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Blast Dynamic Analysis By Single Degree Of Freedom Method For RC Control Room Building

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Abstract: In petrochemical industry, accidental explosion can be produced, as it handles hydrocarbons and other fuels in the process plant. The occurrences of such incidents are minimum due to proper planning and design of process plants. In spite of such incidents are relatively rare, if they occur, consequences can be extremely severe, in terms of personnel casualty, financial loss and public safety. In the view of such type of fatalities, plant buildings are need to design, to withstand explosion effects, to protect people inside it, so that building could not pose an added hazards to its occupants. Most of the companies in the industry, consider blast resistance for critical buildings like Control Room, to minimize impact of explosion on plant operation, even if unoccupied. In this paper, dynamic analysis has been performed using single degree of freedom (SDOF) method for RC control room building, considering 200 mbar blast design pressure and 200 ms time duration.

Index Terms - blast resistant building, dynamic blast analysis, SDOF, blast design pressure, blast time duration.

I. Introduction

The petrochemical plants consist of pipe racks with congested pipe routing, pressure vessels like horizontal exchangers, vertical vessels, technological structures and control room buildings. These plants are involved in the chemical operations and sometime these operations can produce accidental explosion within plant. These explosions can cause to loss of personnel's life, huge amount of property loss and public safety in the surrounded area.

Nowadays, presence of personnel is very rare as most of the plant operations are being operated from control room building. To defy the accidental explosion, control room building is located such that, it can have less impact of blast, which can not be avoidable. In such circumstances, it is requisite to design control room building as blast resistant building.

II. Blast Resistance Design Process

The comprehensive process involved in the analysis and design of petrochemical plant buildings for blast hazards is represented in figure 1. This flow diagram presents fifteen key steps in the complete blast analysis and design process, as follows.

Steps 1 & 2	-	These steps outlined the owner's prerequisites and necessities for the buildings.
Steps 3 & 4	-	These steps are to determine the explosion scenarios to be used to compute the design blast overpressure.
Step 5	-	This step is to ascertain how the building should perform in the course of the explosion scenario.
Step 7	-	This step is to evaluate the blast loadings for the different elements of the building.
Step 6, 8 & 9	-	These steps are to select the structural materials and systems for the building and related structural properties and response limits invariable with performance requirements for the building.

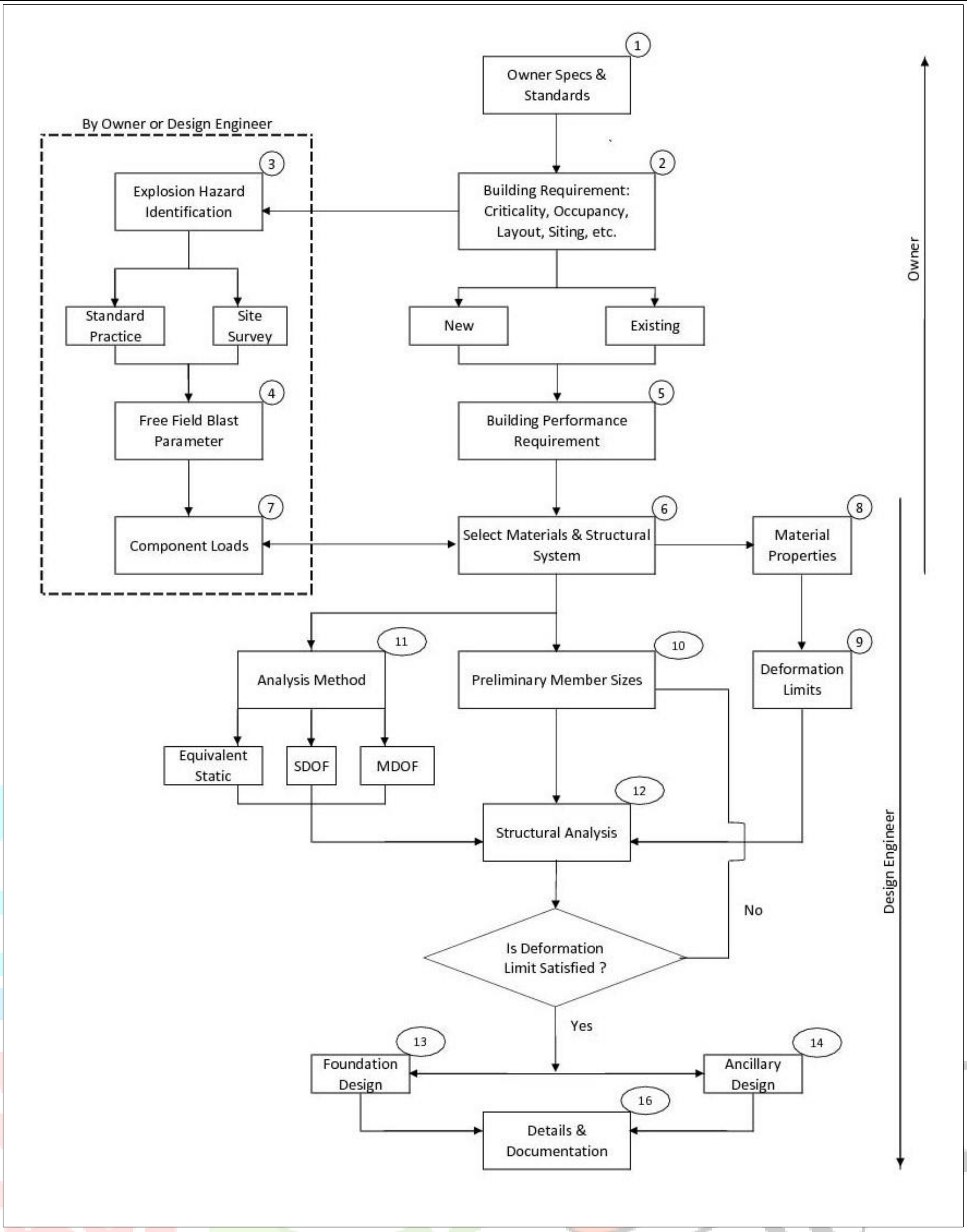


Figure 1 Petrochemical Buildings, Blast Resistance Design Process [6]

Step 10 & 12 - These steps are to choose and execute the level of structural calculations suitable for specific condition.

Step 13 to 15 - These steps are to detail building element design and documentation.

In this design process, it is presumed that owner will furnish the requisite information as mentioned in the steps 1 to 5 and design engineer's duties are outlined in the steps 6 to 15 of the process.

III. BLAST WAVE PARAMETERS FOR BLAST LOADING

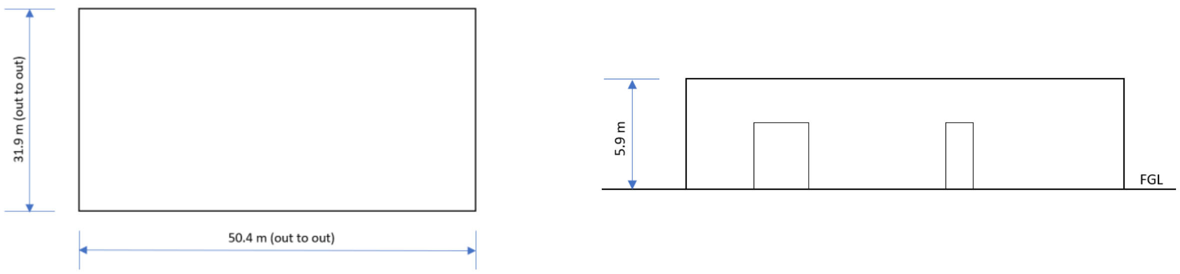
The petrochemical sector is primarily concerned with vapour cloud explosions, despite the fact that there are many other sorts of explosions. The design blast loads are typically provided by the facility owner because there are no laws or industry guidelines for defining what blast overpressures should be employed. It is simple to understand why these overpressures will vary from one owner to the next and even for various areas within a single facility given the huge range of procedures. various plant regions are categorized by various owners using different danger ratings. These risk levels depend on the type of substance handled and the method employed.

The primary parameters of the blast wave to design blast resistant building, are required to determine blast loading for elements of buildings.

- Peak side-on overpressure, P_{so},
- duration, t_d

IV. Description of Building

A single-story RC control building of size 50m x 31.5m in plan and height 5.9 m from finished ground level is analyzed for blast pressure 200mbar and duration 200ms.



PLAN ELEVATION

Figure 2 Building Details

Material
Concrete – M40
Reinforcement – Fe500

V. Blast Load Calculation

In this paper, it is considered that, blast loading will be applied normal to short side of building and determined the blast loading on components of building, such as, front wall, side wall, roof slab etc.

Building Dimension

Width $B_W := 31.9 \cdot m$

Length $B_L := 50.4 \cdot m$

Height $B_H := 5.9 \cdot m$

Blast Loading

Peak side-on overpressure $P_{so} := 20 \cdot kPa$

Duration $t_d := 200 \cdot ms$

Shock Front Velocity

$$U := 345 \cdot \left(1 + 0.0083 \frac{P_{so}}{kPa}\right)^{0.5} \cdot \frac{m}{s} \qquad U = 372.536 \frac{m}{s} \qquad \text{(Eq. 3.5)}$$

Length of pressure wave

$$L_w := U \cdot t_d \qquad L_w = 74.507\, m \qquad \text{(Eq. 3.6)}$$

Peak dynamic wind pressure

$$q_o := 0.0032 \cdot \left(\frac{P_{so}}{kPa}\right)^2 \cdot kPa \qquad q_o = 1.28 \cdot kPa \qquad \text{(Eq. 3.4)}$$

Front Wall Loading

The front wall is assumed to span vertically from foundation to roof. The design will be for a typical wall segment one foot wide.

Peak Reflected Pressure, Pr

$$P_{rfw} := \left(2 + 0.0073 \cdot \frac{P_{so}}{kPa}\right) \cdot P_{so} \qquad P_{rfw} = 42.92 \cdot kPa \qquad \text{(Eq. 3.2 \& 3.3)}$$

Clearing disance, S

$$S_c := \min\left(B_H, \frac{B_W}{2}\right) \qquad S_c = 5.9\, m \qquad \text{(Sec. 3.5.1)}$$

Reflected overpressure clearing time, tc

$t_{cfw} := 3 \cdot \left(\frac{S_c}{U}\right)$ $t_{cfw} = 0.048\,s$ (Eq. 3.8)

$Check_1 := if(t_{cfw} < t_d, "OK", "NOTOK")$ $Check_1 = "OK"$

Drag coefficient, Cd $C_{dfw} := 1.0$ (Sec. 3.3.3)

Stagnation Pressure, Ps

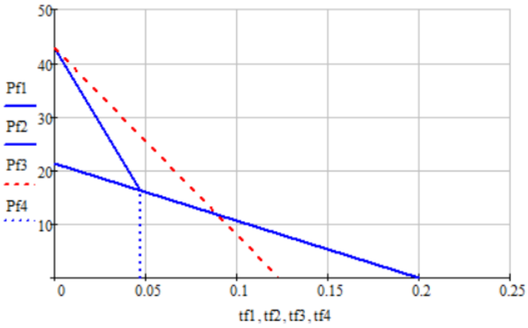
$P_{sfw} := P_{so} + C_{dfw} \cdot q_o$ $P_{sfw} = 21.28 \cdot kPa$ (Eq. 3.7)

Front Wall Impulse, Iw

$I_{wfw} := 0.5 \cdot (P_{rfw} - P_{sfw}) \cdot t_{cfw} + 0.5 \cdot P_{sfw} \cdot t_d$ $I_{wfw} = 2.642 \cdot kPa \cdot s$ (Eq. 3.9)

Effective Duration, te

$t_{efw} := 2 \cdot \frac{I_{wfw}}{P_{rfw}}$ $t_{efw} = 0.123\,s$ (Eq. 3.10)



Front Wall Loading

Side Wall Loading

The side wall is the same as the front wall, spanning vertically from foundation to roof. Because the highest loads are on the front wall, a side wall analysis would only be necessary to check the interaction of in-plane and out-of-plane shear wall forces.

This calculation will be for wall segment, L1, 1 foot wide (0.3m)

Drag coefficient, Cds $C_{dsw} := -0.4$ (Sec. 3.3.3)

Equivalent load coefficient

$\frac{L_w}{B_L} = 1.478$ Reduction factor $C_{esw} := 0.5$ (Fig. 3.9)

Equivalent Peak Overpressure, Pa

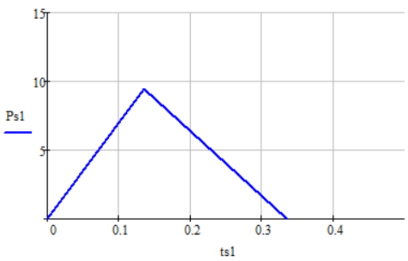
$P_{asw} := C_{esw} \cdot P_{so} + C_{dsw} \cdot q_o$ $P_{asw} = 9.488 \cdot kPa$

Rise Time, tr

$t_{rsw} := \frac{B_L}{U}$ $t_{rsw} = 0.135\,s$

Total positive phase duration

$t_{osw} := t_{rsw} + t_d$ $t_{osw} = 0.335\,s$



Side Wall Loading

Roof Loading

The roof is a slab spanning between roof beams. For design of the roof, a section 1 foot wide by 8 feet long will be used.

Drag coefficient $C_{drf} := -0.4$

Equivalent load coefficient

$\frac{L_W}{B_L} = 1.478$ Reduction factor $C_{erf} := 0.5$ (Fig. 3.9)

Equivalent Peak Overpressure, Pa

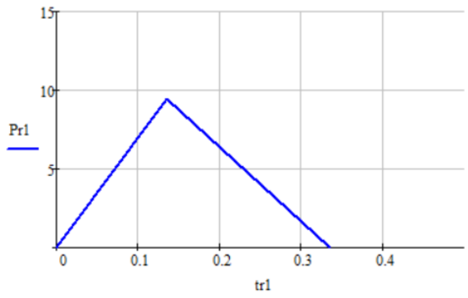
$P_{arf} := C_{erf} \cdot P_{so} + C_{drf} \cdot q_o$ $P_{arf} = 9.488 \cdot kPa$

Rise Time, tr

$t_{rrf} := \frac{B_L}{U}$ $t_{rrf} = 0.135\,s$

Total positive phase duration

$t_{orf} := t_{rrf} + t_d$ $t_{orf} = 0.335\,s$



Roof Loading

Rear wall loading

Normally, rear wall loading is only taken into account when calculating net overall frame loading. The addition of the rear wall load helps to lessen the total lateral blast force since it is directed in the opposite direction as the front wall load. Rear wall effects are frequently conservatively ignored for structures where a blast load might come from either direction.

Blast pressure intensity considered on structural elements is tabulated below.

Element	Blast pressure (kN/m^2)
Front Wall	21.28
Side Wall	9.5
Roof	9.5
Rear Wall	-

VI. Deformation Limits

To provide a suitable response to blast loads, response deformation limits are applied. These restrictions are determined by the type of building or component, the materials used in construction, the location of the structure, and the intended level of protection.

Ductility ratio (μ) – It is ratio of maximum displacement of member to elastic limit displacement.
Hinge rotation (θ) – It is relate to maximum deflection to span.
Response Criteria – Low - Building can be used with minor local component damage.

Table 1 Response Limit for Reinforced Concrete (R/C) [6]

Component.	Low Response	
	μ	θ
R/C Beams, Slabs & Wall Panels (no shear reinforcement)		1
R/C Beams, Slabs & Wall Panels (compression face steel reinforcement and shear reinforcement in maximum moment areas)		2
R/C Walls, Slabs, & Columns (in flexure & axial compression load)		1
R/C & R/M Shear Walls & Diaphragms	3	
R/C & R/M Components (shear control, without shear)	1.3	
R/C & R/M Components (shear control, with shear)	1.6	

VII. Dynamic Analysis Method

A dynamic blast analysis' main goal is to assess a structure's ability to withstand a given blast load. In order to achieve this objective, the analysis should be able to reasonably forecast the structure's dynamic reaction. A specific structural configuration, which comprises the kind of material, span length, support circumstances, and applied loading, serves as the basis for the study of a typical member. Based on the member configuration, the projected section capacities, and the postulated failure mechanisms, a resistance function—or applied force against displacement relationship is produced.

The analysis ought to offer -
a. Each structural element's maximum relative deflections.
b. Relative rotation angles at the positions of the plastic hinges.
c. Dynamic responses transmitted to the auxiliary components.
d. Reactions and deflections brought on by rebound.

After the analysis is finished, the design can move on to assess the member's suitability by using the acceptance criteria.

VIII. Single Degree of Freedom System (SDOF)

SDOF approximations are used for the majority of dynamic assessments in blast resistant design of petrochemical facilities. SDOF systems are approximations of common construction types such single story plane frames, cantilever barrier walls, and compact box-like structures.

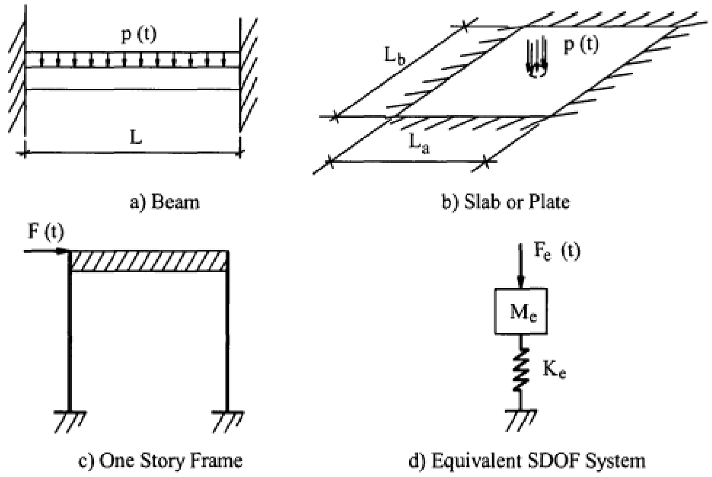


Figure 3 Typical Structures represented as equivalent SDOF system [6]

IX. FRONT WALL DYNAMIC ANALYSIS BY SDOF METHOD

In this paper, only dynamic analysis for front wall for out-of-plane blast loading is presented.

Front Wall Loading

The front wall is assumed to span vertically from foundation to roof. The design will be for a typical wall segment one foot wide.

Peak Reflected Pressure, Pr

$P_{rfw} := \left(2 + 0.0073 \cdot \frac{P_{so}}{kPa}\right) \cdot P_{so}$ $\quad$ $P_{rfw} = 42.92 \cdot kPa$ $\quad$ (Eq. 3.2 & 3.3)

Clearing disance, S

$S_c := \min\left(B_H, \frac{B_W}{2}\right)$ $\quad$ $S_c = 5.9\,m$ $\quad$ (Sec. 3.5.1)

Reflected overpressure clearing time, tc

$t_{cfw} := 3 \cdot \left(\frac{S_c}{U}\right)$ $\quad$ $t_{cfw} = 0.0475122\,s$ $\quad$ (Eq. 3.8)

$Check_1 := if\left(t_{cfw} < t_d, "OK", "NOTOK"\right)$ $\quad$ $Check_1 = "OK"$

Drag coefficient, Cd $\quad$ $C_{dfw} := 1.0$ $\quad$ (Sec. 3.3.3)

Stagnation Pressure, Ps

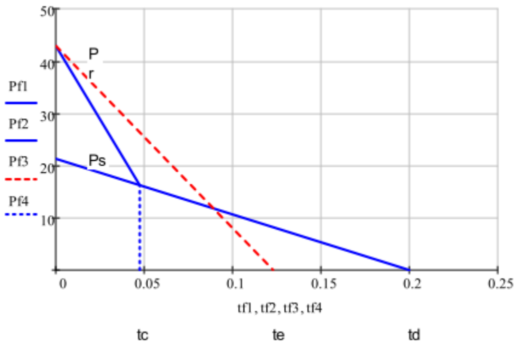
$P_{sfw} := P_{so} + C_{dfw} \cdot q_o$ $\quad$ $P_{sfw} = 21.28 \cdot kPa$ $\quad$ (Eq. 3.7)

Front Wall Impulse, Iw

$I_{wfw} := 0.5 \cdot \left(P_{rfw} - P_{sfw}\right) \cdot t_{cfw} + 0.5 \cdot P_{sfw} \cdot t_d$ $\quad$ $I_{wfw} = 2.642 \cdot kPa \cdot s$ $\quad$ (Eq. 3.9)

Effective Duration, te

$t_{efw} := 2 \cdot \frac{I_{wfw}}{P_{rfw}}$ $\quad$ $t_{efw} = 0.123\,s$ $\quad$ (Eq. 3.10)



Front Wall Loading

1.0) Input Data

Compressive strength of concrete $f_{ck} := 35 \cdot MPa$

Specified compressive strength of concrete $f_c := 0.8 \cdot f_{ck}$ $f_c = 28 \cdot MPa$

Specified yield strength of reinforcement $f_y := 500 \cdot MPa$

Modulus of elasticity for reinforcement $E_s := 200000 \cdot MPa$ (cl. 20.2.2.2 ACI 318-14)

Modulus of elasticity for concrete $E_c := 4700 \cdot \sqrt{\frac{f_c}{MPa}} \cdot MPa$ (cl. 19.2.2.1.b ACI 318-14)

$E_c = 24870.062 \cdot MPa$

Modular ratio $n := \frac{E_s}{E_c}$ $n = 8.042$

Acceleration due to gravity $g = 9.807 \frac{m}{s^2}$

Density of concrete $\gamma_c := 24 \cdot \frac{kN}{m^3}$

Strength Increase Factor for Materials Appendix 5.A, Table 5.A.1, ASCE Manual

Strength Increase Factor for concrete $SIF_c := 1.0$

Strength Increase Factor for reinforcement $SIF_s := 1.1$

Dynamic Increase Factor for Materials Appendix 5.A, Table 5.A.2, ASCE Manual

Dynamic Increase Factor for concrete in flexure $DIF_{flc} := 1.19$

Dynamic Increase Factor for concrete in compression $DIF_{coc} := 1.12$

Dynamic Increase Factor for concrete in diagonal tension $DIF_{dtc} := 1.0$

Dynamic Increase Factor for concrete in direct shear $DIF_{dsc} := 1.1$

Dynamic Increase Factor for concrete in bond $DIF_{boc} := 1.0$

Dynamic Increase Factor for reinforcenent in flexure $DIF_{fls} := 1.17$

Dynamic Increase Factor for reinforcenent in compression $DIF_{cos} := 1.10$

Dynamic Increase Factor for reinforcenent in diagonal tension $DIF_{dts} := 1.0$

Dynamic Increase Factor for reinforcenent in direct shear $DIF_{dss} := 1.1$

Dynamic Increase Factor for reinforcenent in bond $DIF_{bos} := 1.17$

2.0) Blast Loading

The applied load is divided into two triangular components: "Reflection Load" & "Stagnation Load".

Reflection Load $F_{RL} := Lw \cdot b_w \cdot (P_{rfw} - P_{sfw})$ $F_{RL} = 41.554 \cdot kN$

Stagnation Load $F_{SL} := Lw \cdot b_w \cdot P_{sfw}$ $F_{SL} = 40.863 \cdot kN$

Total pressure on wall $P_{rfw} + P_{sfw} = 64.2 \cdot kPa$

Total load $F_o := F_{RL} + F_{SL}$ $F_o = 82.417 \cdot kN$

3.0) Trial sizing

Wall thickness $Tw := 400 \cdot mm$

Clear cover $c_c := 50 \cdot mm$

Diameter of bar $d_{bv} := 20 \cdot mm$ (vertical bar)

$d_{bh} := 20 \cdot mm$ (horizontal bar)

Spacing of bar $s_b := 200 \cdot mm$

Area of single bar $A_b := \frac{\pi}{4} \cdot d_{bv}^{\,2}$ $A_b = 314.159 \cdot mm^2$

Effective depth $d_{effw} := Tw - c_c - \frac{d_{bv}}{2}$ $d_{effw} = 340 \cdot mm$

4.0) Computing Bending Resistance

for dynamic loading

$F_{dy} := SIF_s \cdot DIF_{fls} \cdot f_y$ $F_{dy} = 643.5 \cdot MPa$

$F_{dc} := SIF_c \cdot DIF_{flc} \cdot f_c$ $F_{dc} = 33.32 \cdot MPa$

for bending tension on the inside face

$d_{effb} := Tw - c_c - d_{bh} - \frac{d_{bv}}{2}$ $d_{effb} = 320 \cdot mm$

Minimum area of reinforcement

$$A_{sminf} := \max\left(\frac{0.25\cdot\sqrt{\frac{f_c}{MPa}}}{\frac{f_y}{MPa}}\cdot b_w\cdot d_{effb}, \frac{1.4\cdot MPa}{f_y}\cdot b_w\cdot d_{effb}\right)$$ (cl. 9.6.1.2 ACI-318-14)

$$A_{sminf} = 273.101\cdot mm^2$$

for the nominal design width

$$A_{sf} := \frac{A_b\cdot b_w}{s_b} \qquad A_{sf} = 478.779\cdot mm^2$$

$$Check2 := if\left(A_{sf} > A_{sminf}, "OK", "NOTOK"\right) \qquad Check2 = "OK"$$

Depth of compression block

$$a_{cf} := \frac{A_{sf}\cdot F_{dy}}{0.85\cdot F_{dc}\cdot b_w} \qquad a_{cf} = 35.69\cdot mm$$

Plastic Moment, Mp = Mn

$$M_p := A_{sf}\cdot F_{dy}\cdot\left(d_{effb} - \frac{a_{cf}}{2}\right) \qquad M_p = 93.092\cdot kN\cdot m$$

positive (inward) bending resistance based on pinned ends, (Table 6.1, ASCE Manual)

$$R_b := \frac{8\cdot M_p}{Lw} \qquad R_b = 118.212\cdot kN$$

for bending tension on the outside face

$$d_{efffo} := Tw - 2c_c - d_{bh} - \frac{d_{bv}}{2} \qquad d_{efffo} = 270\cdot mm$$

Minimum area of reinforcement

$$A_{sminfo} := \max\left(\frac{0.25\cdot\sqrt{\frac{f_c}{MPa}}}{\frac{f_y}{MPa}}\cdot b_w\cdot d_{efffo}, \frac{1.4\cdot MPa}{f_y}\cdot b_w\cdot d_{efffo}\right)$$ (cl. 9.6.1.2 ACI-318-14)

$$A_{sminfo} = 230.429\cdot mm^2$$

Plastic Moment, Mp = Mn

$$M_{pfo} := A_{sf}\cdot F_{dy}\cdot\left(d_{efffo} - \frac{a_{cf}}{2}\right) \qquad M_{pfo} = 77.687\cdot kN\cdot m$$

rebound (outward) bending resistance based on pinned ends

$$R_{bfo} := \frac{8\cdot M_{pfo}}{Lw} \qquad R_{bfo} = 98.651\cdot kN$$

5.0) Computing Shear Resistance

for dynamic shear

$F_{dcs} := SIF_c \cdot DIF_{boc} \cdot f_c$ $\quad\quad F_{dcs} = 28 \cdot MPa$

Because positive or rebound bending can occur, calculate shear resistance based on the smaller of deff based on inside tension or outside tension

$d_{efffs} := \min(d_{efffb}, d_{efffo})$ $\quad\quad d_{efffs} = 270 \cdot mm$

Nominal shear strength $\quad \lambda := 1$ $\quad$ (cl.22.5.5.1 ACI-318-14)

$$V_n := 0.17 \cdot \lambda \cdot \sqrt{\frac{f_c}{MPa}} \cdot b_w \cdot d_{efffs} \cdot MPa \quad\quad V_n = 74.03 \cdot kN$$

the critical section for is d from the support

$$R_s := \frac{V_n \cdot Lw}{(0.5 \cdot Lw - d_{efffs})} \quad\quad R_s = 161.94 \cdot kN \quad\quad R_s = 36.406 \cdot kip$$

6.0) Computing SDOF Equivalent System

Governing Resistance $\quad R_u := \min(R_b, R_s)$

$R_u = 118.212 \cdot kN$

Check3 := if($R_{bfo} < R_s$, "Rb < Rs, Bending Controls" , "Rs < Rb, Shear Controls")

Check3 = "Rb < Rs, Bending Controls"

Allowable ductility ratio $\quad \mu_a := 1.6$

Allowable support rotation $\quad \theta_a := 1.0 \cdot deg$ $\quad$ Low response condition

(Appendix 5, Table 5.B.1, ASCE Manual)

Moment of Inertia will be based on positive (inward) bending.

gross moment of inertia

$$I_g := \frac{b_w \cdot Tw^3}{12} \quad\quad I_g = 1.626 \times 10^9 \cdot mm^4$$

transformed rebar area

$$n \cdot A_{sf} = 3.85 \times 10^{3} \cdot mm^{2}$$

location of transformed neutral axis

$$d_{na} := \frac{-(n \cdot A_{sf}) + \sqrt{n \cdot A_{sf} \cdot (n \cdot A_{sf} + 2 \cdot b_w \cdot d_{effb})}}{b_w} \qquad d_{na} = 78.165 \cdot mm$$

cracked moment of inertia

$$I_{cr} := \frac{b_w \cdot d_{na}^{3}}{3} + n \cdot A_{sf} \cdot (d_{effb} - d_{na})^{2} \qquad I_{cr} = 2.737 \times 10^{8} \cdot mm^{4}$$

averaged moment of inertia

$$I_a := \frac{(I_g + I_{cr})}{2} \qquad I_a = 9.496 \times 10^{8} \cdot mm^{4}$$

effective stiffness

$$K_e := \frac{384 \cdot E_c \cdot I_a}{5 \cdot Lw^{3}} \qquad K_e = 7.254 \cdot \frac{kN}{mm}$$

beam mass M = (wall weight)/g

$$M := \frac{\gamma_c \cdot Tw \cdot b_w \cdot Lw}{g} \qquad M = 1.88 \times 10^{-3} \cdot \frac{kN \cdot s^{2}}{mm}$$

Because of the expected response, use an average of elastic & plastic values for KLM

Elastic Uniform Mass Factor $K_{Me} := 0.5$

Plastic Uniform Mass Factor $K_{Mp} := 0.33$

Elastic Load Factor $K_{Le} := 0.64$

Plastic Load Factor $K_{Lp} := 0.5$

Elastic KLM $K_{LMe} := \frac{K_{Me}}{K_{Le}}$ $K_{LMe} = 0.781$

Plastic KLM $K_{LMp} := \frac{K_{Mp}}{K_{Lp}}$ $K_{LMp} = 0.66$

Average KLM $K_{LM} := \text{mean}(K_{LMe}, K_{LMp})$ $K_{LM} = 0.721$

equivalent mass $M_e := K_{LM} \cdot M$ $M_e = 0.00135 \cdot \frac{kN \cdot s^{2}}{mm}$

period of vibration

$$t_n := 2 \cdot \pi \cdot \sqrt{\left(\frac{M_e}{K_e}\right)} \qquad t_n = 0.0859\, s$$

Blast load duration for front wall $t_{efw} = 0.123\, s$

Required time step $t_{stepr} := \min\left(\frac{t_{efw}}{10}, \frac{t_n}{10}\right)$ $t_{stepr} = 0.008586\, s$

Consider time step $t_{step} := 0.002 \cdot s$

Support Reaction

$F_{re} := 0.39$ $F_{fe} := 0.11$

$F_{rp} := 0.38$ $F_{fp} := 0.12$

$F_r := \mathrm{mean}(F_{re}, F_{rp})$ $F_f := \mathrm{mean}(F_{fe}, F_{fp})$

$F_r = 0.385$ $F_f = 0.115$

Time upto to which response is considered $t_d = 0.2\,s$

7.0) Numerical Integration

Time period for blast loading $t_d = 0.2\,s$

Peak loading $F_o = 82.417 \cdot kN$

Span of wall $Lw = 6.3\,m$

Effective stiffness $K_e = 7.254 \cdot \frac{kN}{mm}$

Effective Mass $M_e = 0.00135 \cdot \frac{kN \cdot s^2}{mm}$

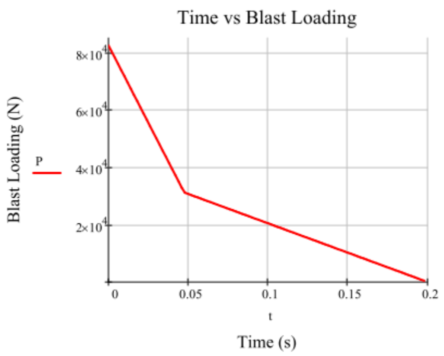
Dampig constant $C_{damp} := 0$

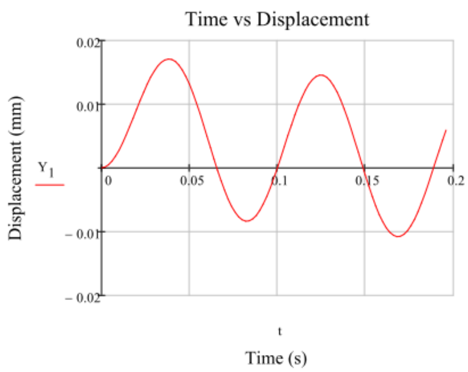
Time step consideration $t_{step} = 0.002\,s$

Yield deflection $y_e := \frac{R_u}{K_e}$ $y_e = 16.296 \cdot mm$

Positive resistance $R_{ut} := R_u$ $R_{ut} = 118.212 \cdot kN$

Negative resistance $R_{uc} := -R_u$ $R_{uc} = -118.212 \cdot kN$





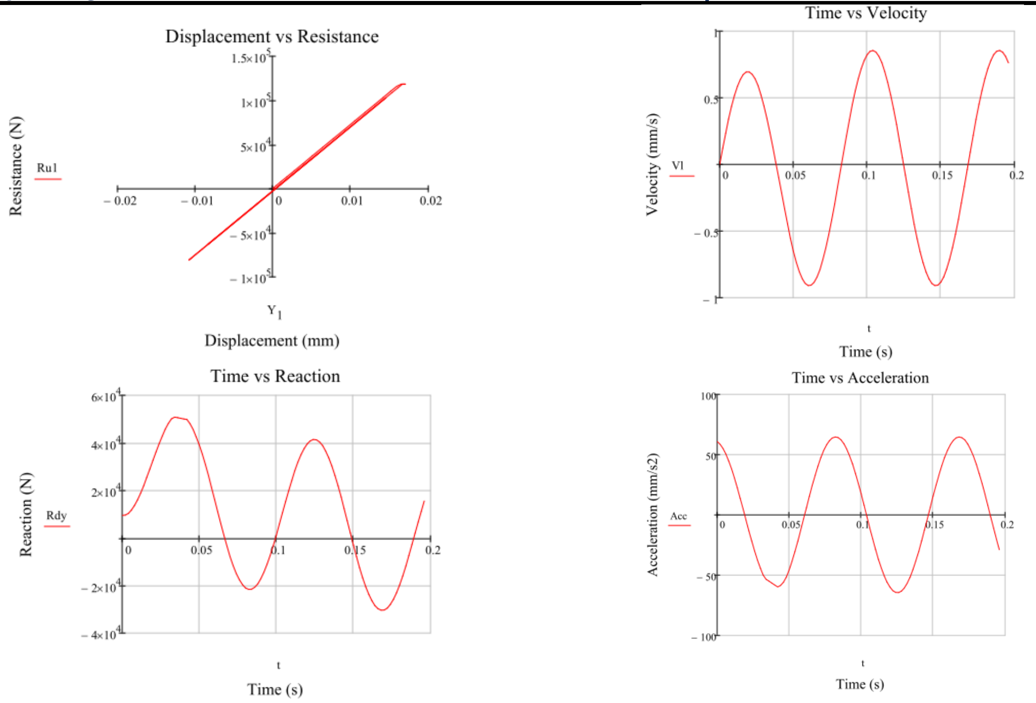


Figure 4 Graphs from Numerical Integration Method

Maximum Dynamic Reaction

$$\mathrm{maxRdy} := \max(\mathrm{Rdy})$$

$$\mathrm{maxRdy} = 50.771 \cdot \mathrm{kN}$$

Corresponding time

$$\mathrm{maxRdyTime} := \mathrm{vlookup}\left(\frac{\mathrm{maxRdy}}{\mathrm{kN}}, \mathrm{tableTRdy}, 1\right) \cdot \mathrm{s}$$

$$\mathrm{maxRdyTime} = (0.034)\ \mathrm{s}$$

Maximum Rebound Reaction

$$\mathrm{minRdy} := \min(\mathrm{Rdy})$$

$$\mathrm{minRdy} = -30.438 \cdot \mathrm{kN}$$

Corresponding time

$$\mathrm{minRdyTime} := \mathrm{vlookup}\left(\frac{\mathrm{minRdy}}{\mathrm{kN}}, \mathrm{tableTRdy}, 1\right) \cdot \mathrm{s}$$

$$\mathrm{minRdyTime} = (0.168)\ \mathrm{s}$$

Maximum Displacement

$$y_m := \max(Y_1)$$

$$y_m = 17.067 \cdot \mathrm{mm}$$

Corresponding time

$$\mathrm{maxDisTime} := \mathrm{vlookup}\left(\frac{y_m}{\mathrm{mm}}, \mathrm{tableTY}_1, 1\right) \cdot \mathrm{s}$$

$$\mathrm{maxDisTime} = (0.038)\ \mathrm{s}$$

The Plastic deformation

$$y_p := y_m - y_e$$

$$y_p = 0.771 \cdot \mathrm{mm}$$

Rebound Displacement $y_{mr} := \text{vlookup}\left(\frac{\text{minRdy}}{\text{kN}}, \text{tableRdyDis}, 1\right)\cdot\text{mm}$

$y_{mr} = (-10.79)\cdot\text{mm}$

Rebound elastic deformation $y_{er} := -y_{mr} - y_p$

$y_{er} = (10.019)\cdot\text{mm}$

Support Rotation $\theta_d := \text{atan}\left(\frac{y_m}{0.5\cdot Lw}\right)$ $\theta_d = 0.31\cdot\text{deg}$

$\text{Check4} := \text{if}(\theta_d < \theta_a, \text{"OK"}, \text{"Revise"})$ $\text{Check4} = \text{"OK"}$

Ductility ration $m_d := \frac{y_m}{y_e}$ $m_d = 1.047$

$\text{Check5} := \text{if}(m_d < \mu_a, \text{"OK"}, \text{"Revise"})$ $\text{Check5} = \text{"OK"}$

X. Results

Blast loading normal to short side of building

Sr. No.	Wall	Checks	Actual values	Limiting values
1	Front Wall (out-of-plane)	Support rotation	0.31 deg	1.0 deg
		Ductility ratio	1.047	1.6
2	Side Wall (out-of-plane)	Support rotation	0.168 deg	1.0 deg
		Ductility ratio	0.567	1.6
3	Side Wall (in-plane)	Support rotation	0.00324 deg	1.0 deg
		Ductility ratio	0.228	1.0
4	Roof (out-of-plane)	Support rotation	0.278 deg	1.0 deg
		Ductility ratio	0.819	1.3
5	Roof (in-plane)	Ductility ratio	0.135	1.3

XI. Conclusions

The RC control room building has been analyzed and designed for 20 kPa blast peak side-on overpressure and 200ms blast duration. It is found that the structural response of building are within deformation limits. Also based on results obtained, following conclusion are drawn.

1) The front wall has been checked for out-of-plane blast loading and has utilization ratio 30% in support rotation and 65% in ductility.
2) The side wall has been checked for out-of-plane blast loading and has utilization ratio 16% in support rotation and 35% in ductility.
3) The side wall has been checked for in-plane blast loading and has utilzatio ratio 0.3% in support rotation and 22% in ductility.
4) The roof slab has been checked for out-of-plane blast loading and has utilization ratio 27% in support rotation and 63% in ductility.
5) The roof slab has been checked for in-plane blast loading and has utilization ratio 10% in ductility.

XII. References

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