ANALYSIS AND DESIGN OF A SHELTER SUBJECTED TO BLAST LOADING

A DISSERTATION
submitted in partial fulfilment of the
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in
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CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the dissertation entitled "ANALYSIS AND DESIGN OF A SHELTER SUBJECTED TO BLAST LOADING" in the partial fulfilment of the requirements for the award of the degree of MASTER OF ENGINEERING in EARTHQUAKE ENGINEERING, of the University of Roorkee, Roorkee, is an authentic record of my own work carried out for a period from August 86 to February 87, under the supervision of Dr. S. Basu, Professor and Dr. G.I. Prajapati, Reader, Department of Earthquake Engineering, University of Roorkee, Roorkee (India).

The matter embodied in this dissertation has not been submitted by me for the award of any other degree or diploma.

Dated March 2 , 1987 (B.K. CHOUDHURY)

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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ABSTRACT

The objective of this thesis is to carry out analysis and design of an above ground close rectangular shelter subjected to conventional above ground explosion. The structure consists of a series of steel frame supporting the outer shell of roof and wall slab. The variation of the blast parameters have been studied and the design blast parameters have been found out from the data available for 1 tonne of explosive using scaling laws.

An average loading has been considered for design of roof slab and wall slab. The load transmitted by roof and wall slabs to roof girder and column has been idealized considering the dynamic reaction of roof and wall slab. The dynamic analysis of each element has been carried out by first transforming it into an equivalent single degree of freedom system.

The dynamic analysis and design have been carried out by using the charted solution available for a number of standard load-time functions. The numerical integration has been found necessary for the analysis of the roof girder and the frame.

Except the roof girder, connections and foundation, each element of the structure have been allowed to deform in the plastic range considering a ductility factor of 5. The limit state method has been used for the design of R.C.C. members and the design criteria given by the IS:4991-1968, IS:456-1978 and IS:800-1984 has been followed in the design.

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NOTATIONS

Notation	Description
A_s	AREA OF STEEL SECTION
A_{st}	AREA OF TENSILE STEEL
A_{tb}	AREA OF TENSION BOLT
A_w	AREA OF WEB OF I-SECTION
a	LENGTH OF SHORTER EDGE OF TWO-WAY SLAB
a'	DISTANCE BETWEEN CENTER LINE OF ANCHOR BOLT AND CENTER LINE OF COLUMN
B	WIDTH OF FRONT FACE
b	WIDTH OF RECTANGULAR SECTION
b_f	WIDTH OF FLANGE OF I-SECTION
b_p	WIDTH OF BASE PLATE
C	TOTAL COMPRESSION
C_d	DRAG CO-EFFICIENT
D	TOTAL DEPTH OF RECTANGULAR SECTION
D_p	TOTAL DEPTH OF BASE PLATE
d	EFFECTIVE DEPTH OF RECTANGULAR SECTION
d_ω	DEPTH OF WEB OF I-SECTION
E	MODULUS OF ELASTICITY
E_c	MODULUS OF ELASTICITY OF CONCRETE
E_s	MODULUS OF ELASTICITY OF STEEL
F	DYNAMIC FORCE AT ANY TIME 't'
F_e	EQUIVALENT DYNAMIC FORCE AT ANY TIME 't'
f_{ck}	CHARACTERISTIC COMPRESSIVE STRENGTH OF CONCRETE
f_b	BENDING STRESS
f_y	CHARACTERISTIC STRENGTH OF REINFORCEMENT

f_s	TENSILE STRESS IN ANCHOR BOLT
H	HEIGHT OF RECTANGULAR BOX-TYPE STRUCTURE
I_a	AVERAGE MOMENT OF INERTIA
I_{cr}	CRACK-SECTION MOMENT OF INERTIA
I_G	GROSS MOMENT OF INERTIA
I_{xx}	MOMENT OF INERTIA ABOUT THE AXIS OF MAXIMUM STRENGTH
j	CONSTANT
K_E	EFFECTIVE STIFFNESS
K_e	EQUIVALENT STIFFNESS
K_L	LOAD FACTOR
K_M	MASS FACTOR
K_{LM}	LOAD-MASS FACTOR
L	LENGTH
M	BENDING MOMENT
M_c	MACH NO.
M_e	EQUIVALENT MASS
M_t	TOTAL MASS
M_p	PLASTIC MOMENT
M_{pfa}	TOTAL POSITIVE ULTIMATE BENDING CAPACITY ALONG MID SPAN SECTION PARALLEL TO SHORT EDGE OF SLAB
M_{pfb}	TOTAL POSITIVE ULTIMATE MOMENT CAPACITY ALONG MIDSPAN SECTION PARALLEL TO LONG EDGE OF SLAB
M_{psa}	TOTAL NEGATIVE ULTIMATE MOMENT CAPACITY ALONG SHORT EDGE SUPPORT OF SLAB
M_{psb}	TOTAL NEGATIVE ULTIMATE MOMENT CAPACITY ALONG LONG EDGE SUPPORT OF SLAB
m	MODULAR RATIO

m_o	HIGHEST ORDINATE OF TRIANGULAR MASS DISTRIBUTION
N	CONSTANT
P_A	AXIAL LOAD
p_{ro}	PEAK REFLECTED OVERPRESSURE
p_{so}	PEAK SIDE ON OVERPRESSURE
p_a	AMBIENT AIR PRESSURE
p_o	HIGHEST ORDINATE OF TRIANGULAR LOAD DISTRIBUTION
p_r	REFLECTED OVERPRESSURE
p_s	SIDE ON OVERPRESSURE
q_o	PEAK DYNAMIC PRESSURE DUE TO BLAST WIND
q	DYNAMIC PRESSURE DUE TO BLAST WIND
R_m	MAXIMUM RESISTANCE
R	RESISTANCE WITHIN ELASTIC LIMIT
R_s	REACTION AT SUPPORT
S_t	ANY TIME STATION
T	NATURAL TIME PERIOD
T_t	TOTAL TENSION
t_a	ARRIVAL TIME
t_b	THICKNESS OF BASE PLATE
t_c	CLEARANCE TIME
t_d	DURATION OF IDEALIZED TRIANGULAR PULSE
t_f	THICKNESS OF FLANGE
t_m	TIME OF MAXIMUM DEFLECTION
t_{min}	MINIMUM REQUIRED THICKNESS
t_o	POSITIVE PHASE DURATION OF SHOCK WAVE
t_r	RISE TIME
t_s	THICKNESS OF DIAGONAL STIFFENER

t_t	TRANSIT TIME
t_w	THICKNESS OF WEB
$t_{(el)}$	TIME TOF ELASTIC DEFLECTION
U	SHOCK FRONT VELOCITY
V_A	DYNAMIC REACTION AT SHORT EDGE OF SLAB
V_B	DYNAMIC REACTION AT LONG EDGE OF SLAB
V	DYNAMIC REACTION OF BEAM OR GIRDER
V_u	TOTAL SHEAR FORCE
V_{us}	REQUIRED STRENGTH OF SHEAR REINFORCEMENT
V_y	MAXIMUM SHEAR CAPACITY AT YIELD FOR STRUCTURAL STEEL
v_s	VELOCITY OF SOUND IN AIR
W_d	TOTAL DEAD LOAD
W_t	TOTAL LOAD
x_u	DEPTH OF NEUTRAL AXIS
y	DEFLECTION
$\ddot{y}$	ACCELERATION
Z_{xx}	ELASTIC SECTION MODULUS ABOUT THE AXIS OF MAXIMUM STRENGTH
Z_p	PLASTIC SECTION MODULUS
α	DECAY PARAMETER
μ	DUCTILITY RATIO
δ	DEFLECTION
ω	NATURAL CIRCULAR FREQUENCY
$\phi(x)$	CHARACTERISTIC SHAPE
σ_{bf}	PERMISSIBLE BEARING STRESS OF CONCRETE
σ_{cbc}	PERMISSIBLE BENDING COMPRESSIVE STRESS OF CONCRETE

σ_{tf}	PPERMISSIBLE TENSILE STRESS IN ANCHOR BOLT
σ_y	CHARACTERISTIC STRENGTH OF STRUCTURAL STEEL
τ_c	PERMISSIBLE SHEAR STRESS OF CONCRETE
τ_{vs}	PERMISSIBLE AVERAGE SHEAR STRESS OF STRUCTURAL STEEL
τ_{vf}	PERMISSIBLE SHEAR STRESS OF ANCHOR BOLT
Δt	TIME INTERVAL

CONTENTS

CHAPTER - 1

INTRODUCTION

1.1 General

An explosion is a phenomenon resulting from a sudden release of energy. The sources of an explosion may be of various types, it may come from explosive such as gunpowder, from pressurized steam in a boiler, sudden release of gas from a gas pipe, from an uncontrolled nuclear transformation, from bursting of atomic or hydrogen bomb, or from detonation of conventional explosive device, etc. (In each case, as mentioned above, the energy release must be sudden one and happends so rapidly that a local accumulation of energy occurs surrounding the point of explosion). The detonation of a conventional explosive device releases a large amount of energy within a fraction of a second. During this time the components of the device volatilized into a sphere of hot compressed gas at a temperature of thousands of degrees and pressure of order of thousands of atmospheres. The parent sphere of intensely hot gas expands and radiates thermal energy in a band of short wave lengths range. Air is nearly opaque in this range of wave length, consequently, the shell of air surrounding the parent gas sphere absorbs radiated energy until it is approximately as hot as the sphere. Then it is turn radiates energy to another shell of air and so on until the temperature is reduced to a value at which this process is less rapid than the transmission of energy by molecular collision. After this energy is transmitted by shock wave or rapid pressure rise which moves radially outward from the point of detonation.

Relative to the position of a structure, a blast can take place at different position and according to this, blast can broadly be classified as (i) air blast (ii) at or near ground surface blast (iii) under ground blast and (iv) direct hit.

Explosive weapons are rated in terms of their energy yield which is expressed in terms of weight of TNT (trinitro-toluene) that will give the same amount of total energy yield as in the explosion to be rated. The weight of TNT corresponding to explosive energy release of one million kilo calories is defined as one ton of TNT.

Depending upon the source and location of detonation as mentioned above, amount of energy yield and types of target an explosion can cause wide spread damages resulting into major disasters. Damages are caused as a result of energy transmission from the source of explosion to a target. The most severe form of transmission of energy to the target is a direct mechanical coupling resulting into the shattering of the target depending upon the shattering power of the explosion. The indirect transmission of energy from an explosion to a target occurs through the action of flying missiles, thermal radiation and in the form of blast wave.

Here, the energy transmitted to the target depends on the nature of explosive, size of the explosion and the medium in which it occurs. In case of detonation of conventional device, the blast wave causes the violent frontal blow followed by tremendous enveloping crushing action while the target is simultaneously subjected to a blast wind of super hurrican velocity. However, for effective blast resistant design, the possibility of damages that may be caused by other damage mechanism such as fire hazards resulting from thermal radiation, impact of missiles, etc., should be taken into account.

1.1.1 Past damages to the structures caused by the conventional blast:

During World War-II, many bulletins and reports were prepared by different countries on the damage by high explosive bombs but serurity considerations prevented their wide circulation. However, the damages by atomic bombs have been more widely published. Different types of past structural damages can be described as follows -

(a) Collapse of R.C.C. roof: A bomb can penetrate an R.C.C. roof and then explode inside the building. In this case surrounding walls sway out and the roof caves in.

(b) Scabbing of R.C.C. roof: If a bomb explodes above a R.C.C. roof, the blast strikes the roof front-on and the roof experience a reflected overpressure. This will generate a stress wave within the roof slab which will reflect as a tension wave from free surface below. When a certain portion of cement concrete cannot stand the tension this would get detached which is called scabbing of R.C.C. roof.

(c) Damage of wall: Fracture may appear where the wall joins the roof and other walls. Fissures in wall are common phenomenon of blast damage.

(d) Factory type building: If a shed having roof of steel trusses with CGI sheets is subjected to blast loading, the steel trusses at the points of junction with the wall get lifted up. This results in cracks at the junction of steel trusses with the wall.

1.1.2 Past damages to the structures due to atomic blast:

To describe the large scale damage caused by blast, the two case histories on the 20 K.T. bombing incident on Hiroshima and Nagasaki prepared by U.S. Strategic Bombing Survey (1947) can be considered. However, it is necessary to interpolate the results of this data to the smaller yield based on sound theoritical principles. The two case histories are described very briefly in the following paragraphs.

The damage study due to atomic blast in Hiroshima included almost every R.C.C. structure, steel frame structures, structures having load bearing brick wall and also wood frame structures within the damage area. This study reveals that blast damage to building spread uniformly in all directions resulting in an approximately circular area of devastation. The limiting distance to which buildings were damaged depended upon their strength and type of construction. The strong and heavy multistory steel and concrete frame structures damaged only in an area relatively nearer to the point of detonation. At greater distances, steel and brick structures remained undamaged, where only the weakest wood frame building were damaged.

In addition to the extensive blast damage, Hiroshima also suffered an extensive fire storm which was initiated by the direct radiated heat and from secondary sources. The fire damage extended over an area of about 11.4 square km.

In Nagasaki, damage to building occurred over an area of 4.66 sq.km. The blast completely demolished dwellings within a radius of 1 km of ground zero and caused various degrees of damage to other structures within a radius of 4 km. The damage area was irregular in the shape partly because of the hilly terrain and partly because of the areas which were built up. The degree of damage to building varied according to their relative distance from ground zero, type and design of construction, their orientation to the direction of blast wave propagation and shielding effect of hills and man made structures. The specially designed R.C.C. and steel frame structures to withstand earthquake forces suffered less damage than the other buildings designed for normal loading. Structures of all kinds with their long axes parallel to the direction of blast wave propagation suffered less damage than the structures with their long axes

Perpendicular to the direction of blast wave propagation.

1.1.3 Importance of blast resistant design:

From the past experience as mentioned above it is seen that a large explosion can cause wide spread damage to the properties and human lives. The main objective of a blast resistant structure is to save life and its contents from the destructive effect of an explosion. Though it is not possible to protect a surface structure from a direct hit of an explosion, there are large areas surrounding an explosion in which conventional structures would collapse or suffer severe damage while balst resistant structure would suffer little or no damage and offer protection to its content.

The importance of the blast resistant capability of a structure depends upon the criticality of its location and the importance, cost and nature of the contents to be protected.

A nuclear power plant is a very important structure and its failure due to blast will not only cause damage to national property but will also cause great loss of lives due to secondary effect.

The airport hangers (both military and civil) houses the costly aircrafts. Loss of aircrafts due to collapse of shelters under blast loading must be avoided.

Protective structures such as ammunition magazines, high explosive stores,etc. must be strong enough to resist any damage due to blast in order to avoid secondary effect.

1.2 Resistant Structure and Important features in Planning & Design

1.2.1 Resistant structure:

Depending upon the function of the structure, its location, importance, etc., the resistant structure may take any one of many forms. It may be a small boxlike personal shelter buried undergoound designed for very high pressure levels or a boxlike above ground structure for a relatively low pressure or an extensive low-rise-surface dome designed for high pressure levels to protect some critical military equipment or function.

A number of considerations involve in the design problem of blast resistant structure. The important considerations are —

(a) The magnitude of forces imposed on a structure are very large in comparison with the forces for which the structure would normally be designed.

(b) The geometry of the structure, the amount of openings and the type of wall covering affect the magnitude of blast forces imposed on the structure.

(c) The danger of fire caused by thermal radiation or by secondary effects such as the shorting of electrical circuits or rupture of heating units or fuel lines is so great that special fire precaution to reduce fire risk is needed.

1.2.2 Level of protection:

The main objective of a blast resistant design is that the structure itself, its equipment, contents and occupants are as invulnerable to the effects of various weapons as is physically and economically feasible. The primary decision, as to whether or not protective construction should be employed in a new project and the level of protection needed, are an extremely complex problem. Consideration which will contribute to the formation of these decisions involve:

(a) The probability of attack.

(b) The size of weapon and likely distance to the structure.

(c) The vulnerability of the installation to attack.

(d) The evaluation of the relative importance of the activity or operation to be housed.

(e) The feasibility and cost of various levels of protective construction.

1.2.3 Selection of structural system:

Several factors affect the selection of the structural system. The selection of the structural system depends on its function. However, requirements necessary for frame, wall, openings, etc., under blast loading will make a basis for the preliminary selection of the type of a structure.

Another additional consideration affecting the selection of the structural system is the advantage to be gained by the use of geometric shapes of building which tend to reduce the intensity of blast load on the structure as mentioned earlier. Arch and dome constructions possess two advantages which can be utilized to obtain a high level of blast resistance. The first advantage is the reduction of blast load due to a curved surface facing the incident blast wave. The second advantage is the high efficiency of the structure from strength point of view. However, disadvantages arise of this type of construction from restricted space arrangement and demand of a higher cost of construction.

From strength point of view, though a rectangular box type blast resistant structure is less efficient than the dome or arch type construction, this type of structure may be desirable from the following considerations:

(a) Due to the inherent uncertainities in the blast loading that may be imposed on a structure, the actual behaviour of the structure under consideration also becomes uncertain. The expected behaviour becomes more uncertain if the structure is more complex in its shape.

(b) The dynamic analysis of a simple box type rectangular structure will be less complicated in comparison to other complex structure and due to simplicity it will demand less cost and time of construction which make this type of construction more desirable in an emergency.

1.2.4 Choice of underground construction :

Though the function of the structure and ground condition may prohibit buried construction or even semiburied construction, advantage should be taken, if practicable, of the high blast resistance of earth-covered structure either above or below the ground since this type of construction is extremely resistant to high blast pressure and penetration of flying splinters.

1.3 Definitions

For the purpose of this study, the following definitions shall apply —

1.3.1 Blast Wind: It is the moving air mass along with the overpressures resulting from pressure difference behind the shock wave front.

1.3.2 Clearance Time: This is time in which the reflected pressure decays down to the sum of the sideon overpressure and the drag pressure.

1.3.3 Decay Parameter: It is the coefficient of the negative power of exponent e governing the fall of pressure with time in the pressure time curve.

1.3.4 Drag Force: It is the force on a structure or structural element due to the blast wind. On any structural element, the drag force equals dynamic pressure multiplied by the drag co-efficient of the element.

1.3.5 Dynamic Pressure: It is the pressure effect due to blast wind.

1.3.6 Ground Zero: It is the point on earth surface vertically below the explosion.

1.3.7 Mach Number: It is the ratio of the speed of the shock front propagation to the speed of sound in standard atmosphere at sea level.

1.3.8 Over-Pressure: It is the rise in pressure above atmospheric pressure due to shock wave from an air blast.

1.3.9 Reflected Overpressure: It is the over pressure resulting due to reflection of a shock wave front striking any surface. If the shock front is parallel to the surface, the reflection is normal.

1.3.10 Shock wave front: It is the discontinuity between the blast wave and the surrounding atmosphere.

1.3.11 Side-on Overpressure: It is the overpressure if it is not reflected by any surface.

1.3.12 Transit time: It is the time required for the shock front to travel across the structure or its element under consideration.

1.3.13 Yield: It is the measure of the size of the explosion expressed in equivalent weight of reference explosive.

1.4 Objective of the study

The objective of the study is to analyse and design the rectangular box type over ground shelter subjected to blast loading caused due to the explosion of 0.1 tonne charge on the ground surface at a distance of 20.0 m, as per the guidelines given in the IS:4991-1968 (Blast Resistant Design of Structures). The analysis is carried out by using approximate method combined with equivalent single degree freedom system approach.

1.5 Out line of thesis

An outline of the matter presented in different chapters are given in the following paragraphs.

A brief description of the blast wave phenomenon and its behaviour striking upon an above-ground close rectangular structure resulting from a conventional blast explosion is given in the chapter-2. The blast considered is a surface contact burst or an air burst very near to ground surface in a normal homogeneous atmosphere of infinite extent. In the same chapter, variation of different blast parameters at different distances from ground zero are computed for different yield of burst by interpolation of data available for 1 tonne explosive (2) using scaling laws and the results are plotted to show the variations. For interpolation of data, a computer programme is developed. All the charts and table used for estimation of blast parameters are included for ready reference. Also idealized loading on different parts of a close rectangular structure is studied as per IS:4991-1968 (2).

Chapter - 3 describes the load calculations, general assumptions, design consideration and analysis and detail blast resistant design of an above ground close rectangular shelter for a blast loading of 100 kg explosive at a distance 20 m from ground zero by approximate design method. All the table used in the design and charted solution available for analysis of elastic and elasto-plastic SDOF (single degree of freedom) system subjected to triangular load time variation are included for ready reference. A computer programme has been developed to find out displacement response of an undamped elasto-plastic SDOF system with bi-linear resistance function subjected to load time variation (as encountered in the design of roof girder and column) for which charted solution is not available. Numerical integration has been carried out by constant velocity method which takes less time for computation and gives sufficiently accurate results for practical purpose if time interval considered is less than 1/10th of the natural period of the system.

The conclusions based on the present study are given in the chapter-4.

1.6 Review of Literature

Kinney (19) had described in detail the origin of blast wave, propagation of blast wave and the damage mechanism. An approximate design procedure for the above ground diffraction and drag type structure subjected to the above ground blast loading and the idealization of blast loading was discussed by Newmark (24). Francy (13) developed a method to determine the load-time relationship for the blast load transmitted by wall covering to the supporting frame. The triangular load pulse was considered for wall loading. Haltiwagner (16) investigated the problem of the blast loading on drag type and semi drag type above ground industrial building experimentally and analytically.

Norris et al. (23) discussed the blast wave phenomenon, its nature, two case histories on the bombing incident on Hiroshima and Nagasaki and general considerations regarding planning and design of blast resistant structure. Biggs (8) described the procedure of designing the structures for idealized blast loading by using the concept of equivalent SDOF.

Using available blast data, Misra et al.(21) investigated about the required impulse to demolish a 34.0 cm thick brick wall made with 1:6 or 1:8 (cement:sand) mortar. Chandrasekaran and Saini (10) discussed the method of finding an equivalent linear system from the nonlinear system usually encountered in the design of above ground structure subjected to explosive load.

Longinow et al. (20) studied the probability of survival of people located in burried personnel shelters subjected to blast effects produced due to detonation of one Mega Ton weapon near the ground surface treating roof slab as the weakest structural link. He also considered variations in the geometric parameters of the roof slab, material properties and airblast parameters. Dowding et al. (11) used SDOF approach to consider simultaneous efforts of air blast and ground motion on a structure from conventional explosion in mining. Ginsburg et al. (15) presented some aspects of analysis and design of protective structures using optimization approach.

Goyal(14) carried out inelastic dynamic analysis of an arch shelter subjected to blast load due to conventional explosion by idealizing the structure by two dimensional straight beam elements.

CHAPTER – 2

BLAST WAVE PHENOMENON, ITS PARAMETERS AND LOADING ON STRUCTURE FOR CONVENTIONAL EXPLOSION

2.1 Blast wave in infinite homogeneous atmosphere

To simplify the presentation, the explosion of a nominal size weapon in a normal homogeneous atmosphere of infinite extent is considered.

Immediately after the detonation occurs, the expansion of the hot gases initiates a pressure wave in the surrounding air, as shown very roughly by the curve in the fig. 2.1a. This shows the general nature of the variation of the air overpressure with distance from the explosion (ground zero) at a given instant. As the pressure wave is propagated away from the center of the explosion, the following or inner part moves through a region which has been previously compressed and heated by the leading or outer parts of the wave. The disturbances moves with the velocity of sound which increases with the temperature and pressure of the air through which the wave is moving. The innerpart of the wave moves more rapidly and catches up with the outerpart as shown in the fig. 2.1b. The wave front thus gets steeper and steeper and within a very short period it becomes abrupt as shown in the fig. 2.1c. At the advancing front of the wave, called the shock front, there is a very sudden increase of pressure from normal atmospheric to the peak shock pressure. The shock front thus behaves like a moving wall of highly compressed air.

Initially, in the hot central region of the bomb, the pressure exceeds atmospheric pressure by many hundred times. The pressure distribution behind the shock front is somewhat as shown in the fig. 2.1c (23). It shows the peak overpressure at the shock front usually represented as p_{so}, dropping rapidly in a relatively small distance to a value about one-half of the shock-front over-pressure. The magnitude of the over-pressure is uniform in this interior portion.

As the expansion proceeds, the pressure distribution in the region behind the shock front gradually changes. The over-pressure is no longer constant but drops of continuously nearer the center, causing a drop in pressure below the initial atmospheric value. Thus, a suction phase develops. The front of shock wave weakens as it progresses outward and its velocity reduces to the velocity of sound in the initial cooler air. The fig. 2.2 shows the over-pressure distribution in the shock wave as a function of the distance from the explosion at different stages in the expansion.

The behaviour of the shock wave from this time can be considered in two different ways as shown in the fig. 2.3 and 2.4. The fig. 2.3, shows distancewise variation of over-pressure at a particular time and the fig. 2.4 shows the timewise variation of over-pressure at particular distance from ground zero.

2.2 Idealization of over-pressure Vs time and dynamic pressure Vs time

Any surface encountered by the blast wave described above will also be subjected to a dynamic pressure in addition to the over-pressure. The incident blast wave characteristics are described by the peak initial over-pressure or side on over-pressure p_{so}, the over-pressure p_s versus time t curve, the maximum dynamic pressure q_o, the dynamic pressure q versus time t curve and the duration of positive phase t_o as shown in the fig. 2.5.

The pressure varies with time according to the following relations:

$$p_s = p_{so} \left(1 - \frac{t}{t_o} \right) e^{-\alpha \frac{t}{t_o}} \qquad (2.1)$$

$$q = q_o \left(1 - \frac{t}{t_o} \right) e^{-\alpha \frac{t}{t_o}} \qquad (2.2)$$

From the eqn.(2.1) and (2.2), it can be noted that dynamic pressure q decays much faster with time than the side-on over-pressure p_s. The pressure time relationship in the positive phase are idealized by using a straight line (24) starting with the maximum pressure value but terminating at a time t_d for over-pressure and t_q for dynamic pressure such that the impulse value remains same. The negative phase of the loading is neglected because the maximum negative over-pressure is much smaller than the peak positive over-pressure. The negative phase duration is 2 to 5 times as long as that of the positive phase. The peak negative over-pressure is approximately one-eighth of the peak positive over-pressure for air blast of low over-pressure.

2.3 Scaling laws of blast phenomenon

It has been found that blast parameters such as over-pressure and duration are related for different strength of explosion according to their ratio of cube root of the equivalent weight of TNT. These relationships are referred to as scaling laws. The scaling law states that if a given over-pressure is experienced at a radial distance of r_1 for an explosion of yield w_1, the same over-pressure for another explosion of yield equal to w_2 will be experienced at a radial distance r_2 from the center of explosion, where

$$r_2 = r_1 \left(\frac{w_2}{w_1} \right)^{1/3}$$

The same scaling law also states that if the peak pressure from two explosions of different yields are equal at the radial distances r_1 and r_2, then the positive phase duration at these two points is given by

$$t_{02} = t_{01} \left(\frac{w_2}{w_1} \right)^{1/3}$$

Now, if the yield of reference explosive is expressed in tonne and magnitude of reference explosive is 1 tonne, the above two relationship can be expressed as:

$$\text{Scaled distance} = \frac{\text{Actual distance}}{(w)^{1/3}} \qquad (2.3)$$

$$\text{and Scaled time} = \frac{\text{Actual time}}{(w)^{1/3}} \qquad (2.4)$$

where w is the yield of explosion in tonnes under consideration.

2.4 Variation of blast parameters

As mentioned above, blast parameters vary with total yield of explosion and distance from ground zero. The blast parameters at different distances from ground zero for 1 tonne explosive is given in table 2.1,(2), which will be considered as reference explosive in this study. The ambient air pressure is considered as 1 kg.cm^2 at mean sea level. By interpolation of the data presented in Table 2.1, blast parameters for different yield of explosion and at different radial distances from ground zero have been calculated using scaling laws given in the eqn.(2.3) and (2.4). The calculations are carried out with the help of computer program. The results are plotted in the fig. 2.6 to 2.11 from which p_{so}, p_{ro}, q_o, t_o, t_d and Mach No. can be calculated for three different yields of explosion (0.1t, 0.2t and 0.3t) at various distances ranging from 20 m to 35 m from ground zero.

To calculate the blast load at different points of a structure, it is necessary to calculate the various blast parameters at that points. To illustrate this, an industrial type structure is considered as shown in the fig. 2.12. Two cases can be considered, i.e.,(i) direction of blast wave propagation perpendicular to y-axis (Ref. fig. 2.12a) and

(ii) direction of blast wave propagation perpendicular to x-axis. Here, only case (i) will be illustrated. The radial distances from ground zero for different significant points are calculated as shown in the fig. 2.12b. The blast parameters calculated for different points on the structure are given in the tables 2.2 through 2.4.

2.5 Blast load on the above ground structures

2.5.1 General:

The manner in which the blast wave loads a structure is a function of the distance of the structure from ground zero, the height of the burst of the weapon and the weapon size. The shock front from a contact burst is almost vertical as shown in the fig. 2.3 and the effect of yield of a contact burst is almost the double of an equal explosion high in the air.

The most severe blast loading on any face of a structure is produced when the structure is oriented with the face normal to the direction of propagation of the shock front. However, for the lack of known orientation of future explosion, every face of a structure must be considered as a front face.

The total blast load that may be experienced by a structure depends also on the type of structure. There are mainly two types of structures that may be considered for blast loading (i) diffraction type structure and (ii) drag type structure. The first one is the closed structure without openings, with the total area opposing the blast. This type of structure will be subjected to both the shock wave pressure p_s and the dynamic pressure q caused by the blast wind. The structures with opening area less than 5% of the total wall area can be considered as a closed structure (24). The other type of structures are composed of elements like beams, columns, etc., which have small area opposing the shock wave. These are mainly subjected to dynamic pressure q.

2.5.2 Loading on a closed rectangular structure:

The procedure given below will be applied to calculate the blast loading on the structure considered for analysis (2).

To calculate the loading on different parts of the structure, the same idealization of the pressure time variation as shown in the fig. 2.5 is considered for the incident wave. It is assumed that p_{so} in the incident blast wave does not diminish in strength as the wave passes over the structure (23).

Front face: when the shock wave strikes the vertical face of the structure, the pressure in the front face instantenously increases to the reflected over-pressure p_{ro} which is given by

$$p_{ro} = p_{so} \left(2 + \frac{6p_{so}}{p_{so} + 7\ p_a}\right) \qquad (2.5)$$

The net pressure acting on the front face at any time t is the reflected over-pressure p_r or $(p_s + C_d\ q)$

whichever is greater.

The reflected over-pressure p_r drops from peak value p_{ro} to over-pressure $(p_s + C_d\ q)$ in a clearance time t_c which is given by

$$t_c = \frac{3S}{U} \text{ or } t_d \text{ whichever is smaller} \qquad (2.6)$$

where

S = height of the building H or 1/2 the width of front face perpendicular to the direction of wave propagation whichever is smaller

and U = shock front velocity

$$= M_c \ v_s \qquad (2.7)$$

where

v_s = velocity of sound in air which may be taken as 344 m/s at mean sea level at 20^{o}C

M_c = Mach mnumber of the incident pulse given by $\sqrt{1 + 6\ p_{so}/\ 7\ p_a}$

The over-pressure on the front face of a structure is not uniformly distributed. The maximum value occurs at the midpoint of the base and the minimum value occurs along the edges. Those portion of the front face nearest to the edges are cleared or relieved of the reflection effects in a shorter time than the remainder of the front face and the over-pressure existing at those points is lower, following the clearing stage. The net effect of this vertical and horizontal variation is of questionable value (23) for design purpose.

Hence, the front wall loading is assumed to be distributed uniformly over the front-wall face. For the same reason the rear-wall loading discussed subsequently is assumed to be uniformly distributed over the surface.

The net average loading on the front face as a function of time is shown in the fig. 2.13. The pressures p_{ro}, p_{so}, q_o and time t_d for actual explosion can be calculated from the data given for the reference explosion using scaling laws.

Rear face: when the shock front crosses the rear edge of the structure, the foot of the shock spills down the back wall. The over-pressures on the back wall behind this differential wave are considerably less than those in the incident blast wave because of the vortex which develops at the top and travels down the wall. A period of time longer than that required for the passage of this diffracted shock to the bottom of the wall must pass before the back wall average over-pressure reaches its peak value. The time required to build up the pressure on the back wall is measured from the instant the shock front reaches the back wall. This time is called the rise time.

The average pressure-time variation on back face can be considered as shown in the fig. 2.14 where the time has been measured from the instant the shock first strikes the front face. The time intervals of interest are as follows -

$\frac{L}{U}$ = travel time required for the shock front to travel from front to rear face, where L is the length of the structure along the direction of propagation of blast wave = t_t

and $\frac{4S}{U}$ = t_r = rise time (2.8)

The value of C_d, i.e., the drag coefficient used to incorporate the dynamic pressure on the structure are given in the table 2.5 for different shape of elements.

Roof and side walls: During the passage of the blast wave across the structure low pressure areas develop on the roof and side walls because of formation of vortex. This causes pressure variation in both the directions of the roof and side walls. But for design purposes, an average pressure time variation will be considered as shown in the fig. 2.15a. The rise in pressure is considered to be initiated at a time equal to L/2U and from this time pressure rises to its peak value within a time equal to L/2U. When transit time t_t is greater than the duration t_d of the shock wave, this average over-pressure on the roof can not be considered because some portion nearest to the front edge of the roof or wall will be free from over-pressure at the time when peak average value is considered to be reached, i.e., at time L/U. In this case the load on roof or side wall may be considered as moving triangular pulse having the peak value of over-pressure $(p_{so} + C_d\, q_o)$ and time t_d as shown in the fig 2.15b.

Overturning of structure: under the action of the blast loading, the structure is subjected to large horizontal force which tends to cause overturning and sliding of the structure. The net average force causing this overturning and sliding can be obtained as a function of time by substracting the average loading on back wall from that on the front wall.

CHAPTER - 3

ANALYSIS AND DESIGN

3.1 General

The structure considered here for analysis is a single storey, above ground closed rectangular structure as shown in the fig. 3.1 which consist of a series of steel rigid frames supporting the outer shell of reinforced concrete slabs.

It is seen from the discussions presented in the previous chapters that the structures designed to resist blast forces are subjected to completely different type of load than that considered in conventional design. The blast wave loads the exposed surface of the structure and then the load is transmitted to the other elements. Thus, the response of each individual element is important unlike the ground motion where the whole structural system is simultaneously subjected to inertia forces. It is, thus, seen that the structure to be analysed is complex as a dynamic system which consists of a group of elements, each with distributed mass and hence infinite degrees of freedom, all interacting with one another in a complicated manner under the action of rapidly moving shock wave. A rigorous solution is impractical in this case. So an approximate method has been applied for analysis and design of the structure which is more convenient for practical design purpose considering each element of the total structure to be an independent one degree freedom system. Several assumptions and idealizations have to be made to carry out the analysis and design validity of which are discussed here.

3.2 Assumptions and general design consideration

The structure is assumed to be situated at a distance of 20 m from ground zero and yield of explosion is considered as 100 kg (2).

The blast wave is considered to be propagated perpendicularly to the long axis of the building as this condition will cause severe effect for the frame. But for design of wall slab, all faces will be designed considering it as a front face. The structure is windowless as its function is to protect its contents from blast effect. If the structure is fitted with the doors, these doors should be also designed for the front face blast loading, however, this has not been included in this study.

Considering the probability of occurrence of blast loading to be small, the structural element may be permitted to deform in the plastic range for economical design as the plastic deformations increases the energy absorption capacity of the structural elements with a further advantage that the effective time period of the structural element is elongated thereby reducing the effective load for design. Therefore, all elements except foundation and roof girder have been permitted to undergo plastic deformation corresponding to a ductility ratio of 5,(2).

The damping effect has been neglected as it will have less importance in controlling the maximum response of a structure to this impulsive loading.

The design criteria given in IS:4991-1968(2), IS:456-1978 (1) and IS:800-1984 (3) are followed for designing the structure.

3.3 Resistance function

Depending upon the types of materials used, the resistance function of a structure may have different forms as shown in the fig. 3.2a where the curve A represents a structure of brittle materials, the curve B represents the structure of a ductile material such as reinforced concrete and the curve C represents the structure where resistance decreases after a certain deflection but before complete failure which may be the case in plain concrete. For approximate design, these resistance functions are idealized as shown in the fig. 3.2b and fig. 3.3.

For the most structure, the resistance function can be idealized as a bilinear function as shown in the fig.3.2b where area under the bilinear curve up to maximum permissible deflection has been kept same to that of actual curve, thus keeping the energy absorption capacity same.

But for some other cases as in the case of fixed beam, the resistance function can not be bilinear because in this case first plastic hinges form at supports and further deflection is required to form midspan hinge and hence attain maximum resistance. Therefore the resistance function becomes as shown by the solid line in the fig.3.3. This resistance function can further be idealized by the broken line as shown in the fig. 3.3 the slope of which will give effective stiffness K_E. This idealization will require the area OABC is equal to the area ODBC.

The maximum resistance R_m, for the present purpose is defined as the total load having the same distribution (for which the analysis being made) which the element could support statically. The stiffness is defined as the total load of the same distribution which can clause a unit deflection at the significant point where the deflection is equal to that of the equivalent system.

As stated above, K_E will be calculated as follows —

Area OABC = Area ODBC

Let $(y_{el})_2 - (y_{el})_E = y$

$$\therefore \frac{1}{2}(y_{el})_1 \times R_1 + \left[(y_{el})_2 - (y_{el})_1\right] \times R_1 + \frac{1}{2}\left[(y_{el})_2 - (y_{el})_1\right]\left[R_m - R_1\right] = \frac{y + (y_{el})_2}{2} \times R_m$$

$$\text{or } \left[y + (y_{el})_2\right] R_m = (y_{el})_1 R_1 + 2R_1\left[(y_{el})_2 - (y_{el})_1\right] + \left[R_m - R_1\right]\left[(y_{el})_2 - (y_{el})_1\right]$$

$$\text{or } y = \frac{1}{R_m}\left[(y_{el})_1 R_1 + 2R_1\left\{(y_{el})_2 - (y_{el})_1\right\} + \left\{R_m - R_1\right\}\left\{(y_{el})_2 - (y_{el})_1\right\} - R_m (y_{el})_2\right] \quad \ldots(3.1)$$

$$\therefore \quad (y_{el})_E = (y_{el})_2 - y \quad \ldots(3.2)$$

$$\therefore \quad K_E = R_m/(y_{el})_E \quad \ldots(3.3)$$

3.4 Stress-strain condition

The stress and strain diagram shown in the fig.3.4 will be considered to find out the ultimate bending capacity of a rectangular R.C.C. section.

3.5 Load calculation

As per assumptions mentioned above, basic loading data are calculated using scaling laws and by interpolation of the data presented in the table 2.1 (2). The calculated blast parameters for the incident wave are given below —

p_{so} = 0.72371 kg/cm^2

Mach No. = M_c = 1.2691

t_o = 14.058 m.sec.

t_d = 9.6118 m. sec.

q_o = 0.17003 kg/cm^2

$$p_{ro} = 1.8575 \text{ kg/cm}^2$$

$$p_a = 1 \text{ kg/cm}^2$$

Shock front velocity,

$$U = M_c \; v_s$$

$$v_s = 344 \text{ m/sec.}$$

$$\therefore \; U = 1.2691 \times 344 \times 100 \times \frac{1}{1000}$$

$$= 0.4366 \text{ m/m.sec}$$

$$t_c = \frac{3S}{U}$$

Here $H = 4\text{m}$

and $\frac{B}{2} = 9\text{m}$

$$\therefore \; S = H = 4.00 \text{ m}$$

$$\therefore \; t_c = \frac{3 \times 4}{0.4366} = 27.49 \text{ m.sec}$$

3.5.1 Front face loading

As $t_c > t_d$, therefore a triangular pressure-time variation will be considered as shown in the fig. 3.5a which gives the average front face loading as per idealization made in the Chapter - 2.

3.5.2 Roof loading

$$t_t = \frac{L}{U} = \frac{5}{0.4366} = 13.74 \text{ mili. sec.}$$

$$> t_d$$

$\therefore$ Load on roof may be considered as a moving pulse as shown in the fig. 3.5b.

Peak value of overpressure

$$= p_{so} + C_d\, q_o$$

for $q_o = 0.17003$ kg/cm^2, value of

$C_d = -0.4$ (Table – 2.5)

$\therefore$ Peak value of over pressure

$$= 0.72371 - 0.4 \times 0.17003$$

$$= 0.656 \text{ kg/cm}^2$$

3.5.3 Loading on Side Wall

All the side walls have been designed as front wall subjected to the front face loading.

3.5.4 Rear Face Loading

Rise time $t_r = \frac{4S}{U} = \frac{4 \times 4}{0.4366} = 36.65$ m.sec $> t_d$

Since t_r is greater than t_d no rear face loading will be considered.

3.6 <u>Design of the roof slab</u>

3.6.1 Loading

It is seen from the fig 2.6 that the overpressure decreases as the radial distances from the ground zero increases. Due to this reason, the overpressure variation on the roof can be represented as shown in the fig 3.6. As per design criteria given in the IS:4991-1968(2), the blast loading on the roof is to be considered as Moving Pulse as shown in the fig 3.6(a–c) however, the actual blast loading considered for the purpose of the analysis is taken as average roof loading as shown in the fig 3.6d. (Detail calculations have been shown in the Appendix – A).

3.6.2 Design considerations

The roof slab has been analysed and designed as a two way slab with all four edges restrained. The edges can be considered as restrained because the adjacent monolithic roof panels will be loaded simultaneously and the top edges of the front and rear walls, which are monolithic with roof slab, have much larger thickness than the roof slab. The dimension of the roof slab perpendicular to the direction of propagation of wave is 6.00 m and the dimension along the direction of wave propagation is 5.00 m (fig 3.1).

3.6.3 Analysis and design

The roof slab has been analysed and designed following the steps given in the Appendix – A. The characteristic strengths of the concrete, f_{ck} and steel, f_y, have been taken as 150 kg/cm^2 and 2500 kg/cm^2, respectively.

These values have been increased by 25% for blast loading. The total thickness of the slab has been calculated as 13 cm for the steel ratio $p_s = 0.0016$ to achieve the ductility ratio $\mu = 5$. The roof slab has been checked for shear considering the dynamic reactions at long edge and has been found safe without shear reinforcement. The details of roof slab has been shown in the fig 3.12.

3.7 Design of wall slab

3.7.1 Design considerations

The footing of the wall is assumed to have less rigidity than the required rigidity to provide the rotational restraint to the wall. Again the roof slab supporting the top edge of the wall slab has less thickness and hence insufficient strength to restrain the wall efficiently. Therefore, the wall slab has been analysed and designed as one way slab (neglecting the support provided by vertical steel column) being simply supported at the top and bottom edges.

3.7.2 Design

The following material properties which have been increased by 25% for the blast load are considered in the design.

f_{ck} = 150 kg/cm^2, f_y = 4150 kg/cm^2 for main reinforcement and f_y = 2500 kg/cm^2 for shear reinforcement.

The analysis and design of the wall slab have been carried out following the steps given in the Appendix-B. The total thickness of the wall has been calculated as 40.0 cm for the steel ratio 0.0026 to achieve the ductility ratio $\mu = 5$. The wall slab has been checked for the shear considering dynamic reactions at supports and necessary shear reinforcement has been provided. The details of wall slab is shown in the fig 3.13.

3.8 Design of the roof girder

3.8.1 Design considerations

The roof girder has to perform two functions. It has to support the slab and at the same time it has to perform its part of the frame action in resisting the horizontal force. Thus, the design of the girder becomes complicated under these two facts. Considering its two-fold function, it is seen that if the girder is allowed to deform in the plastic range under the vertical load coming from roof, its ability to restrain the column under horizontal frame action may be impaired. Since the horizontal response causes the plastic hinges in the column at the girder connection, the column may not be able to provide effective moment restraint for the girder acting under vertical load. Under these facts, the girder can be designed elastically as a simply supported beam. For more accurate method of design, one has to consider the horizontal and vertical responses simultaneously. But for a simplified design procedure, the girder can be designed as mentioned above without much error.

3.8.2 Loading

The vertical load on the girder will be the sum of the dynamic reactions coming from the adjacent roof panels. Dynamic reactions will vary with time and, therefore, the load-time variation for the girder is required. The load time variation for design of girder is shown in the fig 3.8a. Calculation of this is given in Appendix-C.

3.8.3 Design

The analysis and design of the roof girder have been carried out following the steps given in the Appendix-C. For the design of the roof girder, first it has been tried to further idealized the load time function into a triangular load time function (as shown by the dotted line in the fig 3.8a) to make use of the charted solution available for this type of loading. As this idealization is found invalid, the maximum response of the girder has been found out by the numerical analysis given in the Appenxix-C.

The ISMB 300 @ 46.1 kg/m having the characteristic strength σ_y = 2400 kg/cm^2 is found safe for the design loading. The calculated maximum deflection and the yield deflection are found to be 1.32 cm and 2.38 cm, respectively which are within the safe limits as per the design considerations mentioned above. The details of the ISMB 300 is given in the Appendix-C.

3.9 Design of the frame

3.9.1 Loading

The total horizontal force on the frame is the sum of the dynamic reaction of the front wall and the rear wall. Since in this case, there is no rear face loading, the total horizontal dynamic force acting on the frame will be given by the dynamic reaction of the front wall only. To calculate the total load time variation of the frame, the same idealization as in the case of the girder has been made. The design load time variation is shown in the fig 3.8b, the detail calculations of this is shown in the Appendix-D.

3.9.2 Design

The detail design calculations are given in the Appendix-D. The plastic hinges form only in the columns of the frame and the plastic bending strength of the columns have been calculated accordingly. To calculate the characteristic shape of the frame, the deflected shape of the frame under the horizontal loading has been assumed as shown in the fig 3.10,(8).

Since the charted solution for the design load-time function of the frame is not available, the analysis has been carried out by using the same numerical method as in the case of the roof girder and the column section of ISWB 200 @ 20.8 kg/m, having characteristic strength, σ_y = 2200 kg/cm^2, has been found safe for the design loading. The yield deflection, $y_{(el)}$ and maximum deflection, y_{max}, are found to be 4.0 cm and 10.25 cm, respectively. The calculated ductility ratio, μ, is found to be 2.56 which is less than 5. The column section has been checked for the shear force at base and for the stability under the axial load.

3.10 Design of beam column connection

3.10.1 General

From the frame design it is seen that plastic hinges form at the both ends of the column. As the ductility of connection is questionable or it is difficult to design fully ductile connection due to the uncertainities in the stress distribution and uncertainities in the complete control of the quality of welds, the plastic hinges are desirable to form in the column. For this reason and because the frame design requires considerable ductility, it has been recommended that (23) the connection be made slightly stronger than the member itself by which it can be ensured that plastic hinge will form in the member where desired ductility behaviour is more certain.

Referring to the frame design, the column-girder connection and column base details are both designed for the maximum bending moment and direct stress carried by the column.

3.10.2 Design

The detail arrangement of beam-column connection is shown in the fig 3.14. The beam is taken straight over the column. The outer flange of the column is connected to the beam by a cover plate of size 46 cm x 12 cm x 12 mm having σ_y = 4000 kg/cm^2 considering the thickness of web of beam, diagonal stiffener of zise 100 mm x 16 mm is found necessary and they are provided on both sides of the web in the joint as shown in the fig 3.14. Detail design calculations are shown in the Appendix-E.

3.11 <u>Design of base plate and connection to column</u>

The base plate has been designed for the axial load and plastic moment at the base of the column. The anchor bolts have been designed to resist tension in the base plate. The base plate has been placed on concrete foundation having characteristic strength, f_{ck} = 200 kg/cm^2. The size of base plate is found to be 300 x 400 x 38 mm having σ_y = 4200 kg/cm^2. To design the base plate and the anchor bolts, the stress and strain diagrams shown in the fig 3.15a and fig 3.15b, respectively, have been used.

The column has been connected to the base plate by four cleat angles (σ_y = 4200 kg/cm^2) fixed by fillet weld as shown in the fig 3.15. The detail design of base plate and connections are given in the Appendix-F.

3.12 Design of footing

The general arrangement of common footing of the column and the wall slab is shown in the fig 3.13. The inner side of the footing is covered by the concrete filling as shown in the fig 3.13 to prevent the direct contact of the steel column with foundation soil. The characteristic strength of concrete, f_{ck} and steel f_y have been taken as 200 kg/cm^2 and 2500 kg/cm^2, respectively. The unit weight and permissible static bearing capacity of soil have been assumed as 1.8 t/m^2 and 10 t/m^2, respectively. The dynamic bearing capacity of soil have been considered twice the static bearing capacity (2). The pressure distributions on the footing parallel to the direction of blast wave propagation are shown in the fig 3.16. The footing has been designed considering the loads and moments given in the table 3.3 and the total required depth of footing is found to be 30 cm for steel ratio, $p_s = 0.0016$. In the design of the footing, the blast loading that may be transmitted to the footing through soil have been neglected (2). The detail design calculations are given in the Appendix-G.

3.13 Design of end wall

The design and analysis of the end wall have been carried out by considering it as the two way simply supported slab at all the four edges. The same load time function (fig 3.5a) as in the case of front wall has been considered for the design. The characteristic strengths of concrete, f_{ck}, and steel, f_y have been considered as 150 kg/cm^2 and 2500 kg/cm^2, respectively. The total required thickness is calculated as 38 cm for steel ratio, $p_s = 0.0016$. The analysis and detail design calculations are given in the Appendix - H and details of reinforcement are shown in the fig 3.17.

CHAPTER - 4

CONCLUSIONS

The analysis and design of the shelter subjected to the blast load have been carried out by idealizing the elements of the structure such as the front wall, roof slab, girder and frame into an individual equivalent single degree of freedom system using the approximate method. The roof slab and the front wall have been analysed by using the available charted solution for the standard load time functions. For analyzing the girder and the frame, the load time variations to which they are subjected to have been obtained by using numerical analysis.

The front wall and the roof have been designed for the ductility ratio $\mu = 5$. The deflection of the roof slab at yield and the maximum deflection are 0.43 cm and 2.15 cm, respectively. The maximum deflection of the roof occurs after the duration of 34.2 m-sec. The girder, which has been analysed on the basis of elastic behaviour, undergoes the maximum deflection of 1.3 cm. This maximum deflection occurs after the duration 46.4 m-sec. Since the maximum girder deflection is less than the maximum roof slab deflection, the effect of the girder deflection on the roof slab can be neglected.

The approximate method which taken into account the dynamic properties of each structural element is good enough for the analysis of the shelter subjected to blast loading. This approach also consideres the reactions (which vary with time) transmitted from one structural element to the another. This approach is justified because in reality the distance of the structure from the ground zero and the yield of the explosion which governs the blast loading on the structure, can never be known exactly. Such uncertainity of the loading for which the structure is to be designed justifies the use of the analysis and design procedures used in this investigation.

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AAPPENDIX - A

DESIGN OF THE ROOF SLAB

A.1 Pressure distribution on roof

At time $t = 0$, over-pressure along the front edge of the roof at point A of the fig. 3.6(a) will be equal to 0.632 kg/cm^2 which will vary with time and reduce to zero in 9.798 m. sec. The wave front velocity, U is 0.434 m/m-sec.

The leading edge of the wave front takes 7.067 m-sec to travel from point A to point B as shown in the fig. 3.6(a). At this instant, the pressure at A is 0.176 kg/cm^2.

When the leading edge of the wave front travels the distance of 4.12m measured from A in 9.798 m-sec, the pressure at the leading edge becomes 0.468 kg/cm^2 while the pressure becomes zero at A as shown in the fig. 3.6(b).

When the leading edge of the wave front reaches point C as shown in the fig. 3.6 (c) in 14.402 m-sec, the pressure at C becomes 0.409 kg/cm^2.

Average roof pressure for design

At no time the entire length of the roof in the direction of the propagation of the wave front is loaded by the moving pressure pulse and, therefore, the roof slab has to be designed for moving pressure pulse. However, to simplify the analysis, the average pressure pulse on the roof slab is calculated by considering the pressure pulse $p_{so} + C_d q_o$ $= 0.656$ kg/cm^2 covering the length of 4.2 m of the roof slab.

At $t = 9.6118$, total load on roof

$$= \frac{1}{2} \times 0.656 \times 6 \times 4.2 \times 100 \times 100$$

$$= 82656 \text{ kg}$$

$\therefore$ Average load on roof $= \frac{82656}{5 \times 6 \times 100 \times 100} = 0.28$ kg/cm^2

$\therefore$ The average pressure diagram considered for the design will be as shown in the fig 3.6d

where

$$t_d = 9.6118 + \frac{0.8}{0.4366} = 11.44 \text{ m.sec}$$

A.2 Design calculations

To find out the first trial value of R_m, let

$$T = 0.05 \text{ sec.}$$

$$\therefore \quad \frac{t_d}{T} = \frac{11.44}{0.05 \times 1000} = 0.23$$

From the fig 3.7a, for μ = 5

$$\frac{R_m}{F_1} = 0.23$$

$$\therefore \quad R_m = F_1 \times 0.23$$

$$= 0.28 \times 1.2 \times 0.23$$ (The load has been increased by 20% (1))

$$= 0.08 \text{ kg/cm}^2$$

$\therefore$ Total estimated maximum resistance

$$= 0.080 \times 5.00 \times 6.00 \times 10^4$$

$$= 24000 \text{ kg}$$

Now from the table 3.1, the maximum resistance of the slab for an aspect ratio = 5.0/6 = 0.83, is given by

$$R_m = \frac{1}{a}\left[12\ (M_{pfa} + M_{psa}) + 10.3\ (M_{pfb} + M_{psb})\right] \quad \ldots (3.4)$$

Considering M_{pfa}, M_{psa}, M_{pfb} and M_{psb} are equal, i.e, bending resistance are equal at all points of the slab, equation (3.4) can be written as

$$R_m = 1/a\left[12(2M_p x5.0x100) + 10.3(2M_p x6x100)\right]$$

where a is the shorter edge of the slab and M_p is the ultimate bending strength per cm width of the slab.

$$\therefore \quad R_m = \frac{1}{5x100}\left[12000\ M_p + 12360\ M_p\right]$$

$$= 48.72 \times M_p$$

$$\therefore \quad \text{Required } M_p = \frac{24000}{48.72} = 492.61 \text{ kg-cm/cm}$$

$$f_{ck} = 150 \text{ kg/cm}^2 \times 1.25 = 187.5 \text{ kg/cm}^2$$

$$f_y = 2500 \text{ kg/cm}^2 \times 1.25 = 3125 \text{ kg/cm}^2$$

Assuming $p_s = 0.0016$

From the fig 3.4

Total tension = Total compression

or $A_{st} \times 0.87 f_y = 0.36 \times f_{ck} \times b \times x_u$

or $p_s \times b \times d \times 0.87 \times f_y = 0.36 \times f_{ck} \times b \times x_u$

$$\therefore \frac{x_u}{d} = \frac{0.87 f_y p_s}{0.36 f_{ck}} \qquad \ldots (3.5)$$

Moment of resistance in terms of tensile steel

$$M_p = A_{st} \times 0.87 \times f_y (d - 0.416 x_u)$$

Substituting the value of x_u from the equation (3.5) in the above expression, we get

$$M_p = p_s bd^2 0.87 f_y \left[1-1.005 \frac{f_y p_s}{f_{ck}}\right] \qquad \ldots (3.6)$$

$$\therefore 492.61 = 0.0016 \times 1 \times d^2 \times 0.87 \times 3125 \left[1-1.005 \frac{3125 \times 0.0016}{187.5}\right]$$

$$\therefore d = 10.79 \text{ cm}$$

Provide overall depth of slab = 13 cm

Now, the slab will be checked for maximum response as follows

From the table 3.1

$$R_1 = 26.4 M^o_{psb}$$

or $R_1 = 26.4 M_p$

and $K_1 = \dfrac{705 EI_a}{a^2}$

$$K_2 = \frac{212 EI_a}{a^2}$$

From the fig 3.3

$$(y_{el})_1 = \frac{R_1}{K_1} = \frac{26.4\ M_p\ a^2}{705\ EI_a} = 0.0374\ \frac{M_p\ a^2}{EI_a}$$

$$(y_{el})_2 = (y_{el})_1 + \frac{R_m - R_1}{K_2}$$

$$= \frac{0.0374\ M_p\ a^2}{EI_a} + \frac{(49.75\ M_p - 26.4\ M_p)\ a^2}{212\ EI_a}$$

$$= 0.1485\ \frac{M_p\ a^2}{EI_a}$$

From the equation (3.1)

$$y = \frac{1}{49.75\ M_p}\Bigg[\frac{0.0374\ M_p\ a^2}{EI_a} \times 26.4\ M_p + 2 \times 26.4\ M_p \times \left(\frac{0.1485\ M_p a^2}{EI_a} - \frac{0.0374\ M_p a^2}{EI_a}\right) + (49.75\ M_p - 26.4 M_p)\left(\frac{0.1485 M_p a^2}{EI_a} - \frac{0.0374 M_p a^2}{EI_a}\right) - 49.75 \times M_p \times 0.1485\ \frac{M_p a^2}{EI_a}\Bigg]$$

$$= \frac{0.0414\ M_p\ a^2}{EI_a}$$

$$\therefore (y_{el})_E = (y_{el})_2 - y = \frac{0.1071\ M_p a^2}{EI_a}$$

$$\therefore K_E = \frac{R_m}{(y_{el})_E} = \frac{464.52\ EIa}{a^2} \qquad \ldots (3.7)$$

The moment of inertia I_a will be calculated by taking the average of the uncracked and cracked transformed section moment of inertias (8). Thus,

$$I_a = \frac{I_G + I_{cr}}{2} \qquad \ldots (3.8)$$

where

$$I_{cr} = \frac{1}{12} b x_u^3 + bx\ x_u \times \left(\frac{x_u}{2} \right)^2 + m\ A_s\ (d - x_u)^2 \qquad ..\ (3.9)$$

and $$I_G = \frac{1}{12} bD^3 \qquad ...\ (3.10)$$

From the equation (3.5)

$$\frac{x_u}{d} = \frac{0.87x3125.0x0.0016}{0.36 \times 187.5} = 0.06$$

$$\therefore x_u = 10.79 \times 0.06 = 0.65 \text{ cm}$$

$$m = \frac{2800}{3x50} = 19 \quad \text{where} \quad \sigma_{cbc} = 50 \text{ kg/cm}^2$$

Now

$$I_G = \frac{1}{12} \times 1 \times (13)^3 = 183.08 \text{ cm}^4/\text{cm}$$

$$I_{cr} = 33.82 \text{ cm}^4/\text{cm}$$

$$\therefore I_a = 0.5 \times (183.08 + 33.82) = 108.45 \text{ cm}^4/\text{cm}$$

$$E_c = 5700 \sqrt{f_{ck}} x1.25 \text{ N/mm}^2 = 275950.0 \text{ kg/cm}^2$$

From the equation (3.7)

$$K_E = \frac{464.52x275950x108.45}{(5.0x100)^2} = 55606.35 \text{ kg/cm}$$

From the table 3.1

$$K_{LM} = 0.69, \text{ for elastic range}$$

$$K_{LM} = 0.54, \text{ for plastic range}$$

For $\mu = 5$, the value of K_{LM} can be considered as 0.57 (8)

$$T = 2\pi \sqrt{\frac{K_{LM} \times M_t}{K_E}}$$

$$M_t = (5.0x6.0x0.12x2.4x1000)/981$$

$$= 8.81 \text{ kg-sec}^2/\text{cm}$$

$\therefore$ T = 0.06 sec

$\therefore \frac{t_d}{T} = \frac{11.44}{0.06\text{x}1000} = 0.19$

Total weight of the slab

= 5.0x6.0x0.13x2.4x1000

= 9360 kg

$\therefore$ Resistance of slab against blast load only

= 24000 – 9360

= 14640 kg

$\therefore \frac{R_m}{F_1} = \frac{14640}{5.0\text{x}6.0\text{x}10^4\text{x}0.28}$

= 0.18

$\therefore$ From the fig 3.7a

$\frac{y_m}{y_{(el)}} = \mu = 5.0$

Hence O.K.

A.3 <u>Check for shear</u>

From the table 3.1

Dynamic reaction at short edge = V_A = 0.07F + 0.13 R_m

Dynamic reaction at long edge = V_B = 0.1F + 0.2 R_m

Now, it is seen that $V_B > V_A$

V_B = 0.1 (0.28x1.2x5x6x10^4) + 0.2x24000

= 14880 kg

$\tau_{c(cal)} = \frac{14880}{6\text{x}100\text{x}10.8}$

= 2.3 kg/cm^2 < 3.5 kg/cm^2

Hence O.K.

APPENDIX - B

DESIGN OF THE WALL SLAB

B.1 Design calculations

Assuming the footing of wall at a depth of 45 cm below ground level, the effective length of one way slab is taken as —

L_w = 439 cm

Assuming T= 0.03 sec, we get

$$t_d/T = \frac{9.6118}{0.03x1000} = 0.32$$

From the fig. 3.7, for a ductility rario $\mu = 5$ and $t_d/T = 0.32$, the value of R_m/F_1 is equal to 0.29 and $F_1 = p_{ro} = 1.8575\ kg/cm^2$

Using a safety factor of 1.2 (1) we get,

$$F_1 = 1.8575 \times 1.2 = 2.229\ kg/cm^2$$

$$\therefore R_m = 0.29 \times F_1 = 0.29 \times 2.229$$

$$= 0.65\ kg/cm^2$$

Considering 1 cm strip of the slab,

$$M_p = \frac{R_m L_w^2}{8}$$

From the equation (3.6) —

$$\frac{0.65x(435)^2}{8} = 0.0026xd^2x0.87x5187.5 \quad 1-1.005\ \frac{5187.5x0.0026}{187.5}$$

where, $p_s = 0.0026$

$f_y = 5187.5\ kg/cm^2$

$f_{ck} = 187.5\ kg/cm^2$

$\therefore$ d = 37.50 cm

Provide overall depth, $D = 40$ cm

$\therefore$ $d = 38$ cm

From equation (3.10) —

$$I_G = \frac{1}{12} \times 1 \times (40)^3 = 5333.33 \text{ cm}^4/\text{cm}$$

$$x_u = 6.61 \text{ cm}$$

From the equation (3.9)

$$I_{cr} = \frac{1}{12} x1x(6.61)^3 + \frac{(6.61)^3}{4} x1 + 19x0.0026x38(38-6.61)^2$$

$$= 1946.00 \text{ cm}^4$$

$$I_a = 0.5x(5333.33+1946.00) = 3639.67 \text{ cm}^4/\text{cm}$$

From the table 3.2

$$K_E = \frac{384\ EI_a}{5L_w^3} = 912.00 \text{ kg/cm per cm width}$$

$$M_t = (0.40x0.01x4.39x2.4x1000)/981$$

$$= 0.04 \text{ kg-sec}^2/\text{cm per cm width}$$

$$K_{LM} = 0.68$$

$$\therefore T = 2\pi \sqrt{\frac{K_{LM}\ M_t}{K_E}} = 0.03 \text{ sec}$$

which is equal to the assumed value. Hence $\mu = 5$

O.K.

From the fig 3.7(b), $t_d/T = 0.32$, we get

$$t_m/t_d = 2$$

$$\therefore t_m = 9.6118 \times 2 = 19.22 \text{ m sec}$$

B.2 Check for shear

Maximum shear force = dynamic reaction at support

From the table 3.2, the dynamic reaction is given by

$$V = 0.38 R_m + 0.12 F$$

The maximum dynamic reaction will occur when the slab reaches its elastic limit

$$R_m = 0.65x1x439 = 285.35 \text{ kg/cm width}$$

$$y_{(el)} = \frac{2.85.35}{912.00} = 0.3 \text{ cm}$$

$$F = 2.229 \times 439 \times 1 = 978.53$$

$$\omega = 2\pi/T = 0.209 \text{ m-sec}$$

Elastic response of a S.D.F. system subjected to triangular loading is given by

$$y = \frac{F}{K_E}\left[\left\{1-\cos(\omega t)\right\} + \frac{1}{t_d}\left\{\frac{1}{\omega}\sin(\omega t)-t\right\}\right]_{t \leq t_d} \quad ...(3.11)$$

The response at $t = t_d = 9.6118$ m-sec. will be 0.94 cm which is greater than $y_{(el)}$.

The time of the yield response will be calculated from the equation (3.11) by substituting

$$y = y_{(el)} = 0.3 \text{ and } t = t_{(el)}$$

$$\therefore t_{(el)} = 4 \text{ m. sec}$$

$\therefore$ Loading at the time of yield

$$= \frac{2.229 \times 4}{9.6118} = 0.928 \text{ kg/cm}^2$$

$$\therefore F = 0.928 \times 400 \times 1 = 371.2 \text{ kg/cm width}$$

$$\therefore V = 0.38x285.35+0.12x371.2 = 152.98 \text{ kg/cm width}$$

$$\tau_{c(cal)} = \frac{152.98}{1x38} = 4.03 \text{ kg/cm}^2 > 3.5 \text{ kg/cm}^2$$

hence shear reinforcement is necessary.

Strength of shear reinforcement (1)

$$= V_{us} = V_u - \tau_c \text{ bd}$$

$$= \frac{0.87 \; f_y \; A_{sv} \; d}{S_v}$$

$$V_u = V = 152.98 \text{ kg/cm}$$

$$\therefore \quad V_{us} = 152.98 - 3.5 \times 1 \times 38 = 19.98 \text{ kg}$$

$\therefore$ Provide 10 mm ∅ (f_y = 2500x1.25 = 3125 kg/cm^2)

nominal stirrups, with spacing equal to the spacing of horizontal distribution steel of the slab-wall.

APPENDIX-C

DESIGN OF FORCE GIRDER

C.1 Load calculations

From the load-time curve considered in the design of slab, we have

$$F_e = (0.28x5x6.0x10^4)xK_L$$

$$= (84000 \times K_L) \text{ kg}$$

and $K_e = (55606.35 \times K_L)$ kg/cm

The natural frequency of the slab is

$$\omega = 2\pi/T = 0.105 \text{ rad/m.sec}$$

$$y_{(el)} = R_m/K_E = 24000/55606.35 = 0.43 \text{ cm}$$

The deflection of the slab at 11.44 m.sec is given by using the equation (3.11)

$$y_{(11.44)} = 0.72 \text{ cm}$$

The time at which the slab reaches its elastic limit can be obtained by using the equation (3.11).

$$y_{(el)} = \frac{R_m}{K_E} = \frac{F_1}{K_E}\left[(1-\cos\omega\, t_{(el)}) + \frac{1}{t_d}\left(\frac{1}{\omega}\sin(\omega t_{el}) - t_{(el)}\right)\right]$$

$$\text{or } \frac{R_m}{F_1} = \left[\left\{1-\cos\omega t_{(el)}\right\} + \frac{1}{t_d}\left\{\frac{1}{\omega}\sin(\omega t_{(el)}) - t_{(el)}\right\}\right]$$

$$\therefore 0.18 = 1 - \cos(0.105xt_{el}) + \frac{1}{11.44}\left[\frac{1}{0.105}\sin(0.105xt_{el}) - t_{el}\right]$$

$$\therefore t_{(el)} = 6.5 \text{ m.sec}$$

From the table 3.1, the dynamic reaction at the shorter edge of the roof is given by

$$V_A = 0.07F + 0.13\ R_m \quad \text{(plastic)}$$

From the above relation, it is seen that maximum reaction will occur when the slab reaches its elastic limit. The time at which the roof slab reaches its elastic limit, i.e., $t_{(el)}$ is very small in comparison to the expected natural period of the beam. The time required to produce this reaction can be neglected.

$$\text{Max. reaction} = 0.07 \times 84000 + 24000 \times 0.13 \times 2$$
$$= 18000 \text{ kg}$$

With the above idealization, the load time variation for girder can be considered as shown in the fig 3.8a. In the load time variation, the duration of the loading is considered as 34.32 mili-sec which is the time of maximum response of the roof slab. This time of maximum response is calculated. From the design of roof slab, we have

$$t_d/T = 0.19 \quad \text{and} \quad R_m/F_1 = 0.18$$

Using the fig 3.7b, the corresponding value of t_m/t_d is obtained and this value is equal to 3.

$$\therefore \quad t_m = 3 \times 11.44 = 34.32 \text{ m.sec}$$

The girder load F_2 at 11.44 m.sec (which is the duration of roof loading) will be given by the maximum resistance of the two adjacent roof panels only as roof loading at 11.44 m.sec time is zero. Then loading on girder will remain constant upto the time of the maximum response of the roof slab and the loading is assumed to be decreasing suddenly after this time, i.e. 34.32 m. sec. The spanwise distribution of this loading can be assumed as triangular as shown in the fig 3.9a because this is the type of edge reaction developed along the short edge of a two way slab in the plastic range (8). The loading from the roof slab on the girder is considered as shown in the fig 3.9b.

C.2 Design calculations

Load time variation of girder as shown in the fig 3.8a can further be idealized as shown by the dotted line provided the time of maximum response remains within the range of 11.44 m.sec. For a preliminary design this condition is assumed to be true. Assuming natural time period of the girder, T = 0.05 sec , we have

$$\frac{t_d}{T} = \frac{17.51}{0.05x1000} = 0.35$$

From the fig 3.11a, $(D.L.F.)_{max} = 0.95$

Thus, estimated required strength of girder is

Dynamic load	= 18000 x 0.95	= 17100.00
Weight of slab	= 0.13x5.2x2.6x2.4x1000	= 4218.00
	Total	= 21318.00 kg

Considering the triangular live load and dead load as shown in the fig 3.9, let highest ordinate of the loading is p_o

and effective span of the girder is L_b. The total load (L.L. + D.L.) which will act on the mid-span of the girder will be equal to $L_b p_o/2$.

$\therefore$ Reaction at the support $= R_s = L_b p_o/4$

Mid span moment $= \frac{p_o L_b^2}{12}$

If total load acting at midspan $= W_t$

Then $p_o = \frac{2W_t}{L_b}$

$\therefore$ Mid span moment $= \frac{W_t L_b}{6}$ ... (3.12)

$\therefore$ Required moment $= M = \frac{W_t L_b}{6}$

If it is assumed that ISWB 200 will be provided for column then L_b can be considered as (3)

$$L_b = 5.2 - 0.20 = 5.0 \text{ m}$$

We have

$$\frac{f_b}{y} = \frac{M}{I}$$

$$\therefore f_b = M/Z$$

In this case, the beam will be designed such that the maximum stress (bending) in the beam is just equal to its permissible yield stress or less than it.

$$\therefore f_b = \sigma_y$$

and $M_y = \sigma_y \times Z$

where Z is the elastic section modulus about the axis of maximum strength.

Provide steel grade of σ_y $= 2400\ kg/cm^2$

For the blast loading, $\sigma_y = 2400x1.25 = 3000\ kg/cm^2$

The maximum moment developed due to dynamic loading and load from slab

$$= \frac{21318x5x100}{6} = 1776500\ kg\text{-}cm$$

Required $Z = \frac{1776500}{3000} = 592.17\ cm^3$

For a trial solution, let the section provided is ISMB 300 @ 46.1 kg/m; sectional properties of which are as follows

$D = 30\ cm$

$b_f = 14\ cm$, $t_f = 1.31\ cm$

$t_w = 0.77\ cm$

$A_s = 58.7\ cm^2$

$Z_{xx} = 599\ cm^3$

$I_{xx} = 8985\ cm^4$

Self weight of the girder $= 46.1 \times 5 = 230.5$ kg

Total load including self weight of girder

$W_t = 21318 + 230.5 = 21548.5$ kg

$$\therefore\ M = \frac{21548.5x5.0x100}{6} = 1795708.3\ kg\text{-}cm$$

$$\therefore\ \text{Actual required } Z_{xx} = \frac{1795708.3}{3000}$$

$$= 598.56\ cm^3 < 599.0\ cm^3$$

Now, the natural period of the beam and the maximum response will be calculated to check the validity of the idealization made in the load-time variation diagram in the fig 3.8a.

Calculation of K_M and K_L:

For a triangular load distribution, the characteristic shape of the beam is given by

$$\phi(x) = \frac{x}{8L^5}(5L^2 - 4x^2)^2 \qquad (x \leq L/2) \qquad \ldots(3.13)$$

If m_o and p_o are the highest ordinate of the triangular mass and load distribution then —

$$m = m_o \frac{2x}{L}$$

and $$p = p_o \frac{2x}{L}$$

where m and p are the mass and load at any distance x from the support

$$\therefore \quad M_e = \int_0^{L/2} m\, \phi_{(x)}^2\, dx$$

$$= \frac{m_o \times L \cdot 2154}{16 \times 384}$$

$$M_t = \frac{1}{2} m_o L$$

$$\therefore \quad K_M = \frac{M_e}{M_t} = 0.7$$

$$F_e = 2 \int_0^{L/2} p \quad \phi(x)\ dx$$

$$= \frac{17\ p_o\ L}{42}$$

$$F_t = \frac{1}{2} L \, p_o$$

$$\therefore \quad K_L = \frac{F_e}{F_t} = 0.81$$

The stiffness of the girder for triangular loading is given by

$$K = \frac{60 \, EI_b}{L_b^3}$$

For the structural steel, $E = 2.1 \times 10^6$ kg/cm^2 (7)

$$\therefore \quad K = \frac{60 \times 2.1 \times 10^6 \times 8985.0}{(5.0 \times 100)^3} = 9056.88 \text{ kg/cm}$$

Total dead load, W_d = slab weight + self weight

= 4218.00 + 230.5 = 4449 kg

$$\therefore \quad M_t = \frac{4448}{981} \text{ Kg-sec}^2/\text{cm}$$

$$\therefore \quad T = 2\pi \sqrt{\frac{K_M \, M_t}{K_L \, K}} = 0.13 \text{ sec}$$

$$\therefore \quad \frac{t_d}{T} = \frac{17.51}{0.13 \times 1000} = 0.13$$

From the fig 3.11b, for $t_d/T = 0.13$,

$t_m/T = 0.257$

$$\therefore \quad t_m = 0.257 \times 0.13 \times 1000 = 33.4 \text{ m sec} > 11.44 \text{ m.sec}$$

Hence, the idealization of a triangular loading will not be valid.

Therefore, numerical analysis has to be done to find out the maximum response and time of maximum response. In order to do this, the actual girder system has been transformed into an equivalent S.D.F. System where,

$$M_e = K_M M_t = 3173904.2 \text{ Kg-m.sec}^2/\text{cm}$$

$$K_e = K_L \times K_E = 7336.07 \text{ kg/cm}$$

$$F_e(t) = F(t) \times 0.81$$

The equation of motion of the system is given by

$$M_e \ddot{y} + K_e y = F_e(t)$$

(Neglecting damping)

The numerical integration of this equation is carried out by constant velocity method which is simple and takes less time in computation than other methods. By this method displacement at any time station (S_t+1) can be calculated as

$$y^{(S_t+1)} = 2y^{(S_t)} - y^{(S_t-1)} + \ddot{y}^{(S_t)} (\Delta t)^2 \quad \ldots(3.14)$$

Though the equation (3.14) is approximate, it gives sufficiently accurate results provided the time interval considered is small in relation to the variation of acceleration. It has been found (8) that sufficiently accurate results can be obtained for practical purpose if time interval is not higher than one tenth of the natural period of vibration 'T'. Here, time interval considered is 1.144 m. sec only which is much less than T/10 of the system.

$\ddot{y}^{(S_t)}$ in the equation (3.14) can be found out from the equation of motion (as $y^{(S_t)}$, i.e., displacement at previous time station will be known) which is given by

$$\ddot{y}^{(S_t)} = F_e(t) - K_e\, y^{(S_t)} \times \frac{1}{M_e} \qquad \ldots (3.15)$$

The displacement after the first time interval is calculated considering the acceleration to remain constant whithin this time interval. Thus,

$$y^{(1)} = 1/2\, \ddot{y}^{(o)} (\Delta t)^2 \qquad \ldots (3.16)$$

The system is considered to be started at rest but $y^{(o)}$ can be obtained from equation (3.15) as the force is not zero at t=0. After calculating the $y^{(1)}$, the solution can be obtained step by step by using the equation (3.14).

To carry out the numerical solution, a computer programme has been developed which can be used to find out maximum response for both elastic and elasto-plastic SDF system having bi-linear resistance function and system starting at rest.

From the results of the numerical analysis, we have

y_{max} = 1.32 cm

t_m = 46.9 m.sec

C.3 Check for maximum resistance

Maximum moment that can be carried by the beam

$= \sigma_y \times Z_{xx} = 3000 \times 599.0 = 1797000$ kg-cm

From the equation (3.12), the maximum resistance is given by

$$R_m = \frac{6 \times 1797000}{5.0 \times 100} = 21564 \text{ kg}$$

$$y_{(el)} = R_m/K = 21564/9056.88$$

$$= 2.38 \text{ cm} > y_{max}$$

Hence O.K.

Maximum dynamic force $= y_{max} \times K$

$$= 1.32 \times 9056.88 = 11956.00 \text{ kg}$$

Total load on the girder $= 11956 + 4219 + 231.00$

$$= 16406 \text{ kg} < 21564 \text{ kg}$$

Hence O.K.

C.4 <u>Check for shear</u>

Since $\frac{d_w}{t_w} = \frac{27.38}{0.77} = 35.56 < 85$,

Unstiffened web can be provided (7).

Permissible average shear stress for unstiffened web

$$= \tau_{va} = 0.4 \times \sigma_y$$

For blast loading

$$\tau_{va} = 0.4 \times \sigma_y \times 1.25$$

$$= 0.4 \times 3000 \times 1.25 = 1500 \text{ kg/cm}^2$$

From the table (3.2), the dynamic reaction in the elastic range is given by

$$V = 0.39 R + 0.11 F$$

$$R = 11956.00 \text{ kg}$$

Since $t_m > t_d$

$$V = 0.39 R = 11956.00 \times 0.39 = 4663 \text{ kg}$$

Average shear stress $= \frac{V}{t_w \times D} = \frac{4663}{0.77x30}$

$= 220\ kg/cm^2 < 1500\ kg/cm^2$

C.5 Check for deflection

$$\delta = \frac{W_d\ L_b^3}{60\ EI_a}$$

$$= \frac{4449\ x\ (5x100)^3}{60x2.1x10^6x8985}$$

$$\therefore \quad \delta/L_b = \frac{4448\ x\ (500)^2}{60x2.1x10^6x8985}$$

$$= \frac{1}{1018} < \frac{1}{325}$$

Hence O.K.

APPENDIX-D

DESIGN OF FRAME

D.1 Load calculation

From table 3.2, the dynamic reaction of the simply supported slab is given by

$V = 0.38\ R_m + 0.12\ F$

From the design of wall slab, total maximum resistance of wall slab

$R_m = 0.65 x 4.45 x 6.00 x 10^4 = 173550$ kg

$F = 1.8575 x 4.45 x 6.00 x 10^4 = 495952.5$ kg

$\therefore V_{(max)} = 0.38 x 173550 + 0.12 x 495952.5 = 125463.30$ kg

At the end of the duration of front wall loading, i.e, at 9.6118 m. sec,

$V = 0.38 x 173550 = 65949.00$ kg

From these data the idealized load time variation of the frame has been constructed and shown in the fig 3.8b.

In the fig 3.8b, 19.22 m.sec is the time of maximum response of wall slab.

D.2 Analysis and design calculations

Assume R_m/F_1 as 0.05 for the first trial and taking $F_1 = V_{max}$

$\therefore R_m = 0.05 x F_1$

$= 0.05 x 125463.30 = 6273.17$ kg

To ensure the formation of plastic hinge in the column, M_p can be calculated as

$$M_p = R_m \, L_c/4$$

where

L_c = unbraced height of column x 1.2

= effective length

$\therefore M_p = 6273.17 \times 430/4 = 674365.78$ kg-cm

Provide σ_y = = 2200 kg/cm^2

For blast loading σ_y = 2200 x 1.25

= 2750 kg/cm^2

$\therefore$ Required $Z_p = \frac{M_p}{\sigma_y}$ (If the effect of axial load on column is negligible)

$$= \frac{674365.78}{2750} = 245.22 \text{ cm}^3$$

For a trial solution let the column section be ISWB 200 @ 20.8 kg/m, the sectional properties are as follows

Weight	= 21.0 kg/m
A_s	= 36.71 cm^2
D	= 20.0 cm
t_f	= 0.90 cm
d_w	= (20.0 − 0.90 x 2) = 18.20 cm
t_w	= 0.61 cm
b_f	= 14 cm
I_{xx}	= 2624.5 cm^4
Z_{xx}	= 360.8 cm^3

The plastic modulus, Z_p, of the assumed section is 291.174 cm^3

The axial load, P_A, on the column will be the dynamic reaction of the girder plus the dead load from the roof slab and girder including self weight of column.

$$P_A = 4663+(4218/2)+(5.2x46.1/2)+4.02x21+100 = 7105.77 \text{ kg}$$

and the axial load carryigg capacity, P_y, at yield is

$$P_y = A_s \times \sigma_y = 36.71x2750 = 100952.5 \text{ kg}$$

$$\therefore \quad P_A/P_y = 7105.77/100952.5 = 0.05 < 0.15$$

$$M_p = Z_p \times \sigma_y$$

$$\therefore \quad M_p = 291.174 \times 2750 = 800728.5 \text{ kg-cm}$$

$$L_c = 3.58 \times 1.2 = 4.30 \text{ m}$$

Using the factor of safety 1.3 (3), the resistance of the assumed section will be

$$R_m = \frac{4 \times M_p}{1.3\ L_c}$$

$$= \frac{4x800728.5}{1.3 \times 430} = 5729.72 \text{ kg} < 6273.17 \text{ kg}$$

D. 2.1 Numerical Analysis

The following data has been used in the analysis

(1) Stiffness of the frame (K_E):

The stiffness of the frame corresponding to a horizontal load at the top level of the frame will be the stiffness of the equivalent S.D.F. system of the frame. Using the moment of inertia of the girder and moment of inertia considered in the column, the stiffness of the frame by conventional frame analysis method is given by

$$K_E = \frac{20.64 \times EI_c}{L_c^3}$$

$$\therefore \quad K_E = K_e = \frac{20.640 x 2.1 x 10^6 x 2624.5}{(430)^3}$$

$$= 1430.77 \text{ kg/cm}$$

(2) Equivalent Mass (M_e):

From the fig 3.1, displacement at any distance from from base is

$$\delta_x = \frac{\delta}{L_c} x$$

$$\therefore \quad \phi(x) = \frac{x}{L_c}$$

Let

m_1 = mass per unit length at top of beam which includes self weight of beam and load coming from roof slab.

m_2 = mass per unit length of column including wall masses

$$M_e = m_1 \times L_b + 2 \int_0^{L_c} m_2 \, \phi^2(x) \, dx$$

$$M_e = m_1 L_b + 2 \left(\frac{1}{3} m_2 L_c \right)$$

From the above relation the equivalent mass M_e is calculated as follows

Roof slab	– 0.13x6x5.2x2.4x1000	= 9734.40 kg
Roof Girder	– 46.1 x 5.0	= 230.00 kg
Wall slab	– 2 x $\frac{1}{3}$x0.40x4.45x6x2.4x1000	= 17088.00 kg
Columns	– 20.8 x 3.58 x 2 x $\frac{1}{3}$	= 49.50 kg
	Total load	= 27101.90 kg

$$\therefore \quad M_e = \frac{27101.90}{981} = 27.626813 \text{ kg-sec}^2/\text{cm}$$

(3) (i) Maximum Resistance of the section

$$= \frac{4\ Z_p\ \sigma_y}{1.3\text{x}430} = \frac{4\text{x}291.174\text{x}2750}{1.3 \text{ x } 430} = 5729.72 \text{ kg}$$

(ii) Time interval (TINT)

TINT = 1.9224 m.sec

and K_L = 1

From the results of the numerical analysis it is found that

$$y_{(max)} = 10.25 \text{ cm}$$

$$t_m = 324.89 \text{ m.sec}$$

$$= 0.325 \text{ sec}$$

$$\therefore \quad y_{(el)} = \frac{R_m}{K_E} = \frac{5729.72}{1430.77} = 4.0 \text{ cm}$$

$$\therefore \quad \mu = \frac{y_m}{y_{(el)}} = \frac{10.25}{4.0} = 2.56 < 5 \qquad \text{O.K.}$$

$$T = 2\pi\sqrt{\frac{M_e}{K_e}} = 2\pi \text{ x } \sqrt{\frac{27.626813}{1430.77}} = 0.873 \text{ sec}$$

D.4 <u>Check for shear</u>

The maximum shear capacity V_y is given by

$$V_y = 0.55\ A_w = 18.2\text{x}0.61 = 16788.75 \text{ kg}$$

The maximum dynamic force = 5560.34 kg

The maximum shear force at base of column

$$= \frac{5560.34}{2} = 2780.17 \text{ kg} < 16788.75 \text{ kg}$$

Hence O.K.

D.5 Check for stability

For stability, the compressive force should be less than

$$0.33 \frac{\pi^2 E_s I_c}{L_c^2}$$

Compressive force in each column $= P_A = 7105.77$ kg

$$\frac{0.33\,\pi^2 EI_c}{L_c^2} = \frac{0.33 \times \pi^2 \times 2.1 \times 10^6 \times 2624.5}{(430)^2}$$

$$= 97082.88 \text{ kg} > P_A$$

Hence O.K.

APPENDIX - E

DESIGN OF BEAM COLUMN CONNECTION

E.1 Design of cover plate

Design moment = Plastic moment in column

= 800728.5 kg-cm

$\therefore$ The cover plate will be designed for a force of $\frac{800728.5}{(20-0.9)}$ = 40036.43 kg

Assuming 12 mm thick plate, the approximate L/r ratio of the plate = $\frac{300-13.1x2}{12}$

= 22.8

Consider σ_y of steel = 4000 kg/cm^2

By interpolation of data given in the table 5.1 of IS:800-1984 (3), the permissible compressive stress of the plate

= 1.25x232 = 2900 kg/cm^2 (25% increase due to dynamic loading)

Area required = $\frac{40036.43}{2900}$ = 13.8 cm^2

Provide 12 cm wide and 12 mm thick plate.

The plate will be welded to the web of the beam and flange of the column.

Use 6 mm size fillet weld,

permissible shear stress of weld = 1250x1.25 kg/cm^2

Throat thickness = 0.6 $\sqrt{2}$ = 0.424 cm

$\therefore$ Length of weld required = $\frac{40036.43}{1.25x1250x0.424}$ = 74 cm

Provide a 46 cm long plate.

Effective length of weld available

$$= 2\ (30 - 2 \times 1.3 - 0.6 \times 2 + 16 - 0.6 \times 2) = 82 \text{ cm}$$

Hence O.K.

The bottom flange of the beam is connected to the flange of the column by full penetration buttweld.

The force at the junction of beam and column

$$= \frac{800728.5}{30-1.31} = 27909.7 \text{ kg}$$

The force taken by the weld connecting the column flange to the bottom flange of beam

$$= 1.25 \times 1025 \times 2\ 14 \times 0.9$$

$$= 32287.5 \text{ kg} > 27909.7 \text{ kg}. \quad \text{Hence O.K.}$$

A nominal weld size of 5 mm is provided on both side of the web of the column to connect it with the bottom flange of the beam.

E.2 <u>Design of diagonal stiffener</u>

The minimum thickness of the web in the connection (web of beam in this case) which will be sage against shear is given by (7) —

$$t_{min} = \sqrt{3}\ \ Z_{xx}/D^2$$

where Z_{xx} is the elastic modulus of the column whose plastic moment governs the design and D is the overall depth of column section.

$$\therefore\ t_{min} = \sqrt{3} \times 360.8/(20)^2 = 1.56 \text{ cm}$$

Since the thickness of web of beam (1.31 cm) is less than the t_{min} calculated above, diagonal stiffener is necessary.

Thickness of diagonal stiffener is given by (7).

$$t_s = \frac{\sqrt{2}}{b_s} \left[Z_{xx}/D - t_w\ D/2 \right]$$

where b_s is the width of diagonal stiffener.

As the moment is reversible, diagonal stiffners will be provided on both side of the web as shown in the fig 3.15.

Take b_s = 100 mm

∴ Required thickness of each stiffener plate

$$= \frac{\sqrt{2}}{10} \left[360.8/20 - 0.77x20/2 \right] = 1.46 \text{ cm}$$

Provide 100 mm x 16 mm thick plate on each side.

APPENDIX - F

DESIGN OF BASE PLATE AND CONNECTION TO COLUMN

F.1 Design of base plate

Total vertical load = P_A = 7105.77 kg

Moment at the base of column = M_p = 800728.5 kg-cm

Consider M 200 Grade of concrete for the pedestal.

Permissible bearing stress of concrete is given by -

$$\sigma_{bf} = 0.45 \times f_{ck} \times 1.25 = 112.5 \text{ kg/cm}^2$$

$$\sigma_y \text{ of base plate} = 4200 \text{ kg/cm}^2$$

$$= 1.25 \times 4200 \text{ for blast loading.}$$

As the moment is large, anchor bolts will be designed to resist tension due to applied moment.

From the stress and strain diagram (fig. 3.15) the following relationship can be established.

Taking moment about tension bolt

$$C \times j_d = P_A \times a' + M_p$$

or $\frac{1}{2} Nd \times \sigma_{bf} \times b_p \times (d - \frac{Nd}{3}) = P_A \times a' + M_p$

where b_p = width of base plate

N = constant

Simplifying the above equation, we get

$$\frac{1}{2} Nd^2 \sigma_{bf} b_p (1 - \frac{N}{3}) = P_A \times a' + M_p \quad \ldots(3.17)$$

Equating all the vertical force to zero

$$T_t = C - P_A \quad \ldots(3.18)$$

From the strain diagram —

$$\frac{T_t}{A_{tb}\ E_s} = \frac{d-Nd}{Nd} \times \sigma_{bf}/E_c$$

$$\text{or} \quad f_s = \frac{1-N}{N}\ m\ \sigma_{bf} \qquad \ldots(3.19)$$

where f_s = stress (tensile) in anchor bolt

m = modular ratio

Now the design of the base plate can be carried out by trial and error basis first assuming the size of base plate and the position of anchor bolts.

Assume plate size = 30 x 40 cm

distance from center line of anchor holes to the nearest edge of the plate = 5 cm > 3.8 cm (Ref. 3, table 8.2).

Referring to the fig 3.15

a' = 15 cm

d = 35 cm

From the equation (3.17)

$$\frac{1}{2} \times N \times (35)^2 \times 112.5 \times 30\ (1 - \frac{N}{3}) = 7105.77 \times 15 + 800728.5$$

$\therefore N = 0.534$

$\therefore Nd = 35 \times 0.534 = 18.69$ cm

From the equation (3.18)

$$T_t = \frac{1}{2} \times 18.69 \times 112.5 \times 30 - 7105.77 = 24433.61 \text{ kg}$$

From the equation (3.19)

$$f_s = \frac{1 - 0.534}{0.534} \times 14 \times 112.5$$

$$= 1374.44 \text{ kg/cm}^2 < 1500 \text{ kg/cm}^2$$

Area of steel bolt required on one side

$$= 24433.61/1374.44 = 17.78 \text{ cm}^2$$

(permissible tensile stress in turned bolt $\sigma_{tf} = 1200 \times 1.25 = 1500 \text{kg/cm}^2$)

Provide 4 bolts of 25 mm diameter on each side.

Centre to centre distance of bolts $= \frac{30-4.5\times2}{3} = 7$ cm

Required minimum pitch $= 2.5\times2.5 = 6.25$ cm

Hence O.K.

To design the thickness of plate, critical section will be considered at a distance of $\frac{D_p - 0.95 \text{ D}}{2}$ from the edge of the plate

$\therefore$ distance of critical section from

$$\text{edge} = d_c = \frac{40 - 0.95 \times 20}{2} = 10.5 \text{ cm}$$

The compressive stress at critical section

$$= \frac{\sigma_{bf}}{Nd} \times (Nd - d_c)$$

$$= \frac{112.5}{18.69} (18.69 - 10.5) = 49.30 \text{ kg/cm}^2$$

Considering 1 cm width of the base plate

moment at the critical section due to compressive force

$$= \frac{49.30+112.5}{2} \times 10.5 \times 1 \times \frac{49.30+2\times112.5}{49.30 + 112.5} \times \frac{10.5}{3}$$

$= 5040.26$ kg-cm/cm

moment at the critical section due to tension in anchor bolt

$= 24433.61 (10.5 - 5)/30$

$= 4479.53$ kg-cm/cm

Plate will be designed for B.M. $= 5040.26$ kg-cm

Permissible bonding stress of the base plate

$= 1850 \times 1.25 = 2312.5\ kg/cm^2$

The thickness of the base plate is given by

$$\therefore \quad \frac{5040.26}{t_b^2/6} = 2312.5$$

$$\therefore \quad t_b = 3.67\ cm$$

Provide base plate of size 300 x 400 x 38 mm

F.2 <u>Check for shear in the anchor bolt</u>

Permissible shear stress in anchor bolt

$= \tau_{vf} = 1000 \times 1.25 = 1250\ kg/cm^2$

Maximum shear force at base of column = 2669.84 kg

$\therefore$ Shear stress in the bolt $= 2669.84/(\frac{\pi}{4} \times (2.5)^2 \times 8)$

$= 67.98\ kg/cm^2 < 1250\ kg/cm^2$

Again

$$\frac{\tau_{vf,cal}}{\tau_{vf}} + \frac{f_s}{\sigma_{tf}} = \frac{67.98}{1250} + \frac{1374.44}{1500}$$

$= 0.97 < 1.4$ Hence O.K.

F.3 <u>Design of connection of base plate to column</u>

The column will be connected to the base plate by four cleat angles, one in each flange of column and two in the web of column (one in each side of web) fixed by fillet weld as shown in the fig 3.15.

Load carried by each cleat angle in flange $= \frac{800728.5}{20} = 40036.43$ kg

Permissible stress of steel in bearing

$= 0.75 \times 4200 \times 1.25 = 3937.5$ kg/cm^2

Bearing area required $= \frac{40036.43}{3937.5} = 10.17$ cm^2

Thickness of cleat angle required $= \frac{10.17}{30} = 0.34$

where length of each angle is 30 cm

Provide 200 x 100 x 10 mm cleat angle on each flange.

Provide size of filled weld = 8 mm

Effective length of the weld

$= (14-2x0.8) + 2x(20-2x0.8) + 2x(6-2x0.8)$

$= 58.0$ cm

Permissible shear stress of weld $= 1025 \times 1.25$

$= 1281.25$ kg/cm^2

Strength of the weld in shear $= \frac{0.8}{\sqrt{2}} \times 58.0 \times 1281.25$

$= 42037.5$ kg $>$ 40036.43 kg

Hence O.K.

Provide 200 x 100 x 10mm cleat angle of length 14 cm on each side of the web of column.

APPENDIX - G

DESIGN OF FOOTING

G.1 Calculation of base pressure

Assume self wt. of footing = 10% of superimposed load given in the table 3.3.

∴ Total load on footing including self weight

$$= 20336.00 + 0.1 \times 20336.00 = 22370.00 \text{ kg}$$

For the reversible nature of moment at column base, two cases for design moment will have to be considered.

Case I

Design vertical load = 22370.00 kg

Design moment = 10225.43 kg-m

The maximum compressive stress will occur at point A of the footing (fig. 3.16).

Case II

Design vertical load is same as in the Case I.

Design moment = 5789.15 kg-m

The maximum compressive stress will occur at point B of footing (fig. 3.16).

G.1.1 Check for base Pressure

Case I

$$\text{Base pressure} = \frac{22370}{3 \times 2} \pm \frac{10225.43 \times 6}{(2.0) \times (3)^2}$$

$$= 3728.33 \pm 3408.48$$

$$= 7137 \text{ kg/m}^2 \text{ or } 319.85 \text{ kg/m}^2$$

$$< 2 \times 10000 \text{ kg/m}^2$$

Case II

$$\text{Base pressure} = \frac{22370}{3x2} \pm \frac{5789.15}{2x(3)^2}$$

$= 4150\ kg/cm^2$ or $3406.7\ kg/m^2$

Hence O.K.

G.2 Design of slab of the footing

Concrete grade M200

Steel grade of $f_y = 2500\ kg/cm^2$

The pressure distribution along the length of the footing is shown in the fig 3.16 for both the cases.

Case I

Considering 1m width of footing, moment at point C

$$= \frac{7137+4864.64}{2} x1x1x \frac{2x7137+4864.64}{7137+4864.64} x \frac{1}{3}$$

$= 3189.77$ kg-m/m

Case II

Moment at point D

$$= \frac{4150+3827.9}{2} x1.3x1x \frac{2x4150+3827.9}{4150+3827.9} x \frac{1.3}{3}$$

$= 3416.00$ kg-m/m

∴ The Case II will govern the thickness of the footing

using a load factor = 1.2

design moment = 3416.00 x 1.2

= 4099 kg-m

Provide steel ratio = 0.0016

From the equation (3.6)

$$4099 = 0.0016x1xd^2x0.87x(2500x1.25x10^4)\left[1-1.005\ \frac{2500x0.0016}{200x1.25}\right]$$

∴ d = 28.0 cm

Provide total depth = 30 cm

G.3 Check for shear

Maximum shear force $= \frac{4150+3827.9}{2} \times 1.3 \times 1$

$= 5185.64$ kg/m - width

Shear stress $= \frac{5185.64}{1 \times 0.3 \times 100 \times 100} = 1.73 \text{ kg/cm}^2 < 3.6 \text{ kg/cm}^2$

Hence O.K.

Provide the footing of the size 3000 x 2000 x 300 mm with steel ratio of 0.0016 in each direction.

G.4 Footing of R.C.C. wall

The wall carry only vertical load of 4320.00 kg/m. Considering the bearing capacity of soil, a 80 cm wide concrete footing of thickness 30 cm will be sufficient. The details of reinforcement are shown in the fig. 3.13.

APPENDIX-H

DESIGN OF END WALL

H.1 Analysis and design calculations

The same load time function as in the case of front wall will be considered for the end wall also.

Length of shorter edge = a = 439 cm

Length of long edge = b = 560 cm

$\therefore \frac{a}{b} = 0.8$ $\qquad f_y = 3125\ kg/cm^2$

$f_{ck} = 187.5\ kg/cm^2.$

To provide safety against splinters, a minimum thickness of 38 cm is necessary for wall slab. So, first trial solution will be carried out by considering the total thickness of wall as 38 cm.

As mentioned above —

$D = 38$ cm

$\therefore \quad d = 38 - 2.5 = 35.5$ cm

From the equation (3.5), (Assuming $p_s = 0.0025$)

$$x_u = \frac{0.87 \times 3125 \times 0.0025}{0.36 \times 187.5} \times 35.5 = 3.57 \text{ cm}$$

From the equation (3.9)

$$I_{cr} = \frac{1}{12} \times (3.57)^3 + \frac{(3.57)^3}{4} + 19 \times 0.0025 \times 35.5(35.5-3.57)^2$$

$$= 1734.34 \quad cm^4/cm \text{ width}$$

$$I_G = \frac{1}{12} \times 1 \times (38)^3 = 4572.7 \text{ cm}^4/\text{cm width}$$

$$\therefore I_a = (I_{cr} + I_G)/2 = 3153.52 \text{ cm}^4/\text{cm width}$$

From the table 3.4

$$K_E = \frac{212 \text{ EIa}}{a^2}$$

$$\therefore K_E = \frac{212 \times 275950 \times 3153.52}{(439)^2} = 1038543.7 \text{ kg/cm}$$

$$K_{LM} = 0.63$$

$$M_t = 4.39 \times 5.6 \times 0.38 \times 2.4 \times 1000/981$$

$$= 22.85 \text{ kg-sec}^2/\text{cm}$$

$$\therefore T = 2\pi \sqrt{\frac{K_{LM} M_t}{K_E}}$$

$$= 2\pi \sqrt{\frac{0.63 \times 22.85}{1038543.7}} = 0.023 \text{ sec}$$

Ultimate moment capacity per unit width, M_p of the assumed section is obtained by using the equation (3.6)

$$M_p = 0.0025 \times 1 \times (35.5)^2 \times 0.87 \times 3125 \quad 1-1.005 \frac{3125 \times 0.0025}{187.5}$$

$$= 8207 \text{ kg-cm/cm width}$$

From the table 3.4 for a value of a/b = 0.8, we have

$$R_m = \frac{1}{a} (12 M_{pfa} + 10.3 M_{pfb})$$

If the resistance at all points of slab is considered equal, the above equation can be written as

$$R_m = \frac{1}{a} (12 M_p \times a + 10.3 M_p \times b)$$

∴ Maximum resistance of the section considered, will be

$$R_m = \frac{1}{439} \quad 12x8207.0x439+10.3x8207.0x560$$

$$= 206315.38 \text{ kg}$$

$$\therefore R_m/F_1 = \frac{206315.38}{1.8575x439x560x1.2} = 0.38$$

$$t_d/T = 9.6118/0.023 \times 1000 = 0.42$$

∴ From the fig 3.7a we have

$$\mu = 5$$

Hence O.K.

H.2 <u>Check for shear</u>

From the table 3.4

$$V_A = 0.07F + 0.13 R_m$$

$$V_B = 0.10F + 0.20 R_m$$

Neglecting decay in overpressure,

$$V_B = 0.10x1.8575x439x560x1.2+0.20x206315.38$$

$$= 96060.81 \text{ kg}$$

$$\tau_{c(cal)} = \frac{96060.81}{560 \times 35.5}$$

$$= 5.0 \text{ kg/cm}^2 > 3.5 \text{ kg/cm}^2$$

Hence the shear reinforcement is necessary

TABLE 2.1 BLAST PARAMETERS FROM GROUND BURST OF 1 TONNE EXPLOSIVE

DISTANCE, m x	PEAK SIDE ON OVER-PRESSURE RATIO p_{so}/p_a	MACH No. M	POSITIVE PHASE DURATION t_o, milli-secs	DURATION OF EQUIVALENT TRIANGULAR PULSE t_d, milli-secs	DYNAMIC PRESSURE RATIO q_o/p_a	PEAK REFLECTED OVERPRESSURE RATIO p_{ro}/p_a
(1)	(2)	(3)	(4)	(5)	(6)	(7)
15	8·00	2·80	9·50	5·39	10·667	41·60
18	5·00	2·30	11·00	7·18	5·208	22·50
21	3·30	1·96	16·38	9·33	2·643	12·94
24	2·40	1·75	18·65	11·22	1·532	8·48
27	1·80	1·60	20·92	13·30	0·920	5·81
30	1·40	1·48	22·93	15·39	0·583	4·20
33	1·20	1·42	24·95	16·31	0·439	3·45
36	1·00	1·36	26·71	17·94	0·312	2·75
39	0·86	1·32	28·22	19·20	0·235	2·28
42	0·76	1·28	29·74	20·20	0·186	1·97
45	0·66	1·25	31·25	21·60	0·142	1·66
48	0·59	1·23	32·26	22·70	0·115	1·46
51	0·53	1·20	33·52	23·70	0·093	1·28
54	0·48	1·19	34·52	24·70	0·077	1·14
57	0·43	1·17	35·53	26·40	0·062	1·01
60	0·40	1·16	36·29	26·60	0·054	0·93
63	0·37	1·15	37·30	27·80	0·046	0·85
66	0·34	1·14	38·05	28·76	0·039	0·77
69	0·32	1·13	38·81	29·25	0·035	0·72
72	0·30	1·12	39·56	29·87	0·031	0·67
75	0·28	1·11	40·32	30·71	0·027	0·62
78	0·26	1·104	40·82	31·85	0·023	0·58
81	0·25	1·100	41·58	31·92	0·022	0·55
84	0·24	1·098	42·34	32·00	0·020	0·53
87	0·23	1·095	42·84	32·26	0·018	0·50
90	0·22	1·086	43·60	33·39	0·016	0·47
93	0·20	1·082	44·35	34·70	0·014	0·43
96	0·19	1·077	45·46	35·37	0·013	0·41
99	0·18	1·072	45·61	36·22	0·012	0·40

x Taken from IS:4991–1968 (2)

TABLE - 2.2

BLAST PARAMETERS ALONG THE HEIGHT (FRONT WALL) OF THE INDUSTRIAL TYPE BUILDING

DISTANCE FROM GROUND ZERO (m)	p_{so} (kg/cm^2)	MAC No.	t_o (mili-sec)	t_d (mili-sec)	p_d (kg/cm^2)	p_{ro} (kg/cm^2)
20.00	0.724	1.269	14.058	9.612	0.170	1.858
20.40	0.695	1.261	14.260	9.799	0.157	1.729
21.54	0.627	1.241	14.725	10.265	0.129	1.566
29.12	0.373	1.151	17.272	12.855	0.047	0.857
30.46	0.344	1.141	17.618	13.293	0.040	0.780
32.25	0.317	1.128	18.07	13.623	0.034	0.712

TABLE - 2.3

BLAST PARAMETERS ALONG THE ROOF OF INDUSTRIAL TYPE BUILDING (HEIGHT = 8 m)

DISTANCE FROM GROUND ZERO (m)	p_{so} (Kg/cm^2)	MAC No.	t_o (mili-sec)	t_d (mili-sec)	p_d (Kg/cm^2)
21.54	0.627	1.241	14.725	10.265	0.129
25.30	0.471	1.187	16.102	11.598	0.074
29.12	0.373	1.151	17.272	12.855	0.047
52.61	0.132	1.048	21.503	18.698	0.007
56.57	0.104	1.034	21.701	19.820	0.004
60.53	0.075	1.020	21.899	20.942	0.002

Note: Direction of Blast Wave Propagation Perpendicular to y-y axis

TABLE - 2.4

BLAST PARAMETERS ALONG THE ROOF THE INDUSTRIAL TYPE BUILDING (HEIGHT = 16 m)

DISTANCE FROM GROUND ZERO (m)	p_{so} (kg/cm^2)	MAC No.	t_o (mili-sec)	t_d (kg/cm^2)	p_d (kg/cm^2)
32.25	0.317	1.128	18.070	13.623	0.034
35.78	0.266	1.106	18.876	14.622	0.024
39.40	0.237	1.097	19.721	14.889	0.019
43.08	0.201	1.082	20.564	16.068	0.014
46.82	0.174	1.069	21.214	17.058	0.011
50.60	0.147	1.055	21.403	18.129	0.008
54.41	0.119	1.042	21.593	19.208	0.006

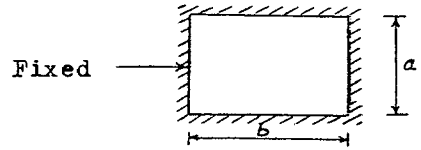
Note: Direction of Blast Wave Propagation Perpendicular to y-y axis.

TABLE 2.5 DRAG COEFFICIENT C_d

SL No.	SHAPE OF ELEMENT	DRAG COEFFICIENT C_d	REMARKS
(1)	(2)	(3)	(4)
	For Closed Rectangular Structures		
i)	Front vertical face	1·0	For above ground structures
ii)	Roof, rear and side faces for		
	q_0 = 0 to 1·8 kg/cm²	−0·4	
	q_0 = 1·8 to 3·5 kg/cm²	−0·3	
	q_0 = 3·5 to 9·0 kg/cm²	−0·2	
iii)	Front face sloping		
	4 to 1	Zero	For semi-buried structures
	1½ to 1	0·4	
	For Open, Drag Type Structures		
iv)	Sphere	0·1	—
v)	Cylinder	1·2	This covers steel tubes used as columns, truss members, etc
vi)	Structural shapes	2·0	This covers flats, angles, tees, I sections, etc
vii)	Rectangular projection	1·3	This covers beam projections below or above slabs, cantilever walls standing freely above ground, etc

* Taken from IS:4991-1968 (2)

Table 3.1 Transformation Factors for Two-way Slabs: Fixed Four Sides, Uniform Load

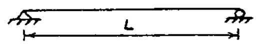


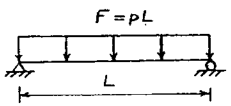
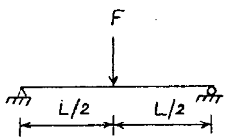
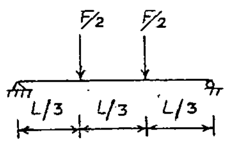
V_A = total dynamic reaction along short edge,
V_B = total dynamic reaction along long edge

Strain range	a/b	Load factor K_L	Mass factor K_M	Load-mass factor K_{LM}	Maximum resistance	Spring constant k	Dynamic reactions V_A	Dynamic reactions V_B
Elastic	1.0	0.33	0.21	0.63	$29.2M^{o}_{psb}$	$810EI_a/a^2$	0.10F+0.15R	0.10F+0.15R
	0.9	0.34	0.23	0.68	$27.4M^{o}_{psb}$	$742EI_a/a^2$	0.09F+0.14R	0.10F+0.17R
	0.8	0.36	0.25	0.69	$26.4M^{o}_{psb}$	$705EI_a/a^2$	0.08F+0.12R	0.11F+0.19R
	0.7	0.38	0.27	0.71	$26.2M^{o}_{psb}$	$692EI_a/a^2$	0.07F+0.11R	0.11F+0.21R
	0.6	0.41	0.29	0.71	$27.3M^{o}_{psb}$	$724EI_a/a^2$	0.06F+0.09R	0.12F+0.23R
	0.5	0.43	0.31	0.72	$30.2M^{o}_{psb}$	$806EI_a/a^2$	0.05F+0.08R	0.12F+0.25R
Elastic-Plastic	1.0	0.46	0.31	0.67	$(1/a)\ 12(M_{pfa}+M_{psa})+12(M_{pfb}+M_{psb})$	$252EI_a/a^2$	0.07F+0.18R	0.07F+0.18R
	0.9	0.47	0.33	0.70	$(1/a)\ 12(M_{pfa}+M_{psa})+11(M_{pfb}+M_{psb})$	$230EI_a/a^2$	0.06F+0.16R	0.08F+0.20R
	0.8	0.49	0.35	0.71	$(1/a)\ 12(M_{pfa}+M_{psa})+10.3(M_{pfb}+M_{psb})$	$212EI_a/a^2$	0.06F+0.14R	0.08F+0.22R
	0.7	0.51	0.37	0.73	$(1/a)\ 12(M_{pfa}+M_{psa})+9.8(M_{pfb}+M_{psb})$	$201EI_a/a^2$	0.05F+0.13R	0.08F+0.24R
	0.6	0.53	0.39	0.74	$(1/a)\ 12(M_{pfa}+M_{psa})+9.3(M_{pfb}+M_{psb})$	$197EI_a/a^2$	0.04F+0.11R	0.09F+0.26R
	0.5	0.55	0.41	0.75	$)1/a)\ 12(M_{pfa}+M_{psa})+9.0(M_{pfb}+M_{psb})$	$201EI_a/a^2$	0.04F+0.09R	0.09F+0.28R
Plastic	1.0	0.33	0.17	0.51	$(1/a)\ 12(M_{pfa}+M_{psa})+12(M_{pfb}+M_{psb})$	0	$0.09F+0.16R_m$	$0.09F+0.16R_m$
	0.9	0.35	0.18	0.51	$(1/a)\ 12(M_{pfa}+M_{psa})+11(M_{pfb}+M_{psb})$	0	$0.08F+0.15R_m$	$0.09F+0.18R_m$
	0.8	0.37	0.20	0.54	$(1/a)\ 12(M_{pfa}+M_{psa})+10.3(M_{pfb}+M_{psb})$	0	$0.07F+0.13R_m$	$0.10F+0.20R_m$
	0.7	0.38	0.22	0.58	$(1/a)\ 12(M_{pfa}+M_{psa})+9.8(M_{pfb}+M_{psb})$	0	$0.06F+0.12R_m$	$0.10F+0.22R_m$
	0.6	0.40	0.23	0.58	$(1/a)\ 12(M_{pfa}+M_{psa})+9.3(M_{pfb}+M_{psb})$	0	$0.05F+0.10R_m$	$0.10F+0.25R_m$
	0.5	0.42	0.25	0.59	$(1/a)\ 12(M_{pfa}+M_{psa})+9.0(M_{pfb}+M_{psb})$	0	$0.04F+0.08R_m$	$0.11F+0.27R_m$

* Taken from "Introduction To Structural Dynamics" by John M. Biggs (8)

Table 3.2 Transformation Factors for Beams and One-way Slabs


Simply-supported

Loading diagram	Strain range	Load factor K_L	Mass factor K_M: Concentrated mass*	Mass factor K_M: Uniform mass	Load-mass factor K_{LM}: Concentrated mass*	Load-mass factor K_{LM}: Uniform mass	Maximum resistance R_m	Spring constant k	Dynamic reaction V
	Elastic	0.64	...	0.50	...	0.78	$\frac{8M_P}{L}$	$\frac{384EI}{5L^3}$	$0.39R+0.11F$
	Plastic	0.50	...	0.33	...	0.66	$\frac{8M_P}{L}$	0	$0.38R_m+0.12F$
	Elastic	1.0	1.0	0.49	1.0	0.49	$\frac{4M_P}{L}$	$\frac{48EI}{L^3}$	$0.78R-0.28F$
	Plastic	1.0	1.0	0.33	1.0	0.33	$\frac{4M_P}{L}$	0	$0.75R_m-0.25F$
	Elastic	0.87	0.76	0.52	0.87	0.60	$\frac{6M_P}{L}$	$\frac{56.4EI}{L^3}$	$0.62R-0.12F$
	Plastic	1.0	1.0	0.56	1.0	0.56	$\frac{6M_P}{L}$	0	$0.52R_m-0.02F$

* Equal parts of the concentrated mass are lumped at each concentrated load

x Taken from "Introduction To Structural Dynamics" by John M. Biggs (8)

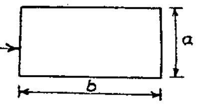
TABLE 3.3 LOADS AND MOMENTS ON COLUMN FOOTING

No.	PARTICULARS	VERTICAL LOAD (kg)	DISTANCE FROM C.G. OF FOOTING (m)	MOMENT (Kg-m)
1.	Vertical load from column	7105.77	–	–
2.	Vertical load from wall = 4320x2.0	8640.00	(–) 0.3	(–) 2592.00
3.	Load due to concrete filling —			
	(i) $\frac{1}{2}$x1.1x0.45x2x2200	1089.00	(+) 0.77	(+) 838.53
	(ii) 0.5x0.45x2x2200	990.00	(+) 0.15	(+) 148.50
4.	Load due to earth on footing —			
	(i) 1.0x2x0.45x1800	1620.00	(–) 1.0	(–) 1620.00
	(ii) $\frac{1}{2}$ x1.1x0.45x2x1800	891.00	(+) 1.13	(+) 1006.83
5.	Moment at base of column	–	–	(±) 8007.29
	Total	20336.0	–	(–)10225.43 (+) 5789.15

Table 3.4 Transformation Factors for Two-way Slabs: Simple Supports – Four Sides, Uniform Load

V_A = total dynamic reaction along short edge, V_B = total dynamic reaction along long edge

Simple support



Strain range	a/b	Load factor K_L	Mass factor K_M	Load-mass factor K_{LM}	Maximum resistance	Spring constant k	Dynamic reactions V_A	Dynamic reactions V_B
ELASTIC	1.0	0.45	0.31	0.68	$\frac{12}{a}(M_{pfa} + M_{pfb})$	$\frac{252EI_a}{a^2}$	0.07F+0.18R	0.07F+0.18R
	0.9	0.47	0.33	0.70	$\frac{1}{a}(12M_{pfa}+11M_{pfb})$	$\frac{230EI_a}{a^2}$	0.06F+0.16R	0.08F+0.20R
	0.8	0.49	0.35	0.71	$\frac{1}{a}(12M_{pfa}+10.3M_{pfb})$	$\frac{212EI_a}{a^2}$	0.06F+0.14R	0.08F+0.22R
	0.7	0.51	0.37	0.73	$\frac{1}{a}(12M_{pfa}+9.8M_{pfb})$	$\frac{201EI_a}{a^2}$	0.05F+0.13R	0.08F+0.24R
	0.6	0.53	0.39	0.74	$\frac{1}{a}(12M_{pfa}+9.3M_{pfb})$	$\frac{197EI_a}{a^2}$	0.04F+0.11R	0.09F+0.26R
	0.5	0.55	0.41	0.75	$\frac{1}{a}(12M_{pfa}+9.0M_{pfb})$	$\frac{201EI_a}{a^2}$	0.04F+0.09R	0.09F+0.28R
PLASTIC	1.0	0.33	0.17	0.51	$\frac{12}{a}(M_{pfa}+M_{pfb})$	0	$0.09F+0.16R_m$	$0.09F+0.16R_m$
	0.9	0.35	0.18	0.51	$\frac{1}{a}(12M_{pfa}+11M_{pfb})$	0	$0.08F+0.15R_m$	$0.09F+0.18R_m$
	0.8	0.37	0.20	0.54	$\frac{1}{a}(12M_{pfa}+10.3M_{pfb})$	0	$0.07F+0.13R_m$	$0.10F+0.20R_m$
	0.7	0.38	0.22	0.58	$\frac{1}{a}(12M_{pfa}+9.8M_{pfb})$	0	$0.06F+0.12R_m$	$0.10F+0.22R_m$
	0.6	0.40	0.23	0.58	$\frac{1}{a}(12M_{pfa}+9.3M_{pfb})$	0	$0.05F+0.10R_m$	$0.10F+0.25R_m$
	0.5	0.42	0.25	0.59	$\frac{1}{a}(12M_{pfa}+9.0M_{pfb})$	0	$0.04F+0.08R_m$	$0.11F+0.27R_m$

x Taken from "Introduction To Structural Dynamics" by John M. Biggs (8)

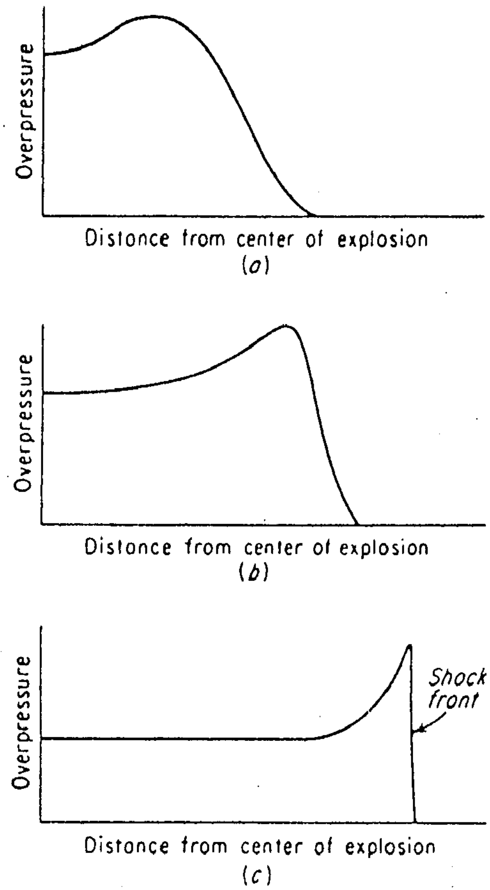


FIG. 2.1 - OVERPRESSURE DISTRIBUTION IN EARLY STAGES OF SHOCK-FRONT FORMATION

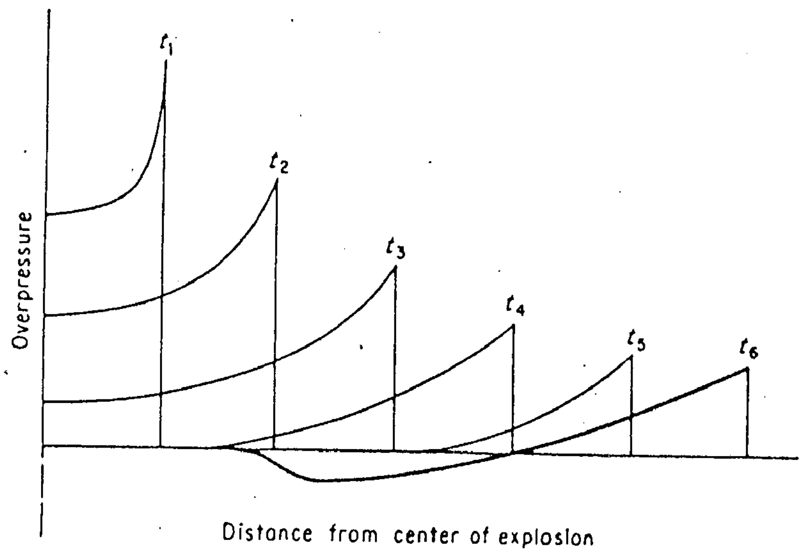


FIG. 2.2 - VARIATION OF OVERPRESSURE WITH DISTANCE FROM CENTER OF EXPLOSION AT VARIOUS TIMES

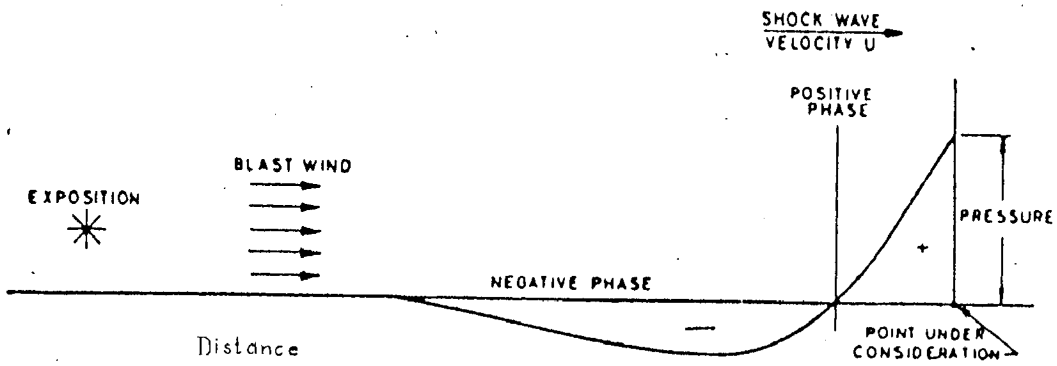


FIG. 2.3 VARIATION OF OVERPRESSURE WITH DISTANCE AT A GIVEN TIME

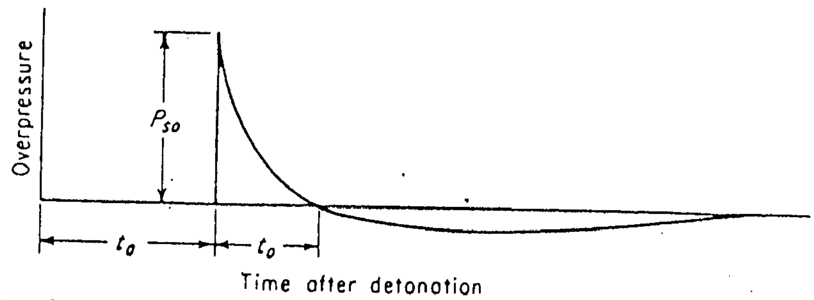


FIG. 2.4 - VARIATION OF OVERPRESSURE WITH TIME AT A GIVEN LOCATION

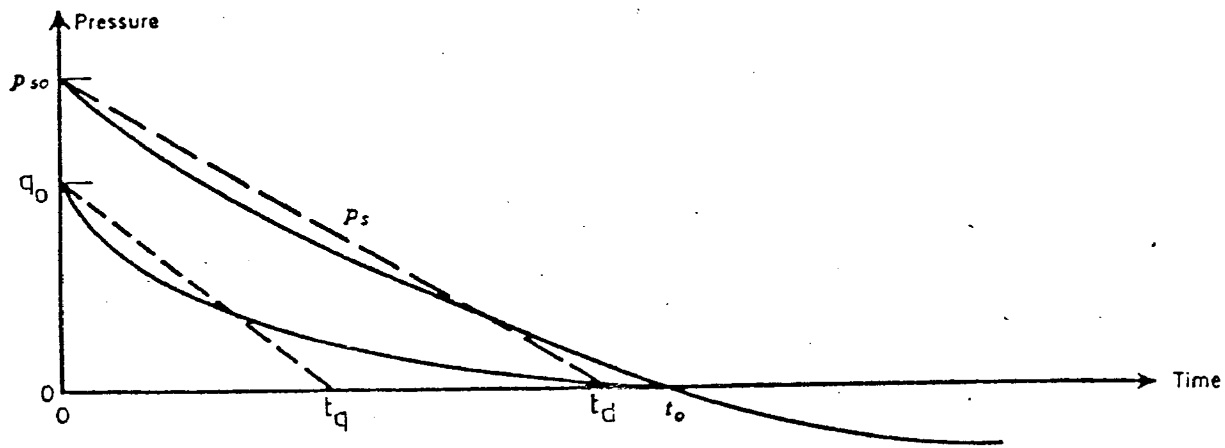


FIG. 2.5 IDEALIZED OVERPRESSURE AND DRAG PRESSURE VARIATION WITH TIME

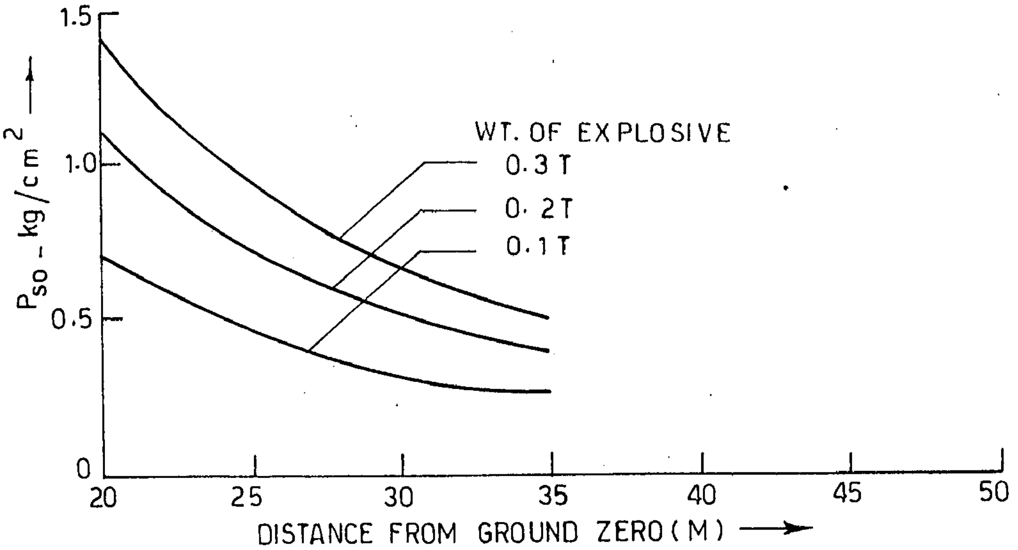


FIG. 2.6 _ P_{SO} VS DISTANCE

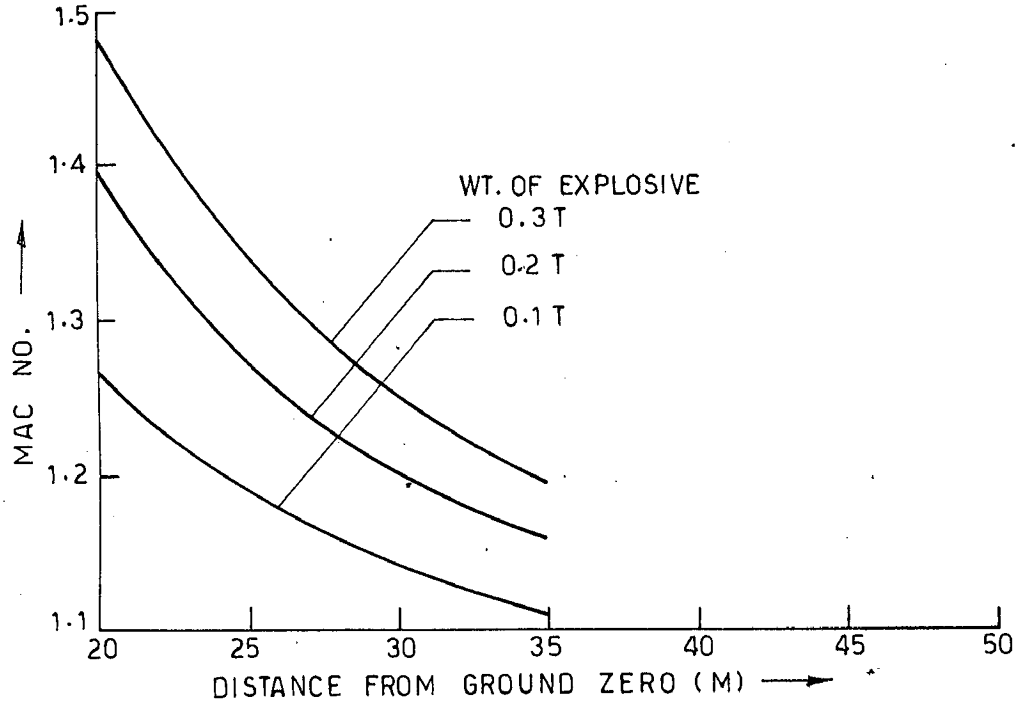


FIG. 2.7 _ MAC NO. VS DISTANCE

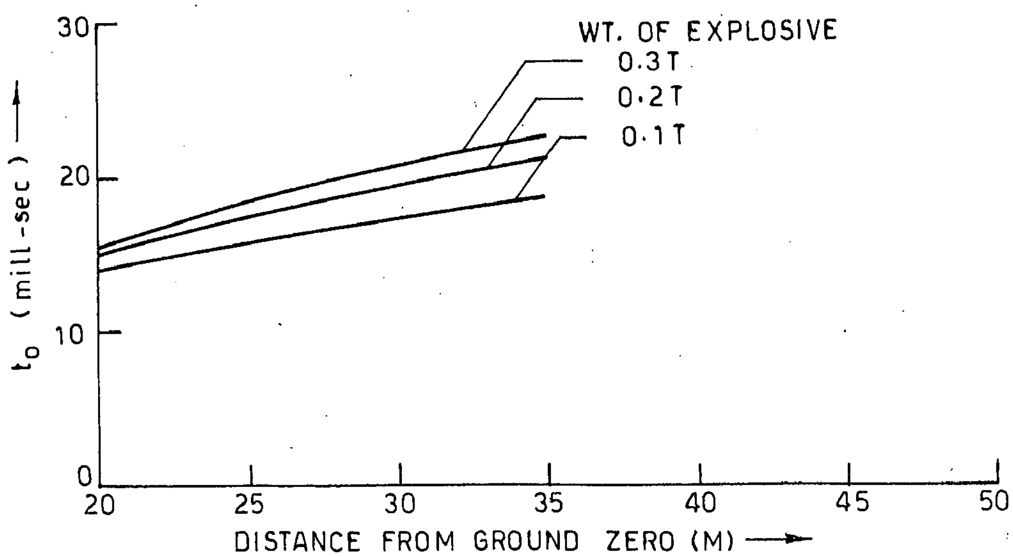


FIG. 2.8 _ t_o VS DISTANCE

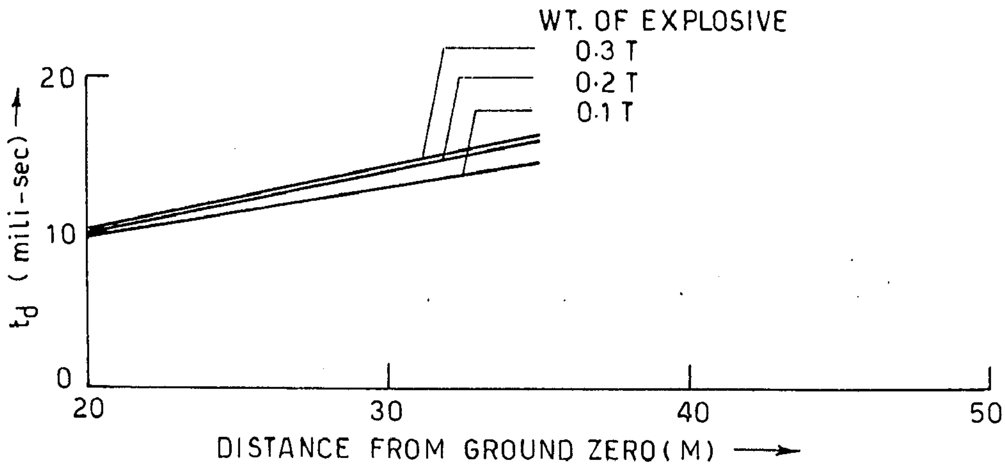


FIG. 2.9 _ t_d VS DISTANCE

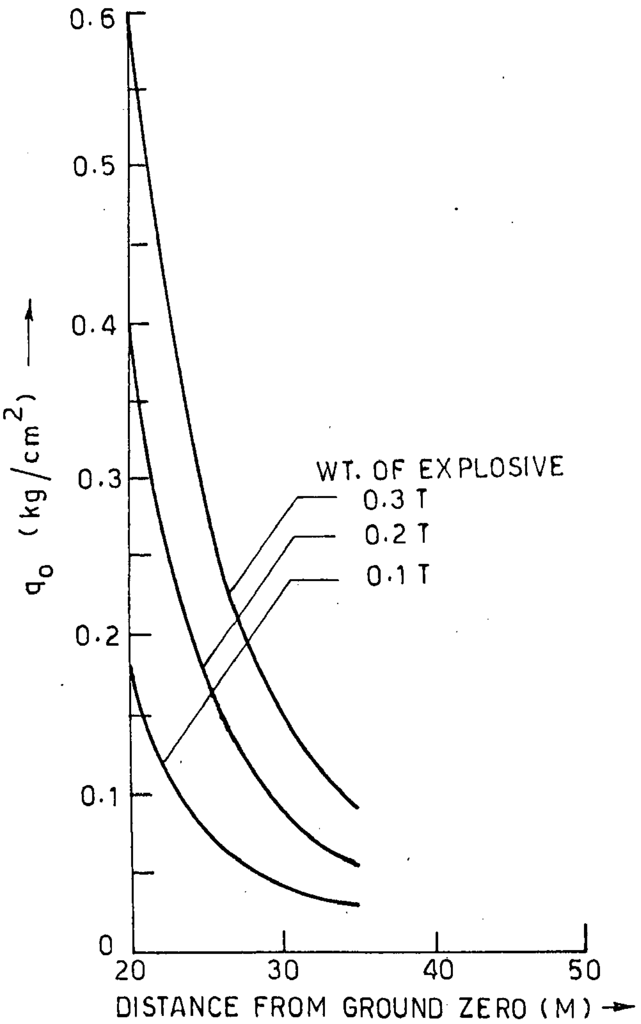


FIG.2.10_ q_o VS DISTANCE

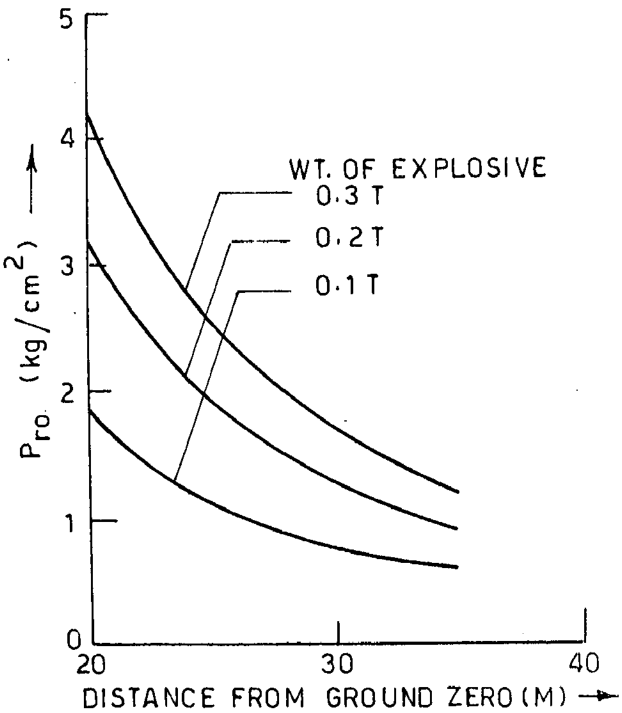
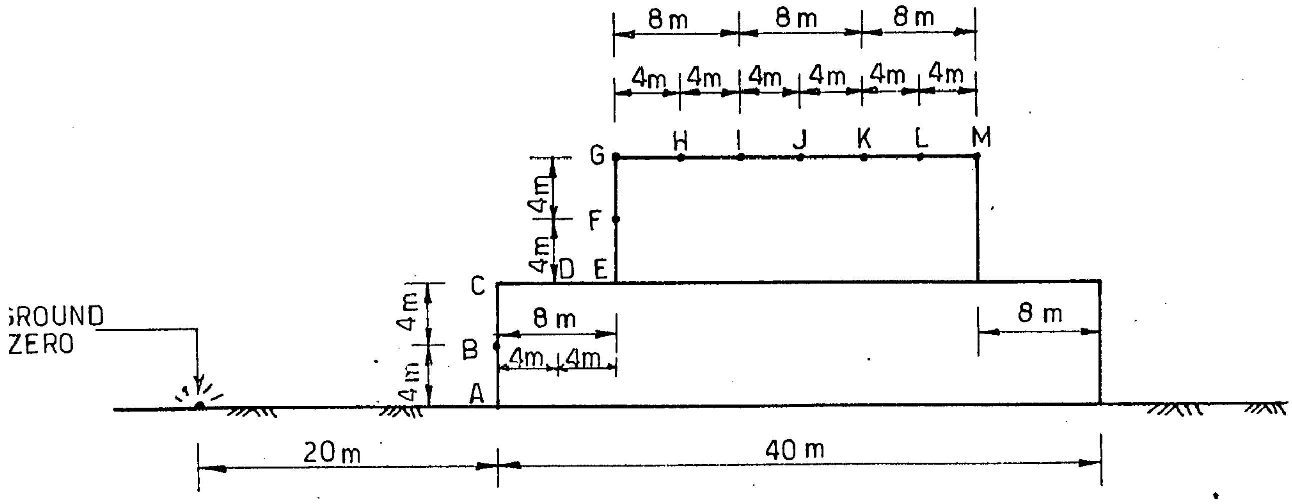
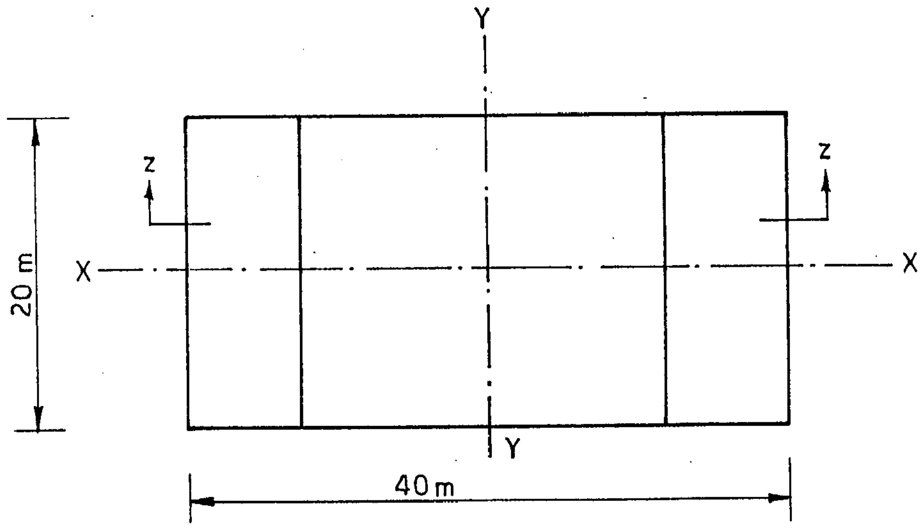


FIG.2.11 _ P_{ro} VS DISTANCE



(b) SECTION: Z-Z



(a) PLAN

FIG.2.12_INDUSTRIAL TYPE BUILDING (SHOWING THE POINTS CONSIDERD FOR BLAST PARAMETER CALCULATION)

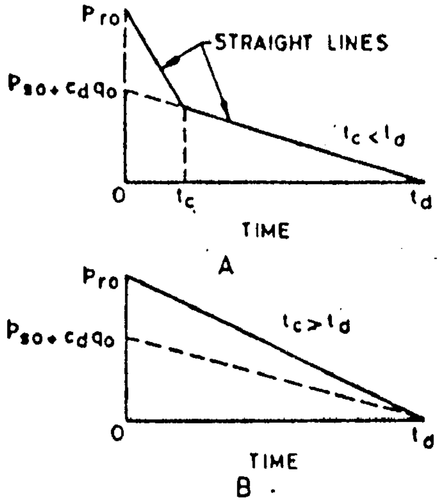


FIG. 2.13. PRESSURE VS TIME FOR FRONT FACE

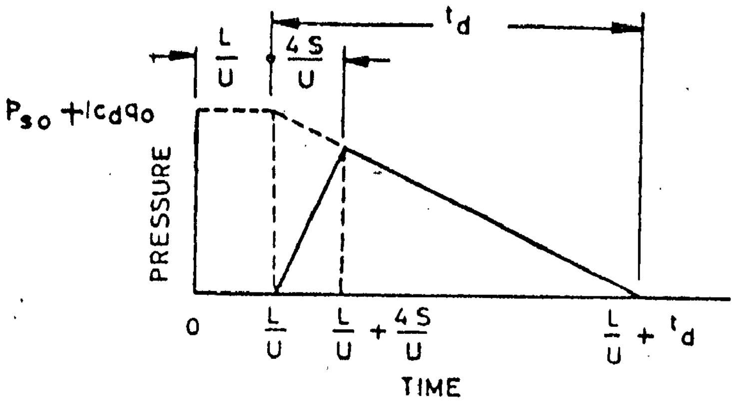


FIG. 2.14 PRESSURE VS TIME FOR REAR FACE

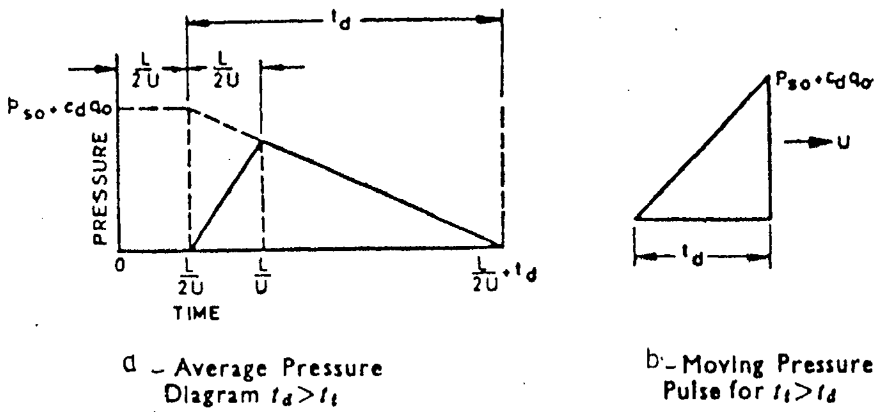
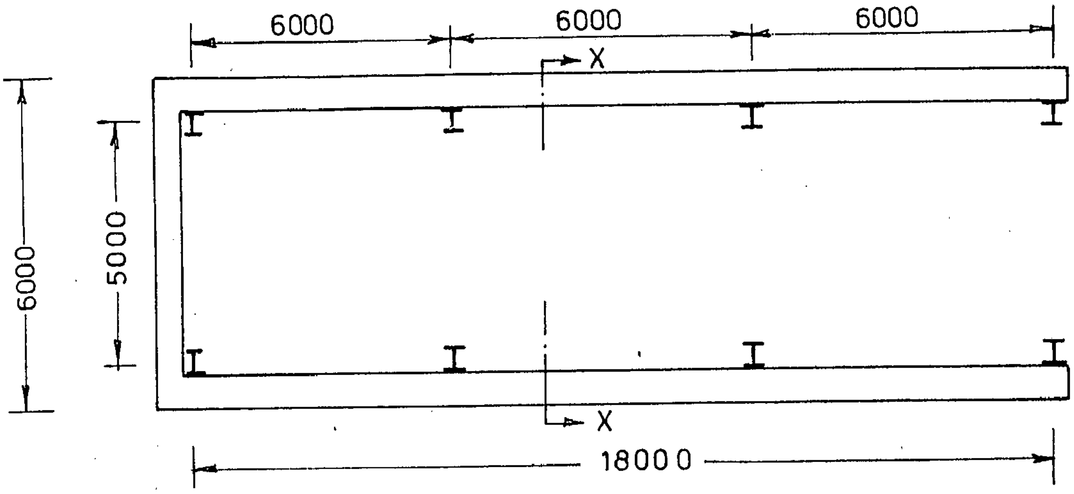
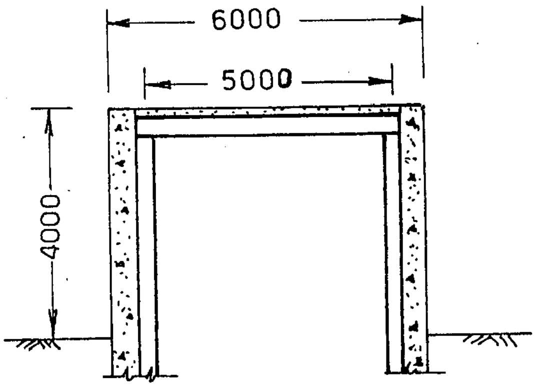


FIG. 2.15. PRESSURE VS TIME FOR ROOF AND SIDE WALLS



PLAN



SECTION: X-X

ALL DIMENSIONS ARE IN mm

FIG.3.1 _ GENERAL ARRANGEMENT OF THE SHELTER

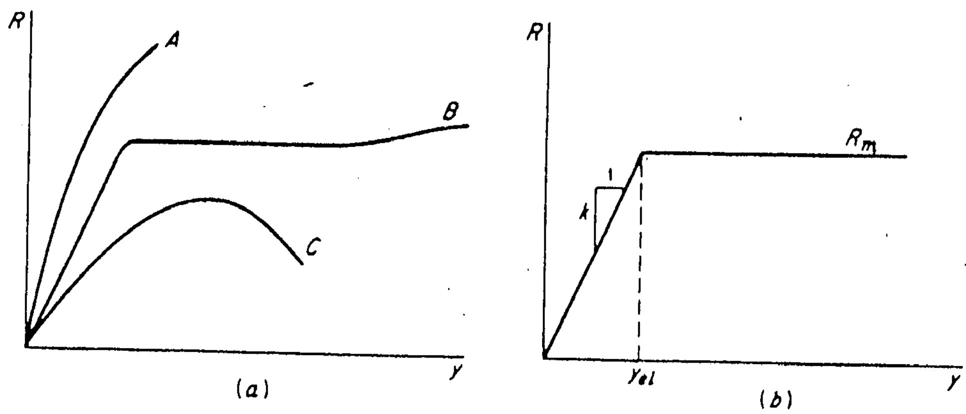


FIG 3.2 _ RESISTANCE FUNCTIONS

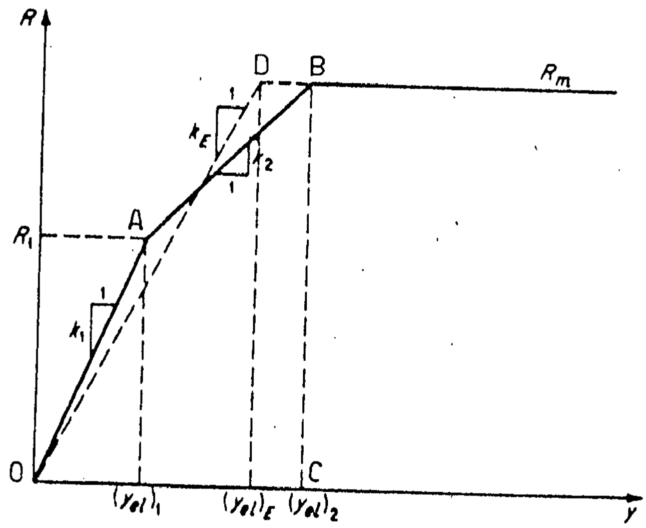


FIG 3.3 _ EFFECTIVE BILINEAR RESISTANCE FUNCTION

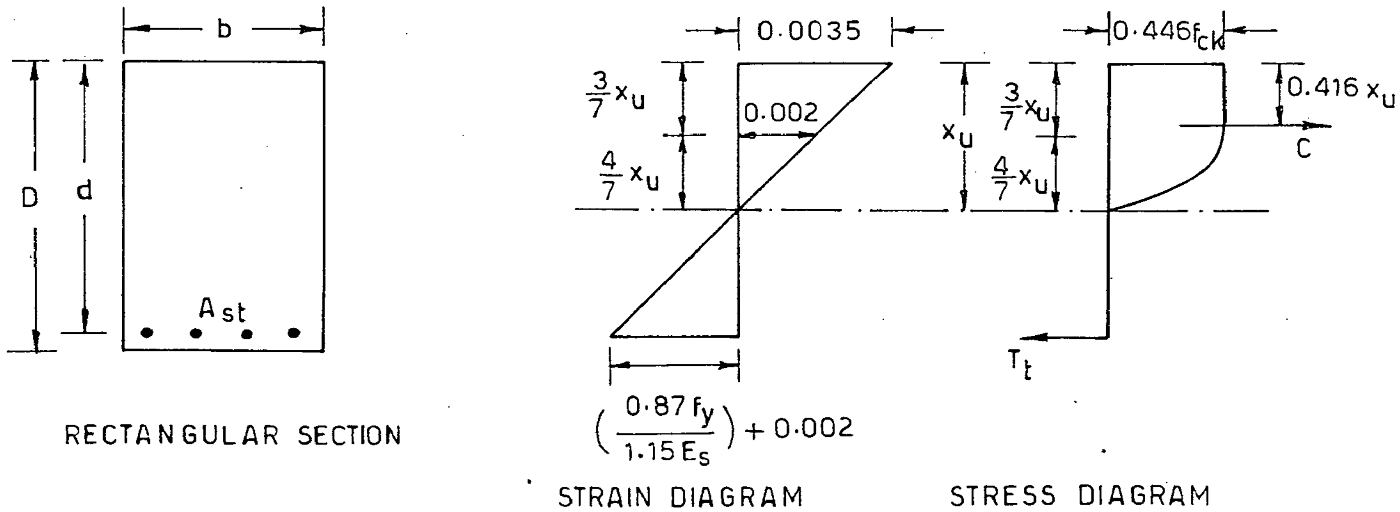
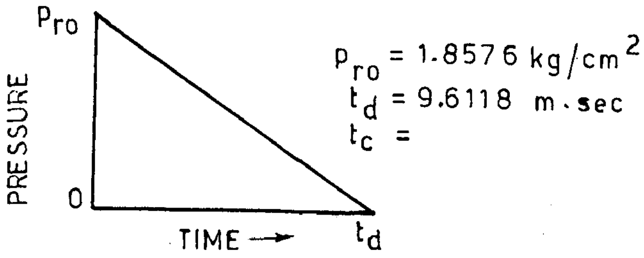
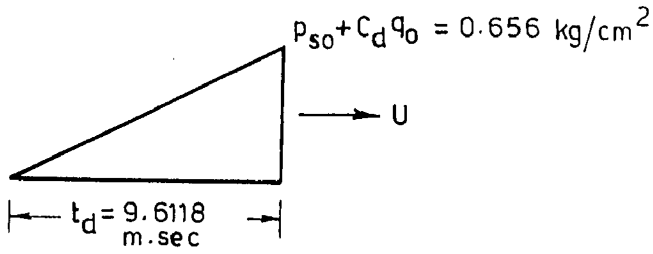


FIG. 3.4 _ STRESS STRAIN DIAGRAM

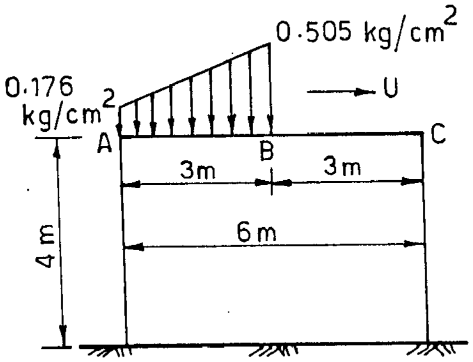


(a) DESIGN AVERAGE FRONT FACE LOADING

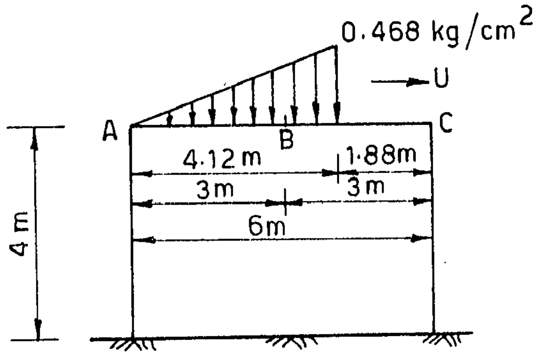


(b) DESIGN MOVING PULSE ON ROOF

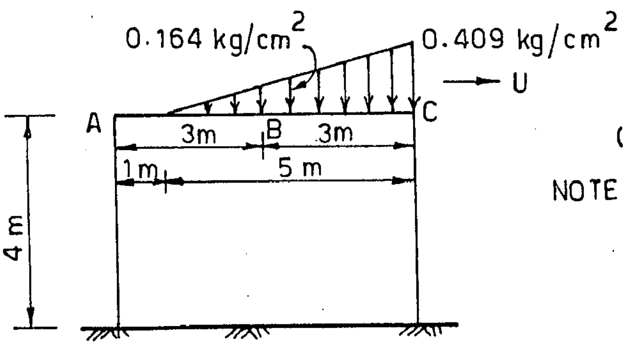
FIG.3.5 _ DESIGN AVERAGE FRONT FACE LOADING AND MOVING PULSE ON ROOF



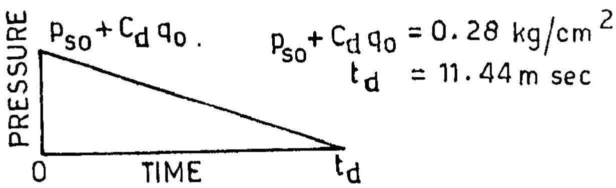
(a) PR. DISTRIBUTION AT $t = 7.067$ m.sec



(b) PR. DISTRIBUTION AT $t = 9.798$ m.sec



(c) PR. DISTRIBUTION AT $t = 14.402$ m.sec



(d) DESIGN AVERAGE ROOF LOADING

NOTE: THE INSTANT AT WHICH THE SHOCK FRONT FIRST REACHES AT POINT A IS CONSIDERED AS REFERENCE TIME ($t = 0$)

FIG. 3.6 _ PR. DISTRIBUTION ON ROOF

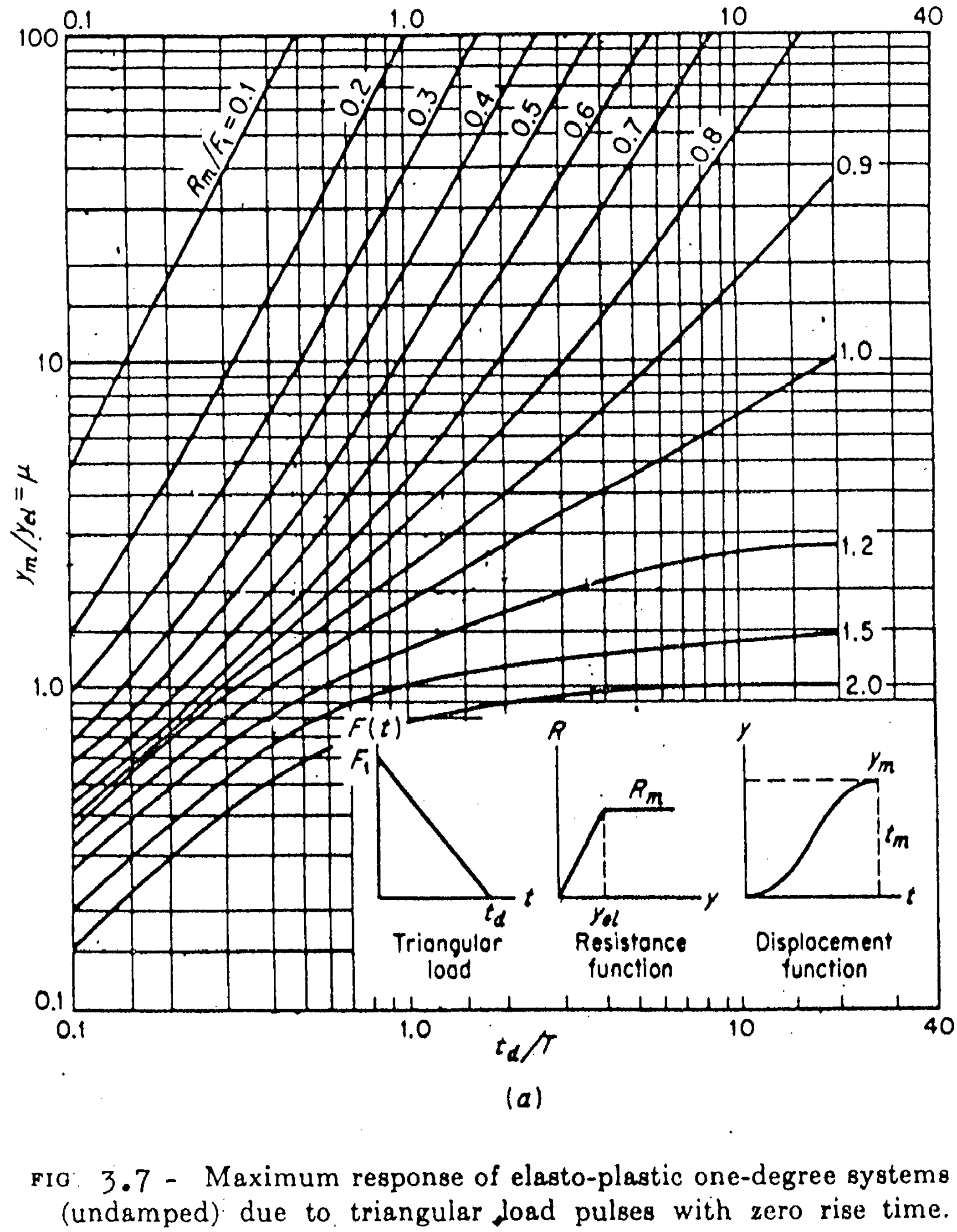


FIG. 3.7 - Maximum response of elasto-plastic one-degree systems (undamped) due to triangular load pulses with zero rise time.

x Taken from "Introduction to structural dynamics" by John M. Biggs (8)

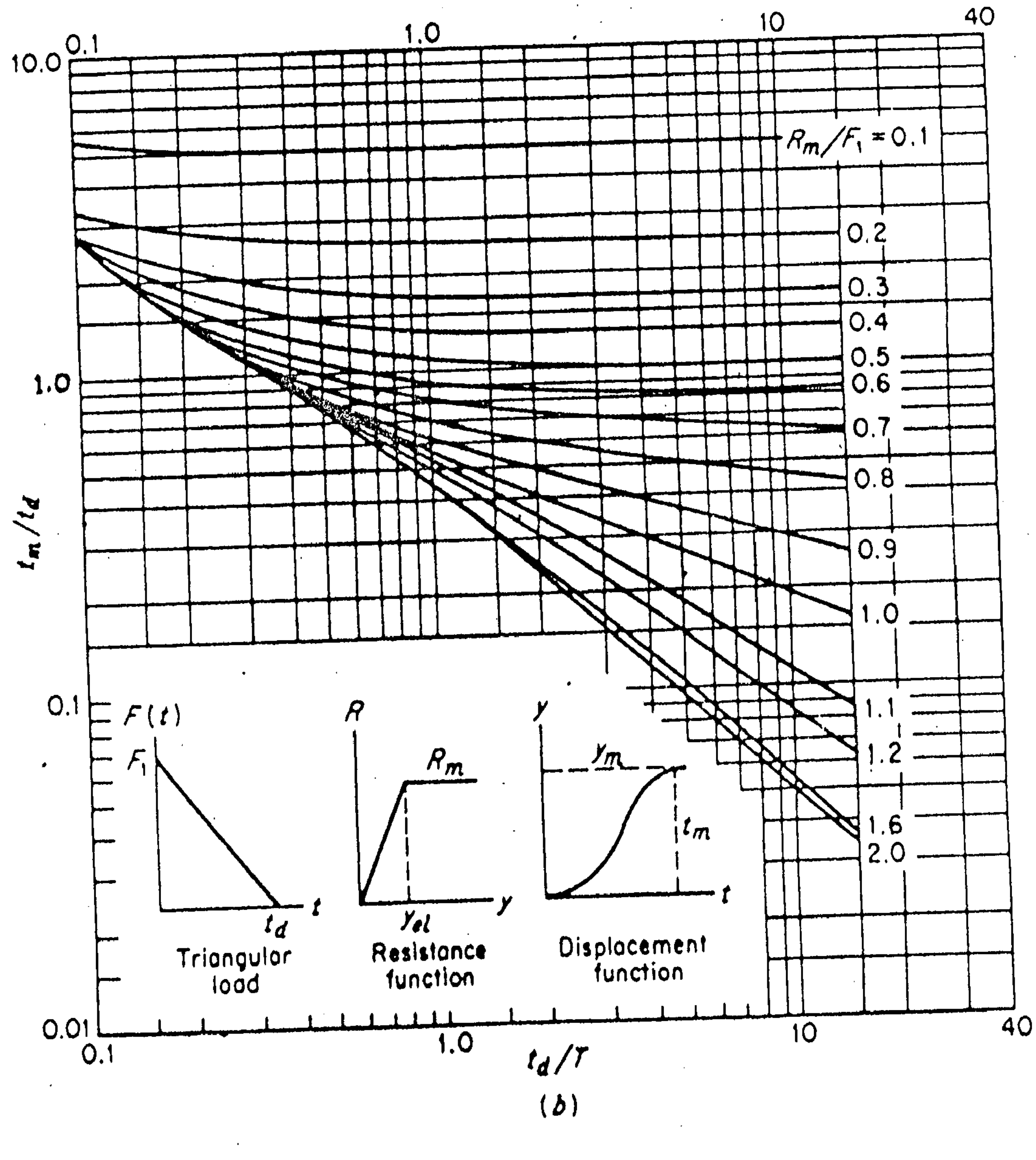
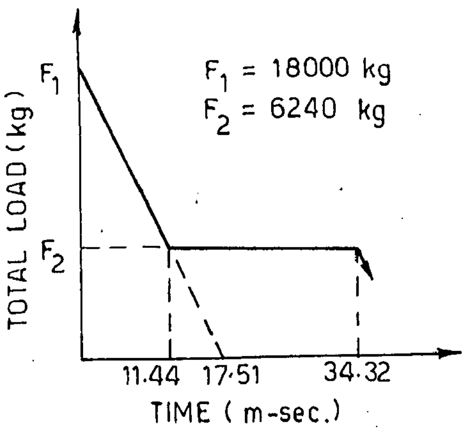
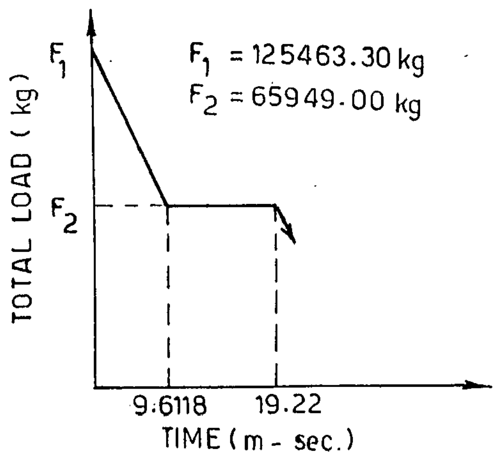


FIG 3.7 Maximum response of elasto-plastic one-degree systems (undamped) due to triangular load pulses with zero rise time.

x Taken from "Introduction to structural dynamics" by John M. Biggs (8).



(a) LOAD TIME FUNCTION FOR ROOF GIRDER



(b) LOAD TIME FUNCTION FOR STEEL FRAME

FIG.3.8 _ DESIGN LOAD TIME FUNCTION FOR ROOF GIRDER AND STEEL FRAME

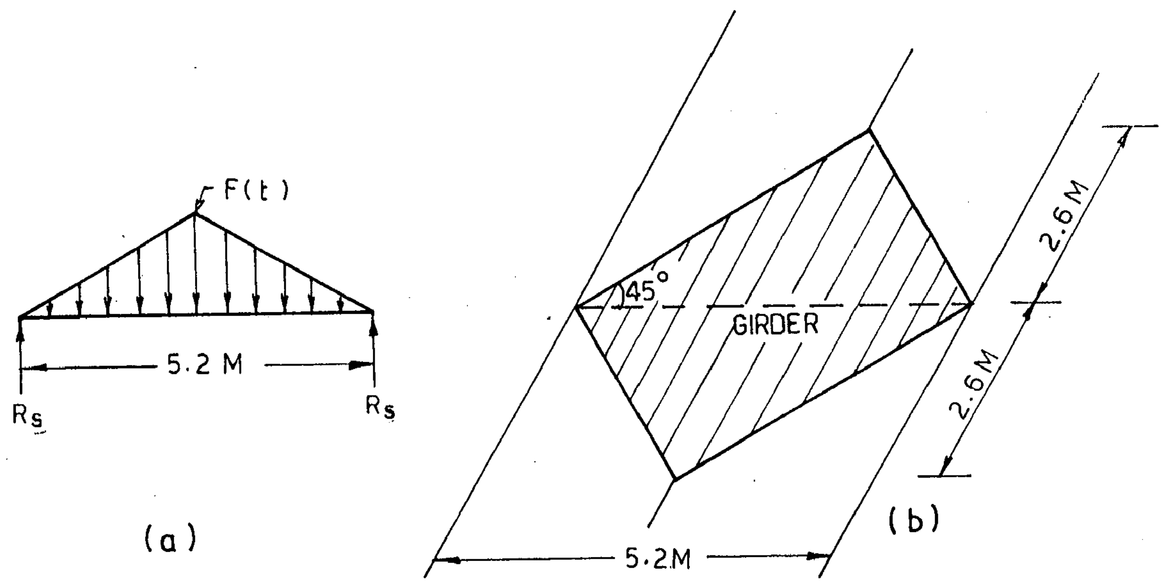


FIG.3.9 _ SPACE WISE DISTRIBUTION OF STATIC AND DYNAMIC LOAD ON ROOF GIRDER

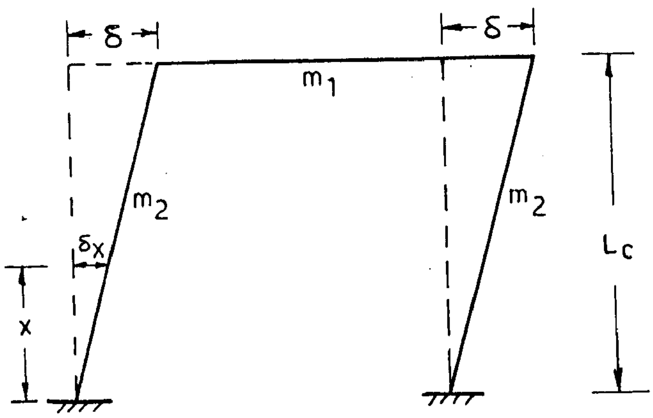
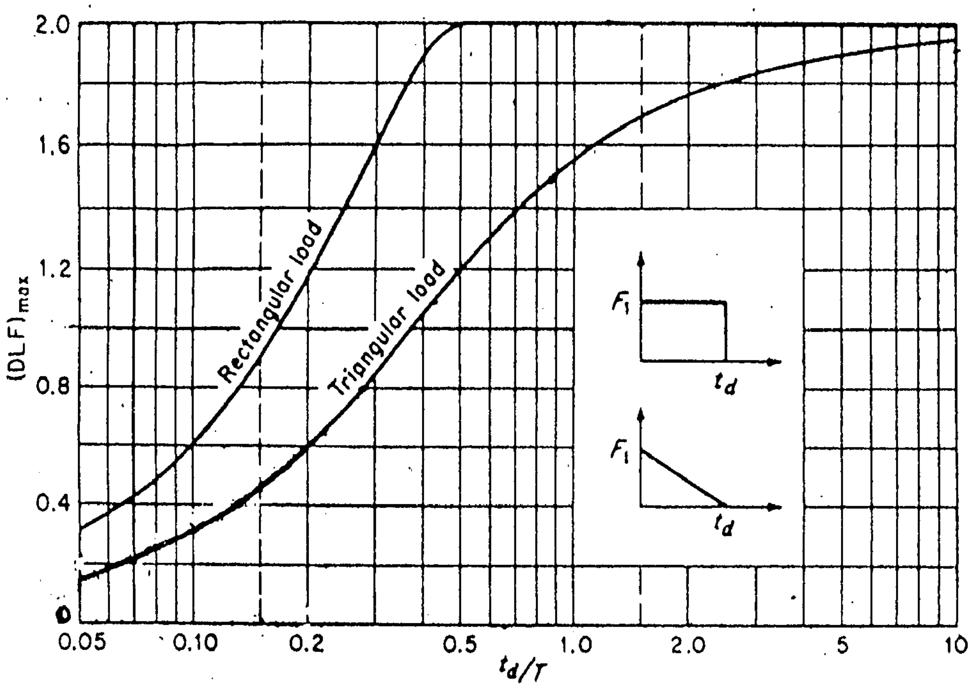
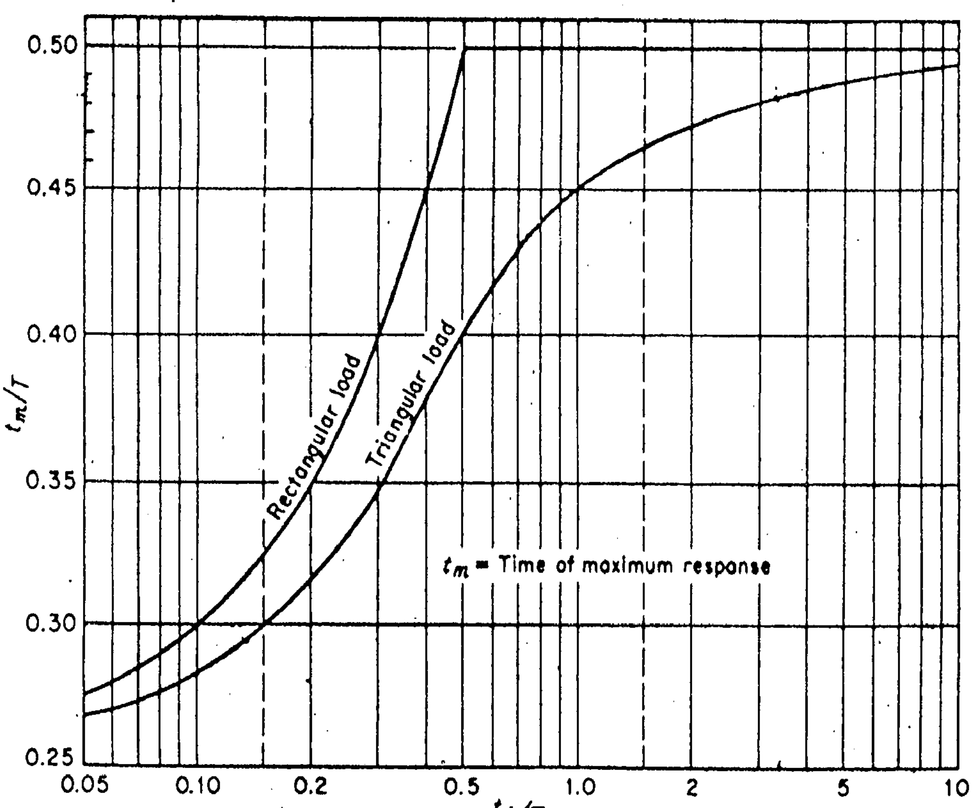


FIG.3.10 _ ASSUMED DEFLECTED SHAPE OF FRAME TO CALCULATE $\phi(x)$



(a)



(b)

FIG 3.11 Maximum response of one-degree elastic systems (undamped) subjected to rectangular and triangular load pulses having zero rise time.

* Taken from "Introduction to structural dynamics" by John M. Biggs (8)

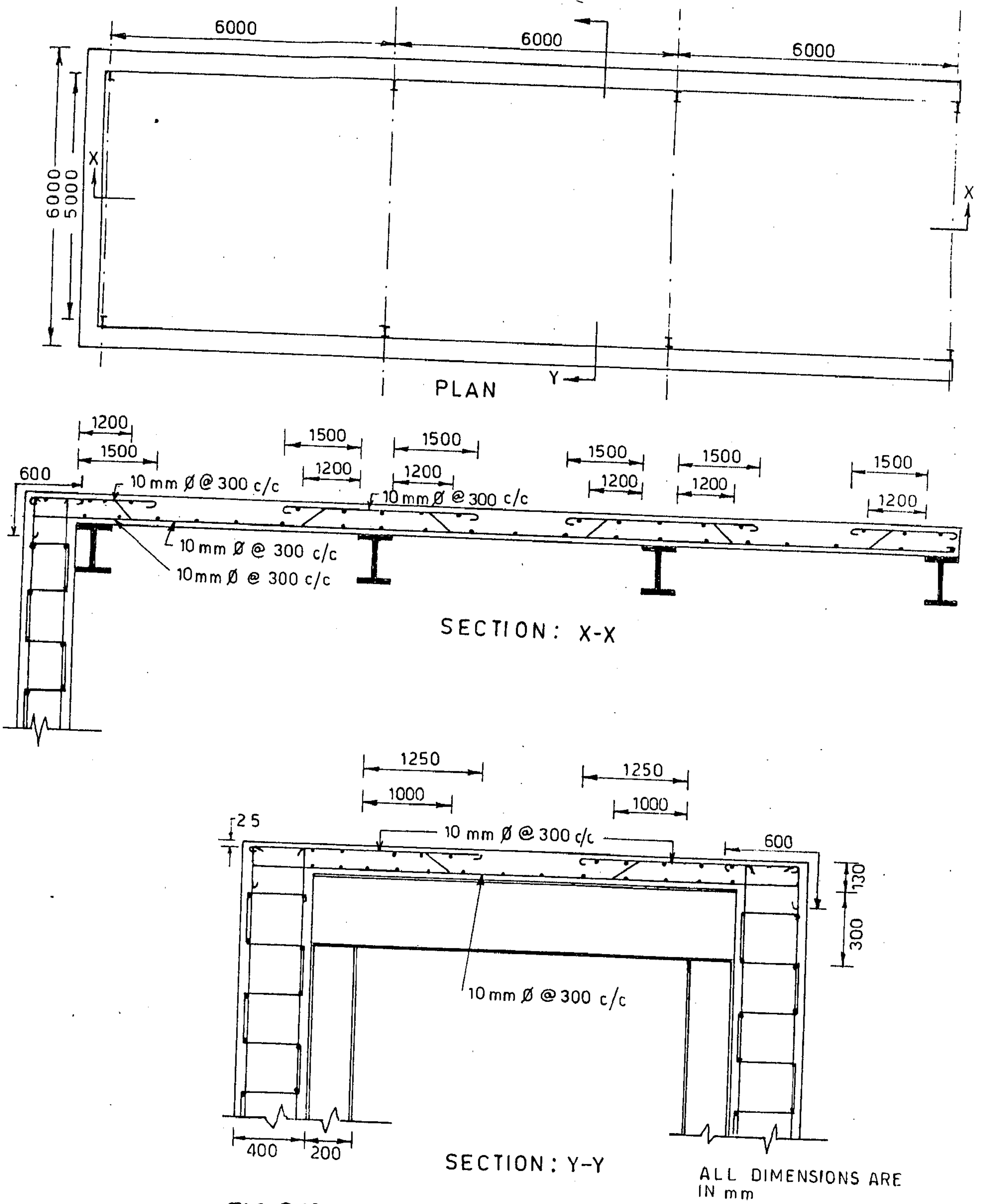


FIG.3.12_DETAILS OF ROOF SLAB

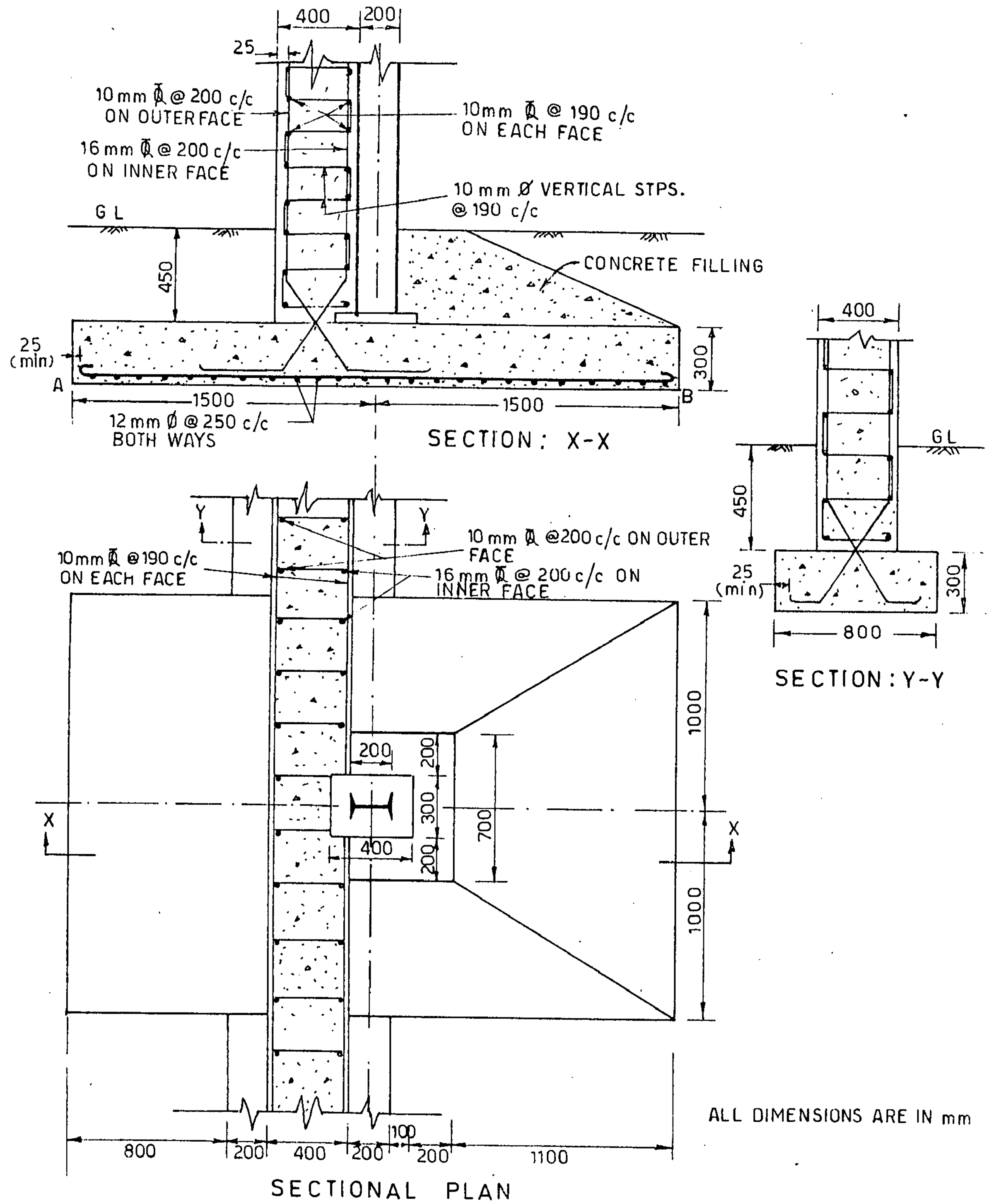


FIG.3.13 DETAILS OF R.C.C WALL, COLUMN FOOTING AND FOUNDATION OF R.C.C. WALL

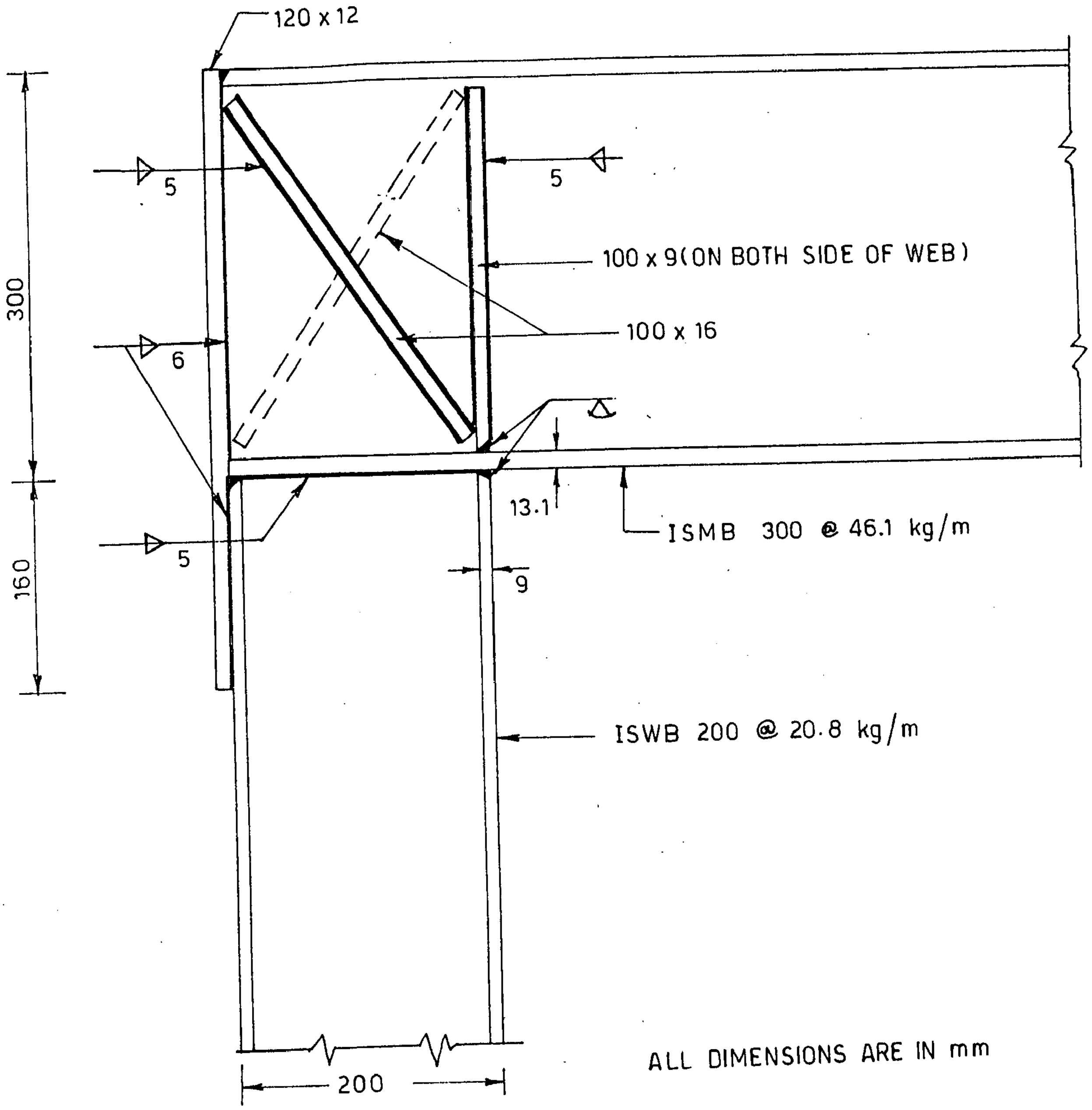


FIG.3.14_DETAILS OF FRAME CONNECTION

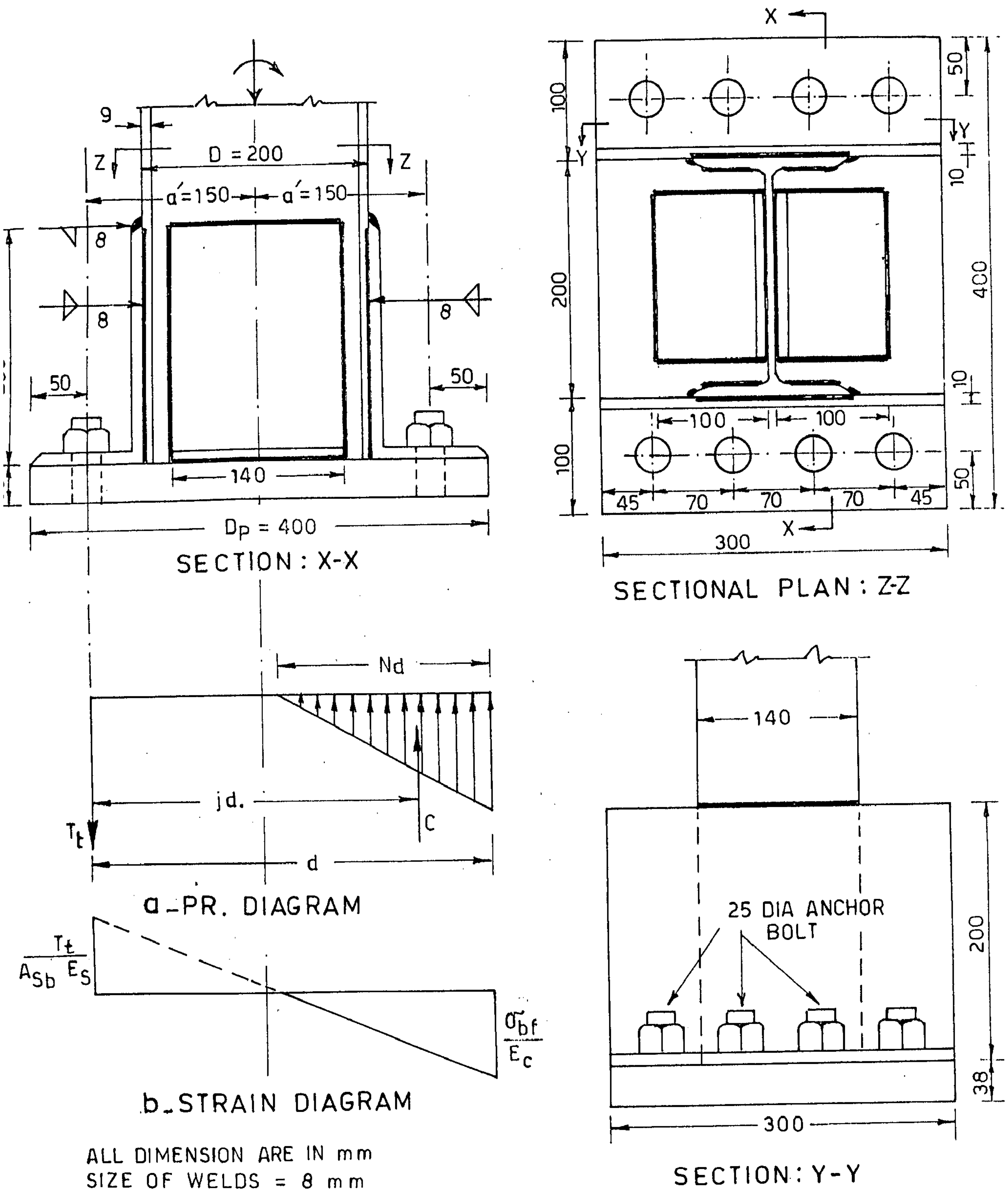


FIG. 3.15 – DETAILS OF BASE PLATE AND CONNECTION

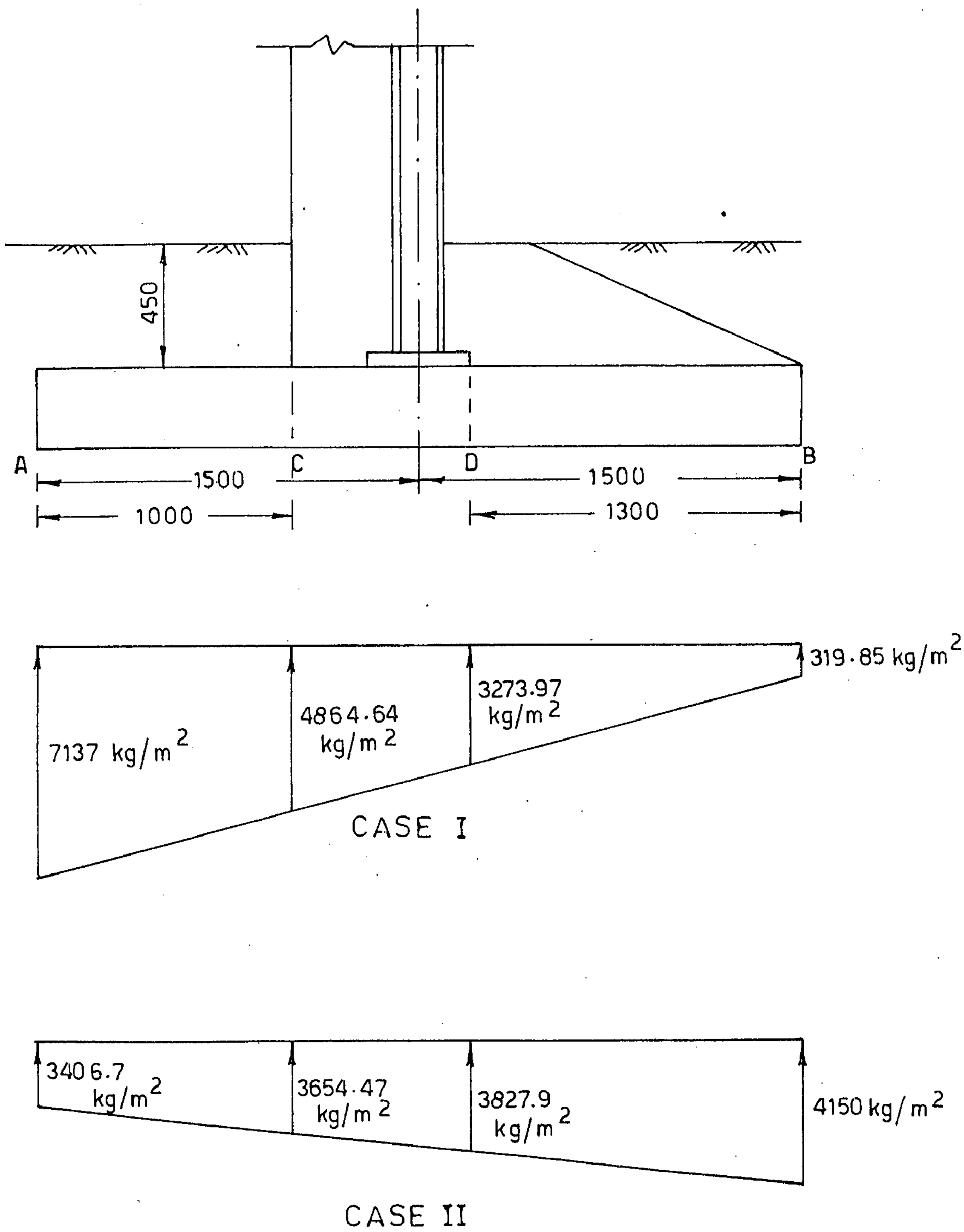
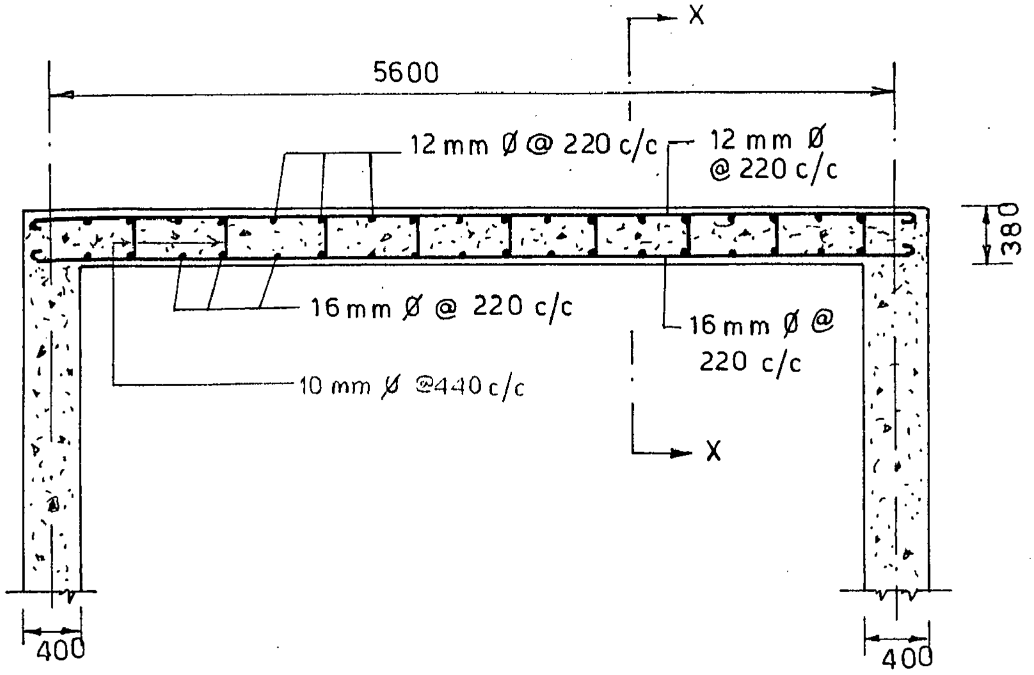
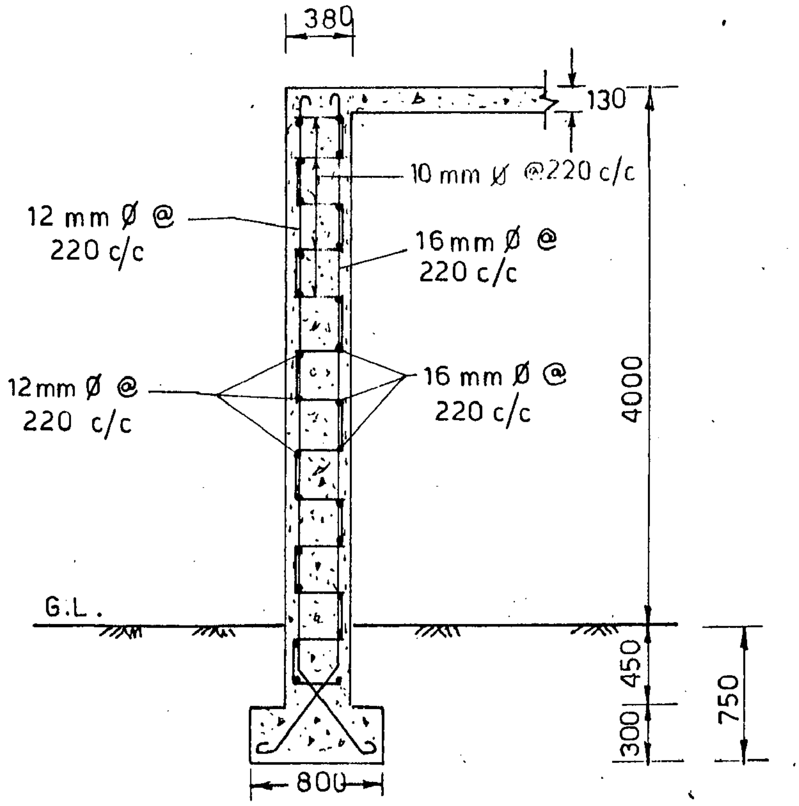


FIG.3.16_PRESSURE VARIATION ON FOOTING(STATIC)



SECTIONAL PLAN



SECTION : X-X

ALL DIMENSIONS ARE IN mm

FIG.3:17_ DETAILS OF END WALL