the source of the explosion and the target, and the comparative strength/ductility of the structural members.
localized shear failure occurs in the form of localized punching and spalling, which generate low and high-speed debris. The punching effect is commonly referred to as bleaching. It is frequent in high velocity impact applications and the case of explosions near the surface of structural members. Bleaching failures in general occur simultaneously by spalling and scabbing of concrete covers as well as fragments and debris. (Figure 3.14).

3.11.3 Pressure-Impulse (P-I) Diagrams
The pressure-impulse (P-I) diagram is generally used to numerically describe a exact damage level to joint blast pressures and impulses applied on a specific structural member. An example of a P-I diagram is shown in Figure 3.15 to illustrate levels of damage of a structural element.
Region (I) describes significant structural damage and region (II) displays no or minor damage. There are other P-I diagrams that relates with human response to blast in which case there are three categories of blast-induced injury, namely: primary, secondary, and tertiary injury.

3.12 Blast Wave-Structure Interaction
The structural response of a structural member subjected to shock wave may be evaluated in two phases.
Firstly, blast-loading effects, i.e., pressures that are generated directly from the action of the shock pressure;
Secondly, the structural behaviour, or the probable damage criteria related with such loading effects.
Figure 3.14: Breaching failure due to a close-in explosion of 6000 kg TNT equivalent (Grote 2001)

Figure 3.15: Typical pressure-impulse (P-I) diagram (Biggs 1964)
It is imperative to visualize the application of the blast waves with the target structures. It is complex in the case of complicated structural scenarios. It is likely to utilize a SDOF model. Consequently, two types of target structures can be analyzed to determine the dynamic response to blast loading, diffraction-type and drag type structures. As is evident, the earlier would be influenced mainly by diffraction loading and the second by drag loading. It is stressed that actual structures will respond to both types of loading and the difference is observed to carry out the analysis. The structural behaviour is based on the size, shape and weight of the target, how tightly it is fastened to the ground, and also on the availability of openings in each face of the structure. Above ground or shallow-buried structures can be exposed to ground shock owing to ignition of explosives that are at the ground surface. The energy passed on to the ground by the explosion is the chief cause of ground shock. A portion of this energy is directly conveyed through the ground as directly-induced ground shock, while fraction is conveyed through the air as air-induced ground shock. Air-induced ground shock occurs when the air-blast wave applies pressure on the ground surface and transmits a stress pulse into the ground under layers. Normally, motion due to air-induced ground shock is peak at the ground surface and decays with depth (TM5-1300 (1990)). The direct shock is the outcome of the direct transmission of explosive energy through the soil. The net ground shock happens owing to a combination of both the airinduced and direct shocks with reference to a certain location.

### 3.12.1 Air-induced Ground Shock Loading

One-dimensional wave propagation theory has been forwarded in order to resolve complications of predicting actual ground motion, to calculate the maximum displacement, velocity and acceleration in terms of the already known blast wave parameters (Agbabian 1985). The maximum vertical velocity at the ground surface, $V_v$, is stated in terms of the peak incident overpressure, $P_{so}$, as:

$$V_v = \frac{P_{so}}{\rho C_p}$$  \hspace{1cm} (3.63)

where $\rho$ and $C_p$ are, respectively, the mass density and the wave seismic velocity in the soil.

The integral of Eq 3.63 with time, the maximum vertical displacement at the ground surface, $D_v$, can be derived as:
\[ D_v = \frac{i_x}{1000 \rho C_p} \]  

(3.64)

Considering the depth of soil layers, a numerical equation is provided to calculate the vertical displacement in meters so that

\[ D_v = 0.09W^{\frac{1}{5}}(H / 50)^{0.6}(P_{wo})^2 \]  

(3.65)

where \( W = \) the explosion yield in \(10^9\) kg, and \( H = \) the depth of the soil layer in meters.

### 3.12.2 Direct Ground Shock Loading

The ground surface experiences vertical and horizontal motions owing to direct transmission of the explosion energy. Some experimental equations were established (TM5-1300 (1990)) to calculate the direct-induced ground motions in three different ground media; dry soil, saturated soil and rock media.

The peak vertical displacement in m/s at the ground surface for rock, \( D_{v_{rock}} \) and dry soil, \( D_{v_{dry}} \) are expressed as

\[ D_{v_{rock}} = \frac{0.25R^\frac{1}{3}W^\frac{1}{3}}{Z^\frac{1}{3}} \]  

(3.66)

\[ D_{v_{dry}} = \frac{0.17R^\frac{1}{3}W^\frac{1}{3}}{Z^{2.3}} \]  

(3.67)

The peak acceleration, \( A_v \), in vertical direction, in m/s² for all ground media is stated as

\[ A_v = \frac{1000}{W^{\frac{1}{8}}Z^2} \]  

(3.68)

### 3.13 Blast-Resistant Design Manuals

This section provides review of the generally used military design manuals and computational techniques to calculate blast loads and the responses of structural systems. Most of these design guidelines deal with military applications. Their know how is essential for civil engineers.
Structures to Resist the Effects of Accidental Explosions, TM 5-1300 (U.S. Departments of the Army, Navy, and Air Force, 1990). This manual is the most popular document by both military and civilian organizations for the analysis and design of structures against explosions and to protect personnel and precious equipment. It incorporates step-by-step analysis and design procedures, including information on such items as (1) blast, fragment, and shock-loading; (2) principles of dynamic analysis; (3) reinforced and structural steel design; and (4) a number of special design considerations. It includes literature on tolerances and fragility, as well as shock isolation. Enough information is available for selection and design of security windows, doors, utility openings, and other components against blast and forced-entry effects.

A Manual for the Prediction of Blast and Fragment Loadings on Structures, DOE/TIC-11268 (U.S. Department of Energy, 1992). This manual provides information to the designers of facilities against accidental explosions. It is used for the evaluation of the explosion-resistant capabilities of existing structures.

Protective Construction Design Manual, ESLTR-87-57 (Air Force Engineering and Services Center, 1989). This manual demonstrates methodologies for the analysis and design of protective structures against the effects of conventional (non-nuclear) weapons and is useful for engineers with fundamental knowledge of weapons effects, structural dynamics, and hardened protective structures.

Fundamentals of Protective Design for Conventional Weapons, TM 5-855-1 (U.S. Department of the Army, 1986). This manual provides methodologies for the design and analysis of protective structures against the impact of conventional weapons. It is useful for engineers occupied in designing hardened facilities.


Structural Design for Physical Security—State of the Practice Report (ASCE, 1995). This report is a wide-ranging guide for civilian designers and planners who considers physical security parameters into the building retrofitting works.
3.14 Computer Programs for Blast and Shock Effects (Grote 2001)

The numerical softwares for blast calculation and structural response employ first-principle and semi-empirical methods. Programs utilizing the first principle method can be classified into uncouple and couple analyses. The uncouple analysis determines shock loads as if the structure were inflexible and then applying these loads to a numerical simulation of the structure. However, when the shock field is obtained with a rigid structural simulation, the loads on the structure are often over-estimated, mainly if considerable motion or failure of the structure takes place during the loading phase.

The blast simulation module is associated with the structural response module for a coupled analysis. Accordingly, the Computational Fluid Mechanics model for blast-load calculation is solved concurrently with the Computational Solid Mechanics model for structural response. By considering the structural motion during the blast computation, the pressures occur owing to motion. Accordingly, failure of the structure can be calculated more precisely. Examples of this type of computer codes are AUTODYN, DYNA3D, LSDYNA and ABAQUS. Table 3.1 provides a list of computer programs that may be employed to simulate blast-effects on structural components. CONWEP has been used in the present research work.

3.15 Scope and Review of Computer Programs

Computer programs for the calculation of blast-effects can be subdivided in two groups: first-principle and semi-empirical. In first-principle programs, mathematical equations are solved that describe the basic laws of physics governing a particular problem. These principles are conservation of matter, momentum, and energy. In addition, mathematical relationships i.e. constitutive equations, which describe the physical behavior of materials, are required. If these equations are solved accurately with suitable mathematical models, they should calculate the blast loads and structural response. However, there are certain limitations to accurate prediction of the effects of an explosion through the use of first-principle programs. Among them are the following:

- During calculation of blast due to explosions in air, the response of the air often involves complex phenomena, e.g. dust-air mixtures, boundary effects, and turbulence. Turbulent flow, for instance, cannot be calculated without the addition of models governed by empirical parameters.
Calculation of the pressures imparted by a detonating explosive on the structure involves multiscale occurrence that are difficult to visualize.

In the calculation of structural failure, the behavior of the materials is neither well understood nor readily characterized. Also, accurate constitutive equations are not available for the materials, mostly in fracture or fragmentation. These deficiencies in first-principle codes may be adjusted through engineering judgment. However, the main objective of first-principle techniques is to provide predictions in new domains where the experience that makes engineering judgment possible is not available.

Semi-empirical computational methods are based on simplified models of physical phenomena. These are developed through analysis of test results and application of engineering judgment. These methods rely on extensive data and case studies. They involve fewer equations and require less computer time, which makes them more practical than first-principle codes for design purposes.

Computer models and programs have become vital in engineering design and development. The complexity and dependability of the models varies considerably. In order to provide a review for blast programs, the following paragraph describes several classifications.

A significant classification of computer simulation models and analyses is whether the governing equations and response are linear or nonlinear. Linear analysis is applicable when the displacements of a structure or medium are small and the stresses can be related to the deformation by linear relationships.

Examples of linear analyses include acoustic-wave propagation and stress analyses of structures and machines under normal operating loads. In a linear analysis, there is generally no need for validation of a computer program. It is due to the fact that the equations are vigorous, and modern computer programs can accurately predict the linear behavior if sufficient resolution is achieved in the model.

Nonlinear analysis, however, is needed when the displacements of a medium or structure are large and when the strains and stresses exceed the range in which linear relationships hold. Behavior such as fracture, fragmentation, and flow due to high-pressure sources is also nonlinear. Thus the problems of blast evolution and structural response are highly nonlinear and there are many mathematical and physical complications and phenomena whose underlying physics are not well understood. For the nonlinear computations required for most blast-effects problems, validation of computer programs by experiments
in similar scenarios is vital. Even the more highly developed and user-friendly programs are not readily usable by non experts for several reasons:

- Many parameters need to be input, including artificial viscosities and choices among methodologies, and their selection requires considerable experience.
- The development of inputs requires construction of detailed finite- element or finite-difference models.
- Substantial skill is required to evaluate the output, both as to its correctness and its appropriateness to the situation modeled. Without such judgment, it is possible through a combination of modeling errors and poor interpretation to obtain wrong or meaningless results.

Therefore, true computational modeling of specific scenarios by engineers unfamiliar with these programs is difficult, if not unfeasible. Current research is seeking ways to make these and other sophisticated computer programs more perceptive and user-friendly. (SAIC, 1994). Such tools would make the programs developed for blast-effects simulation more handy for engineers who are not experts in the programs themselves.

Another hindrance to the use of these programs is the magnitude of computational resources required. Some of the simulations take 10–90 hours on the most powerful supercomputers. Small-scale, two-dimensional calculations can often be executed on workstations in a matter of minutes or hours. However, the time required grows rapidly as the model increases in detail. A concise commentary about the computer programs listed in Table 3.1 are as follows.

BlastX code (Version 3.0) (SAIC 1994) is a semi empirical computer code. It calculates the propagation of blast shock waves and detonation product gases in multi room structures. The code provides calculation of the pressure time and temperature time histories in these structures. The 3.0 version includes

- A variety of room shapes that may be used throughout a structure
- An interactive menu driven input module
- An enhanced version of the burning, venting and wall failure models from the Naval Surface Warfare Center INBLAST code.
- Failure models using the total shock and quasi static gas pressure on a wall.
- Heat detonation to walls
- A more accurate model of shock propagation through openings
- Modelling of blast effects within and outside of explosive storage magazines.
Table 3.1: Examples of Computer Programs Used To Simulate Blast Effects and
Structural Response (Ngo 2007)

<table>
<thead>
<tr>
<th>Name</th>
<th>Purpose</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>BlastX</td>
<td>Blast prediction</td>
<td>Semi empirical</td>
</tr>
<tr>
<td>CTH</td>
<td>Blast prediction</td>
<td>First Principle</td>
</tr>
<tr>
<td>FEFLO</td>
<td>Blast prediction</td>
<td>First Principle</td>
</tr>
<tr>
<td>FOIL</td>
<td>Blast prediction</td>
<td>First Principle</td>
</tr>
<tr>
<td>SHARC</td>
<td>Blast prediction</td>
<td>First Principle</td>
</tr>
<tr>
<td>DYNA3D</td>
<td>Structural response</td>
<td>First Principle</td>
</tr>
<tr>
<td>ALE3D</td>
<td>Coupled analysis</td>
<td>First Principle</td>
</tr>
<tr>
<td>LS-DYNA</td>
<td>Coupled Analysis</td>
<td>First Principle</td>
</tr>
<tr>
<td>Air3D</td>
<td>Blast prediction, CFD code</td>
<td>First Principle</td>
</tr>
<tr>
<td>CONWEP</td>
<td>Blast prediction</td>
<td>Empirical</td>
</tr>
<tr>
<td>AUTODYN</td>
<td>Structural response + CFD (Couple analysis)</td>
<td>First Principle</td>
</tr>
<tr>
<td>ABAQUS</td>
<td>Structural response + CFD (Couple analysis)</td>
<td>First Principle</td>
</tr>
</tbody>
</table>

FEFLO is a general-purpose Computational Fluid Dynamics code based on adaptive/unstructured grids. It deals all aspects of a comprehensive Computational Fluid Dynamics capability: gridding, solvers, mesh movement techniques, effective use of supercomputer architectures. FEFLO was conceived as a general-purpose CFD code based on the following general principles:

- Use of unstructured grids (automatic grid generation and mesh refinement)
- Finite element discretization of space
- Separate flow modules for compressible and incompressible flows
- Formulation for moving grids
- Edge-based data structures for speed
- Optimal data structures for different architectures

One of the distinguishing features of FEFLO has been its ability to carry out automatic element removal, remeshing and interpolation for problems with considerable body/surface motion.
The year 2001 has seen a number of important developments and improvements for FEFLO (Rainold et al. 2002). Notable among these are:

i. Boolean operations of discrete surfaces;
ii. Advances in the gridding of surfaces given as discrete data;
iii. Improvements in grid system
iv. Inclusion of vorticity confinement to track vortices over large distances;
v. Combination of Baldwin-Lomax and Smagorinsky turbulence models;
vi. Arbitrary-gas equation of state lookup;
vii. Combustion modeling;
viii. Local remeshing
ix. Embedded grids for complex fluid-structure interaction applications;
x. Complete port to OpenMP;
xi. Port to NEC-SX5;
xii. New renumbering techniques to minimize indirect addressing.
xiii. Advances in general design methodologies;

ALE3D is a three-dimensional finite-element code that utilizes arbitrary Lagrangian-Eulerian techniques to simulate fluid dynamics and elastic-plastic response on an unstructured mesh. The grid consists of arbitrarily connected hexahedra, beam, and shell elements. The mesh can be constructed from disjoint blocks of elements which interact at the boundaries via slide surfaces. ALE3D is currently being applied to a number of studies involving the effects of explosive events.

LS-DYNA is an explicit and implicit finite element program for analyzing the nonlinear dynamic response of structures. LS-DYNA has many features to simulate the physical behavior of 2D and 3D structures. nonlinear dynamics, thermal, failure, contact, quasi-static, Eulerian, Arbitrary-Lagrangian-Eulerian (ALE), Fluid-Structure-Interaction (FSI), multi-physics coupling, etc. LS-DYNA's most important application areas are crashworthiness analysis and sheet metal forming analysis. Besides, LS-DYNA is broadly used to simulate impacts on structures from drop tests, underwater shock, explosions or high-velocity impacts. Explosive forming, process engineering, accident reconstruction, vehicle dynamics, thermal brake disc analysis or nuclear safety are further areas in the broad range of possible applications. LS-DYNA simulates impact analysis of different containers. Sizes range from small shock absorber castings to heavy nuclear containers or spent fuel tanks. Cross-sections vary from impact of thin-gauge steel storage drums to thick-walled transport containers. Impact simulations are not only limited to consumer and industrial goods. High-risk buildings, such as nuclear plants, embankment dams or bridges
are investigated with respect to impact, from aircrafts for example. Simulation is of great help in these scenarios as it is impossible to carry out real tests.

AIR3D is a version of the ground-water flow code MODFLOW to simulate three dimensional air-flow in a heterogeneous, anisotropic unsaturated zone. Although the code was developed principally for this purpose, it can also be employed to simulate natural air flow in the unsaturated zone resulted due to atmospheric-pressure variations.

AUTODYN is an explicit software package for non-linear dynamics. It incorporates finite element analysis and computational fluid dynamics.

ABAQUS is a commercial software package for Finite Element Analysis (FEA). Abaqus/Standard is a general-purpose solver using a conventional implicit integration scheme to resolve finite element analyses. Abaqus is used in the automotive, aerospace, and industrial product industries. The product is popular in academic and research circles due to the wide material modeling capability, and the program's ability to be customized. ABAQUS was initially designed to address non-linear physical behavior; as a result, the package has an extensive range of material models. ABAQUS includes the following features:

Capabilities for both static and dynamic problems
  o Modelling of very large shape changes in solids, in both two and three dimensions
  o The library includes a full set of continuum elements, beam elements, shell and plate elements, among others.
  o Capability to model contact between solids
  o Advanced material library which includes the usual elastic and elastic – plastic solids; models for foams, concrete, soils, piezoelectric materials, and many others.
  o Capabilities to model a number of phenomena of interest, including vibrations, coupled fluid/structure interactions, acoustics, buckling problems, and so on.

The ABAQUS shell element library provides elements that permits the modeling of curved, intersecting shells that can display nonlinear material response.
Chapter 4

Experimental Evaluation of Impulsive Loading on Concrete Structure

4.1 Introduction
The purpose of this experimental work is to establish relationships between different parameters of ground shock and airblast against impulsive loading. These parameters are peak air pressure, peak reflected airblast pressure along the height of the structure, peak ground acceleration, arrival time of ground shock, duration of ground shock, time lag between ground shock and airblast pressure reaching the structure.

For this purpose, a 1/10th scaled model of reinforced concrete containment structure was constructed and subjected to explosions of various charge weight at varying distances. Using these relationships of 1/10th scale model, the structural safety analysis of a typical reactor containment has been done in Chapter 5. The generation and the effects of blast wave on the shell structure in the plastic range are discussed. Critical distances have been determined for different amount of blast charges for a typical shell structure.

The developed methodology of analysis may be adopted for evaluation of the effect of an external explosion on the reinforced concrete containments of other reactors.

The importance of the design of critical structures against terrorist or military attack has been under consideration in recent times. This work deals with the scenario of surface explosions at various distances against a structure. A surface explosion generates both ground shock and airblast pressure on nearby structures. The blast loading on a structure caused by a high-explosive detonation is dependent upon several factors:

1. The magnitude of the explosion.

2. The location of the explosion relative to the structure in question
   - i.e unconfined or confined explosions. The unconfined explosions include free air burst explosion, surface burst explosion. The confined explosions include fully vented explosions, partially confined explosions, fully confined explosions.
(3) The geometrical configuration of the structure.
(4) The structure orientation with respect to the explosion and the ground surface.

A charge located on or very near the ground surface is considered to be a surface burst. The initial wave of the explosion is reflected and reinforced by the ground surface to produce a reflected wave. Unlike an air burst, the reflected wave merges with the incident wave at the point of detonation to form a single wave, similar in nature to the mach wave of the air burst but essentially hemispherical in shape (Figure 4.1).

The basic characteristics of ground shocks induced by surface explosions are short duration, large amplitude and high frequency excitations (Dhakal 2003). The response in the forced vibration phase includes high frequency vibration modes with small displacements but large accelerations, thus inducing high inertial shear force. However, the free vibration response is dominated by lower frequency modes with larger displacements but smaller accelerations.

![Figure 4.1: Surface Burst Blast Environment (TM5-1300 (1990))](image)

### 4.2 Effect of Structural Configuration

The available empirical relations (Henrych 1979, Baker et al 1983, TM5-855-1 (1991), Bulson 1997) for structural analysis against impulsive loading have limitations and are usually inconvenient to use. The pressure is also assumed uniformly distributed along the
such simplifications are expected to introduce errors in estimation of air
blast loads on structures with curved surfaces also e.g. domes, arches, cylindrical vessels, etc.

4.3 Defining the threat

To resist blast loads, the first requirement in the assessment of a structure is to determine the
threat.

The threat for a conventional bomb is defined by two equally important elements, the bomb
size, or charge weight, and the standoff distance, the minimum guaranteed distance between
the blast source and the target. To standardize the criteria, the charge weight of an explosive
device is defined in terms of equivalent Trinitrotoluene (TNT) weight. The relative effect on
pressure and impulse can be scaled to an equivalent amount of TNT.

4.4 Scaling Effects

Scale-model experiments may be used to study the explosions effect, but scaling effects must
be considered in interpreting the results. A brief summary of the relationships between scale-
model (model dimension = scale factor S times full scale-dimension) and full scale
parameters is given in Table 4.1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Full-scale value</th>
<th>Scale-model value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dimension</td>
<td>$x$</td>
<td>$Sx$</td>
</tr>
<tr>
<td>Area</td>
<td>$A$</td>
<td>$S^2A$</td>
</tr>
<tr>
<td>Volume</td>
<td>$V$</td>
<td>$S^3V$</td>
</tr>
<tr>
<td>Charge weight</td>
<td>$Q$</td>
<td>$S^3Q$</td>
</tr>
<tr>
<td>Charge standoff</td>
<td>$R$</td>
<td>$SR$</td>
</tr>
<tr>
<td>Scaled standoff</td>
<td>$R/Q^{1/3}=Z$</td>
<td>$SR/(S^3 Q)^{1/3}=Z$</td>
</tr>
<tr>
<td>Pressure</td>
<td>$P$</td>
<td>$P$</td>
</tr>
<tr>
<td>Impulse / unit area</td>
<td>$I$</td>
<td>$SI$</td>
</tr>
<tr>
<td>Velocity</td>
<td>$v$</td>
<td>$v$</td>
</tr>
</tbody>
</table>
The most widely used approach to blast wave scaling is the cube root scaling law proposed independently by Hopkinson (1915) and Cranz (1926). The law states that, similar blast waves are produced at the same scaled distances when two explosive charges of similar geometry and of same explosive but of different sizes are detonated in the same atmosphere. Thus, if charges of weights \( Q_1 \) and \( Q_2 \) are detonated then the same peak pressure is produced at distance of \( R_1 \) and \( R_2 \), respectively. The distances \( R_1 \) and \( R_2 \) are related as given below:

\[
\frac{R_1}{R_2} = \left( \frac{Q_1}{Q_2} \right)^{1/3}
\]

(4.1)

The duration of the positive phase of a pressure wave \( T \) against the detonation of charge weights of \( Q_1 \) and \( Q_2 \) is also given by a similar equation.

\[
\frac{T_1}{T_2} = \left( \frac{Q_1}{Q_2} \right)^{1/3}
\]

(4.2)

Equation 4.2 is valid when the scaled distance is the same for both charge weights.

### 4.5 Scope of Impulse Loads in ACI Standard 359 (2006)

ACI Standard 359 (2006) “Code for Concrete Reactor Vessels and Containments” deals with the impulse loads as time dependent loads e.g. the dynamic effects of accidental pressure \( P_a \), the effects of pipe rupture reactions \( R_{rr} \) and Jet impingement loading \( R_{rj} \) etc.

The impulsive loads in the design of safety related structures are of the following types:

- **Accidental Explosions:** A possible source of an accidental explosion is the release of explosive liquefied gas from off site storage tanks during transport or land storage. The released gas when combined with air forms a vapor cloud which can be ignited, resulting in a deflagration or an explosion. Examples of explosive gases include the hydrocarbon gases such as Ethylene, Ethylene oxide, methane, butane, ethane, propane and propylene

- **Intentional Explosions:** Studies have been performed for the event when a penetrating warhead is detonated inside containment. Also a pressure wave in the form of blast loading is applied to simulate the detonation of explosives close to the Nuclear Power Plant structure. The present paper describes results which relate to intentional explosions.
Jet Impingement: Jets resulting from a postulated rupture of high pressure piping exert an impulsive load on the affected area. The ruptured pipe may contain steam water mixture, steam or sub cooled water. The impulse load due to jet impingement is localized to the affected area.

Pipe Whip Restrain Action: Postulated breaks in high energy piping which result in the escape of fluid or gas from the pipe will induce motion to the pipe. The “whipping” pipe may impact other pipes and/or other Structures, Systems and Components (SSC). Impulsive forces are developed at the anchor or restraining of the whipping pipe during impact or rebound. Also the momentum of the escaping fluid or gas from the pipe induces impulsive forces at the pipe restraints.

Tornado Induced Pressure Drop: The tornado loads, postulated for design of safety related SSC, consist of a sustained pressure due to rotational and translational wind speed and the tornado generated missiles. In addition, the effect of the pressure drop when the lower pressure center of the tornado passes over a building, is considered in the design.

Although the U.S. Regulation does not require consideration of explosions due to terrorist attack and other act of war in the design of Nuclear Power Plant structures, other countries postulate and consider such explosions (Saeed et al 2005).

4.6 Experimental Setup

A 1:10 scale model of a typical nuclear power plant containment structure was constructed to determine the experimental relationships of airblast pressure time history as a function of surface explosion charge weight, distance to structure, structure height, as well as the simultaneous ground shock wave history. The experimental setup with explosion scenario is shown in Figure 4.2. The similarity of soil parameters at the experimental site with the actual site conditions was ensured in order to determine the ground shock relationship. The success of the experimental programme was dependent on the ability to accurately measure reflected pressure and to collect acceleration data from wall-mounted accelerometers.
The following equipment was calibrated and used in the tests.

(1) A self-contained shock-hardened data acquisition, HDAS system placed 10 m away from the structure was used to record the data. The entire system, including transducer and two battery packs, was fitted inside a circular canister 6-cm diameter and 15-cm deep. Each gauge used in the test was connected to an HDAS system. The four buried external wires Arm, Calibration, Trigger, and Ground—were expendable at zero time. Therefore, the system was capable of obtaining data even though the cables may be destroyed during the event. The system used internal battery power, thus allowing for non volatile memory retention during and after the test. The system captured the beginning of the waveform by using 16 ms of pretrigger data. A g-sensitive switch acted as a trigger and closed at 350 g’s. The system was triggered by a 5 volt pulse. The frequency range of the waves measured using the shock hardened data acquisition system was 100 kHz to 1 MHz.

(2) Three Kulite HKS-375 pressure transducers were used to measure pressure time-domain waveforms on the containment wall. The gauge was 16 mm long x 8 mm diameter. The gauges were screwed into a steel mount and then mounted into a pipe flush-mounted with the wall section. The pipe was cast into the wall sections during construction. Each pressure gauge was mounted at the one third point of the containment wall in front of the blast wave.
(3) A seismograph Geosonic 2000DK was used to record the blast event placed at 10 m from the structure and provided three different vibration readings on three channels. Each channel represented an axis of particle movement. The axes recorded by the seismograph were: radial/longitudinal, vertical and transverse. The vertical axis represented the vertical movement of the ground particles. The radial or longitudinal axis represented the ground movement that ran from the blast to the transducer at right angle to the radial channel.

![Configuration of Blast Experiment Scheme](image)

**Fig. 4.3: Configuration of Blast Experiment Scheme**
RESULTS AND DISCUSSION

4.7 Overpressure in the Free Air from Surface Explosions

The experimental values of shock wave propagation in the air from explosions were obtained. The explosive (TNT) in a spherical shape was placed on the surface of the saturated sandy clay. Axial symmetry of the hydrodynamic effects of the explosion was required and a spherical charge single point detonation at the center of the charge provided this scenario. The TNT charge weights used in experimental work varied from 1 kg to 25 kg, and the increase was gradual till the appearance of cracks on the concrete surface of the scaled containment model. The charge distances were varied 5m to 25m. Fig. 4.1 provides the configuration of blast resistant scheme. The pressure time histories in each case in the air at the target points directly above the ground surface were recorded. The experimental output provided the shock wave front arrival time $T_a$, the rising time from arrival time to the peak value $T_r$, the peak pressure $P_{so}$, the decreasing time from peak to the ambient pressure $T_d$ and other ground shock parameters such as peak particle acceleration (PPA). Table 4.2 provides typical experimental values of ground shock wave propagation in the air from various charge weights.

<table>
<thead>
<tr>
<th>$R$ (m)</th>
<th>$Q$ (kg)</th>
<th>$P_{so}$ (MPa)</th>
<th>$T_a$ (msec)</th>
<th>$T_r$ (msec)</th>
<th>$T_d$ (msec)</th>
<th>$T$ (msec)</th>
<th>$P_{ro}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>1</td>
<td>0.047</td>
<td>8</td>
<td>13</td>
<td>1</td>
<td>14</td>
<td>0.033</td>
</tr>
<tr>
<td>10</td>
<td>3</td>
<td>0.025</td>
<td>15</td>
<td>17</td>
<td>3</td>
<td>20</td>
<td>0.015</td>
</tr>
<tr>
<td>15</td>
<td>5</td>
<td>0.016</td>
<td>22</td>
<td>22</td>
<td>4</td>
<td>26</td>
<td>0.008</td>
</tr>
<tr>
<td>20</td>
<td>15</td>
<td>0.018</td>
<td>25</td>
<td>20</td>
<td>7</td>
<td>27</td>
<td>0.010</td>
</tr>
<tr>
<td>25</td>
<td>25</td>
<td>0.017</td>
<td>29</td>
<td>21</td>
<td>9</td>
<td>30</td>
<td>0.008</td>
</tr>
</tbody>
</table>

Table 4.2: Experimental values of shock wave propagation in the air from Various charge weights
4.8 Relationship between Peak Air Pressure and Scaled Distance ($R/Q^{1/3}$)

Many empirical formulae for predicting peak pressures in the air are available in the literature. Most of them were obtained from field blast tests (Henrych 1979, TM5-855-1 (1991), Brode 1959). Brode’s (1959) empirical formulae for peak pressure at shock wave front in an unlimited atmosphere are in the form of

$$p_{so} = 0.67 \left( \frac{R}{Q^{1/3}} \right)^{1/3} + 0.1, \quad p_{so} > 1 \text{ (MPa)}$$  \hspace{1cm} (4.3)

$$p_{so} = 0.098 \left( \frac{R}{Q^{1/3}} \right)^{-1} + 0.1465 \left( \frac{R}{Q^{1/3}} \right)^{2} + 0.585 \left( \frac{R}{Q^{1/3}} \right)^{-3} - 0.0019, \quad 0.1 \leq p_{so} \leq 1 \text{ (MPa)}$$  \hspace{1cm} (4.4)

Henrych’s (1979) empirical formulae in an unlimited atmosphere are

$$p_{so} = 1.4072 \left( \frac{R}{Q^{1/3}} \right)^{-1} + 0.554 \left( \frac{R}{Q^{1/3}} \right)^{2} - 0.0357 \left( \frac{R}{Q^{1/3}} \right)^{-3} + 0.00000625 \left( \frac{R}{Q^{1/3}} \right)^{-4}, \quad 0.1 \leq \frac{R}{Q^{1/3}} < 0.3 \text{ (MPa)}$$  \hspace{1cm} (4.5)

$$p_{so} = 0.619 \left( \frac{R}{Q^{1/3}} \right)^{-1} - 0.033 \left( \frac{R}{Q^{1/3}} \right)^{2} + 0.213 \left( \frac{R}{Q^{1/3}} \right)^{-3}, \quad 0.3 \leq \frac{R}{Q^{1/3}} \leq 1 \text{ (MPa)}$$  \hspace{1cm} (4.6)

Wu et al. (2007) determined the peak values at each point in the air using the simulated pressure time histories. He determined empirical attenuation relations for peak air pressure at a hemispherical front on the basis of these data.

$$p_{so} = 1.059 \left( \frac{R}{Q^{1/3}} \right)^{2.56} - 0.051, \quad 0.1 \leq \frac{R}{Q^{1/3}} \leq 1 \text{ (MPa)}$$  \hspace{1cm} (4.7)

$$p_{so} = 1.008 \left( \frac{R}{Q^{1/3}} \right)^{-2.01}, \quad 10 \geq \frac{R}{Q^{1/3}} > 1 \text{ (MPa)}$$  \hspace{1cm} (4.8)

The following relationship was obtained using the experimental data given in Table 4.2.

$$p_{so} = 1.017 \left( \frac{R}{Q^{1/3}} \right)^{-1.91} \text{ (MPa)}$$  \hspace{1cm} (4.9)

Where $R$ is the distance in meters measured from the charge center and $Q$ is the TNT charge weight in kilograms. This relationship is also shown graphically in Figure 4.4. Figure 4.5
shows the comparisons of the peak pressure in the air predicted by the present functions and by other empirical relations. As shown, at scaled distances $R/Q^{1/3}$ larger than 1.0, the numerical results agreed well with Henrych’s (1979) and Wu. et al. (2007) simulated data, but quite different with Brode’s (1959) when $R/Q^{1/3}$ was between 4 and 10.

![Graph showing peak pressure attenuation results](image)

**Figure 4.4:** Peak pressure attenuation results corresponding to different charge weights in the free air along the horizontal direction
Figure 4.5: Comparison of peak pressure attenuation against scaled distance

4.9 The Shock Wave Front Arrival Time $T_a$

The best-fit function of the arrival time in terms of distance and charge weights, determined experimentally, is

$$T_a = 0.40R^{1.2}Q^{-0.2} / C_a \text{ (s)} \quad (4.10)$$

Where $C_a$ is the sound speed in the air, which is 340 m/s.

Figure 4.6 shows the arrival time of shock wave front for different charge weights. It is clear that, at the same scaled distance, the larger the charge weight is, the longer the arrival time is.
4.10 The Duration of the positive pressure phase of the shock wave

The duration of the positive pressure phase of the airblast pressure wave \((T)\) can be written as

\[
T = T_r + T_d \tag{4.11}
\]

Figure 4.7: Typical free air pressure time history (TM5-1300 (1990))
The rising time ($T_r$) for pressure time history is the parameter which rises suddenly from zero to peak value. The decreasing time ($T_d$) for the pressure time history to decrease from its peak value to the ambient pressure is another parameter for modeling the pressure time history. The parameters have been illustrated in Figure 4.7.

The following relationship of rising time ($T_r$) and decreasing time ($T_d$) at different scaled distances was obtained using the experimental data given in Table 4.2. This relationship is also shown in Figure 4.8 & Figure 4.9.

\[
T_r = 0.0026 \left( \frac{R}{Q^{1/3}} \right)^{0.98} \quad (4.12)
\]

\[
T_d = 0.0003 \left( \frac{R}{Q^{1/3}} \right)^{0.89} Q^{0.47} \quad (4.13)
\]

Therefore, $T$ in Eq. 6.11 can be rewritten as

\[
T = 0.0026 \left( \frac{R}{Q^{1/3}} \right)^{0.98} + 0.0003 \left( \frac{R}{Q^{1/3}} \right)^{0.89} Q^{0.47} \quad (4.14)
\]

Figure 4.7 illustrates the duration of positive phase duration against various charge weights using the experimental data given in Table 4.2. The pressure time history is usually assumed starting from the peak value and decreases either exponentially or linearly. But, for a more accurate modeling of the air blast pressure time history, this phase has also been explored.

Figure 4.8: Duration for the pressure to increase from zero to peak value
Figure 4.9: Duration for the pressure to increase from peak value to ambient pressure

Figure 4.10: Duration of the positive pressure with various charge weights
4.11 Shock wave reflection from the concrete structure

The pressures on the front surface of the structure were determined in order to study the influence of simultaneous ground shock and air blast pressure on structures. Sensors were placed along the front surface of the wall to record the pressure time histories. It was found that the reflected pressure time histories have the same shape as those in the free air. Other researchers also made similar observations in the previous studies (Henrych 1979, TM5-855-1 (1991). The ratio of the peak reflected pressure to the peak free air pressure was calculated. Figure 4.11 shows the ratio of horizontal (normal to the wall) peak reflected pressure to peak air pressure. It shows that the ratio increases with the peak free air pressure. Using the experimental data in Table 4.2, the best-fit relation of the peak reflected pressure to the peak free air pressure was

$$ P_{ro} = 1.8 (P_{so})^{1.3} \quad (4.15) $$

where $P_{ro}$ is the normal peak reflected pressure at the bottom of the rigid wall. $P_{so}$ is determined by Eq. 4.9. Therefore, $P_{ro}$ is a function of charge weight and distance.

The comparison of the experimental results with other empirical relations is also shown in Figure 4.11. It showed that the results were lower than the Wu. et al. (2007) simulated results and Henrych’s (1979) experimental results. This was owing to the curved surface of the structure in the experiment.

![Figure 4.11: Ratios of horizontal peak reflected pressure to the peak free air pressure](image-url)
4.12 Ground shock wave from surface explosions

Table 4.3 provides the typical experimental values of ground shock from various charge weights and corresponding CONWEP values. The CONWEP results for 1 Kg explosion at 5 m standoff distance are illustrated in Figure 4.12.

![CONWEP results of 1 Kg explosion at 5 m stand off distance](image)

**CONWEP Review**

CONWEP is a collection of conventional weapons effects calculations from the equations and curves of TM-855-1 (1990), “Fundamentals of Protective design for Conventional weapons”. It is intended for use by engineers involved in designing deliberately hardened facilities. Users should have a basic knowledge of weapons effects and structural dynamics. Depending on the type of facility and the threat, the structure may be required to protect personnel and equipment against the effects of penetrating weapon, a contact detonation, or the blast and fragmentation from a standoff detonation. In the last case, if the structure responds in a predominantly flexural mode, it is assumed that the element can be represented by a single degree of freedom (SDOF) system. Transformation factors needed to represent beams and slabs as equivalent SDOF systems are provided in the manual. If the loading or the geometry of the structure is such that a simple SDOF representation is
inadequate to determine response, the analyst should consider using a multi degree of freedom (MDOF) system or a finite element representation for calculations.

### 4.14 Assumptions/References for Ground Shock in CONWEP

CONWEP is a collection of weapons effects calculation software. The assumptions / references incorporated in CONWEP are as follows.

“Equations for free-field stresses and ground motions from bombs detonating on or in the soil near a structure are found in Chapter 5 of TM5-855-1 (1991). This program calculates the peak free-field stress due to the directly transmitted shock wave, and optionally allows the addition of a reflected wave from a deeper layer and a relief (tension) wave reflected from the ground surface. It is important to note that relief wave effects for high magnitude shocks and/or near surface detonations are not well understood, and inclusion of a relief wave in these situations may lead to unconservative answers. Peak particle velocity, acceleration, and displacement is calculated using the direct path only --reflections from the surface or a lower layer are not included.

Please note that the calculated peak stress is for the free-field environment -- it is NOT necessarily the peak pressure that would load a structure. For wall or roof slabs with a roughly normal orientation to the shock wave, the peak pressure and impulse should be multiplied by a reflection factor of about 1.5”.

Stress and particle velocity pulses can be characterized by exponential type time histories that decay rapidly in amplitude and broaden as they propagate outward from the explosion. The arrival time, \( t_a \), is the elapsed time from the instant of detonation to the time at which the ground shock arrives at a given location, and

\[
 t_a = \frac{R}{c} \tag{4.16}
\]

Where \( R = \) distance from the explosion

\[ c = \text{average seismic or propagation velocity at distance } R \]

Typically, these wave forms rise sharply to their peak value such that the rise time \( (t_r) \) can be expressed as

\[
 t_r = 0.1 \ t_a \tag{4.17}
\]
Table 4.3: Typical Experimental values of ground shock with various charge weights

<table>
<thead>
<tr>
<th>( R ) (m)</th>
<th>( Q ) (Kg)</th>
<th>PPA ( (m/s^2) )</th>
<th>( t_a ) (msec)</th>
<th>( t_d ) (msec)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Exp</td>
<td>CONWEP</td>
<td>Exp</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
<td>5.6</td>
<td>5.41</td>
<td>3</td>
</tr>
<tr>
<td>10</td>
<td>3</td>
<td>6.5</td>
<td>6.12</td>
<td>7</td>
</tr>
<tr>
<td>15</td>
<td>5</td>
<td>6.3</td>
<td>6.41</td>
<td>11</td>
</tr>
<tr>
<td>20</td>
<td>15</td>
<td>12.3</td>
<td>11.9</td>
<td>15</td>
</tr>
<tr>
<td>25</td>
<td>25</td>
<td>14.9</td>
<td>14.5</td>
<td>20</td>
</tr>
</tbody>
</table>

**4.15 Demonstration of CONWEP**

The Annexure A demonstrates the sequence of calculation of ground shock of 1 kg charge weight at 5m stand off. Following are the steps in this regards as evident from the Figures in Appendix A.

1. Double click the CONWEP icon (Figure A-1). It provides the basis of CONWEP i.e. TM5-855-1, “Fundamentals of Protective Design for Conventional Weapons”.
2. Figure A-2 selects Weapons Effects Calculation.
3. Ground Shock in Figure A-3 has been selected in the Weapon Effects Main Menu
4. Bare HE has been selected in Figure A-4 as weapon type.
5. TNT has been selected out of various explosives in Figure A-5.
6. Weight of explosive equal to 1 Kg has been entered in Figure A-6.
7. Saturated Sandy Clays and sands has been selected in Figure A-7 keeping in view the material properties.
8. Figure A-8 demonstrates the following
   (i) Reflection from a deeper layer have not been considered.
   (ii) Tensile reflections from the ground surface have been excluded.
9. Figure A-9 provides the outputs for 1 Kg charge weight at 5 m horizontal range to target. The peak acceleration is 5.406 m/s². The impulse is 0.01744 kPa-sec.
10. In Figure A-10, Stress time history out of various ground shock options has been selected.
11. In Figure A-10, Velocity time history out of various ground shock options has been selected.

12. Table A.1 provides ground shock free field P-T history against TNT explosion of 1 Kg at 5m. The related input parameters are as follows:

Backfill density = 1920 kg/m³, Seismic velocity = 1524 m/sec
Attenuation coefficient = 3.100

Resultantly, the time of arrival is 3.281 msec. The duration of the ground shock wave is 9.839 msec.

13. Similarly, Table A.2 provides ground shock free velocity time history.

14. The Fig. A.12 and Fig. A.13 illustrate the data in Table A.1 and Table A.2 respectively.

4.16 Peak particle acceleration (PPA)

Figure 4.13 shows the experimental acceleration time histories on saturated sandy clay at a distance of 25 m with a charge weight of 25 Kg. The soil possessed the following properties:
Density = 1920 Kg/m³, loading wave velocity c = 1524 m/sec. It showed that the acceleration in the horizontal direction is about three times the values in the vertical direction.

The following relationship provides the surface ground motion as a function of charge weight and distance values determined through data in Table 4.3.

\[ PPA = 4.689 R^{-1.3} Q^{0.95} \text{ (g)} \] \hspace{1cm} (4.18)

![Figure 4.13a: The experimental acceleration time histories on saturated sandy clay with a charge weight of 25 kg (a) Horizontal Direction](image-url)
As shown in Table 4.3, the experimental results of PPA of surface ground motions have been compared with CONWEP (1991) values. As shown, the peak particle acceleration values obtained in saturated sandy clay are more than that obtained through CONWEP (1991). The difference is more pronounced at larger scaled distance $R/Q^{1/3}$. Figure 4.14 illustrates the comparison of the experimental data and CONWEP values against various charge weights.
Figure 4.14: Peak Particle Acceleration of ground shock wave for different charge weights

Figure 4.15: Arrival time of ground shock wave for different charge weights
4.17 Arrival time $t_a$

From the experimental data (Table 4.3), the arrival time at a point on ground surface with a distance $R$ from the charge center can be found by the following relationship

$$t_a = \frac{0.58R^{1.24}}{C_s Q^{0.01}} \text{ (s)}$$

(4.19)

Where $C_s$ is the seismic velocity of the soil. In our case, the seismic velocity of saturated sandy clay is 1524 m/s. Table 4.3 shows the comparison of ground shock arrival time against scaled distance in saturated sandy clay in our case and numerical modeling through CONWEP (1991). The experimental and CONWEP results of ground shock wave arrival time has also been illustrated in Figure 4.15.

The water saturation also affects the wave speed in the soils. In general, the propagation of wave in the saturated soil is much faster than in unsaturated soil. With the decrease of water saturation degree, the wave speed decreases. Moreover, the wave speed remains almost constant with varying distance in saturated soil, whereas in unsaturated soils, the wave speed in the near field is much larger than in the far field. This is because the unsaturated soil is compacted in the near field under high pressure, resulting in an increased saturation degree.
and hence higher wave speed. The low wave speed at larger distances for unsaturated soils can be attributed to low pressure and the subsequent dominating role of the soil skeleton.

### 4.18 Shock Wave Duration $t_d$

Duration of the shock wave significantly affects the structural response. Shock wave duration $t_d$ is the difference of total ground shock history time and the arrival time of ground motion at distance $R$ from the charge centre i.e.

$$t_d = t - t_a$$

From the experimental data, it was established

$$t_d = 0.0056 R^{0.54} \text{ (s)} \quad (4.20)$$

In Table 4.3, the shock wave duration experimentally determined in soil has been compared with the duration determined numerically through CONWEP (1991). The results have also been illustrated in Figure 4.16.

### 4.19 Time lag between ground shock and air blast pressure arrival at structures

From the experimental relationships obtained in (4.10) and (4.20), the time lag between the ground shock and air blast pressure reaching to the structure can be determined by

$$T_{\text{lag}} = T_a - t_a = 0.40 R^{1.2} Q^{-0.2} / C_a - \frac{0.58 R^{1.24}}{C_s Q^{0.01}} \text{ (s)} \quad (4.21)$$

It is evident that the time lag is not only related to distance from the charge center and charge weight, but also to wave propagation velocity in the air and at the site. At the same distance, the larger the charge weight, the shorter the time lag.
4.20 Summary

The chapter provides the relationships between following parameters of overpressure in the free air from external explosions which have been established through experiment on 1/10th scaled reactor containment model.

1. **Overpressure in the Free Air from Surface Explosions**
   
   (a) Peak pressure $P_{so}$ and scaled distance $(R/Q^{1/3})$
   
   (b) Shock wave front arrival time in terms of distance $(R)$ and charge weights $(Q)$
   
   (c) Rising time from arrival time to the peak value $(T_r)$
   
   (d) Decreasing time from peak to the ambient pressure $(T_d)$
   
   (e) Duration of the positive pressure phase of the airblast pressure wave $(T)$
   
   (f) Relation of the peak reflected pressure $(P_{ro})$ to the peak free air pressure $(P_{so})$

2. **Ground shock wave from surface explosions**
   
   (a) Peak Particle Acceleration (PPA)
   
   (b) Arrival Time $(t_a)$
   
   (c) Shock Wave Duration $(t_d)$
   
   (d) Time lag between ground shock and air blast pressure arrival at structures

   The variation in results in comparison with the previous researchers and CONWEP has been attributed to the curvature of the structure.
Chapter 5

Evaluation of Containment Structure against External Explosion

5.1 Introduction
The typical containment is Seismic Category 1 and Safety Class 1 structure. The structures and components are designed to satisfactorily absorb the loads and stresses induced as a result of earthquake ground motion corresponding to 0.1 g.
The nuclear containment shell was subjected to surface blast loading varying from 30 t to 160 t of Trinitrotoluene (TNT) at a detonation distance of 100-200 m. The analysis was performed using the software SAP2000 (2008).

5.2 SAP 2000 Review (SAP 2000 (2008))

5.2.1 Features
Following are the salient features of SAP 2000

- Static and dynamic analysis
- Linear and non linear analysis
- Dynamic seismic analysis and static push over analysis
- Vehicle live- load analysis for bridges
- Geometric nonlinearity, including P-delta and large-displacement effects
- Staged (incremental) construction
- Creep, shrink age, and aging effects
- Buckling analysis
- Steady-state and power-spectral-density analysis
- Frame and shell structural elements, including beam- column, truss, membrane, and plate behavior
- Two-dimensional plane and axisymmetric solid elements
Chapter 5  Evaluation of Containment Structure against External Explosion

5.2.2 General Steps
The following are the general steps to analyze and design a structure through SAP2000
1. Create or modify a model that numerically defines the geometry, properties, loading, and analysis parameters for the structure.
2. Perform an analysis of the model
3. Review the results of the analysis
4. Check and optimize the design of the structure

5.2.3 Time History Analysis
Time-history analysis is a step-by-step analysis of the dynamical response of a structure to a specified loading that may vary with time. The analysis may be linear or non linear.

Time-history analysis is used to determine the dynamic response of a structure to arbitrary loading. The dynamic equilibrium equations to be solved are given by:

\[ Ku(t) + C \ddot{u}(t) + M \dddot{u}(t) = r(t) \]

where \( K \) is the stiffness matrix; \( C \) is the damping matrix; \( M \) is the diagonal mass matrix; \( u, \dot{u} \) & \( \ddot{u} \) are the displacements, velocities, and accelerations of the structure; and \( r \) is the applied load. If the load includes ground acceleration, the displacements, velocities, and accelerations are relative to this ground motion.
Any number of time-history Analysis Cases can be defined. Each time-history case can differ in the load applied and in the type of analysis to be performed.

There are several options that determine the type of time-history analysis to be performed:

- Linear vs. Non linear.
- Modal vs. Direct-integration: These are two different solution methods, each with advantages and disadvantages. Under ideal circumstances, both methods should yield the same results to a given problem.
- Transient vs. Periodic: Transient analysis considers the applied load as a one-time event, with a beginning and end. Periodic analysis considers the load to repeat indefinitely, with all transient response damped out.

In a non linear analysis, the stiffness, damping, and load may all depend upon the displacements, velocities, and time. This requires an iterative solution to the equations of motion.

### 5.2.4 Initial Conditions

The initial conditions describe the state of the structure at the beginning of a time-history case. These include:

- Displacements and velocities
- Internal forces and stresses
- Internal state variables for non linear elements
- Energy values for the structure
- External loads

The accelerations are not considered initial conditions, but are computed from the equilibrium equation. For non linear analyses, the initial conditions are specified at the start of the analysis. You have two choices:

- Zero initial conditions: the structure has zero displacement and velocity, all elements are unstressed, and there is no history of non linear deformation.
- Continue from a previous non linear analysis: the displacements, velocities, stresses, loads, energies, and non linear state histories from the end of a previous analysis are carried forward.

There are some limitations when continuing from a previous non linear case:
Non linear static and non linear direct-integration time-history cases may be chained together in any combination, i.e., both types of analysis are compatible with each other.

- Non linear modal time-history (FNA) cases can only continue from other FNA cases that use modes from the same modal analysis case.

5.2.5 Overview of Shell Element Internal Forces/Stresses Output Sign Convention

Annexure B demonstrates shell element internal forces/Stress Output Sign Convention. A brief summary is as follows.

Figure B.1 shows Faces of a Shell Element. The six faces of a shell element are defined as the positive 1 face, negative 1 face, positive 2 face, negative 2 face, positive 3 face and negative 3 face. The numbers 1, 2 and 3 correspond to the local axes of the shell element. The positive 1 face of the element is the face that is perpendicular to the 1-axis of the element whose outward normal (pointing away from the element) is in the positive 1-axis direction. The negative 1 face of the element is a face that is perpendicular to the 1-axis of the element whose outward normal (pointing away from the element) is in the negative 1-axis direction. The other faces have similar definitions.

The Figure B.2 demonstrates internal F11 forces acting on the midsurface of a shell element. In this figure, the force distribution labeled (a) represents an actual F11 force distribution. The force distribution labeled (b) shows how SAP2000 calculates only the internal forces at the corner points of the shell element. Note that these stresses are calculated at any location on the shell element. We simply choose to calculate them only at the corner points because that is a convenient location and it keeps the amount of output to a reasonable volume.

The force distribution labeled (c) in the Figure B.2 shows how SAP2000 assumes that the F11 forces vary linearly along the length of the shell element between the calculated F11 force values at the element nodes for graphical plotting purposes only.

The Figure B.3 illustrates the positive directions for shell element internal forces F11, F22, F12, V13 and V23. Note that these shell element internal forces are forces per unit length.
acting on the midsurface of the shell element. SAP2000 reports only the value of these forces at the shell element corner points.

The Figure B.4 illustrates the positive direction for shell element principal forces, Fmax and Fmin. It also illustrates the positive direction for the shell element maximum transverse shear force, Vmax

For values of V13 and V23 at any angle, the maximum transverse shear stress, V-Max, can be calculated as:

The Figure B.5 illustrates the positive directions for shell element internal moments M11, M22 and M12. Note that these shell element internal moments are moments per unit length acting on the midsurface of the shell element. SAP2000 reports only the value of these moments per unit length at the shell element corner points. The right-hand rule is used to determine the sense of the moments shown in the Figure B.5.

The Figure B.6 illustrates the positive direction for shell element principal moments, Mmax and Mmin.

The basic shell element stresses are identified as S11, S22, S12, S13, and S23. An S21 might also be expected, but S21 is always equal to S12, so it is not actually necessary to report S21. Sij stresses (where i can be equal to 1 or 2 and j can be equal to 1, 2 or 3) are stresses that occur on face i of an element in direction j. Direction j refers to the local axis direction of the shell element. Thus S11 stresses occur on face 1 of the element (perpendicular to the local 1 axis) and are acting in the direction parallel to the local 1 axis (that is, the stresses act normal to face 1). As another example, S12 stresses occur on face 1 of the element (perpendicular to the local 1 axis) and are acting in the direction parallel to the local 2 axis (that is, the stresses act parallel to face 1, like shearing stresses). The Figure B.7 shows examples of each of these basic types of shell stresses. SAP2000 reports internal stresses for shell elements at the four corner points of the appropriate face of the element. For example, refer to Figure 5.7 a, on the positive 1 face, internal stresses are reported by SAP2000 at points A, B, C and D.
Shell internal stresses are reported for both the top and the bottom of the shell element. The top and bottom of the element are defined relative to the local 3-axis of the element. The positive 3-axis side of the element is considered to be the top of the element.

In Figure B.7 a, internal stresses at the top of the element include stresses at the joints labeled A and C and internal stresses at the bottom of the element include stresses at the joints labeled B and D.

The Figure B.8 clearly illustrates the points where SAP2000 reports the shell element internal stress values.

The transverse shear stresses calculated by SAP2000 (S13 and S23) are average values. The actual transverse shear stress distribution is approximately parabolic; it is zero at the top and bottom surfaces and has its maximum or minimum value at the midsurface of the element. SAP2000 reports the average transverse shear value. An approximation to the maximum (or minimum) transverse shear stress would be 1.5 times the average shear stress.

The Figure B.9 & B.10 illustrates the positive directions for shell element internal stresses S11, S22, S12, S13 and S23. Also shown are the positive directions for the principal stresses, S-Max and S-Min, and the positive directions for the maximum transverse shear stresses, S-Max-V.

For values of S13 and S23 at any angle, the maximum transverse shear stress, S-MaxV, can be calculated from:

\[ S - \text{MaxV} = \sqrt{S_{13}^2 + S_{23}^2} \]

Figures B.11-B.14 illustrate the following concepts.
Local axis 3 is always normal to the plane of the shell element. This axis is directed towards you when the path j1-j2-j3 appears counter-clockwise. For quadrilateral elements, the element plane is defined by the vectors that connect the mid-points of the two pairs of opposite sides.

Default Orientation: The default orientation of the local 1 and 2 axes is determined by the relationship between the local 3 axis and the global Z axis:

- The local 3-2 plane is taken to be vertical, i.e., parallel to the Z axis
- The local 2 axis is taken to have an upward (+Z) sense unless the element is horizontal, in which case the local 2 axis is taken along the global +Y direction
- The local 1 axis is horizontal, i.e., it lies in the X-Y plane

The element is considered to be horizontal if the sine of the angle between the local 3 axis and the Z axis is less than $10^{-3}$.

The local 2 axis makes the same angle with the vertical axis as the local 3 axis makes with the horizontal plane. This means that the local 2 axis points vertically upward for vertical elements.

Element Coordinate Angle: The shell element coordinate angle, ang, is used to define element orientations that are different from the default orientation. It is the angle through which the local 1 and 2 axes are rotated about the positive local 3 axis from the default orientation. The rotation for a positive angle value of ang appears counter-clockwise when the local +3 axis is pointing toward you. For horizontal elements, ang is the angle between the local 2 axis and the horizontal +Y axis. Otherwise, ang is the angle between the local 2 axis and the vertical plane containing the local 3 axis. The Figures from B.11 to B.14 demonstrate the aforementioned concepts.

5.3 Material properties of Concrete & Steel

The material properties of concrete & steel used in the analysis were
5.4 Soil Structure Interaction (SSI)

The dynamic loads influence adversely the foundation supporting soil by densifying it which may, in turn, cause differential settlement of the soil and foundation (Das 2001). The spring represents the elastic properties of the soil. Table 5.1 gives the exact solutions for spring constants for circular foundations derived from the theory of elasticity.

<table>
<thead>
<tr>
<th>Motion</th>
<th>Spring Constant for Circular foundations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical</td>
<td>( k = \frac{4Gr_o}{1 - \mu} ) (ton/m)</td>
</tr>
<tr>
<td>Horizontal</td>
<td>( k_x = \frac{32(1 - \mu)Gr_o}{7 - 8\mu} ) (ton/m)</td>
</tr>
<tr>
<td>Rocking</td>
<td>( k_0 = \frac{(8/3)Gr_o^3}{1 - \mu} ) (ton-m²)</td>
</tr>
<tr>
<td>Torsion</td>
<td>( k_\alpha = \frac{16}{3}Gr_o^3 ) (ton-m²)</td>
</tr>
</tbody>
</table>
$G$ is the shear modulus of soil which is calculated based on the shear wave velocity. $\mu$ is the Poisson’s ratio. $r_o$ is the radius of the circular foundation. In Pakistan, the terrain is mostly sandstone with shear wave velocity of 400 m/sec.

5.5 Model Description

The shell elements size is 1m x 1m. The effect of soil embankment up to 7m has also been considered. The thickness of the cylindrical portion has been taken as 1.2m. Four buttresses 3m wide with 1.8m thickness comprising 1m x 1m shell elements have been modeled. Figure 5.1 illustrate the reactor containment configuration. Figure 5.2 & Figure 5.3 display the structural prestressing tendons considered in the model.
Figure 5.1: Typical Reactor Containment Configuration
Figure 5.2: Section B-B (Refer Fig. 5.1)
Figure 5.3: Section C-C (Refer Fig. 5.1)
Following codes have been utilized during the analysis and design

1. Code for Concrete Reactor Vessels and Containments (ACI 359-07)
2. ACI Code 11.10.5 for shear walls (2005)
3. USNRC Regulatory Guide 1.60 (2001)

- The containment is prestressed with 257 cables and 98 hoop tendons in the cylinder with vertical tendons anchored between the ring girder and the tendon gallery and the hoop tendons anchored between two buttresses 240° apart.
- The dome is prestressed with three groups of 45 cables, oriented at 120° and anchored in the vertical face of ring girder. The hoop tendons are spaced at a vertical interval of 275mm C/C.
- The vertical tendons have an angular spacing of \( \frac{360}{257} = 1.40° \) C/C and the dome tendons are at a horizontal spacing of \( \frac{2\times\pi\times18}{3\times45} = 0.900\text{mm} \) C/C.
- The cylinder is prestressed by means of vertical as well as horizontal cables. The ultimate prestressing strength \( f_{pu} \) of a tendon is 2270 KN. The final prestressing force, after deduction of all losses, shall be taken at no more than 60% of GUTS (1361 KN). Therefore, the force on one tendon is given by:

\[
F = 0.60 \ f_{pu} A = 1361 \text{ KN}
\]

- Where \( A \) is the area of X-section of a tendon. The tendon is composed of uncoated seven wire, stress relieved or stabilized high tensile steel strand.
- There are immediate as well as delayed losses in prestress force due to slip at anchorage, elastic shortening of concrete, friction due to intended or unintended change in curvature of the tendon, etc.
5.6 Forces due to Vertical Tendons

The loads on the ring girder due to vertical tendons were calculated by dividing the total force with no. of nodes. Prestress forces on tendons were proportionally distributed on the nodal points. The vertical tendons are spaced at 450 mm C/C.

\[
\text{Every Node carry load} = \frac{257}{115} \times 1361 = 3042 \text{ KN}
\]

5.7 Forces due to Hoop Tendons

The hoop tendons exert an inward pressure on the cylindrical part of the containment. The forces applied on the anchoring points of the buttresses, being equal in magnitude and opposite in direction, cancel each other’s effects.

The pressure \( P \) per unit height of a cylinder of radius \( r \) subject to a uniform internal pressure \( p \) is given by:

\[
P = p \times r
\]

The hoop tendons are spaced at a distance of 275 mm C/C

\( F = \text{Force on one tendon} \)

\( f_H = \text{Factor accounting for immediate loss in prestress} = 0.85 \)

\[
F \times f_H = P \times r \times 0.92
\]

\[
P = \frac{F \times f_H}{r \times 0.92} = \frac{1361 \times 0.85}{18 \times 0.92} = 69.8 \text{ KN/m}^2
\]

The pressure is applied on all the elements below ring girder for use in SAP2000.
5.8 Forces due to Dome Tendons

5.8.1 Pressure on Dome Elements
The dome cables are grouped in 3 families with the axis of each family turned at 120° to the other. Each family contains 45 cables. The minimum GUTS of the cables shall be 2270 KN and the final prestressing force after deduction of all losses shall be 60% of GUTS (1361 KN).

The cables are \( \frac{2 \times F_{f_0}}{0.9 \times r} \)

\[
= \frac{3 \times 1361 \times 0.90}{0.9 \times 18} = 22683 \text{ KN/m}^2
\]

The pressure was applied on all the elements of dome.

5.8.2 Forces on Ring Girder
The dome tendons are anchored in the vertical face of ring girder.
Applying loads 1361 KN in area elements (Local axis 2) adjacent to cylindrical portions

\[
= \frac{1361}{0.300 \times 1.0} = 4536 \text{ KN/m}^2
\]

The size of the finite element in this portion is 0.3 m x 1.0 m

5.9 Forces due to Temperature Gradient

\( \Delta T = \text{Thermal gradient} = (22 ^\circ \text{C}) \)

Dome \( = \frac{22}{0.48} = 45.83 ^\circ \text{C/m} \)

Cylinder \( = \frac{22}{1.37} = 16.06 ^\circ \text{C/m} \)

Buttress \( = \frac{22}{1.83} = 12.02 ^\circ \text{C/m} \)
5.10 Numerical Results

5.10.1 Deflections

The peak deflection and the time at the peak deflection are given below in Table 5.2, for all the five cases, and are illustrated in Figure 5.4, Figure 5.5, Figure 5.6, Figure 5.7 & Figure 5.8. It is clear that the time at which the peak deflections occurred, decreased with the decrease in the amount of blast. The variation of the deflection at the crown is shown in Figure 5.9 up to 1.0 s duration for all the five cases. The complete period of forced vibration and amplitude increase with the increase in the amount of blast charge. The reason is that with increase in the amount of blast charge, the pressure loading increases and more number of cracks are produced.

<table>
<thead>
<tr>
<th>Blast charge (t)</th>
<th>70.0</th>
<th>60.0</th>
<th>50.0</th>
<th>40.0</th>
<th>30.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak deflection</td>
<td>80</td>
<td>74</td>
<td>64</td>
<td>58</td>
<td>50</td>
</tr>
<tr>
<td>(mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Time at peak Deflection (msec)</td>
<td>340</td>
<td>340</td>
<td>320</td>
<td>300</td>
<td>300</td>
</tr>
</tbody>
</table>
Figure 5.4: Deflection against explosion of 70 T charge weight at 100 m

Figure 5.5: Deflection against explosion of 60 T charge weight at 100 m
Figure 5.6: Deflection against explosion of 50 T charge weight at 100 m

Figure 5.7: Deflection against explosion of 40 T charge weight at 100 m
Figure 5.8: Deflection against explosion of 30 T charge weight at 100 m

Figure 5.9: Variation of deflection with time at top of the shell
Figure 5.10: Variation of hoop stress in element 20 for surface blast of different amounts of charges at a detonation distance of 200 m

Figure 5.11: Variation of meridonal stress with time in element 20 for surface blast charges at a detonation distance of 200 m
5.10.2 Stress in concrete

The peak compressive stress values in the vertical direction for different amounts of the blast charge are given in Table 5.3, and illustrated in Figure 5.12, Figure 5.13, Figure 5.14, Figure 5.15 & Figure 5.16. The maximum and minimum values were 64.76 and 16.22 MPa for surface blast charges of blast of 70.0 and 10.0 t of TNT, respectively. The variation of hoop stress in the vertical direction with time at a gauss point in the element number 20 (near the bottom of the near face) where peak stress occurred for all the blast cases is shown in Figure 5.10. Gauss points are several points inside every element that are used to perform the integration quadrature necessary for the finite element method. Spatial orientation of each gauss point is arbitrary. Element volume is defined as a surface area of uniform thickness.

It is seen from the figure that the first peaks occur earlier for higher blast charges and subsequent peaks occur earlier for the lower blast charges. This is owing to the fact that the increase in the non-linearity, which is more pronounced for higher blast charges the time period of forced vibration increases more for the higher blast charges. The stress in the meridional direction for the same gauss point is plotted in the Figure 5.11 for all the blast charges. Similar trends were observed during the compression phase as for the stress in the vertical direction. The magnitude of stress in the hoop direction was less compared to the vertical stress magnitude.

<table>
<thead>
<tr>
<th>Blast Charge (t)</th>
<th>70.0</th>
<th>60.0</th>
<th>50.0</th>
<th>40.0</th>
<th>30.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak stress</td>
<td>64.76</td>
<td>48.13</td>
<td>38.48</td>
<td>29.02</td>
<td>16.22</td>
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</tbody>
</table>
Figure 5.12: Peak stress against explosion of 70 t charge weight at 100m

Figure 5.13: Peak stress against explosion of 60 t charge weight at 100m
Figure 5.14: Peak stress against explosion of 50 t charge weight at 100m

Figure 5.15: Peak stress against explosion of 40 t charge weight at 100m
Yielding and failure of concrete

Yielding i.e failure in the plastic range began for a large number of points on the structure during nonlinear analysis using SAP 2000 inbuilt bilinear shape function. The material degradation began at volume associated with these points. Table 5.4 gives the number of gauss points yielded for different amounts of blast charges. As seen from the Table 5.4, 16 gauss points yielded for blast of 160.0 t, and this number reduced markedly as the blast charge reduced. Further, it may be mentioned that all the gauss points that yielded lie either within the lowest 10m region along the height or at top of the shell.

<table>
<thead>
<tr>
<th>Blast charge (t)</th>
<th>160.0</th>
<th>140.0</th>
<th>120.0</th>
<th>100.0</th>
<th>80.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of gauss</td>
<td>16</td>
<td>12</td>
<td>09</td>
<td>7</td>
<td>5</td>
</tr>
<tr>
<td>Points yielded</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
5.11 Critical distance for different amount of surface blast charges

A parametric study is carried out to calculate the distance (m) for the surface blast charges of 160.0, 140.0, 120.0, 100.0, 80.0, 60.0 and 40.0 t of TNT that generated blast pressures of intensity enough to develop cracks at more than 90% of the total cracked gauss points out of which more than 70% of the gauss points are doubly cracked. Doubly cracked gauss points are the locations which have been crushed in the plastic range. The summary of results (distance of detonation, maximum blast pressure, arrival time of ground shock and air blast etc.) is given in Table 5.5. In the study, the critical distances vary from 110 to 200 m for above blast charges. Total number of cracks in the critical distance calculation varies from 92.0% to 97.2% of the total gauss points. It is seen that with small increase in the number of cracks for 120.0, 140.0 and 160.0 t of blast charge, greater number of gauss points yielded.
Table 5.5: Results for Different Surface Blast Charges

<table>
<thead>
<tr>
<th>Description</th>
<th>Parametric responses for different blast charges</th>
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</thead>
<tbody>
<tr>
<td>Blast charge (t)</td>
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</tr>
<tr>
<td>Critical distance (m)</td>
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</tr>
<tr>
<td>$P_{x0}$ (MPa)</td>
<td>0.109</td>
</tr>
<tr>
<td>$T_a$ (msec)</td>
<td>40</td>
</tr>
<tr>
<td>PPA (g)</td>
<td>245</td>
</tr>
<tr>
<td>$t_a$ (msec)</td>
<td>116</td>
</tr>
<tr>
<td>$P_{ro}$ (MPa)</td>
<td>0.101</td>
</tr>
<tr>
<td>% of total cracked gauss points</td>
<td>92.0</td>
</tr>
<tr>
<td>% of doubly cracked gauss points</td>
<td>75.2</td>
</tr>
<tr>
<td>Number of gauss points yielded</td>
<td>16</td>
</tr>
</tbody>
</table>
Figure 5.17: Peak stress diagram with the application of blast overpressure only

Figure 5.18: Peak stress diagram with the simultaneous application of blast overpressure and ground shock against surface explosion of 160 t at 200 m
5.12 Simultaneous application of blast overpressure and shock wave

Figure 5.17 gives the stress diagram with the application of blast overpressure only, while Figure 5.18 provides the stress diagram with the simultaneous application of blast overpressure and ground shock against surface explosion of 160 t at 200 m through utilization of equations developed above for $T_a$ and $t_a$. It is clear there is an increase of about 5-20 MPa in peak stress in the second case owing to time lag between ground shock and airblast pressure arrival at the structure. It stresses the significance of consideration of simultaneous application of blast overpressure and shock wave for more accurate estimation of explosive loads in modeling response and damage of structures to surface explosions.

5.13 Summary

The chapter deals with a full scale typical reactor containment structure. The structure was subjected to surface blast loads varying from 30 t to 160 t of Trinitrotoluene (TNT) at a detonation distance of 100 to 200 m using the equations developed earlier. Conclusions were drawn on the location of gauss points that yielded and on the simultaneous application of air blast and shock wave on reinforced concrete containment as compared to that of air blast only. A parametric study was also carried out to calculate the critical distance for various intensities of surface blast enough to cause cracks at more than 90% of the total cracked gauss points. It has been observed that an accurate analysis of structure response and damage of structures to a nearby surface explosion requires simultaneous consideration of ground shock and air blast pressure. The increase of 5-20 MPa has been computed with simultaneous application of airblast and ground shock wave during analysis of typical reactor containment.
Chapter 6

Conclusions and Recommendations

6.1 Conclusions

The conclusions may be summarized as follows.

6.1.1 Experimental Findings of Scaled Model Reactor Containment

6.1.1.1 Overpressure in the Free Air from Surface Explosions

The relationships between following parameters of overpressure in the free air from external explosions have been established through experiment on 1/10\textsuperscript{th} scaled reactor containment model. The detail is as follows.

(a) Peak pressure $P_{so}$ and scaled distance ($R/Q^{1/3}$)

The following relationship was obtained using the experimental data.

$$P_{so} = 1.017 \left(\frac{R}{Q^{1/3}}\right)^{-1.91}$$

Where $R$ is the distance in meters measured from the charge center and $Q$ is the TNT charge weight in kilograms.

The experimental relationship has been compared with the following researchers findings.

(i) Brode’s (1959) empirical formulae for peak pressure at shock wave front in an unlimited atmosphere

(ii) Henrych’s (1979) empirical formulae in an unlimited atmosphere

(iii) Chengqing (2007) determined the peak values at each point in the air using the simulated pressure time histories

It has been concluded that, at scaled distances $R/Q^{1/3}$ larger than 1.0, the numerical results agreed well with Henrych’s (1979) and Chengqing’s (2007) simulated data, but quite different with Brode’s (1959) when $R/Q^{1/3}$ was between 4 and 10. The variation may be attributed to the curvature of the structure.

(b) Shock wave front arrival time in terms of distance ($R$) and charge weights ($Q$)

The shock wave front arrival time $T_a$; which is usually not included in the previous experimental studies, is measured here, and may be expressed through the following empirical relationship.
\[ T_a = 0.40R^{1.2}Q^{-0.2} / C_a \ (s) \]

Where \( C_a \) is the sound speed in the air, which is 340 m/s. It is clear that, at the same scaled distance, the larger the charge weight is, the longer the arrival time is.

(c) Rising time from arrival time to the peak value (\( T_r \))

The rising time (\( T_r \)) for pressure time history is the parameter which rises suddenly from zero to peak value. In most previous studies, this phase in the pressure time history is not modeled because the rising time is very short. The pressure time history is usually assumed starting from the peak value and decreases either exponentially or linearly. But, for a more accurate modeling of the air blast pressure time history, this phase has also been explored through the following empirical relationship in terms of scaled distance (\( R/Q^{1/3} \)).

\[ T_r = 0.0026 \ (R / Q^{1/3})^{0.98} \]

(d) Decreasing time from peak to the ambient pressure (\( T_d \))

The decreasing time (\( T_d \)) for the pressure time history to decrease from its peak value to the ambient pressure is another parameter for modeling the pressure time history which has been explored through the experiment for precise modeling of the pressure time history. The resulting empirical relationship in terms of scaled distance (\( R/Q^{1/3} \)) is as follows.

\[ T_d = 0.0003 \ (R / Q^{1/3})^{0.89}Q^{0.47} \]

It shows that at the same scaled distance, the heavier the charge weight, the longer the decreasing time.

(e) Duration of the positive pressure phase of the airblast pressure wave (\( T \))

The experimental relationships of rising time (\( T_r \)) and decreasing time (\( T_d \)), as discussed above, may be combined to calculate the duration of the positive pressure of the airblast pressure wave, and is as follows.

\[ T = 0.0026 \ (R / Q^{1/3})^{0.98} + 0.0003 \ (R / Q^{1/3})^{0.89}Q^{0.47} \]

(f) Relation of the peak reflected pressure (\( P_{ro} \)) to the peak free air pressure (\( P_{so} \))

The pressures on the front surface of the structure have been determined in order to study the influence of simultaneous ground shock and air blast pressure on structures. The ratio of the peak reflected pressure to the peak free air pressure is calculated. It shows that the ratio increases with the peak free air pressure.

The resulting best-fit relation of the peak reflected pressure (\( P_{ro} \)) to the peak free air pressure (\( P_{so} \)) is as follows

Effects of an External Explosion on a Concrete Structure
Ph.D. Thesis, UET Taxila Pakistan, March 2009
where \( P_{ro} = 1.8 (P_{so})^{1.3} \)

6.1.1.2 Ground shock wave from surface explosions

(a) Peak Particle Acceleration (PPA)

The experimental acceleration time histories on saturated sandy clay shows that the acceleration in the horizontal direction is about three times the values in the vertical direction. The following peak particle acceleration provides the surface ground motion as a function of charge weight and distance

\[
PPA = 4.689 R^{1.3} Q^{0.95} \quad (g)
\]

The experimental results of PPA of surface ground motions have been compared with CONWEP (1991) values. As shown, the peak particle acceleration values measured in saturated sandy clay are more than that obtained through the software. The difference is more pronounced at larger scaled distance \( R/Q^{1/3} \).

(b) Arrival Time \( (t_a) \)

From the experimental data, the arrival time at a point on ground surface with a distance \( R \) from the charge center can be expressed by the following relationship

\[
t_a = \frac{0.58 R^{1.24}}{C_s Q^{0.01}} \quad (s)
\]

Where \( C_s \) is the seismic velocity of the soil. In our case, the seismic velocity of saturated sandy clay is 1524 m/s. The results have been compared with CONWEP results, and have been found in good agreement.

(c) Shock Wave Duration \( (t_d) \)

Duration of the shock wave significantly affects the structural response. From the experimental data, it was established in terms of charge distance, and is as follows.

\[
t_d = 0.0056 R^{0.54} \quad (s)
\]

The shock wave duration experimentally determined in soil has been compared with the duration determined numerically through CONWEP. The results agree well with numerical results.

(d) Time lag between ground shock and air blast pressure arrival at structures
From the experimental data, the time lag between the ground shock and air blast pressure reaching to the structure can be determined by

\[ T_{\text{lag}} = T_a - t_a = 0.40R^{1.2}Q^{-0.2}/C_a - \frac{0.58R^{1.24}}{C_s Q^{0.01}} \] (s)

It is evident that the time lag is not only related to distance from the charge center and charge weight, but also to wave propagation velocity in the air and at the site. At the same distance, the larger the charge weight, the shorter the time lag.

### 6.1.2 Analytical Findings of Full Scale Reactor Containment

a. The surface blast loading varying from 30 t to 160 t of trinitrotoluene (TNT) at a detonation distance of 50-200 m has been applied on a typical reactor containment during analysis through SAP 2000. The calculation of percentage of cracked gauss points provides the location and percentage of cracked gauss points on the surface. It is concluded that all the gauss points that yielded lie either within the lowest 10m region along the height or at top of the shell.

b. A parametric study is carried out to calculate the critical distance for the surface blast charges of 160.0, 140.0, 120.0, 100.0, 80.0, 60.0 and 40.0 t of TNT that generated blast pressures of intensity enough to develop cracks at more than 90% of the total cracked gauss points. The 70% of the gauss points are doubly cracked. Doubly cracked gauss points are the locations which have been crushed in the plastic range. In this study, the critical distances vary from 110m to 200m for above blast charges.

c. It has been observed that an accurate analysis of structure response and damage of structures to a nearby surface explosion requires simultaneous consideration of ground shock and air blast pressure. The increase of 5-20 MPa has been computed with simultaneous application of airblast and ground shock wave during analysis of typical reactor containment.

d. The developed equations and the methodology employed can be pursued to estimate the blast response of concrete shell type containment structure and estimating the extent of cracking.
6.2 Recommendations

The following recommendations are suggested for further research work in the related fields.

(i) Modeling the effect of obstacles on a blast wave is a very complicated task. Therefore, the need of extensive experimental data is prerequisite for further research work.

(ii) The significance of Pressure–impulse (P–I) diagrams in the preliminary design or assessment of protective structures to establish safe response limits for given blast-loading scenarios may be explored.

(iii) The study of the response of protective structures to internal blast waves from high explosive charges is highly emphasized.

(iv) Concrete / masonry walls used in Pakistan may be experimentally and analytically evaluated against impulsive loading.

(v) The relation of the peak strain to the shape of the blast wave may be investigated on the containment structure undergoing one-dimensional motion.
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ANNEXURE A
Calculation of Ground Shock Parameters through CONWEP

Figure A.1: CONWEP Configuration
Figure A.2: Weapons effects calculations
Figure A.3: Ground Shock
Figure A.4: Bare HE
### Table of Explosion Materials and Parameters

<table>
<thead>
<tr>
<th>Name</th>
<th>Mott Scaling Constant Bx (3)</th>
<th>Gurney Constant n/s</th>
<th>Equ. Weight for Pressure</th>
<th>Equ. Weight for Impulse</th>
</tr>
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<tbody>
<tr>
<td>ANFO (AmNi/Fuel Oil) &lt;2&gt;</td>
<td>0.22</td>
<td>2530.</td>
<td>0.82</td>
<td>0.82</td>
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<tr>
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<td>2682.</td>
<td>1.09</td>
<td>1.07</td>
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<tr>
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<tr>
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<td>2316.</td>
<td>1.07</td>
<td>0.96</td>
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**Figure A.5: TNT**
<table>
<thead>
<tr>
<th>Name</th>
<th>Mott Scaling</th>
<th>Gurney Scaling</th>
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<td></td>
<td>Constant Bx</td>
<td>Constant m/s</td>
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<td>User Defined Explosive</td>
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Figure A.6: TNT (Contd.)
Annexure A
Calculation of Ground Shock Parameters through CONWEP

<table>
<thead>
<tr>
<th>Material Description</th>
<th>Density kg/cubic m</th>
<th>Seismic Velocity c m/s</th>
<th>Acoustic Impedance rho-c kPa/n/s</th>
<th>Attenuation Coefficient n</th>
</tr>
</thead>
<tbody>
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<td>Loose, dry sand and gravels with low relative density</td>
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<td>183</td>
<td>271</td>
<td>3.1</td>
</tr>
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<td>Sandy loam, loess, dry sands, and backfill</td>
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<td>995</td>
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Enter backfill density, kg/m\(^2\) = 1920
Enter seismic velocity, m/s = 1524
Enter attenuation coefficient = 3.1

Figure A.7: Material description
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<th>Velocity (m/s)</th>
<th>Impedance (ρc) (kPa/m/s)</th>
<th>Attenuation Coefficient (n)</th>
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Enter backfill density, kg/m^3: 1920
Enter seismic velocity, n/s: 1524
Enter attenuation coefficient: 3.1

Do you want to include reflections from a deeper layer? [ ]
Do you want to include tensile reflections from the ground surface? [ ]

Figure A.8: Reflections status
### Figure A.9: Input and Output

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Press <Enter> to continue ...
Figure A.10: Show Stress-time history
Figure A.11: Show velocity time history
### Table A.1: Ground Shock Free Field P-T History

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<th>Time (msec)</th>
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Table A.2: Ground Shock Free Field Velocity Time History

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Effects of an External Explosion on a Concrete Structure
Ph.D. Thesis, UET Taxila Pakistan, March 2009
ANNEXURE B

Shell Element Internal Forces/Stresses Output Sign Convention

(SAP2000)

Figure B.1: Faces of a Shell Element (SAP 2000 (2008))
Figure B.2: Calculation of internal forces (SAP 2000 (2008))
Figure B.3: Positive Directions for Internal Forces (SAP 2000 (2008))
Figure B.4: Positive Directions for Shell Element Forces (SAP 2000 (2008))
Figure B.5: Positive Directions for Shell Element Internal Moments (SAP 2000 (2008))
Figure B.6: Positive Directions for Shell Element Principal Moments (SAP 2000 (2008))
Figure B.7: Illustration of Shell Element Stresses (SAP 2000 (2008))
Figure B.8: The location of Shell Element internal Stress Values (SAP 2000 (2008))
Figure B.9: Illustration of positive directions of Shell Element internal Stresses

(SAP 2000 (2008))
Figure B.10: Distribution of internal Stresses acting on Shell Element (SAP 2000 (2008))
Figure B.11: Shell Forces (SAP 2000 (2008))
Plane and Asolid Element Stresses

Figure B.12: Plane and Asolid Element Stresses (SAP 2000 (2008))
Figure B.13: Shell Element Internal Forces (SAP 2000 (2008))
Figure B.14: Solid Element Stresses (SAP 2000 (2008))