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Analysis of Reinforced Concrete Building under Blast Loading Using SAP 2000

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Abstract:
Now-a-days terrorist attacks are expanding quickly everywhere throughout the world. They were assaulting the place like open spots, gather buildings, essential structure and so forth. So subsequently their future huge property loss, disappointment of close by structures and some human loss likewise will emerge if the assault is extreme one. The majority of the human loss is happening because of crumple of structures. So on the off chance that we figure out how to plan the structures to oppose the blasting or a blast we can ready to lessen the harm and just as the human loss moreover. So to plan the structure to oppose the blasting, first we should know how the structure or building is acting under the state of blasting or under the impact loading. To decide the conduct of structure we should know the loading parameters on the structure to break down. So the primary objective of this investigation is to decide the shoot loading on the structure and to decide the conduct of RC structure under that loading. With the goal that we can know, how the structure will carry on or can perform under blasting. In this project work, the execution or conduct of a hundred feet structure under the impact loading is resolved. Two blast loads were considered to decide the conduct of 100 feet structure. The loads are of 5000lbs. These blast loads were detonated in various three standoff separations, said to be as 50, 100, and 150 feet's. At every standoff separate the conduct of structure is resolved under two touchy loads. The impact parameters were resolved according to the Army Corps of Engineers, Department of Defense, U.S. Air Force and Indian Code IS:4991-1968. The loading on the structure is dynamic in nature so the came about impact parameters which are resolved is given as the contribution for the structure in Time-History examination of SAP 2000. So thus we can decide the removals, increasing speed and speeds regarding comparing times, which are said to be as the execution of the structure.

Keywords : Terrorist, Response of building, Blast Loading, Blast scaling

Introduction

The standard of building design is to accomplish the doled out objectives under the endorsed interest. Most recent couple of decades have seen colossal harms because of abnormal states erratic loading emerging because of ecological loading, in particular impact loading is one of them. The powerlessness evaluation of quake safe building structures is fairly old, yet the majority of the learning regarding this matter has been aggregated amid the previous fifty years. Likeness and disparity of design objectives under these two loadings are to secure/oppose the auxiliary and non basic execution in the anticipated way. A seismic tremor safe building structures is permitted to take points of interest of ductility amid serious quake loading, be that as it may, similar structures don't take excessively ductility under vast impact loading. The impact issue is fairly new; data about the advancement in this field is made accessible for the most part through production of the Army Corps of

Engineers, Department of Defense, U.S. Air Force and other administrative office and open organizations. A significant part of the work is finished by the Massachusetts Institute of Technology (MIT), the University of Illinois, and other driving instructive establishments and designing firms.

Fiascos, for example, the terrorist bombings of the U.S. international safe havens in Nairobi, Kenya and Dar es Salaam, Tanzania in 1998, the Khobar Towers military sleeping quarters in Dhahran, Saudi Arabia in 1996, the Murrah Federal Building in Oklahoma City in 1995, and the World Trade Center in New York in 1993 have shown the requirement for an intensive examination of the conduct of structures exposed to impact loads. To give satisfactory security against blasts, the design and development of open buildings are accepting reestablished consideration of auxiliary architects in light of the fact that the impact of impact is exceptionally intricate to get it. The point in impact obstruction structure is to maintain a strategic distance from dynamic collapse. Since the serious issue while blast is dynamic is the collapse of structure. So on the off chance that we can't stop the dynamic collapse of building however can lessen the harm of blast: either human loss or a property loss.

The significant risk after a blast is the dynamic collapse. Furthermore, today the focal point of every one of the an architects and designers are to capture the dynamic collapse of building. A definitive objective is that, the structure ought to be shielded from the impact, which is probably going to be the Target of terrorist attacks for the most part. The dynamic reaction of the structure to impact loading is mind boggling to break down, on account of the non-direct conduct of the materials just as the geometry. Consequently, examinations and design of impact loading requires nitty gritty information of impact and its wonders.

Strong explosives are primarily high explosives for which impact are best known. Materials, for example, mercury blasts and lead azide are essential explosives. Auxiliary explosives are those make impact wave which can result in across the board harm to the environment. Precedents incorporate trinitrotoluene (TNT) and ANFO (ammonium nitrate fuel oil). Two blast loads were considered to decide the conduct of 100 feet structure. The loads are of 5000lbs. These blast loads were detonated in various three standoff separations, said to be as 50, 100, and 150 feet's. At every standoff remove the conduct of structure is resolved under two unstable loads. The impact parameters were resolved according to the Army Corps of Engineers, Department of Defense, U.S. Air Force[1].The loading on the structure is dynamic in nature so the came about impact parameters which are resolved is given as the contribution for the structure in Time-History investigation of SAP 2000.

Blast Loading Concept

Explosion

An explosion is characterized as, quick and sudden arrival of vitality. Hazardous materials can be grouped by their physical state as solids, fluids or gases. Strong explosives are chiefly high explosives for which impact are best known. Secondary explosives are those make impact wave which can result in boundless harm to the environment. Models incorporate trinitrotoluene and ammonium nitrate fuel oil.

What exactly happens during blasting

The blasting of consolidated unstable produces hot gases under strain and a temperature of around 3000-4000oC. The

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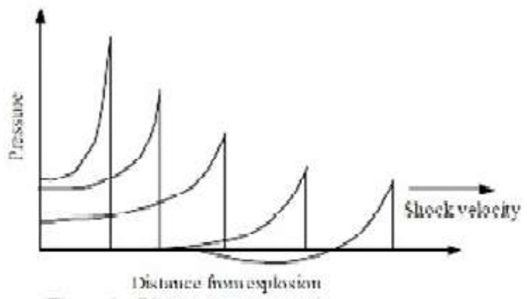
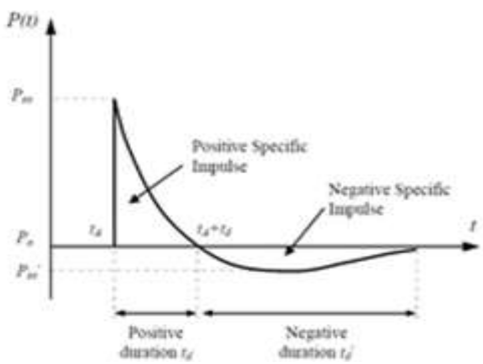
hot gas extends driving out the volume, it involves. As a result, a layer of compacted air(blast wave) shapes before this gas volume the majority of the vitality discharged by the explosion. Impact wave quickly increments to an estimation of weight all the more then the surrounding weight. This is alluded to as the side-on overpressure that rots as the stun wave grows outwards from the explosion source.

A little while later the weight falls beneath the surrounding weight as appeared in Fig-3.2. This stage is only the negative stage. The zone which is having a pinnacle overpressure more than the encompassing weight and directly diminished to the surrounding weight is known as the positive stage.



1. Figure: 3.1. Blast wave propagation



2. Figure: 3.2. Blast wave pressure-time history.

Positive duration is much lesser than the negative duration. The overpressure (p_{so}) in the positive duration is much greater than the pressure in the negative pressure (p_{so}^{-}).

Determination of blast loading Parameters

Blast loading can be determined by some empirical expressions and by some other codes or by some provisions. So in general the blast load is calculated by

- Empirical expressions determined by some number of experiments.
- As per Indian code IS 4991-1968.
- Provisions as per unified facilities criteria (UFC 3-340-02, 5 December 2008.).

Indian code had mentioned only the effect of the positive duration and positive over pressures. The effect of the negative duration and the negative over pressure is not considered. To determine the exact and near to exact analysis of the building, the effect of negative over pressure should also consider.

By Empirical Expressions

Use of the TNT (Trinitrotoluene) as a reference for determining the scaled distance Z, is universal. The first step in quantifying the explosive wave from a source other than the TNT, is to convert the charge mass into an equivalent mass of the TNT. It is performed so that the charge mass of explosive is multiplied by the conversion factor based on the specific energy of the charge and their TNT. Specific energy of different explosive types and their conversion factors of that of the TNT are given in the next table.

Table: 3.1.Conversion factors for different type of explosives.

EXPLOSIVE	Specific Energy	TNT Equivalent
	Qx/ KJ/Kg	Q_x/Q_{TNT}
Compound B (60% RDX and 40% TNT)	5190	1148
RDX (Ciklonit)	5360	1185
HMX	5680	1256
Nitro-glycerine (liquid)	6700	1481
TNT	4520	1000
Explosive gelatine (91 % nitro-glycerine, 7,9% nitrocellulose, 0,9 % antracid, 0,2 % water)	4520	1000
60 % Nitro glycerine	2760	600

dynamite		
Semtex	5660	1250

Explosion wave front speed U = $a_o \sqrt{\frac{6Pso+7Po}{7Po}}$

Where a_o = speed of sound in m/sec
Alternative expression U = $345(1+0.0083P_{so}^2)$ in m/sec
Dynamic (blast wave) pressure $q_o = \frac{5P_{so}^2}{2(P_{so}+7P_o)}$

It can be written also as $q_o = 0.0032P_{so}^2$ in kpa
Where P_{so} = peak over pressure
P_o = ambient pressure
There are various proposals for calculation of the main explosion parameters.
New marks and Hansen's [9] proposed the use of following values

$P_{so} = 6874\frac{W}{R^3}+93\sqrt{\frac{W}{R^3}}$

Mills[10] proposed the following
$P_{so}=\frac{1772}{Z^3}+\frac{114}{Z^2}+\frac{108}{Z}$-0.019 kpa
Brode[11] gives the following expressions for to determine the peak over pressures,
$P_s=\frac{6.7}{Z^3}+1$ ......bars $P_s>10$ bars
$P_s = \frac{0.975}{Z} + \frac{1.455}{Z^2} + \frac{5.85}{Z^3} - 0.019$, $0.1<P_s<10$ bars
Where scaled distance Z = $\frac{R}{\sqrt[3]{W}}$
R = distance from the centre of the spherical charge
W = charge mass expressed in kilogram of TNT
Other important parameters include,
t_o = duration of the positive phase during which the a pressure is greater than the Pressure of the surrounding air
i_s = the specific wave impulse that is equal to the area under the pressure-time Curve from the moment of arrival, t_A, to the end of the positive phase and is given by expression

$i_s=\int_{t_A}^{t_A+t_o} p_s(t)dt$

$i_s=\int_{t_A}^{t_A+t_o} p_s(t)dt$

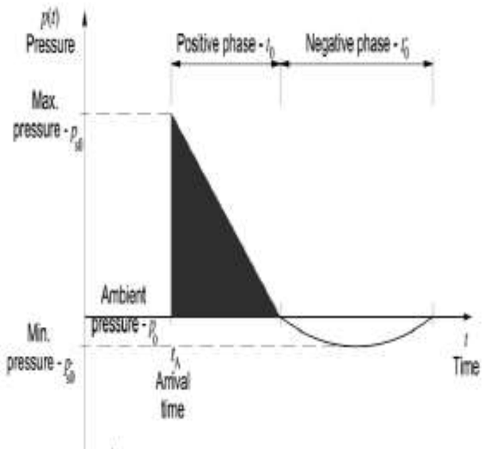


Figure: 3.4. Pressure-Time profile of Explosion Wave
Where p_{so}^- is the maximum value of negative pressure.
Brode [11] proposed the following expression for negative pressure
$p_{so}^- = -\frac{0.35}{Z}$: bars Z>1.6.
And the corresponding negative impulse specific force is given by

$i_s^- = i_s(1-\frac{1}{2Z})$

CALCULATION OF BLAST PARAMETERS ANALYTICAL SOLUTION

We assumed that, the blast wave is considered as plane. The blast parameters are determined as follows;

DESCRIPTION OF DATA FOR TRIAL-I

- Size of building 60fts X 60fts.
- Distance of building from the origin of explosion, R= 150 ft
- Height of the building H= 100ft
- Explosive weight W= 2500 lbs
- Scaled distance Z= $\frac{R}{W^{1/3}}$ = 11.05 ft/$lb^{1/3}$

DETERMINATION OF BLAST PARAMETERS

Determination of following free-field blast wave parameters at Point A:

- peak positive incident pressure P_{so}
- time of arrival of blast wave t_A
- wave length of positive pressure phase L_W
- duration of positive phase of blast pressure t_o.

From fig-2-15[1] for Z= 11.05 ft/$lb^{1/3}$;

- P_{so} = 7.93 psi
- $\frac{t_a}{W^{1/3}}$ = 5.154 ms/$lb^{1/3}$
- $\frac{L_W}{W^{1/3}}$ = 2.53 $ft/lb^{1/3}$
- $\frac{t_o}{W^{1/3}}$ = 2.75 ms/$lb^{1/3}$
- Specific impulsive force i_s = 7.41x$2500^{1/3}$ = 100.57 psi ms

FRONT WALL PEAK POSITIVE REFLECTED PRESSURE

From fig-2-193[1] ;

P_{so} = 7.93 psi and $\alpha = 0^o$, C_{ra} = 2.38.

Therefore reflected peak pressure is given by, $P_{ra} = C_{ra} * P_{so}$ = 2.38*7.93 = 18.87.

Unit positive reflected impulse from fig-2-194[1] ;

$\frac{i_{ra}}{W^{1/3}}$ = 12.81, i_{ra} = 174.05 psi.

FRONT WALL LOADING POSITIVE PHASE

Calculation of sound velocity in reflected over pressure region,

C_r from 2-192[1]; P_{so} =7.93 psi C_r = 1.25 ft/ms

Clearing time for reflected pressure t_c;

$t_c = \frac{4S}{(1+R)Cr} = \frac{4*30}{(1+0.6)\,1.25}$ = 71.00 ms.

Where S= 30ft (60/2 =30);

G= 100/2 =50> 30, so G= 50

R = S/G = 30/50 =0.6

Calculation of fictitious positive phase duration,

$t_{of} = 2i_s/P_{so}$ = (2x100.57)/7.93 = 25.36 ms.

From fig- 2-3[1]; peak dynamic pressure is given by,

P_{so} = 7.93 psi then q_o =1.43 psi.

Drag coefficient based on from suction, C_D =1.0 then $P_{so}+C_Dq_o$ = 7.93+ 1*1.43 = 9.36 psi.

Calculation of factitious duration of the reflected pressure acc to equation

$t_r = \frac{2i_{ra}}{P_{so}} = \frac{2*174.05}{18.87}$ = 18.44 ms

pressure time curve is plotted in fig.

FRONT WALL LOADING NEGATIVE PHASE

Peak positive reflected pressure P_{ra} =18.87 psi, then from fig 2-15[1]

$Z(P_{ra})$ =11.20;

Peak Negative pressure is P_a^- = 1.60 psi for Z = 11.20

Fictitious negative phase duration, t_{rf}^- = 0.0139 *$W^{1/3}$ = 144.56 ms

Negative specific impulsive force is given by, $\frac{i_{ra}^-}{W^{1/3}}$ = 18.80, i_{ra}^- = 246.34 psi ms

Therefore negative phase rise = 0.27 t_{rf}^- = 0.27*144.56 = 39.03 ms

The negative phase time parameter t_o+0.27 t_{rf}^- = 37.32+39.03 = 76.35 ms

Total negative phase duration, $t_o + t_{rf}^-$ = 37.32+144.56 = 181.88 ms.

SIDE WALL LOADING POSITIVE PHASE

Calculation of loading on the rear half of the side wall L =30 ft

Wavelength to span ratio = L_w/L = 38/30 =1.27

Based on fig- 2-196, 2-197 and 2-198 [1] for point on B L_w/L = 1.27,

- P_{sof} = 5.74 psi, C_E =0.53, C_E^-=0.26, $\frac{t_r}{W^{1/3}}$ = 1.78, $\frac{t_{of}}{W^{1/3}}$ = 4.2, and $\frac{t_{of}^-}{W^{1/3}}$ = 11.52.

Where C_E = equivalent load factor, t_d = rise time, t_r = fictitious reflected pressure duration,t_{of} = fictitious positive pressure phase duration

Therefore peak positive pressure P_{so} = C_E*P_{sof} =0.53*5.74 = 3.04 psi

t_r =1.78*$2500^{1/3}$ = 24.15 ms

t_{of}= 5.2 * $2500^{1/3}$ = 27.00 ms.

Peak dynamic pressure from fig-2-3[1]

$C_E\ P_{sof}$= 3.04 then q_o = 0.23 psi.

Drag coefficient is given as C_D = -0.4,

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Calculation of peak positive pressure from equation $C_E\ P_{sof} + C_D\ q_o$ =3.04-0.4*0.23= 2.95 psi

SIDE WALL NEGATIVE PRESSURE PHASE

Peak negative reflected pressure (P_r^-) = C_E^- P_{sof}= 0.26*5.74 =1.50 psi

Negative phase duration t_{of}^- = 11.52*2500$^{1/3}$ =156.35 ms

Negative phase rise time 0.27*t^-_{of} = 42.21 ms

The negative phase time parameter t_o =40.22 ms

Peak rise time t_o +0.27 * t_{of}^- = 40.25+42.21 = 82.46 ms

Total negative duration t_o + t_{of}^- = 40.25+156.35 =196.60 ms

ROOF LOADING – POSITIVE PHASE

Calculation of roof loading, L= 60fts

$\frac{L_W}{L} = 0.68\ and$ P_{sof} =4.44 psi

Based on fig 2-196, 2-197, 2-198[1];

- C_E = 0.35, C_E^- =0.22
- $\frac{t_d}{W^{1/3}}$ = 2.49,
- $\frac{t_{of}}{W^{1/3}}$ = 6.93; $\frac{t_{of}^-}{W^{1/3}}$ = 12.43

Hence peak positive pressure is $C_E P_{sof}$ = 0.35*4.44 =1.56 psi

Rise time t_r =2.49*2500$^{1/3}$ = 33.79 ms and t_{of} = 84.05 ms

Peak dynamic pressure from fig-2-3[1];

For $C_E P_{sof}$ =1.56 psi then q_o =0.13 psi

Calculation of peak positive reflected pressure $C_E P_{sof}$+$C_D q_o$= 1.56-0.4*0.12= 1.51 psi

ROOF LOADING –NEGATIVE PRESSURE

Peak negative reflected pressure P_r^- = C_E^- P_{sof}^- =0.22*4.44= 0.98 psi.

Total Time of peak negative pressure t_{of}^-= 12.43*2500$^{1/3}$ = 168.70 ms

Negative pressure rise time 0.27*t^-_{of} = 0.27*168.70= 45.55 ms.

The negative pressure time parameter, t_o=42.48ms

There fore peak rise is t_o+ 0.27*t_{of}^-= 42.48+45.55 = 88.03 ms.

Total duration is t_o + t_{of}^- = 42.48+168.70= 211.18 ms.

The negative pressure-time curve is plotted in figure.

Table-4.1: Positive and negative peak over and under pressure for various faces without considering atmosphere pressure.

FACE	Peak over (or) under pressure			
	Positive pressure		*Negative pressure*	
	Psi	kN/m^2	Psi	kN/m^2
Front wall	18.87	130.104	-1.60	-11.03
Side wall	2.95	20.340	-1.50	-10.342
On roof	1.51	10.411	-0.98	-6.757

Table-4.2: Positive and negative peak pressures for various faces after considering the ambient pressure (101325 Pascal or 14.7 psi)

Face	Peak over or under pressure			
	Positive pressures		*Negative pressures*	
	Psi	kN/m^2	Psi	kN/m^2
Front wall	33.57	231.43	12.80	88.22
Side wall	17.65	121.65	13.20	90.983
On roof	16.21	111.73	13.72	94.568

The pressure VS time plots are as follows:

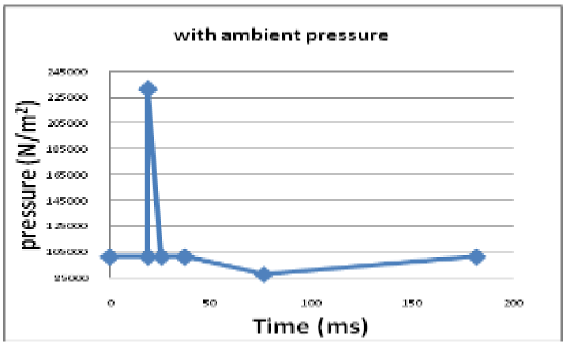

Figure 4.1.a: Pressure-time variation on front wall with and without considering ambient pressure.

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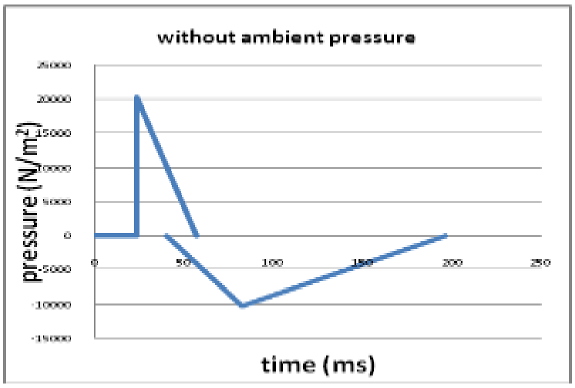


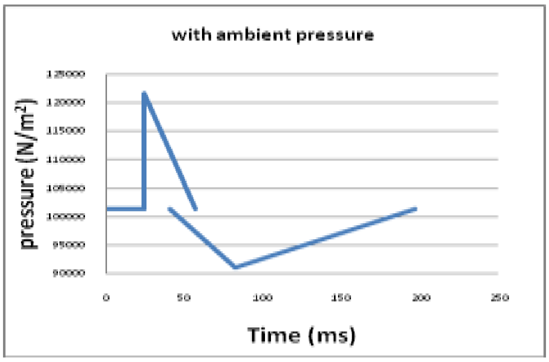


Figure 4.1.b.: Pressure-time plot on side wall with and without considering ambient pressure.

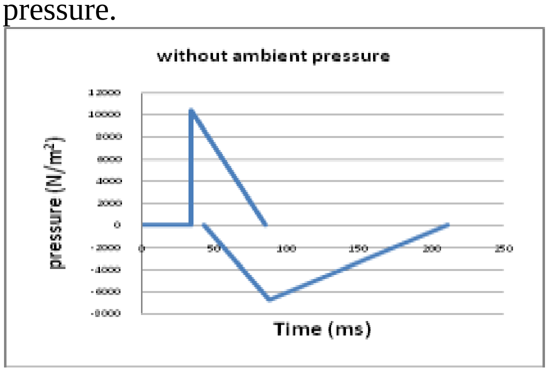


Figure 4.1.C. pressure-time plot on roof with and without ambient pressure.

DESCRIPTION OF DATA FOR TRIAL-2

- Size of building 60fts X 60fts.
- Distance of building from the origin of explosion R= 150 ft
- Height of the building H= 100ft
- Explosion weight W= 5000 lbs
- Scaled distance $Z=\frac{R}{W^{1/3}} = 8.77\ ft/lb^{1/3}$

For 5000lbs explosion weight, the positive and negative pressures are calculated as.

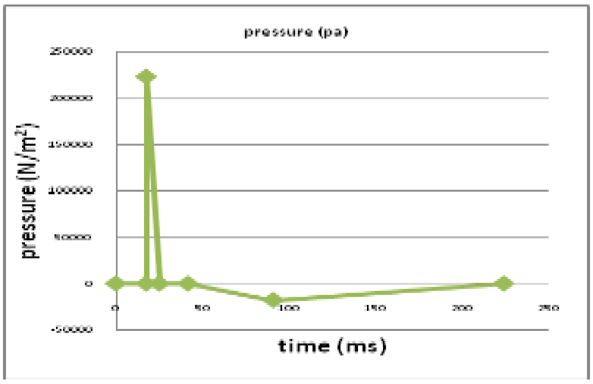
Table 4.3: Positive and negative peak over and under pressure for various faces without considering atmosphere pressure.

FACE	Peak over (or) under pressure			
	Positive pressure		*Negative pressure*	
	Psi	kN/m²	Psi	kN/m²
Front wall	32.40	223390.22	-2.6	-17927
Side wall	4.19	28890	-2.28	-15720
On roof	2.00	13789	-1.51	-10411

Table-4.4: Positive and negative peak pressures for various faces after considering the ambient pressure (101325 Pascal or 14.7 psi)

Face	Peak over or under pressure			
	Positive pressures		*Negative pressures*	
	Psi	kN/m²	Psi	kN/m²
Front wall	47.10	324715	12.10	83398
Side wall	18.90	130215	12.42	85605
On roof	16.70	115114	13.20	90914

The pressure and time plots are as follows:



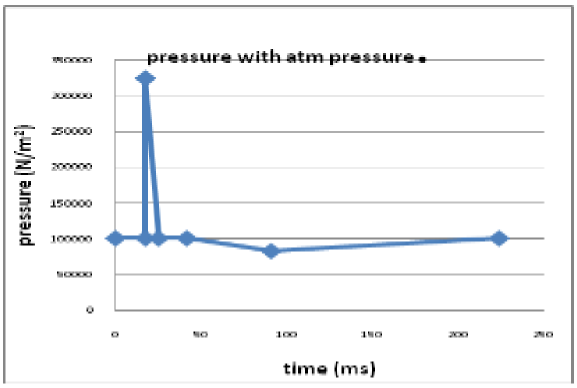


Figure 4.2.a: Pressure-time variation on

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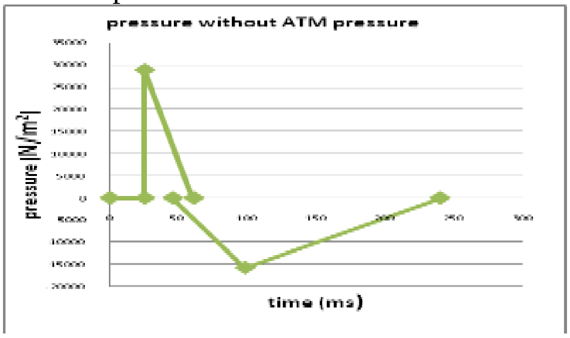
front wall without and with considering ambient pressure.



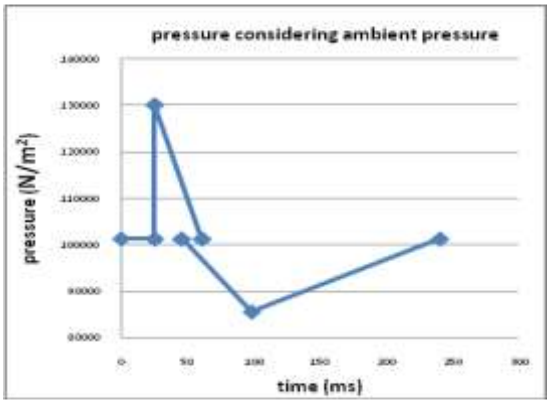
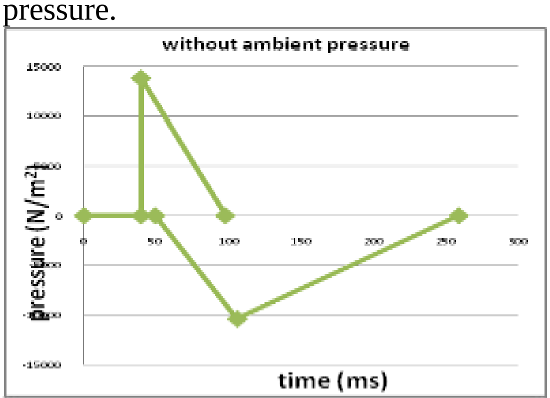


Figure 4.2.b**:** Pressure-time plot on side wall with and without considering ambient pressure.



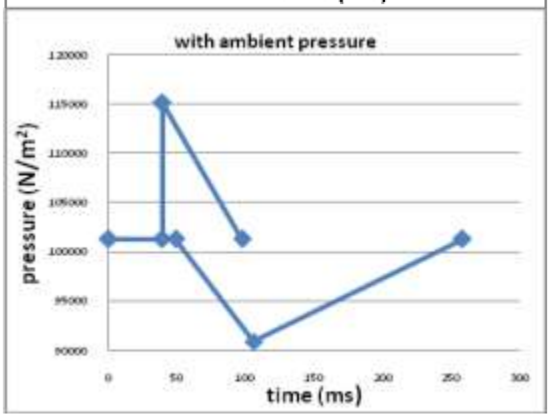


Figure 4.2.c.pressure-time plot on roof with and without ambient pressure

DESCRIPTION OF DATA FOR TRIAL-3

- Size of building 60fts X 60fts.
- Distance of building from the origin of explosion R= 100 ft
- Height of the building H= 100ft
- Explosion weight W= 2500 lbs
- Scaled distance Z= $\frac{R}{W^{1/3}}$ = 7.37 ft/$lb^{1/3}$

Table-4.5: Positive and negative peak over and under pressure for various faces without considering atmosphere pressure.

FACE	Peak over (or) under pressure			
	Positive pressure		*Negative pressure*	
	Psi	kN/m^2	Psi	kN/m^2
Front wall	51.60	355.77	-3.6	-24.82
Side wall	6.98	48.12	-1.70	-11.7
On roof	5.10	35.16	-1.2	-8.27

Table-4.6: Positive and negative peak pressures for various faces after considering the ambient pressure (101.325 kilo Pascal or 14.7 psi)

FACE	Peak over (or) under pressure			
	Positive pressure		*Negative pressure*	
	Psi	kN/m^2	Psi	kN/m^2
Front wall	66.30	457.095	11.10	76.50
Side wall	21.08	148.445	13.00	89.60
On roof	19.20	136.48	13.5	93.05

Pressure and time plots

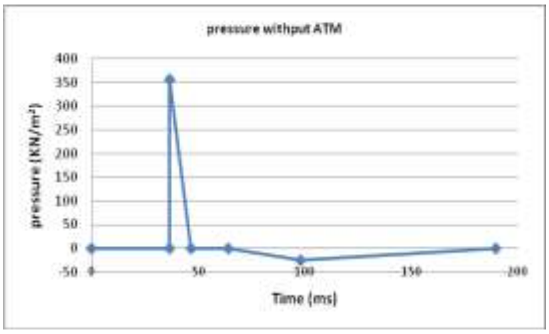


Figure 4.3: Variation of blast pressure on front face without atmospheric pressure.

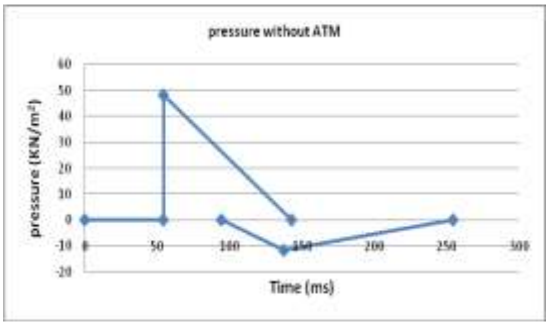


Figure 4.3: Variation of blast pressure on side face without ATM pressure.

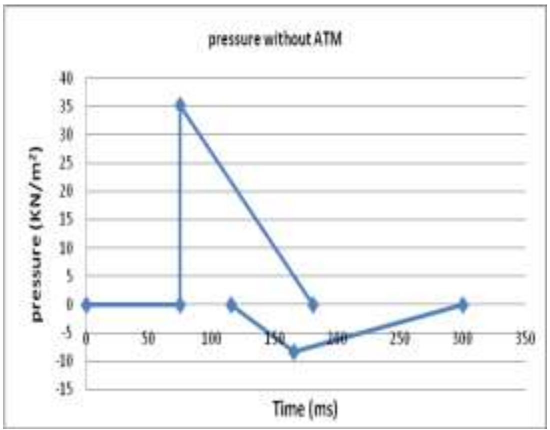


Figure 4.3: Variation of blast pressure on Roof without considering ATM pressure.

DESCRIPTION OF DATA FOR TRAIL-4

- Size of building 60fts X 60fts.
- Distance of building from the origin of explosion R= 100 ft
- Height of the building H= 100ft
- Explosion weight W= 5000 lbs
- Scaled distance Z= $\frac{R}{W^{1/3}}$ = 5.85 ft/$lb^{1/3}$

Table- 4.7 positive and negative peak over and under pressure for various faces without considering atmosphere pressure.

Face	Peak over or under pressure			
	Positive pressures		*Negative pressures*	
	Psi	kN/m^2	Psi	kN/m^2
Front wall	87.73	604.8	-4.60	-31.71
Side wall	18.30	126.1	-3.20	-22.03
On roof	10.18	70.19	-2.34	-16.14

Table-4.8: Positive and negative peak pressures for various faces after considering the ambient pressure (101.325 kilo Pascal or 14.7 psi)

Face	Peak over or under pressure			
	Positive pressures		*Negative pressures*	
	Psi	kN/m^2	Psi	kN/m^2
Front wall	102.43	706.23	10.10	69.62
Side wall	33.00	227.50	11.50	79.03
On roof	24.88	171.51	12.36	85.185

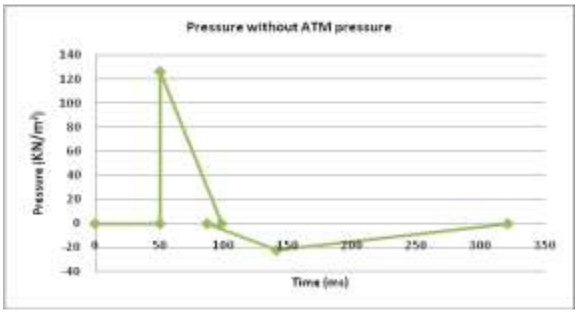


Figure : 4.4.b Pressure-Time plot on side faces without considering ATM pressure.

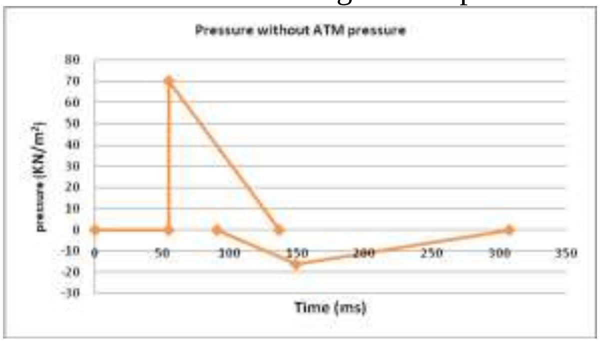


Figure : 4.4.c Pressure-Time plot on Roof without considering ATM pressure.

DESCRIPTION OF DATA FOR TRIAL-5

- Size of building 60fts X 60fts.
- Distance of building from the origin of explosion R= 50 ft
- Height of the building H= 100ft
- Explosion weight W= 2500 lbs
- Scaled distance Z= $\frac{R}{W^{1/3}}$ = 5.85 ft/$lb^{1/3}$

Table 4.9: Positive and negative peak over and under pressure for various faces without considering atmosphere pressure.

Face	Peak over or under pressure			
	Positive pressures		*Negative pressures*	
	Psi	kN/m^2	Psi	kN/m^2

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Front wall	**402.10**	**2772.38**	**-9.20**	**-63.43**
Side wall	**28.91**	**199.32**	**-2.20**	**-15.17**
On roof	**9.98**	**68.81**	**-1.6**	**-11.03**

Table-4.10: Positive and negative peak pressures for various faces after considering the ambient pressure (101.325 kilo Pascal or 14.7 psi)

Face	Peak over or under pressure			
	Positive pressures		*Negative pressures*	
	Psi	kN/m^2	Psi	kN/m^2
Front	416.80	2873.70	5.50	37.89
Side	43.61	300.64	12.50	89.155
On roof	24.68	170.13	13.10	90.30

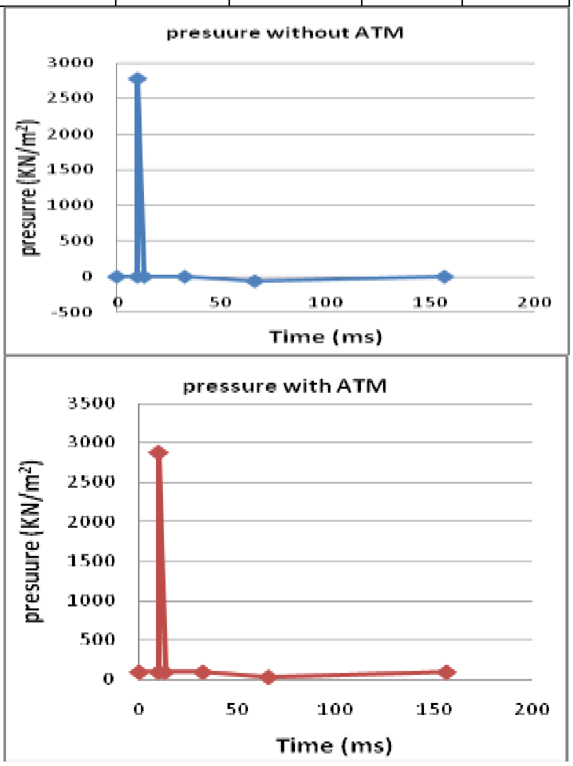
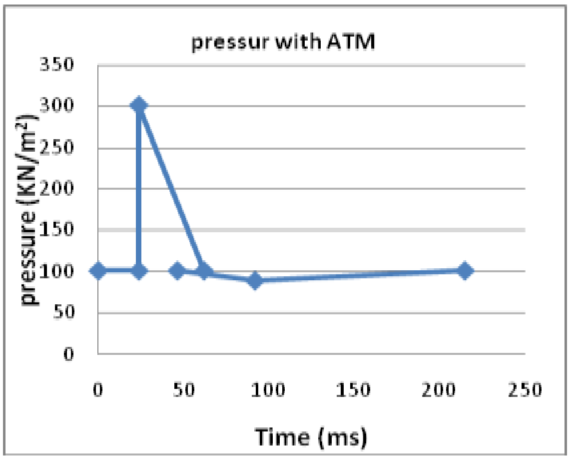


Figure 4.5.a :Pressure variation on front wall with and without ATM pressure



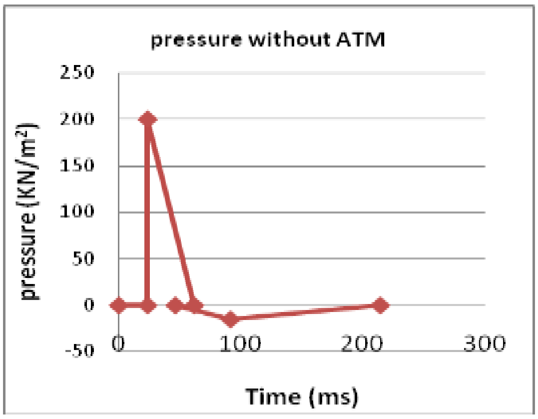
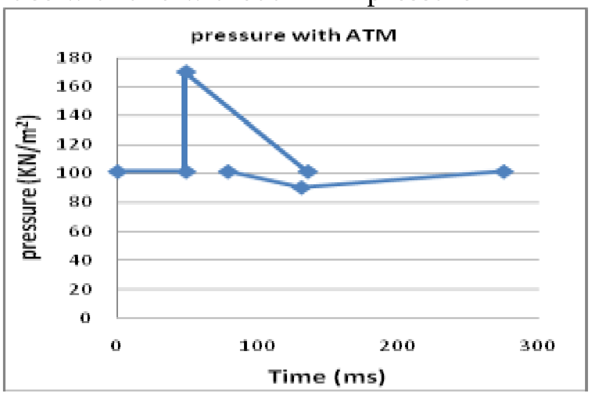


Figure 4.5.b: Pressure variation on side face with and without ATM pressure



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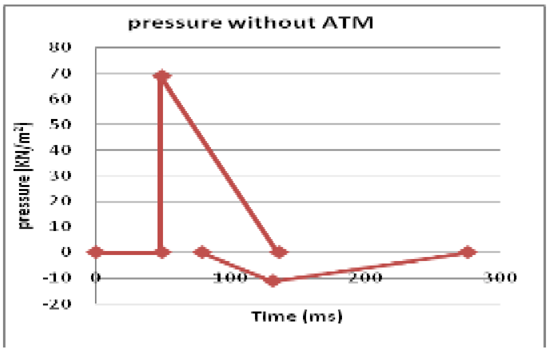

Figure 4.5.c: Variation of pressure on roof with and without ATM pressure

1.1.1. 4.1.6. DESCRIPTION OF DATA FOR TRAIL-6

- Size of building 60fts X 60fts.
- Distance of building from the origin of explosion R= 50 ft
- Height of the building H= 100 ft
- Explosion weight W= 5000 lbs
- Scaled distance Z= $\frac{R}{W^{1/3}}$ = 5.85 ft/$lb^{1/3}$

Table-4.11: Positive and negative peak over and under pressure for various faces without considering atmosphere pressure.

Face	Peak over or under pressure			
	Positive pressures		*Negative pressures*	
	Psi	kN/m^2	Psi	kN/m^2
Front wall	840.0	5791.60	-14.20	-97.90
Side wall	65.00	448.16	-2.70	-18.62
On roof	24.68	170.13	-1.54	-10.62

Table- 4.12 positive and negative peak pressures for various faces after considering the ambient pressure (101.325 kilo Pascal or 14.7 psi)

Face	Peak over or under pressure			
	Positive pressures		*Negative pressures*	
	Psi	kN/m^2	Psi	kN/m^2
Front wall	854.70	5892.9	0.50	8.425
Side wall	79.70	549.485	12.0	82.70
On roof	39.38	271.45	13.16	90.705

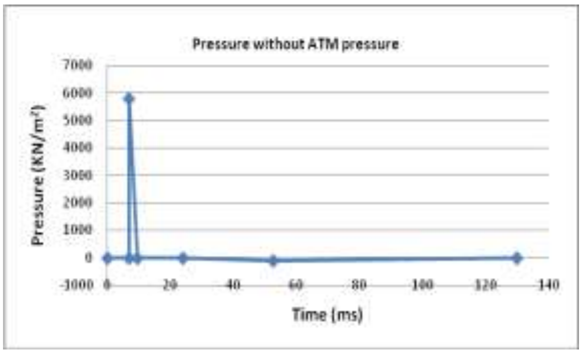

Figure 4.6.a Pressure-Time plot on Front face without considering ATM pressure.

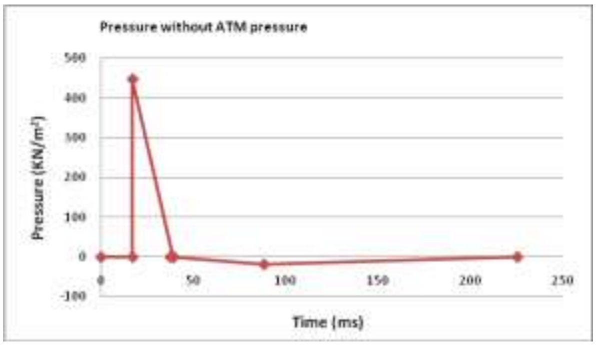

Figure 4.6.b Pressure-Time plot on Side face without considering ATM pressure.

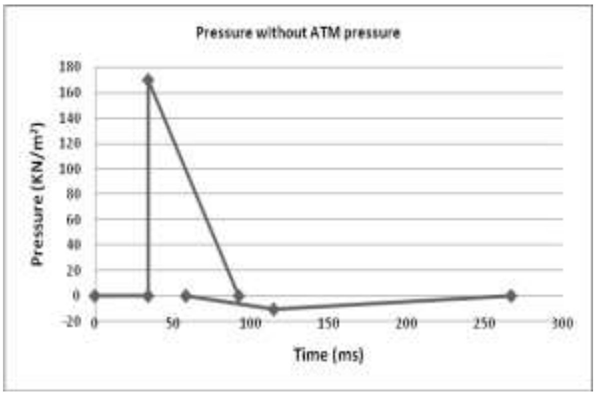

Figure 4.6.c Pressure-Time plot on Roof without considering ATM pressure.

For two explosive weights 2500 and 5000 lbs at a standoff distances of 50, 100 and 150 feet's, the pressure variation (positive and negative pressures) are determined on different faces of the structure or building. In pressure-Time plots the peak positive pressure is much greater than the peak negative pressure. So we can conclude some points from the pressure-Time plots. The

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main points that I have observed is listed below.

- As above said the peak positive pressure is much greater than the peak negative pressure on all the faces of the building.
- The intensity of the peak reflected pressure are much more than the peak positive pressure. So, the effect of the reflected pressure is more on the front face (side where explosion occurred) of the building or structure.

- In case of side face and Roof of the building, the reflected pressure is less then the peak positive pressure. So the effect of the reflected pressure on these face is low when compare with the front face.
- Among the peak positive pressure and the reflected pressure, the greater value is considered on the face in Pressure-Time plots.
- The negative pressure on the front face started after the end of the positive pressure that to not an immediate occurrence, but started after some milliseconds as shown in pressure-Time plots.
- But in case of side faces of the structures, the Negative pressure started before the end of the positive pressure this is clearly observed in the pressure-Time plots.
- In case of the Roof, the Negative pressure is started much before departure of the positive pressure the variation as shown in plots.

CALCULATION OF IMPULSE OR BLAST LOADING

SPECIFIC WAVE IMPULSE (i_s)

The specific wave impulse that is equal to the area under the pressure-time curve from the moment of arrival, t_A, to the end of the positive phase and is given by expression

$$i_s=\int_{t_A}^{t_A+t_0} p_s(t)\,dt.$$

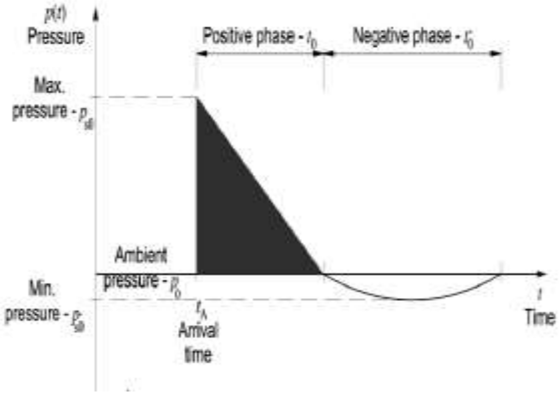
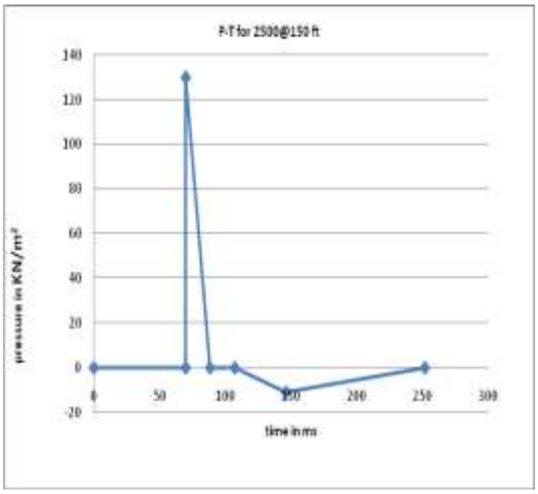


Figure 5.1 :Typical pressure-Time plot

- The specific wave impulse said to be positive whenever the area considered only the positive phase from the Time-pressure plot. It is denoted by the notation (i_s^+).
- If the area taken from the negative phase, then it known as the negative specific wave of impulse (i_s^-).
- This specific wave impulse is used to determine the impulsive force that is acting on the building.
- Impulsive force is further divided into positive impulsive force and negative impulsive force.
- Impulsive force is calculated as, $I_F = i_s*A_b$

Where A_b = blast influence area.

FOR 2500 lbs EXPLOSION WEIGHT



- Positive Specific wave impulse is given by, (i_s^+) = 0.5*18.44*130.10 = 1199.52 KN/m^2-ms.
- Negative specific wave impulse is given by (i_s^-) = 0.5*39.03*11.03 = 215.25 KN/m^2-ms.
- The radius of blast wave on the front face of the structure is given by the condition,

$$\frac{t_r}{t_c} = \frac{R}{S};$$

Where t_r = reflected positive duration = 18.87 ms

t_c = clearance time = 71.00 ms

R = Radius of blast wave on the building.

S = half the width of the building.= 9.15 m

Therefore the Radius is given by, R = $\frac{18.87 * 9.15}{71.00}$ = 2.37 m.

Positive impulsive force $I_f^+ = (i_s^+)* (\frac{\pi}{4} * (2 * 2.37)^2)$ = 21706.03 kN ms.

Similarly negative Impulse force = -3814.35 kN ms.

For different weights of explosion at different standoff distances, the positive and negative Impulsive forces were calculated as mentioned above and they are given in the below table.

Table 5.1 Impulse force and radius of blast wave for two explosion weights.

Explosion weight (lbs)	Standoff distance (ft)	Positive impulsive force (kN-ms)	Negative impulsive force (kN-ms)	Radius of blast wave R (m)
2500	50	32057	7552	1.50
	100	22241	4084	1.96
	150	21706	3814	2.38
5000	50	104585	19689	2.20
	100	68604	11390	2.46
	150	45161	10208	2.71

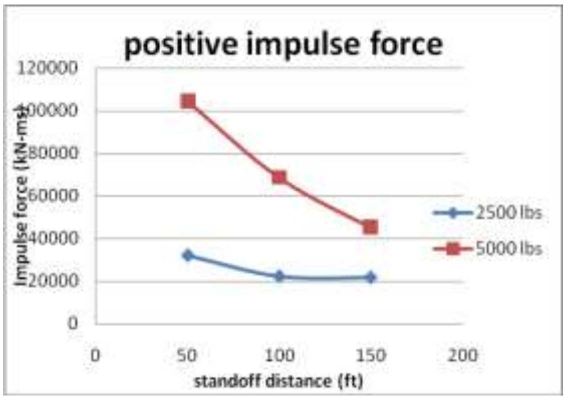


Figure 5.2 Variation of positive Impulsive force with respect to standoff distance.

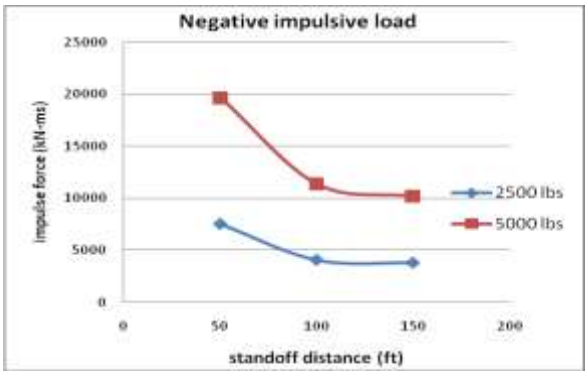


Figure 5.3 Variation of negative Impulsive force with respect to standoff distance.

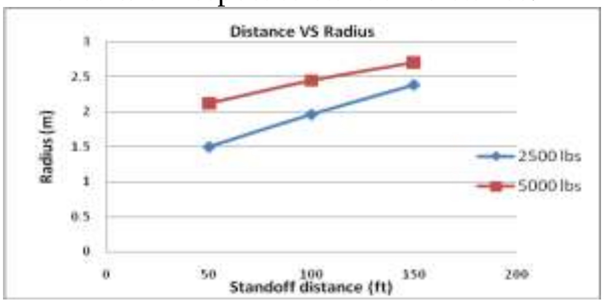


Figure 5.4 Variation of Radius of blast wave VS standoff distance.

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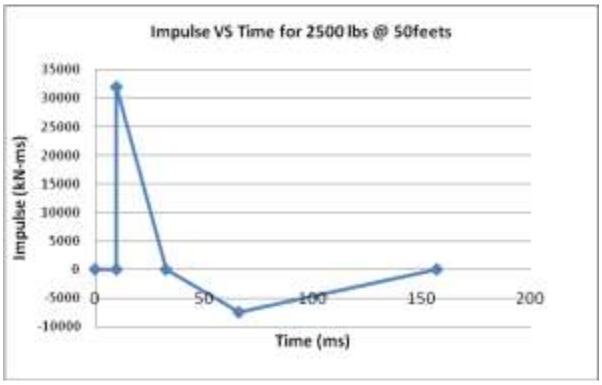


Figure 5.5 a. Typical plots for Impulsive force and Time for 2500 lbs @50 feet

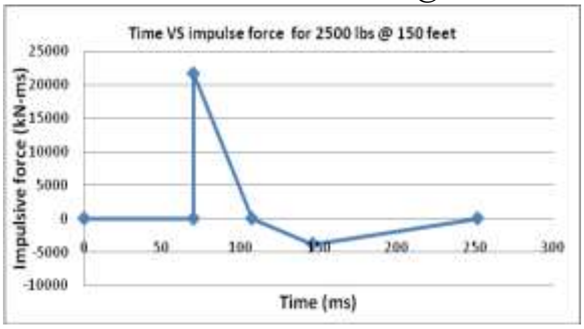
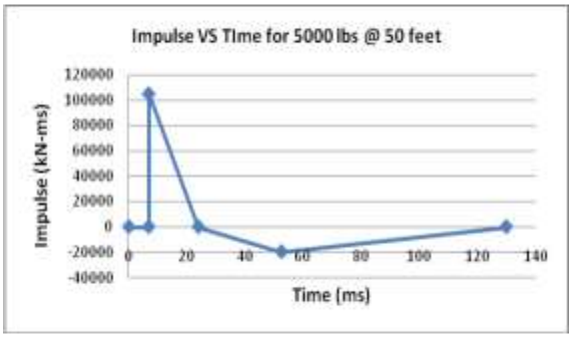
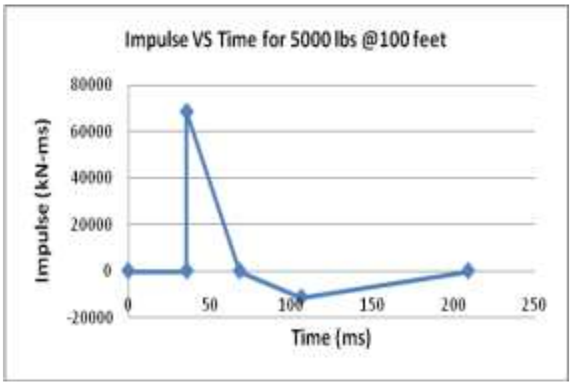
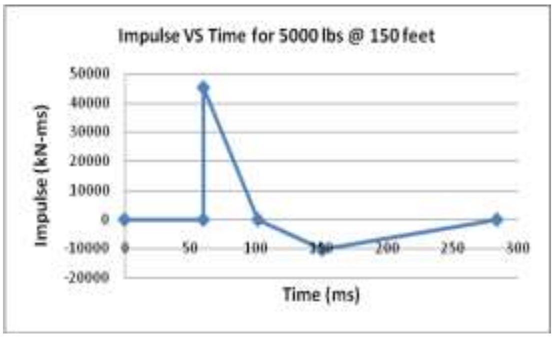


Figure 5.6 Typical plots for Impulsive force and Time for 5000 lbs @ 50, 100, 150 feet's.







MODELLING IN SAP 2000

ABOUT SAP2000:

The SAP name has been synonymous with state-of-the-art analytical methods since its introduction over 30 years ago. SAP2000 follows in the same tradition featuring a very sophisticated, intuitive and versatile user interface powered by an unmatched analysis engine and design tools for engineers working on transportation, industrial, public works, sports, and other facilities. From its 3D object based graphical modelling environment to the wide variety of analysis and design options completely integrated across one powerful user interface, SAP2000 has proven to be the most integrated, productive and practical general purpose structural program on the market today. Now we can harness the power of SAP2000 for all of your analysis and design tasks, including small day-to-day problems.

Complex Models can be generated and meshed with powerful built in templates. Integrated design code features can automatically generate wind, wave, bridge, and seismic loads with comprehensive automatic steel and concrete design code checks per US, Canadian and international design standards. Advanced analytical techniques allow for step-by-step large deformation analysis, Eigen and Ritz analyses based on stiffness of nonlinear cases, catenary cable analysis, material nonlinear analysis with fibre hinges, multi-layered nonlinear shell element, buckling analysis, progressive collapse analysis,

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energy methods for drift control, velocity-dependent dampers, base isolators, support plasticity and nonlinear segmental construction analysis. Nonlinear analyses can be static and/or time history, with options for FNA nonlinear time history dynamic analysis and direct integration. From a simple small 2D static frame analysis to a large complex 3D nonlinear dynamic analysis, SAP2000 is the easiest, most productive solution for structural analysis and design needs.

GENERAL INTRODUCTION

For performing the linear and Non-linear analysis to the framed structure by manually, is very difficult task and also a time consuming process. So huge manual errors will occur when we done by manually. To eliminate this type of errors and recent few decades implemented some software's to eliminate the difficulties. If we want to know the performance of any structure, firstly we should have to model the structure. So for modelling I opted for SAP2000. My intention is to determine the behaviour of the structure under blast loading. So to determine that first we should know the behaviour of explosion and shockwave then to model that building and to provide appropriate structural components. In order to accomplish the desire objectives, linear and nonlinear model time history analysis has been conducted on the building frames model in SAP2000 in this study. Concrete frame buildings have been taken where the frames have been used for performance evaluation and model using the background of software SAP2000. Using unified facilities criteria [1], the blast pressure time functions have been estimated and were applied to the building frames. Linear and nonlinear dynamic modal time history analysis is conducted for the modelled building frames. Subsequently analysis results were recorded for performance evaluation.

EVALUATION OF BLAST RESPONSE

Today in this present era, where the world got advanced with the latest technologies software's that may analyze 2D as well as 3D models with a good accuracy and better simulation with the actual effect of the disastrous loads on the structures. Using the environment of software it is now possible to automobile nonlinear analysis using SAP2000 in this study. Frame works modeled for linear and non linear response were run using the estimated base shear and response spectrum for linear analysis using blast pressure time curves for nonlinear analysis and in this direction the appropriate analysis is carried out.

It seems some odd that the frameworks are modelled previously in linear analysis using response spectrum analysis for considered earthquake ground motion.. As the direction, intensity, blast off distance and type of blast source is erratic. So, in this study a model is taken, which is previously checked for maximum effect of earthquake ground motion as for linear analysis and performance of the structure is the analyzed using nonlinear dynamic model time-history analysis. Details of building frame work are as follows,

- Size of the building 60ft X 60ft.
- Height of the building 100 feet's (10 storey building).
- Explosion weights 2500lbs and 5000lbs.
- Standoff distances are 50, 100, and 150 fts.

RESPONSE SPECTRUM ANALYSIS

Response-spectrum analysis (RSA) is a linear-dynamic statistical analysis method which measures the contribution from each natural mode of vibration to indicate the likely maximum seismic response of an essentially elastic structure. Response-spectrum analysis provides insight into dynamic behaviour by measuring pseudo-

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spectral acceleration, velocity, or displacement as a function of structural period for a given time history and level of damping. It is practical to envelope response spectra such that a smooth curve represents the peak response for each realization of structural period.

Response-spectrum analysis is useful for design decision-making because it relates structural type-selection to dynamic performance.Structural performance objectives should be taken into account during preliminary design and response-spectrum analysis.

DAMPING AND RSA

- RSA provides insight into how damping affects structural response. A family of response curves may be developed with variable levels of damping. As damping increases, response spectra shift downward.
- The International Building Code (IBC) is based on 5% damping. This accounts for incidental damping from hysteretic behaviour, which is not explicitly modelled during RSA.
- Viscous dampers do not affect structural stiffness, are not modelled during RSA, and are not accounted for in the IBC provision for 5% damping.

ADDITIONAL NOTES ON RSA

- All response quantities are positive, therefore RSA is not suitable for torsional irregularity. A static lateral-load procedure is best for measuring accidental torsion. The same applies when considering uplift and compression during foundation design.
- Modal response may be combined using SRSS, CQC, ABS, or GMC methods. CQC is best when periods are closely spaced, with cross-correlation between mode shapes. SRSS is suitable when periods differ by more than 10%.
- Ritz vectors are recommended for RSA because this formulation is computationally efficient. Only pertinent mode shapes which occur in the horizontal plane are identified. Eigen vectors use the full stiffness and mass matrices, which also account for vertical modes. Eigen formulation is useful when considering floor vibration, out-of-plane vibration of shear-wall systems, etc. Eigen application is also useful for locating modelling errors.

TIME-HISTORY ANALYSIS

Non-linear analysis is further divided into two types. They are non-linear static analysis or pushover analysis and nonlinear dynamic analysis or Time-History analysis. Pushover analysis is a broad variety of analysis method both elastic and inelastic, are available for the design of future buildings or for checking structures already exits. Since the most customary inelastic analysis procedure insists in the nonlinear time history analysis, which is regarded as impractical and time consuming, but it will give the accurate results.

Time history analyses are based upon the accelerograms that are applied at the base of buildings. The calculations of the structure performance under dynamic action can be considered either assuming elastic or inelastic (dynamic) behaviour. For the nonlinear time history analysis, as suggested by EN1998:1.2004 the mathematical model shall include the strength of structural element as well as their post-elastic behaviour. In case that realistic model is available, the nonlinear time history analysis is definitely more accurate method; structural behaviour including damage progression effect can be realistically traced, which allows an optimized structure design.

Time history analysis is based upon time-dependent numerical procedures, which are generally very complex and require very powerful calculating capacities. In order to allow statistically secured result, a large number of accelerogram should be used. At the same time, these accelerogram should be representative for the respective building site, .g. in terms of soil condition, distance to the source etc. Since these prerequisite is often not fulfilled, it is allowed to use simulated(synthetically generated) accelerogram that are consistent with source and path mechanisms and the underlying soil conditions, accelerogram shall be chosen accordingly to the provision given in E N 1998-1, section 3.2.3.1.

SOME MOST IMPORTANT POINTS IN TIME-HISTORY ANALYSIS

- Analysis of a structure, applying data over increment time steps as a function of,
  – Acceleration
  – Force
  – Moment, or
  – Displacement
- The smaller is time steps, the more accurate the solution will be.
- Eigen values generated for the structure based on response to time history.
- Considered to be more realistic compared to response spectrum analysis.
- Most useful for very long or very tall structures (flexible structures).

DRAW BACKS IN TIME HISTORY ANALYSIS

- It will take much time to perform analysis.
- Generates and require large quantities of Data.
- May not always reduce seismic forces in structure. Depends on
  -soil properties
  -structural type and available data

AIM OF MODELLING

The main objective of evaluating the performance of a system for unconditional unknowingly blast effect mainly depends on the preliminary design and the method of analysis.. There are two main concerns for modeling a beam-column or any other structural members.

- Force-displacement relationship. A beam-column member exerts a force on the adjacent members and the connections including these members have deformations that contribute to the displacement of the complete structure.
- Demand-capacity measures. Force and deformations are important for modelling the behaviour of the structure, but demand capacity ratios are required to access performance assessment without demand-capacity ratios, however member performance assessment requires demand-capacity ratios.

MODELLING OF BUILDING FRAMES

Frameworks modelled for nonlinear response were run for nonlinear static analysis using the estimated base shear under code based values. Details of the building frame works are listed below,

- Ten storey R C building frame of 18.28m x 18.28 m size.
- Height of each storey is 3.048 m.
- Grade of concrete for beams and columns is M25.
- Internal brick wall thickness is 150mm
- External wall thickness is 230mm.
- Density of concrete is 25 kN/m^3.
- Density of Brick work- 18 KN/m^3.
- Live load on floors- 3.0 kN/m^2.
- Live load on roof – 1.5 kN/m^2.
- Grade of steel used – Fe_{415} and Fe_{500}.
- Floor finishing – 1.0 kN/m^2.

Seismic criteria considered for this building is:

1. Response reduction factor, R= 5.
2. Importance factor, I= 1.5.
3. Zone factor for Zone IV, Z= 0.24.

The building plan consists of 4 X 3 bays. The storey height is 3.04

Table-6.1 The dimensions of all the beams and column.

All beams	300 X 600 mm

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All External columns	300 X 600 mm
All Internal columns	450 X 600 mm
Slab thickness	120 mm

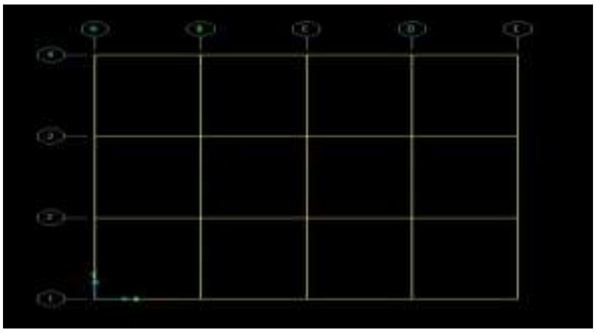

Figure 6.1: Plan of G+10 storey R C building in SAP 2000

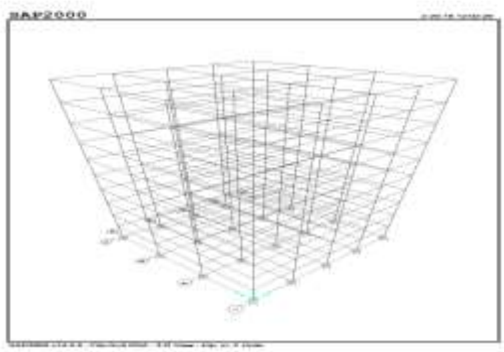

Figure 6.2: 3-D model view of G+10 storey R C building in SAP2000

RESULTS AND DISCUSSION

RC BUILDING SUBJECTED TO BLAST LOADING

A 100 feet height RC building is taken to analysis for Blast loading. The size of the building is 60 ft X 60 ft. The building is modelled in SAP 2000, and performed Response spectrum analysis. The plan of the structure is shown in figure-6.1 and the 3D model in SAP 2000 as shown in figure-6.2 the blast loading on the structure is determined for two individual explosion weights said to be as 2500 lbs and 5000 lbs at a standoff distance of 50, 100, 150 feet's. All the blast parameters are evaluated and the final Impulsive force is determined and plotted. These Impulsive force and time plots to the model in Time-history analysis. To determine the performance of any structure it is necessary to subject the structure to Time-history analysis, if the input function is nonlinear dynamic in nature. Time-History analysis is very complex and time consuming, but it will give accurate values. At every standoff distance we have calculated Impulsive force, and these were used in TH. Blast loading is entirely different from seismic load. The seismic load will said to be vibrative forces where as the blast load is impulsive. The performance of the building is determined at every standoff distances and the plots between Displacement-Time, acceleration-Time were shown in results.

RESULTS SUMMARY

When the explosion weights of 2500 lbs and 5000 lbs are exploded at a standoff distance of 50 100 and 150 feet the Impulsive force-Time plot as shown in figure-5.5 and 5.6 is given as input to the nonlinear dynamic Time history analysis, then the resulted plots are as follows, and the maximum displacements of joint and maximum accelerations are tabulated below:

Table 7.1 maximum Displacements and acceleration

Explosive Weight (lbs)	Standoff Distance (ft)	Max Displacement (mm)	Max. Acceleration (m/s^2)
2500	50	75	215
	100	67	158
	150	44	130
5000	50	648	4930
	100	597	2200
	150	244	498

Figure 7.1: Displacement – Time Plots for 2500 Lbs And 5000 Lbs @50 Feet Stand off Distance Respectively

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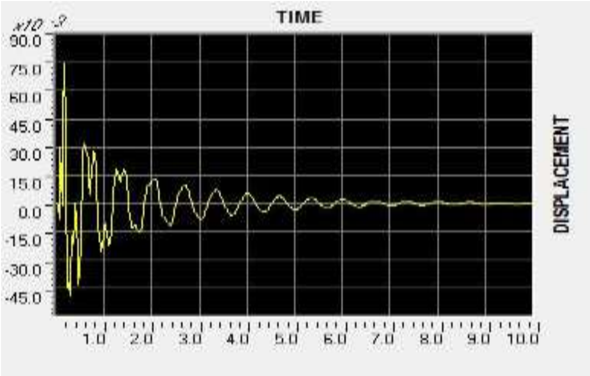


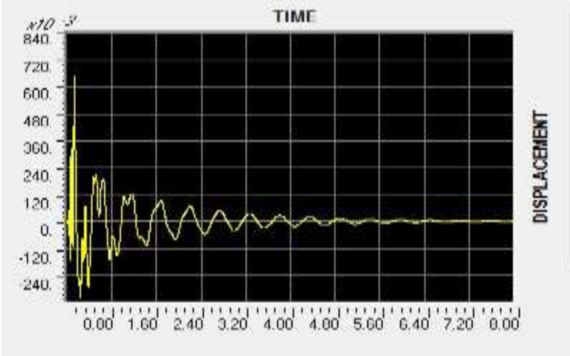


Figure 7.2 Acceleration-Time Plots for 2500 Lbs And 5000 Lbs @50 Feet Standoff Distance Respectively

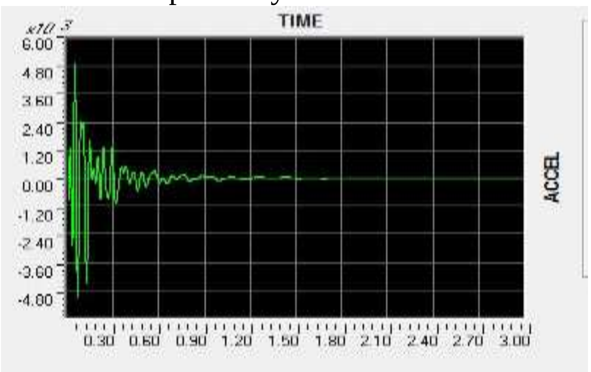
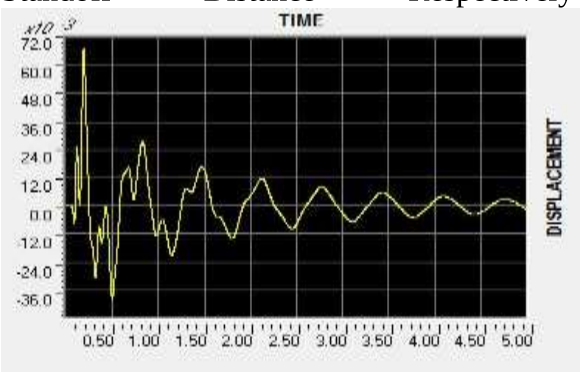


Figure 7.3 Displacement – Time Plots for 2500 Lbs And 5000 Lbs @100 Feet's Standoff Distance Respectively



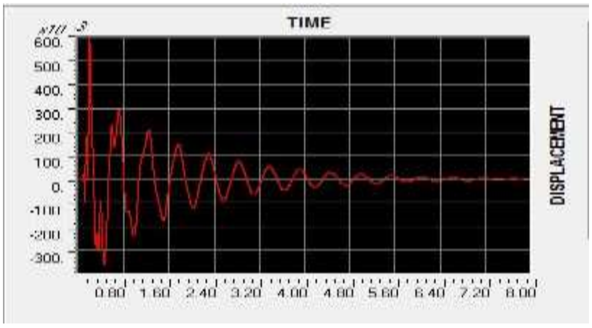


Figure 7.4 Acceleration-Time Plots For 2500 Lbs And 5000 Lbs @100 Feet's Standoff Distance Respectively

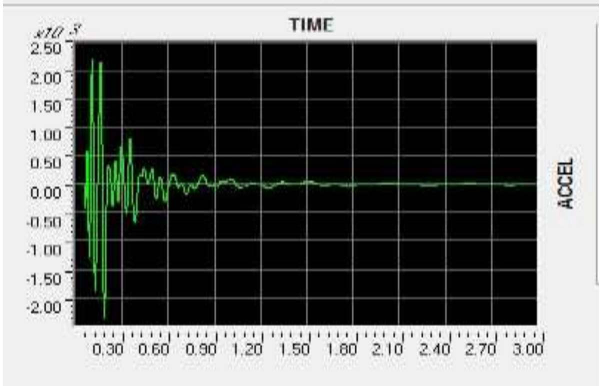
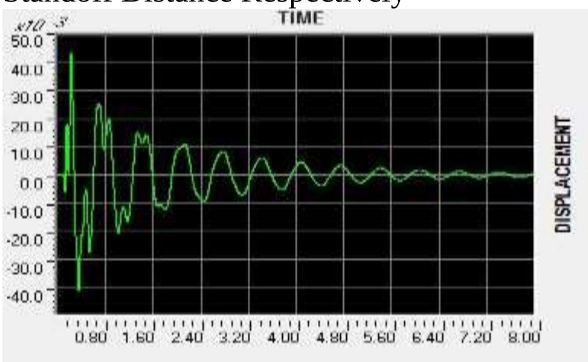


Figure 7.5 Displacement – Time Plots For 2500 Lbs And 5000 Lbs @150 Feet's Standoff Distance Respectively



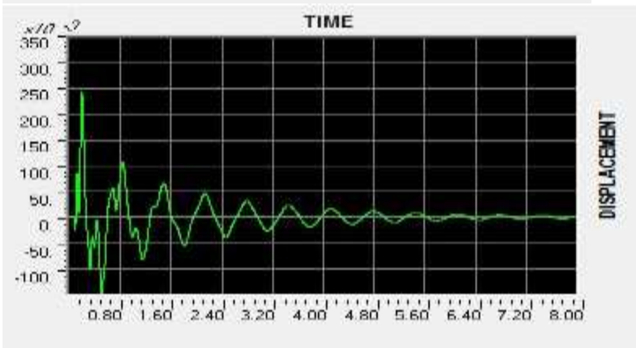


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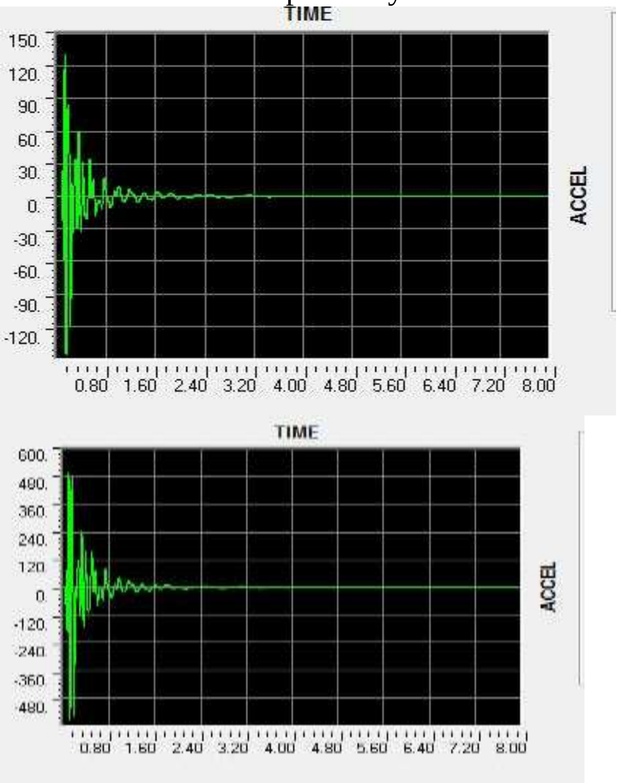
Figure 7.6 Acceleration-Time Plots For 2500 Lbs And 5000 Lbs @150 Feet's Standoff Distance Respectively



CONCLUSIONS

IS 4991-1968 didn't considered the negative pressure in the Pressure-Time plot, why they didn't thought about mean, under the effect of positive stage span, the dislodging of the structure is more than the removal when thought about the Negative pressure. This will go about as a wellbeing factor, so the structure will be exceptionally sheltered in the event that we experienced the Indian code. The variety that I'm watched is given beneath plots.

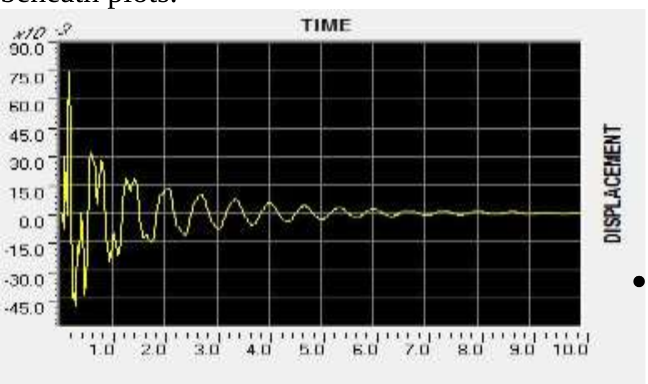


Figure 7.7.a: Displacement of 2500@50 Feet's with Considering Negative Pressure

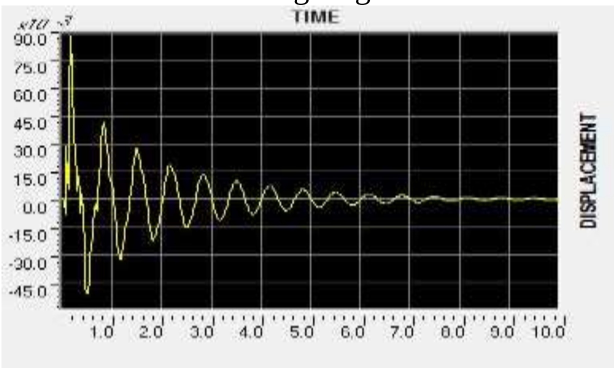


Figure 7.7.b: Displacement of 2500@50 Feet Without Considering Negative Pressure

In above plots the dislodging of 2500lbs @50 feet's is 74mm when the negative stage span is considered. On the off chance that the negative stage term is dismissed, the uprooting is seen as 90mm. So by that we can realize that there is more redirection under the loading without thinking about the Negative stage term. So this issue behind that why our Indian code didn't thought about the negative stage span. In any case, Indian code didn't give the adequate information to deciding the impact parameters when the explosion weight is more than 1 ton. Since the parameters were given just for one ton touchy weight. So our Indian code requires Revision.

- Aside from impact of Negative pressure, a portion of the accompanying focuses were seen amid my investigation. The positive and Negative incautious forces were in expanding request when the standoff separations are in decline. The Impact shoot range on the structure is more at standoff separation of 150 feet's, and after that the sweep is in diminishing request when the standoff remove is likewise diminished.
- The adequacy of vibration were expanded, if the hasty burdens are expanded. i.e., for less load, less redirection and the other way around.

The avoidances in structure, under the explosion weight 5000lbs are about 10 times of redirection under the touchy load of 2500lbs.

- So the harm and the impact under the explosion load of 5000lbs is 10times more than the explosion load of 2500lbs. As I said in over, the presentation of Negative period of loading diminishes the sufficiency of vibration of the building, however it may not impact for high story buildings. Due to sudden change in the loading from positive stage to negative stage, building by then moves inactively, the sections at the best will move aloof and the base segments will move effectively as a result of inversion loading. The impact of tallness of building doesn't make a difference while ascertaining the impact parameters, yet the impact of impact parameter will be more on the tall buildings, so therefore more harm will happen for high raise building.

SCOPE OF FUTURE STUDY

Present investigation has restricted extension with accentuation on structure explosion association. To grow clear understanding, it uncovers that, more research ought to be done on this subject. To decide the correct conduct or the execution of the building requires some model investigation. Yet, it is exceptionally hard to state that, which building is protected and which one isn't. Since the explosion like in World exchange focus (2001) won't ready to withstand by a building on the planet. A portion of the Recommendations for future work are proposed as pursues, to know the better execution of any building, advancement of model is required. Revision of Indian code 4991-1968, is required on the grounds that it does exclude the Negative stage loading, since it may demonstrate impact for tall buildings. Not just hence, IS code gives the information just to one ton unstable weight, so it is extremely hard to discover the shoot parameters for over one ton dangerous load by IS code. Comparative investigation as far as diagnostic methodology ought to be expanded for finding the execution under fluctuating interest emerging because of impact loading.

Study of auxiliary components (shaft, segments, chunks ets) should be given due weight age under impact loading. Non auxiliary components conduct likewise to be examined in subtleties for limiting harm cost under loading emerging because of impact. Interaction of basic and non basic components as far as standardized harm record be produced. Vulnerability Steel building may likewise be assessed under the recommended states of impact loading.

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