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Analysis of Reinforced Concrete 3d Frame under Blast Loading and Check for Progressive Collapse

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Abstract

Objectives: The main objective of this paper is to prevent the structures from blast loading and familiarize with design methods followed as per UFC guidelines. To prevent from these extreme loads we have adopted alternate path method for design. **Methodology:** Blast loads are generally dynamic in nature, so that it varies with time comprising of two phases namely positive phase and negative phase. In this paper for analysis of structure, we have considered only positive phase for the analysis. The load is applied as a triangular impulse loading as a time history variant neglecting the negative phase of the loading. The numerical model of this structure is created and analysed using the STADD. Pro software. The load values required for the model are obtained from IS:4991, the blast parameter values are found for a standoff distance of 20m,30m and 40m by varying charge weight as 100KG, 500KG and 1000KG. Analysis is carried out for different grades of concrete in STADD. Pro and results are compared and analysed. Progressive collapse is also checked by providing plastic hinges in the structure as per FEMA 356 provisions using SAP2000 as per UFC guidelines. **Findings:** The alternate approach followed shows the structure is safe by removing columns at different positions. The results obtained shows the plastic hinges formed in the corner column are higher compared to any internal columns. **Improvements:** Further improvements adopted can be by applying special design for reinforcements apart from general design.

Keywords: Blast Load, Positive Phase, Progressive Collapse, Plastic Hinges, Standoff Distance, Triangular Impulse Loading

1. Introduction

As we know these days there is increase in terrorist threats around the world, which leads to problems related to blast. As our structures are mostly designed to a maximum of earthquake loading, the structure may not with stand these extreme loading subjected to extreme strain rates. From the Reference[1], the Figure 1 shows the difference between different types of loading. Generally a blast can cause damage to a building both internal as well as external structural members. Blast load is dynamic in nature so all the parameters related to blast will vary with time. Blasting can also be used for demolition of structure for progressive collapse without damaging adjacent structures. These types of analysis are generally carried out by military organization or any other special departments. In this paper load calculations and analysis is carried out as per Reference[2].

1.1 Explosion Parameters and Considerations

When explosion occurs it emits lots of energy rapidly within fraction of seconds. The explosion will result in emission of very hot and dense gas under high pressure that produces explosive wave. The wave produced outwards will result in decrease of speed and strength as it reaches the structure or until it reaches atmospheric pressure.

Explosion effect depends on many parameters like pressure, acceleration, velocity etc., apart from the above parameters charge weight is base parameter causing these. The charge weight is assumed in a way that in an external explosion based on previous experiences vehicle bombs are the major reason for blast. So Table-1 shows the capacity of each type of vehicle and Table-2 shows the equivalent charge to TNT for different types of chemical charges.

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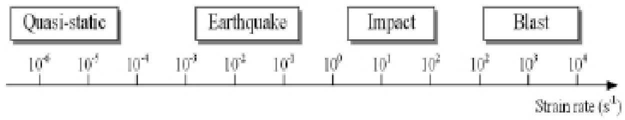

Figure 1. Showing variation of strain for different type of loading.

Table 1. Showing charge weight based on type of vehicle

Vehicle	Charge(KG)
compact car trunk	115
trunk of a large car	230
closed van	680
closed truck	2270
truck with a trailer	13610
truck with two trailers	27220

Table 2. Showing equivalent charge weight for 1TONNE TNT

Explosive	TNT equivalent (KG)
TNT	1000
Compound B	1148
RDX(Ciklonit)	1185
HMX	1256
Nitroglycerin	1480
Gelatin	1000
Nitroglycerin dynamite	600
Semtex	1250
C4	1340

1.2 Loading on Structures

Loading for blast are classified into two types namely confined and unconfined. The following paper deals with unconfined explosion over the ground. When the explosion occurs in air, the pressure released can be applied directly without any variation. But for explosion over or near ground will cause increase or decrease in pressure based on height of explosion and distance of charge from the building. As per Reference[2] the structures are classified as diffraction type and drag type structures. This paper deals with diffraction type of structure these are generally closed structures without openings with total area opposing the blast. The pressure released during blast is applied on the structure as a lateral load on the structure. The pressure released is converted into load by multiplying the tributary area at each joint on the face of explosion side. The force applied as a horizontal one without considering the radial motion of wave, because the refracted wave causes planar wave front before the structure which is called mach stem. The following Figure 2 explains how planar wave front is formed[1-4].

1.3 Load Calculation

The peak pressure and duration of loading are calculated at each joint as specified in IS-4991 from Table-1. This table indicates values for charge of 1 Tonne explosive for various distances. For calculation of peak pressures for different charges we have to follow the scaling laws as specified below.

$$\text{Scaled distance } Z = \frac{Actual\ distance}{W^{\frac{1}{3}}}$$

$$\text{Scaled time } t_0 = \frac{Actual\ tune}{W^{\frac{1}{3}}}$$

Where,

W = yield explosion in equivalent weight of the reference explosive in tonnes.
Z = scaled distance for entering the values.
T_o = scaled time read from table for scaled distance.

The load calculated from IS 4991 is tabulated in the following Tables 3, similarly carried out for other parameters with standoff distance of 20m to various charge weights of 100KG, 500KG and 1000KG respectively[2-4].

Apart from calculation of peak pressure P_{so} from table in Reference it can also be calculated using the following formulas from different scientists

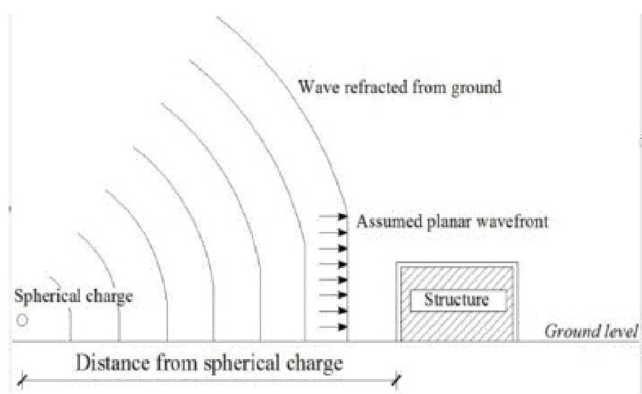

Figure 2. Wave front pattern.

$$P_{so} = 1 + \frac{6.7}{Z^3} \text{ bar } (P_{so} > 10 \text{ bar})$$

$$P_{so} = \frac{5.85}{Z^3} + \frac{1.455}{Z^2} + \frac{0.975}{Z} - 0.019\ bar$$

(0.1 bar < Pso<10bar)

Following equation is introduced by Brode in 1955

Table 3. Showing details of load calculation for 100KG TNT

Floor	Node	Radial distance R(m)	Scaled distance Z(m)	Arrival time Td(ms)	Peak pressure Pr(KN/m²)	Tributary area A(m²)	Load P(KN)
1	7	21.66	46.67	22.21	151.86	5.40	820.04
	8	20.81	44.84	21.53	145.87	10.80	1575.45
	9	20.38	43.90	21.09	173.93	10.80	1878.46
	10	20.38	43.90	21.09	173.93	10.80	1878.46
	11	20.81	44.84	21.53	145.87	10.80	1575.45
	12	21.66	46.67	22.21	151.86	5.40	820.04
2	13	22.54	48.56	22.91	139.91	5.40	755.53
	14	21.73	46.81	22.26	151.00	10.80	1630.75
	15	21.31	45.91	21.93	156.90	10.80	1694.53
	16	21.31	45.91	21.93	156.90	10.80	1694.53
	17	21.73	46.81	22.26	150.98	10.80	1630.54
	18	22.54	48.56	22.91	139.91	5.40	755.53
3	19	23.94	51.57	23.89	122.98	5.40	664.07
	20	23.17	49.92	23.34	131.92	10.80	1424.79
	21	22.78	49.08	23.06	136.89	10.80	1478.44
	22	22.78	49.08	23.06	136.89	10.80	1478.44
	23	23.17	49.92	23.34	131.94	10.80	1425.00
	24	23.94	51.57	23.89	122.98	5.40	664.07
4	25	25.76	55.50	25.55	105.46	5.40	569.47
	26	25.05	53.97	24.69	111.96	10.80	1209.14
	27	24.69	53.19	24.43	115.53	10.80	1247.71
	28	24.69	53.19	24.43	115.53	10.80	1247.71
	29	25.05	53.97	24.69	111.96	10.80	1209.14
	30	25.76	55.50	25.55	105.46	5.40	569.47
5	31	27.93	60.18	26.67	90.76	2.70	245.06
	32	27.28	58.78	26.52	94.44	5.40	509.96
	33	26.95	58.06	26.47	96.31	5.40	520.06
	34	26.95	58.06	26.47	96.31	5.40	520.06
	35	27.28	58.78	26.52	94.44	5.40	509.96
	36	27.93	60.18	26.67	90.76	2.70	245.06

$$P_{so} = 93\left(\frac{W}{R^a}\right)^{\frac{1}{2}} + 6.784\frac{W}{R^2} \text{ bar}$$

Following equation is introduced by Newmark and Hansen in 1961

$$P_{so} = \frac{108}{Z} - \frac{114}{Z^2} + \frac{1172}{Z^3} \text{ bar}$$

Following equation is introduced by Mills in 1987

1.4 Time- History Analysis

In blast loading the pressure time history consists of two phase's namely positive phase and negative phase. Till the arrival of wave to point of contact the pressure will be ambient, at the point of contact the pressure will be maximum then decreases with time. Negative phase is induced due to refraction of waves from structure. At a point t_o the pressure will be nullified. In blast analysis the effect of negative phase will be minimums, so that it will not affect the results much. Therefore for analysis purpose we will be considering only the positive phase which is shaded dark as shown in Figure 3

2. Numerical Modelling

Consider a model for blast load analysis using STADD Pro as shown in the following figure. The dimensions are specified below, the sizes of members are modelled based on the dead load and live load combination without considering blast load initially.

Column size of 0.5m × 0.5m and beam size of 0.4m × 0.6m are considered for the analysis. The storey height is considered as 3.6m and bay of 3m × 3m

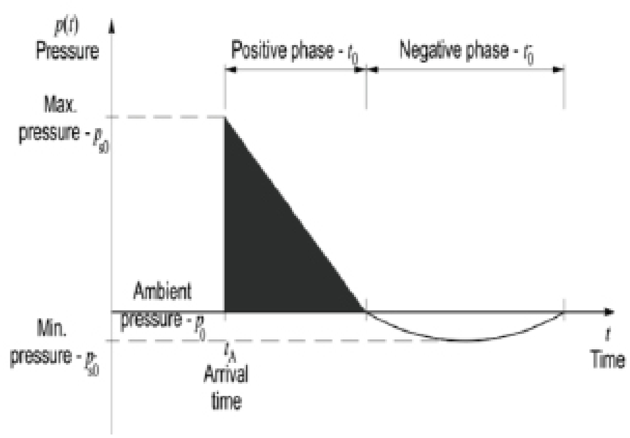

Figure 3. Variation of pressure with time.

3. Analysis Results and Discussion

3.1 Deflection Diagrams

The maximum deflection of the each floor of a structure for a particular charge and distance is mentioned in Figures 4, 5 and 6 in dynamic nature over a time period of 3 seconds. The maximum deflection is occurred at top storey with a displacement of 121.4mm and decreases almost gradually with decrease in floors and deflection decreases cubically with decrease in amount of charge.

4. Check for Progressive Collapse and Plastic Hinges Stability

In general to avoid Progressive Collapse(PC) as per UFC guidelines two direct methods are followed namely specific local resistance, which is most probably not a better method to follow because the loads generated from blast are extreme so that if we design the section dimensions will be very high and it is not an economical approach. Another most common method followed is Alternate path method, which is allowing flow of loading through other paths after damage of the particular column or beam[5].

In this paper we will follow Alternate path method for analysis by applying hinges to beams and columns. Hinges in beams are caused due to moments and hinges in columns are caused due to axial moments, the variation of rotation to moment is mentioned in Figure 7. In flexural members hinges will occur at ends and at the centre, where as in axial members at ends. The application of this hinges in a sap model requires some criteria as per FEMA 356 which is represented in Figure 8.

As per UFC guidelines the check for progressive collapse is carried out in three ways as shown in following Figure 9, 10 and 11 by removing the columns at particular locations. The plastic hinges formed are shown in pink colour which is safe and satisfies requirement for progressive collapse[5-10].

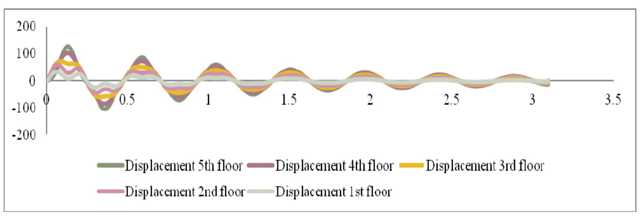

Figure 4. Deflection diagram of 1000KG charge at a standoff distance of 20m.

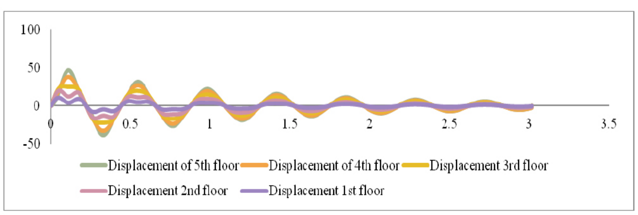

Figure 5. Deflection diagram of 500KG charge at a standoff distance of 20m.

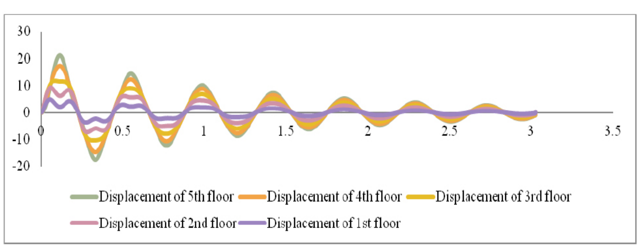

Figure 6. Deflection diagram of 100KG charge at a standoff distance of 20m.

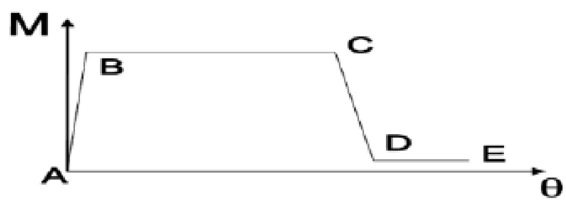

Figure 7. Variation of moment with the rotation.

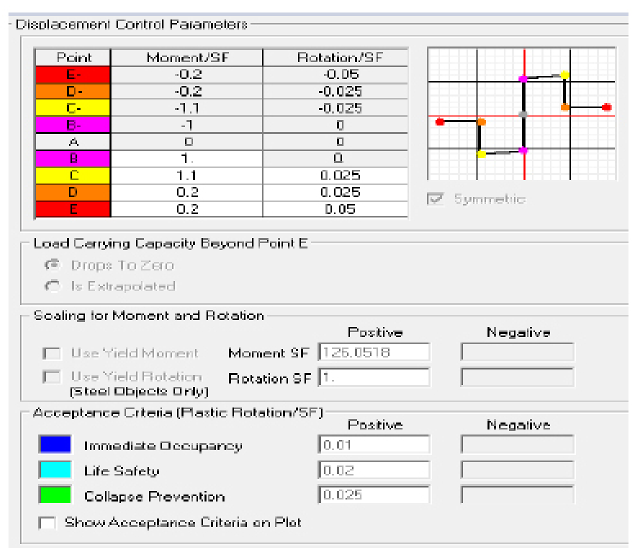

Figure 8. Moment – rotation values according to FEMA 356.

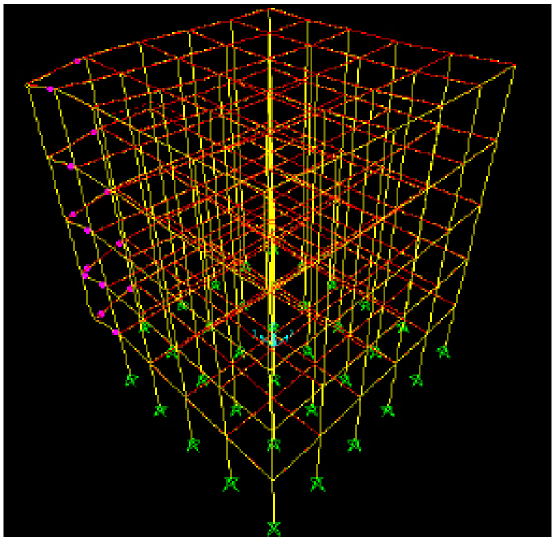

Figure 9. Removal of Corner column.

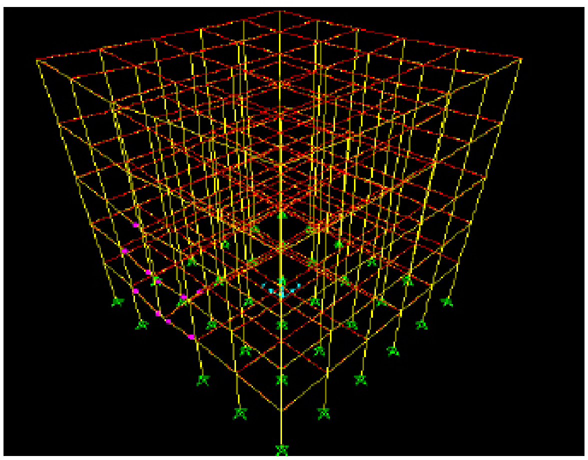

Figure 10. Removal of Exterior column.

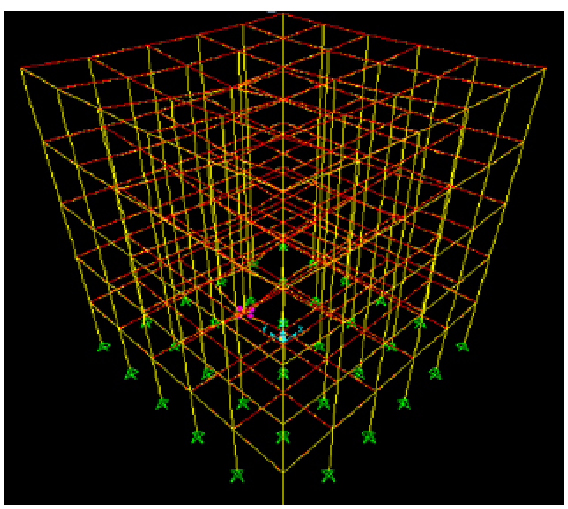

Figure 11. Removal of Interior column.

5. Liable

The total analysis carried out in this paper majorly consists of two parts namely first part consists of analysis of structure with varying charge and distances which tells, with the increase in charge damage incurred will also be on higher side. Here are some methods that can be adopted to design structures to resist against such heavy loads.

- To resist such heavy loads we should adopt dampers or stiffeners which are not an economical approach.
- Joints should be designed to resists such heavy moments, which is sometimes practically not possible because reinforcement exceeds beyond limits.

The second part of this paper explains about progressive collapse which is found through alternate path analysis. Alternate path load analysis shows better economic approach for designing as per Reference.

- In this structure after applying hinges, only localized plastic hinges are formed and it is not failed and satisfies progressive collapse requirements.
- If any of hinges fails, the particular member is removed and analysed until it meets PC requirements

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