Planning, Analysis and Design of a Theare Complex

Project Work

Submitted by :

A. Muthuvel

Guided By :

Prof. R. Krishnamoorthy

1979 - 80

Department of Civil Engineering

Coimbatore Institute of Technology

Coimbatore - 641014

Coimbatore Institute of Technology

Department of Civil Engineering

Coimbatore-641 014.

Project Work

1979-'80

University Register No. ____________

Name ____________ **Roll No.** ____________

Class : Final B.E., Civil

Certifide Bonafide Record of Work Done by

Mr. ____________

In Partial Fulfilment for the Degree of Bachelor of Engineering in Civil Branch of the University of Madras.

Coimbatore.641 014

Date ; ____________

Staff in Charge

Submitted for the University Examination Held in May 1980.

Internal Examiner

External Examiner

ACKNOWLEDGEMENT

I have the privilge of thanking our Professor R. KRISHNAMOORTHY for his valuable suggestions and remarkable guidance.

My esteemed thanks are also due to Professor DR. R.J. GOPALAKRISHNAN, Head of the Department of Civil Engineering and our Principal P.R. RAMAKRISHNAN.

Finally, I feel greatly indebted to one and all of the Faculty of Civil Engineering for their valuable advice regarding this Project Work, and also to my friends helped me in preparing the manuscript of this Project.

A B S T R A C T

This project work describes about the planning, acoustic design, analysis and design of a typical multistoreyed cinema theatre and restaurant complex. Accomodation of this theatre is about 1500. The salient features of this project work are the Pre-Stressed concrete folded plate roof 38m. and 31m. span, Pre-Stressed concrete beam for balcony of 23m. and 21m. span, under ground car parking and a restaurant in the first floor of the structure. The skeleton framed structure is analysed by Moment Distribution method and the slabs are analysed by Yield Line Theory. The design of beam, column and slabs are done by ultimate strength design of reinforced concrete members by V.K.Shankerar, R.Chandra & S.Sarkar. The detailed drawings and estimated for the structure have been prepared. The estimate of this project work is about Rs.

The cost per square meter of plinth area =

As a cinema theatre is a publicplace where the people enjoy the entaiurments like dance, music etc, such places are given and acoustic treatment in the design part. This project work also includes a restaurant in its ground floor. So the place becomes a well-formed public gathering place.

DESCRIPTION

The capacity of this theatre planned is about 1500. The plan area of ground and balcony is 861 m^2. The detailed aretitectural drawings are given. As a change from the conventional type of roof, Pre-stressed concrete folded plate is provided both for appearance and saving of materials. Pre-stressed both for appearance and saving of materials. Pre-stressed concrete beam for balcony is also provided, to give an un-obstructed view. The span is 23m. and is designed as a post-tensioned beam of rectangular section. As the restaurant and underground car parking are provided the structure is analysed as a framed structure. In addition to this to goto first floor and to balcony ramps are provided at a slope of 1:10

The special feature of this project is the provision of Pre-stressed concrete folded plate roof. The analysis and design principles of pre-stressed folded plate is well explained.

The estimation of thetheatre and other auxilliaries is about

CHAPTER-II

PLANNING OF A THEATRE COMPLEX

Introduction

The most important considerations when planning a cenima theatre, restaurant and underground car-parking are 1) Visibility 2) Audibility, 3) Safety, 4) Comfort, 5) Adequate space for parking of cars and other vehicles etc.

VISIBILITY

The clear, un interupted vision of the show is of prime concern while designing a theatre. The seats are aranged such that the vision of the audience is perpendicular to the tangent of the curvature of the screen. The curvature of the screen is convex to the audience in one direction only, i.e. the curvature is horizontal or perpendicular to the vertical plane of the screen. For that a slope is geven (I.S: 2526 - 1963). The slope may be adopted from Brittanica Vol. I. The allowable slope is 8° to 15°. Here an average slope of 12° is adoped. The visibility slop of an audient who is sitting in his position should not exceed 35° (Tamil nadu cenima Autograph rule). Silver Screen is fixed since it is to maintain and has got suitable visibility, especially for colour films.

AUDIABILITY.

There are two types are 1. airborne and 2. Solid borne from outside or inside excluded by using airtight fit doors. A care is takento avoid excessive reverbaration and echoes inside the auditarium. Echoes are avoided by means of the shape And providing inclined walls. For to have the good shape the rearwall in made concave, they it is not visible. So the surface is given convex corugation to avoid the tendency of sound to focus into the hall. Fan shaped or lengthyhexagon type of plan is good due to capacity, slight chages may be done. Ceilings are not provided at the samo level but accustic plaster is provided.

Noise

The main is the elimination of noise. Since aircondition is provided a care is taken to reduce the plant noise and grill noise. If any power cut period or if use of power generators then there may occur noise from generaters for that we have to t take care so that no noise obtained there. For this plant is isolated and ducts are provided to reduce the noise. Depending upon the noise level at the site orientation layout and structurarel design are arranged to provided necessary noise reduction.

COMFORT

The traffic of audience plays good roll in planning of auditorium. For comfortable walking. The seats are arranged such that the distance bwtween row to row as 85cms. andseat to seat 50cms.

The front row is situated morethan the 3/4 of the seree hight. This seating arrangements are done from the plan of GRACE Rainy Rogers Auditorium, New York. The passage with width is given 1m. (near about the allowable limit, Cinemas Rules No. 63) and verified form I.S. specifications. The Urinals are provided comfortably (as per Cinemas Rules No. 55).

SAFETY

The main safty is from fire. From the Tamil Nadu Cinemas (Regulations) Rules No. 91. The fire extingushers bucket of water and sand are provided. The fire extinguishers are checked by the lincied persen for every three months. The internal smok film stopes and ready action for the public fire service are made. Fire warning systems are installed.

SCREEN

Silver Sereen is used. It is having a dimensionof 6mx10m It is convex to the audience. The convex projection to the chord is 1m. at its centre.

MAIN BUILDING

It is of length 48m. and a breadth of 38m. including 8m. of ramp portion. Overall height from ground level is 18m. for the building upto the bottom of the folded plate. The ramp portion height is about 25m. from ground level and 29m. from under ground level. The roof is done as pre- stressed concrete folded plate.

VERANDA

On both sides of the theatre a veranda a having a minimum width of 2m. at the centre and 4.5m. at the ends is provided. The veranda ends at the other end. The veranda floor level is sloped with the inside floor of the theatre.

ENTRANCE.

The main entrance for the theatre is at the front of the main building at theright corner of 4.5m. width. For the restaurant the entrance are of 4.9m. width on both sides of the building. A doorway of 2m. width is provided at 5 places on both the sides of theatre.

RAMP.

The ramp starts from the left hand side of the entrance. The width of the ramp slab is 2m. The slope is 1:10. The ramp has 2m x 4m. flight wherever it is required. The ramp slab is supported by two cantilever beam projecting from the column.

TOILETS

The lavatory and wine urinals are arranged behind the screen in the balcony floor. Toilets are provided more than tha t required in the Cinema Autograph rules.

TICKET COUNTERS

These may be provided accoring to the convenience. Besides, the teatre has a lot of space in the ramp portion. Therefore there is no problem in forming the ticket counters

SOIL INVESTIGATION

The theatre being built in an imaginary place, it is difficult for making the soil investigation. So, the bearing capacity is taken as $10t/m^2$.

FOUNDATION

The plinth beam is provided 2m. above the top ofthe footing The column loads are distributed by means of isolated square footings.

CONCLUSION

Auditorium buildings are essentially public buildings and they are conventient yard sticks to measure the treands in architecture and planning. The design ofauditorium should be flexible so that whatever new ideas are evolved for such structures, they can also be incorperated to the possibleextend.

It may seen to one that the cost to fulfill all the requirements is forbidding. But if a commercial auditorium is built with all the facilities mentioned it is sure to pay.

CHAPTER-III FOLDED PLATE DESIGN

INTRODUCTION:

Since the cinema theatre is having a span of 38m. in breadth wise, it is not possible to design or construct a folded plate to such an extent. So, the pre stressing comes into picture.

Theefficiency of a structure depends up on the effective use of materials in the structure. Shell structures have an inherent advantage in their geometry to distribute the load effectively to all the material instedad of concentrating the load resistances to any particular portion. The load carrying capacity of the shells depends basically on the shape of the structure rather than on the mass of materials used. Even though the shells are very efficient structures, their use is limited to specific purposes due to other utility criteria. They are generally used as roof structures and sometimes as foundation structural elemts. There is a positive aesthetic advantage in using these structures for roof elements. The architect has a great flexibility in adopting any desired shape and curve to express the human imagination through the structures. Conventional shells such as cylindrical and spherical shells are not very difficult to analyse for given standard boundary conditions. Shells of double curvature and shells of arbitary curves accommodate the imagination ofarchitects but techniques are used as a basis of design in such complicated shell structures. Analysis of prestressed concrete shell structures presents much more sophistication as compared to the reinforced concreteshells. The use of prestressing is commonly adopted for conventional shells such as cylindrical, spherical and folded plates without much difficulty

ANALYSIS OF FOLDED PLATES

Folded plates may be broadly classified into three groups such as long, intermediate and short folded plates. When the length to rise ratio of the folded plate is greater than 10, it is called long plate; otherwise it falls into one of the two remaining categories. Methods of analysis of folded platesare generally classified as (i) beam method; (ii) folded plate theory neglecting the relative joint displacements; (iii) folded plate theory considering relative joint displacements; and (iv) elasticity approach. Beam method which comes out of simple statics uses the beam formula, treating the folded plate as a beam. This method, which is based on linear stress variation in cross-section of the fold, neglects the transverse slab bending moments and therefore the method is dependable for long folded plates. Folded plate theory neglecting the relative joint displacements assumes that the plates are relatively deep in their own plane so as to provide transverse supporting reactions at the folds. Thus the folded plate in the transverse direction is treated as continuous one-way slab. This method, even though it makes a better approximation than beam theory, is still an approximate method and likely to give transverse bending moments of the slab higher than the actual ones. The third folded plate theory which considers the relative joint displacements, is much improved and more accurate. It can be applied to most of the folded plates except very short plates. The fourth method which is the most accurate of all, treats the structure as a continuum and uses an elasticity approach. The presentation of this method is beyond the scope of this book. The first two methods are fairly straight forward and do not need much explanation. Therefore only the third method-folded plate theory considering relative joint displacements, is described in this book.

The folded plate theory considering the relative joint displacements separates the structure into two structural actions. The slab in the transverse direction is treated as a continuous slab on elastic supports and the forces in the slab are evaluated using slope-deflection equations. The plate in the longitudinal direction is treated as a beam subjected to the forces in its plane. Displacements at the folds are unsure compatibility conditions between the transverse slab and longitudinal beam actions 1.6.

(i) Slab action ; 1.1 illustrates the two actions of folded plate, (a) slab action, and (b) beam action. Plane of any plate is taken as xy plane while x-axis is considered to be the longitudinal axis of the plate. For the purpose of analysis, all external loads on the plate are resolved into two perpendicular directions. One is transverse to the plate and the other is in the plane of the plate. All external loads are resolved into two perpendicular directions. One is transverse to the plate and the other is in the plane of the plate. All external loads are assumed to be of harmonic function (sin nΠx/L) so as to simplify the analysis of force and deformation relations in the longitudinal section of the folded plate. Any distributed load is approximated as an equivalent harmonic load and the results are deduced from it.

Analysis ; Let the load on any slab connecting i and i+1 joints be represented by

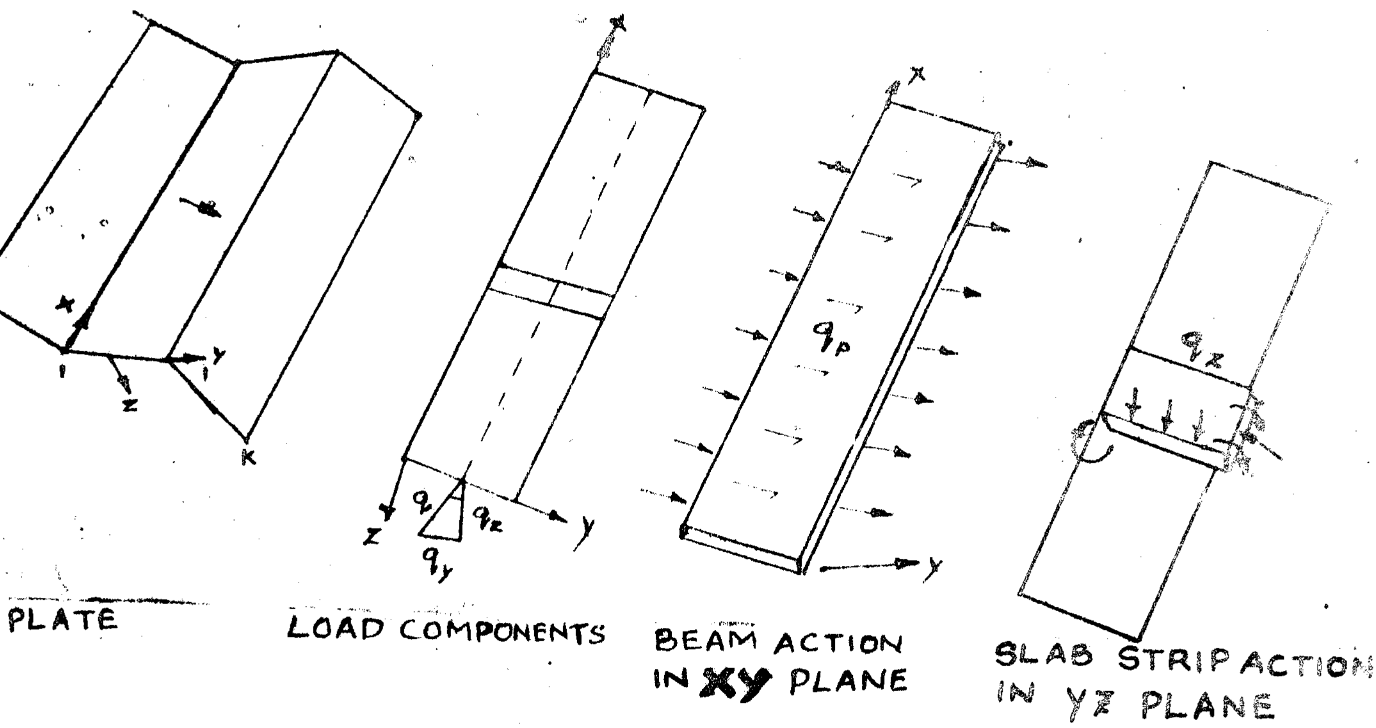

FIG. 1·1: FOLDED PLATE STRUCTURAL ACTION

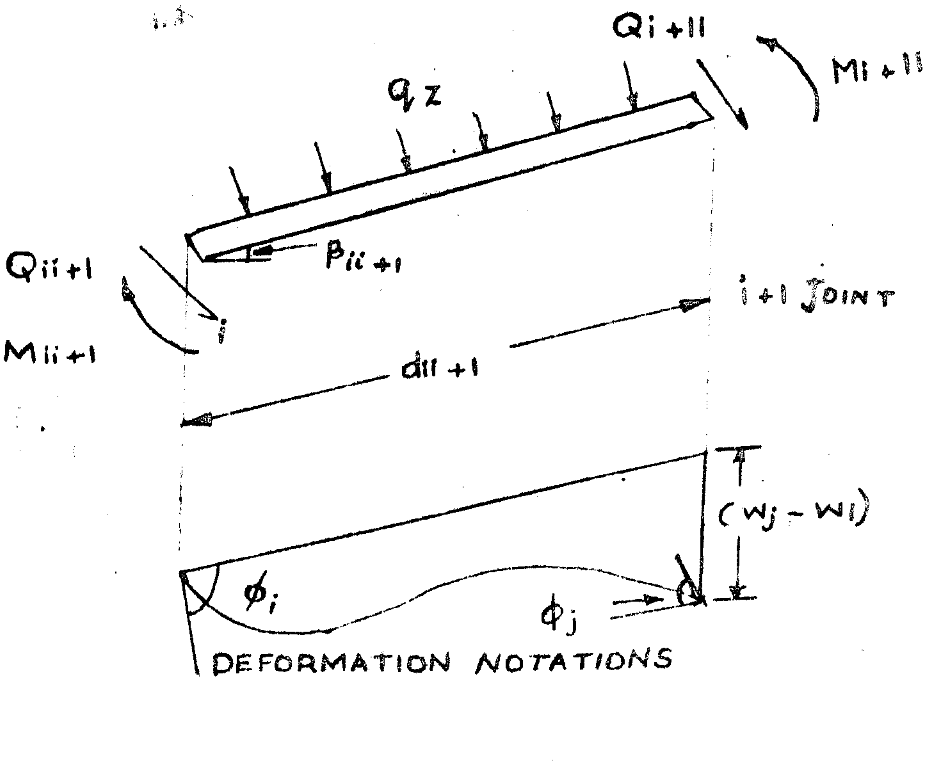

FIG: 1·2
SLAB STRIP SPANNING BETWEEN i AND i+1 JOINTS

$$q_{ii+1} = a \sin \frac{\Pi x}{L}$$

where

L = length of the folded plate

q_{ii+1} = intensity of loadsating vertically downwards

Let q_x and q_y be the components of the distributed load q in x and y-directions respectively. The forces and deformations used in the derivations are shown in fig. 1.2.

Considering a unit width of the slab spanning between i and i+1 joints, the force deformation relations could be written as

$$m_{ii+1} = m^{o}_{ii+1} + k_{ii+1} (2\phi_i + \phi_{i+1} - 3 \;\; _{ii+1})$$

$$m_{i+1i} = m^{o}_{i+1i} + k_{ii+1} (2\phi_{i+1} + \phi_i - 3 \;\; _{ii+1}) \quad \ldots\ldots(1)$$

$$Q_{ii+1} = \frac{(m_{ii+1} - m_{i+1i})}{d_{ii+1}} + \frac{q_y d_{ii+1}}{2}$$

$$Q_{i+1i} = \frac{m_{i+1i} - m_{ii+1}}{d_{ii+1}} - \frac{q_y d_{ii+1}}{2}$$

where

m_{ii+1} = bending moment at joint i of slab connecting i and i + 1 joints

Q_{ii+1} = shear force at joint i of slab connecting i and i+1 joints

ϕ_i = rotation of joint i

$$k = \frac{Et^3}{12d}$$

$$d = \text{width of slab}$$

Moment compatibility at joint i yields

$$m_{ii-1} = m_{ii+1} = m_i$$

The compatibility equations lead to three moment equations as

$$\frac{m_{i-1}}{K_{i-1i}} + m_i \left(\frac{2}{K_{i-1i}} + \frac{2}{K_{ii+1}} \right) + \frac{m_{i+1}}{K_{ii+1}} = 6 \left(\ _{i-1i} - \ _{ii+1} \right)$$

$$+ \frac{1}{K_{i-1i}} \left(2\, m_{fi} + m_{fi-1} \right) + \frac{1}{K_{ii+1}} \left(2m_{fi} + m_{fi+1} \right) \quad ..2$$

Eq.(2) is essentially a governing equation of the ith joint, containing one moment and deflection of the joint. If there are N joints, there will be N equations involving 2N unknowns. N unknowns will have to be determined from the deflection compatibility of the slab and the longitudinal beam action.

(ii)Beam action The three types of loads acting on the beam element are shown in fig. 1.3

(i) Component of the distributed load (q_y) in the plane of the plate;

(ii) component of the slab transverse shear force (Q);

(iii)component of the line loadacting at the joint or fold.

The line loadat the joint is first resolved into components along with

Let P_i = total effective force on the plate ii -1, then

$$P_i = P_{ii-1} + P_{i-ii} + \int q_y dy = (P_i \sin \frac{\Pi x}{L}) \quad .. (5)$$

Considering only shallow plates where depth (d) is small as compared to the length (L), the plate could be treatedas a beam loaded with P_i. Then the mid-span bending moment for the loading is

$$M_{ip} = \frac{P_i L^2}{\Pi^2} \quad .. (6)$$

The corresponding extreme fibre stresses based on simple beam theory are

$$\left.\begin{aligned} f_{pi-1i} &= \frac{M_{ip}}{Z_{i-1i}} \\ f_{pii-1} &= \frac{-M_{ip}}{Z_{i-1i}} \end{aligned}\right\} \quad .. (7)$$

where

M_{ip} = bending moment of mid-span of plate i-1,i.

f_{pi-1i} = ~~modulus-of-secti~~ bending stress due to loadP at i-1 th joint of plate i-1, i and similarly f_{pii-1}

$Z_{i-1}i$ = modulus of section of the plate about x-axis

Bending stresses of the two adjacent plates at the common edge or fold should cause equal strains. If the extreme fibre strains of the two adjacent plates due to P forces are not compatible at the fold, then a secondary set of stresses will be generated within the plate. Such stresses could be generated by a self equilibrium shear force system.

Let T_{i-1} and T_i be the self equilibrium shear forces developed at the i-1 and i folds of the structure. These must vary in sin ΠX/L form to be compatible with the other stress system. The bending moment and normal forces due to the shear forces are given by

$$M_{it} = - \frac{d_{i-1i}}{2} (T_{i-1}+T_i) \qquad .. (8)$$

$$N_{it} = T_{i-1} - T_i \qquad .. (9)$$

where

M_{it} = bending moment on the plate i-1, i due to shear force T

N_{it} = normal force on the plate due to shear force T

Stresses due to the shear force system may be obtained as

$$\left.\begin{aligned} f_{ti-1i} &= \frac{N_{it}}{A_{i-1i}} - \frac{M_{it}}{Z_{i-1i}} \\ f_{tii-1} &= \frac{N_{it}}{A_{i-1i}} + \frac{M_{it}}{Z_{i-1i}} \end{aligned}\right\} \qquad .. (10)$$

f_t = stress in the plate due to shear force T ..

Final extreme fibre stresses can nowbe obtained from eqs..(7) and (10)

$$f_{i-1i} = f_{pi} + f_{ti-1i} = \frac{M_{ip} - M_{it}}{Z_{i-1i}} + \frac{N_{it}}{A_{i-1i}}$$

$$f_{ii-1} = f_{pi-1} + f_{tii-1} = \frac{M_{ip} + M_{it}}{Z_{i+1i}} + \frac{N_{it}}{A_{ii}} \quad .. (11)$$

eq. could also be expressed as

$$f_{i-1i} = \frac{-M_{ip}}{Z_{i-1i}} + \frac{4T_{i-1} + 2T_i}{A_{i-1i}}$$

$$f_{ii+1} = \frac{M_{ip}}{Z_{i-1i}} - \frac{4T_i + 2T_{i-1}}{A_{i+1i}} \quad .. (12)$$

Assuming the Poisson's ratio is zero, the longitudinal strain compatibility reduces to equivalence of the stresses at the fold. A diagrammatic representation of the two adjacent stresses is shown in fig. (1.6)

The strain compatibility then gives

$f_{ii-1} = f_{ii+1}$ or

$$\frac{T_{i-1}}{A_{i-1i}} + 2\left(\frac{1}{A_{i-1i}} + \frac{1}{A_{ii+1}}\right) T_i + \frac{T_{i+1}}{A_{ii+1}} \quad .. 13$$

eq.(13), when applied to each of the folds, will generate sufficient number of equations to determine the shear forces T.

$$v_{ii-1} = \frac{W_i L_i^2}{2 Et} \quad \frac{12}{2E} \quad \left(\frac{M L^2}{td^2}\right)_i \quad .. (14)$$

where

v_{ii-1} = deflection in the plane of the plate i - ii

M_i = $M_{iy} + M_{it}$

Deflections of plates adjacent to joint i should together produce a compatible deflection as shown in fig. 1.7

From fig. 1.7, deflection relation could be written as

$$w_i = (v_{ii+1} - v_{ii-1} \cos \ _i) \operatorname{cosec} \ _i$$

$$\ _i = \left(\frac{w_i - w_{i-1}}{d_{ii-1}} \right)$$

$$= \frac{(v_{ii+1} - v_{ii-1} \cos \ _i) \operatorname{cosec} \ _i (v_{i-1i} - v_{i-1i-2} \cos \ _{i-1})}{d_{ii-1} \qquad \operatorname{cosec} \ _{i-1}}$$

$$= \frac{v_{ii+1} \operatorname{cosec} \ _i (\cos \ _i \operatorname{cosec} \ _{i-1}) \, v_{i-1i} + v_{i-1i}^{-2} \cot \ _{i-1}}{d_{ii-1}}$$

The derivation apparently looks pretty involved. However the solution could be obtained from the series of simultaneous equations, or through an iterated scheme.

spherical and folded plates without much difficulty.

DESIGN

DATA GIVEN:

$L = 31$ m; $H = 5$m.

$\therefore d = 5\sqrt{2} = 7.07$m.

$q_1 = 10$ kg/m^2

SOLUTION:

$t = \frac{L}{250} = \frac{31 \times 100}{250} = 12.5$ say 13cm.

Then $q_d = \sqrt{2} \left(\frac{13}{100} \times 2400 \right) = 441.23$ kg/m^2

say 442 kg/m^2

The uniformly distributed load is converted into a statically equivalent sinusoidal load.

$\therefore$ The external loads are assumed to be as function of harmonic motion.

Harmonic load $q = \frac{4}{\pi} \times$ (U.D.L.)

The longitudinal edges of the plate are assumed to be free so

$T_1 = T_2 = 0$

Let $A_{12} = A_{23} = A$

$t_{12} = t_{23} = t$ $\quad\because$ they are symmetrical

$Z_{12} = Z_{23} = Z$

There is no live load at the fold and the extreme edge of the fold is free, so all the tranverse components of the load will be transmitted directly to the adjacent plate. Being the included angle is 90°, so transverse shear on one plate is directed fully on the adjacent plate. The deformation and transverse shears are un coupled, beeasuse of the symmetry and siggle fold.

$M_{1p} = - M_{2p}$

$\therefore T_2 = 0$

Load at transfer (self weight only)

$$= (9q_g d + Q_g) \frac{4}{\Pi}$$

$$= \left(\frac{442}{\sqrt{2}} \times 7.07 + \frac{442}{\sqrt{2}} \times 7.07 \right) \times \frac{4}{\Pi}$$

$$= 5627 \text{ kg/m.}$$

Load at working load condition;

$$= (q_t d + Q_t) \frac{4}{\Pi} \qquad q_t = 442 + 100 = 542 \text{ kg.}$$

$$= \left(542 \times \sqrt{2} \times 7.07 \times \frac{4}{\Pi} \right) = 6899.9$$

say 6900 kg/m

Let P_{be} = Load balanced at transfer condition.

The allowable compressive stress at working condition is about 0.8 times to that transfer load conditions. Then a good load balance may be obtained by

$$0.8 \, (q_{bt} - 5627) = (6900 - 0.85 \, q_{bt})$$

$$0.8 q_{bt} - 0.8 \times 5627 = 6900 - 0.85 \, q_{bt}$$

$$q_{bt} = \frac{11401.6}{1.65} = 6910 \text{ kg/m}$$

Let the cable profile be parabolic and maximum excentricity

= 3.25m.

Then the equivallent balancing load is given by

$$\frac{8e \, P_t}{L^2} = q_{bt}$$

$$\therefore \quad P_t = \frac{q_{bt} \, L^2}{8e} = \frac{6910 \times 31^2}{8 \times 3.25} = 255404 \text{ kg.}$$

$$A_s = \frac{P_t}{0.7 f_s'} = \frac{255404}{15000 \times 0.7} = 24.32 \text{ cm}^2$$

Provide 7mm diameters wires of 12 nos. cable allowable force on each cable = 49110 kg.

No. of cables required $= \frac{25}{4.62} = 5.4$ cables say 6 cables

Pre-stressing force required = 255404 kg.

$\therefore$ No. of cables required $= \frac{255404}{49110} = 5.2$

say 6 cables

$\therefore$ Applied pre-stressing force = 294660 kg.

DESIGN OF SLAB:

At transfer condition load is not balanced = - 1273 kg.

$\therefore P_2 = 1273$ (Lime load at fold)

$M_{1p} = M_t = 123952$ kgm.

$$q_x = \frac{442}{\sqrt{2}} \times \frac{4}{\pi} = 398 \text{ kg/m}^2$$

B.M. of slab

$$m_2 = \frac{-6 M_{1p} L^2 t^2}{\pi^2 d^4}$$

$$= \frac{-6 \times 123952 \times 961 \times 0.13^2}{\pi^2 \times 7.07^4}$$

$$= -490 \text{ kgm/m}$$

$$\frac{q_x \times d^2}{8} = \frac{448 \times 7.07^2}{\sqrt{2} \times 8} = 1953 \text{ kg.m}$$

Non- Prestressing steel for the folded plate of 31m. span.

from parabola

$$y = \frac{4y_c\, x}{l^2}(l - x) = \frac{4 \times 1953 \times (x)(7.07 - x)}{7.07 \times 7.07}$$

from triangle, $y = \frac{490\, x}{7.07}$

Equating both,

$$69.3\, x = 1105x - 156.28\, x^2$$

$$\therefore x = 6.6 \text{ m}$$

The point ofcontraflexure is 6.6 m from right.

A_t for shear.

$$A_t = \frac{B.M}{\sigma_y \times j \times d} = \frac{490 \times 100}{2500 \times 0.91 \times 11} = 2.13 \text{ cm}^2$$

Provide 8mm ϕ^r of rods of 30 cm C/c.

A_t for mid section;

$$A_t = \frac{1709 \times 100}{0.91 \times 2500 \times 11.5} = 7.09 \text{ cm}^2$$

Provide 10mm ϕ^r rods of 10cm C/c

Distribution steel.

6mm ϕ^r of M.S spacing 20mm C/c on both the faces.

DESIGN OF 38m. SPAN FOLDED PLATE.

$L = 38m.$ $\qquad\qquad H = 6m.$

$\therefore\ d = 6\sqrt{2} = 8.5$

$q_L = 100\ kg/m^2$

Solution.

$t = \frac{38 \times 100}{250} = 15.2\ cm$ Say 16cm.

Then $q_d = \sqrt{2}\ \left(\frac{16}{100} \times 2400 \right)\ 543\ kg/m^2$

Load at transfer (Self weight only)

$$= (q_g\ d + q_g)\ \frac{4}{II}$$

$$= \left(\frac{543}{\sqrt{2}} \times 8.5 + \frac{543}{\sqrt{2}} \times 8.5 \right) \frac{4}{II}$$

$$= 8310\ kg/m$$

Load at working condition. (including live load)

$$= (q_t\ d + q_t)\ \frac{4}{II}$$

$q_t = 543 + 100$
$= 643 kg.$

$$= 643 \times \sqrt{2} \times 8.5 \times \frac{4}{II}$$

$$= 9840\ kg/m.$$

P_{be} = load balanced at transfer.

Check for stress at working condition

$P_e = 0.85\ P_t = 0.85 \times 450922$

$= 383283$ kg.

Load not balanced $= q_{be} = -9993 + 9840 = -153$

$$B.M = \frac{153 \times 38 \times 38}{II^2} = -22385 \text{ kgm.}$$

$$\text{Average stress} = \frac{383283}{16x850} = 28.1 \text{ kg/cm}^2$$

$$\text{Bending stress} = \frac{22385}{16x850^2} = 1.9 \times 10^{-3} \quad \text{Say zero.}$$

The stress are with in permissible limit.

Cable for folded plate of 38m. span

$A_s = 43$ cm^2

Provideing 7mm ø wires of 12 nos. in each cable of fressaynate.

No. of cables required $= \frac{43}{4.62} = 9.3$ Cables

Say 10 cables

Pre-Stressing force required = 450922 kg.

Allowable Pre-Stressing force in each cable = 49110

Applied Pre-Stressing force = 491100 kg.

Non- Pre stressing steel for the folded plate of 38m. is provided as like in 31m span folded plate.

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DESIGN OF PRE-STRESS CONCRETE BEAMS

Introduction

The main point to be taken care of while designing is to provided an un-obstructive view for the patrons. In the past, they followed the conventional method of designing of balcony portion which is uneconomical and an obstructive one. Due to the advancement of pre- stressing eminences in India, the pre-stress concrete beam is an inevit for the public using structures like cinema theatres.

It is uneconomical to provide a beam for a span of 32m. In the conventional design, the beam depth will come to about 2m. which isan undesirable one. If columns are provided, itwill obstruct the view and also reduce the seating capacity.

Here the pre stress concrete beams are provided at 4m. and at 8.1m from the rear wall at a spacing of 21m and 23m respectively of the theatre.

To minimise the work, the longer beam of 23m. span is only designed.

$Z = \frac{1}{6} bd^2 = 148280.3 \ cm^3$

$\frac{1}{6} \times 100 \times d^2 = 148280.3$

$\therefore \quad d = 94.32$ Say 90 cm.

$y_b = y_t = 45$ cm.

Area of section $= 90 \times 100 = 9000 \ cm^2$

Weight of the beam per / m. $= 0.9 \times 2400 = 2160$ kg/m.

$$M_D = \frac{2160 \times 23 \times 23}{8}$$

$$= 14283000 \text{ kgcm.}$$

$$C_{dt} = C_{db} = \frac{M_D}{Z_b} = \frac{14283000}{1/6 \times 100 \times 90 \times 90} = 105.8$$

$$C_{at} = C_{ab} = \frac{19373000}{1/6 \times 100 \times 90 \times 90} = 143.5$$

Check whether C is greater than (C_{dt} + C_{at})

$C_{at} + C_{dt} = 143.5 + 105.8 = 248.3$

$C = 156$ is less than 248.3

The following equation can be usedfor choosing P_i and e

$$\frac{P_i}{A} \left(-1 + \frac{l_{yt}}{r_t^2} \right) - C_{dt} \text{ is less or equal } C_t$$

ie. $\frac{1}{P_i}$ is greater than or equal to $\frac{-1}{A(C_{dt} + C_t)}$ ..(1)

$\frac{1}{P_i}$ isgreater than or equal to $\frac{1}{A(C + C_{db})}$ ...(2)

Elbow is designed by Magnels graphical method. Check for stress are not necessary.

<u>Design of cable profile</u>

Upper Kern distance $K_t = \frac{r^2}{y_b}$

$$r^2 = \frac{I}{A} = \frac{90 \times 100^3}{12 / 9000} = 833.34$$

$$K_t = K_b = \frac{833.34}{50} = 16.67$$

Eccentric from graph = 36.25 cm from nutral axis

$$\frac{M_D}{P_i} = \frac{14283000}{754717} = 19 \text{ cm.}$$

Table for the ordinates of the individual cables

Cabl No.	Position from bottom in cm. 0 m.	4 m.	8 m	11.5 m
lmno	80	44.65	28.3	18.5
jkpg	65	38.3	22.8	18.5
abgh	50	26.15	12.34	8.5
cdef	8.5	8.5	8.5	8.5
Total	203.5	117.6	67.94	54.0
C.G.	50.875	29.4	16.985	13.5
e	0.875	20.6	33.0	36.5

Design of End Block.

Assume plate size = 40 x 60 cm.

Pressure on plate = $\frac{785760}{40 \times 80}$ = 245.55 kg/cm^2

Say 246 kg/cm^2

Let the projection of plate = 3 cm.

Considering 1cm width of plate

B.M. = $\frac{246 \times 3 \times 3}{2}$ = 1107 kg/cm^2

M = f Z

110 f = 1000 x 1/6 x 1 x t^2

t = 2.178 Say 2.5 cm.

Bursting force:

Depth os symmetrical poristion = 2 x 50 = 100 cm

Bursting force Vertical plane.

Considering of span as deep beam (Vertical plane)

= 0.3 P (1- d/D)

= 0.3 x 785760 (1- $\frac{40}{90}$)

= 141436.8 kg.

Force per unit length of the breadth of bearing plate = $\frac{141436.8}{40}$ = 3535.82

a' / a = d/D = 40 / 100 = 0.4

From chart the zero stress position is 0.28D

0.28 x 40 = 11.2

Total tensile force =

$2/3 \times f_y$ max. $\times 88.8 = 3536$

f_y max $= 59.729$

Allowable tensile force = 18 kg/cm^2

Using the parabolic equ. = $\dfrac{4 \times f_y \text{ max.} \times (x)\ (L-x)}{L^2}$

$$18 = \frac{4 \times 59.729 \times (x)\ (88.8 - x)}{88.8^2}$$

$\therefore \quad x = 6.25$

Force for which tensile reinforce is necessary

$= 2/3 \times 41.729 \times 76.3$ / 1cm breadth

Total force $= 2/3 \times 41.729 \times 76.3 \times 40$

$= 84304.6$ kg.

Assuming 20mm ø" 2 legged stirrup.

No. of stirrups necessary = $\dfrac{84304.6}{1400 \times \pi/4 \times 2 \times 2} = 9.6$

Say 10 nos.

Provide at 8.5 cms. C/c.

<u>Bursting Force for horizontal plane</u>

$= 0.3 \times 785\text{?}60\ (1 - \frac{60}{100})$

$= 94231.2$ Kg.

Force per unit length of the breadth of beam plate $= \dfrac{94231.2}{60} = 1571.52$

a'/a = 60 / 100 = 0.6

From chart zero stress position is 0.44 D

0.44 x 60 = 26.4

$2/3 \times f_y \max \times 73.2 = 1571.52$

$f_y \max = 32.2 \ kg/cm^2$

Allowable tensile force = 18 kg/cm^2

Using the paraboly

$$= \frac{4 \times f_y \max \times (x) (L-x)}{L^2}$$

$$18 = \frac{4 \times 32.2 \times (x) (73.2 - x)}{73.2^2}$$

x = 9.49. cm.

Tensile fomede reinforce is necessary.

= 2/5 x 14.2 x 54.22 x 60

= 30785 kg.

Assuming 10mm ø" 2 legged stirrup at 5cm c/c.

oooooo

Lateral ties

Not less than 1/4 of ø of man. reinforcement $= \frac{16}{4} = 4$mm.

It should not be less than 6mm.

So provide 6mm ø

Pitch of ties

i) Least lateral dimension = 40cm.

ii) 16 times the dia of longitudinal bar = 16 x 1.6 = 25.6 cm.

iii) 48 time the dia of transverse reinforcement = 48 x 0.5 = 24 cm.

Hence pitch of 5mm ø at 24 cm / C/C .

FOOTINGS

Load on column	=	76360 kg.
Wt. of footing	=	7636 kg.
Total Load	=	83996 kg.

∴ Area of footing $= \frac{83996}{10000} = 8.39\ m^2$

∴ Side of footing = 2.89 m.

Say = 3.2 m.

Depth from B.M. Consideration

Critical section for bending moment is shown in fig.

Projection beyond the critical section for bendingmoment

$= \frac{320 - 40}{2} = 140$ cm.

Maximum bending moment $= 8788 \times 3.2 \times 1.4 \times \frac{1.4}{2} \times 100$ kg.cm

= 2,756,000 kg.cm.

.·. Effective depth at the critical section

$= 54.20 - 7.80$ cm

$= 46.40$ cm.

Shear force at the critical section

$= 8788 \times 3.20 \times 0.498$ kg.

$= 14,010$ kg.

Breadth of footing at the top at the critical section

$= b' = 40 + 2 \times 90.2$ cm.

.·. Shear stress $= q = \frac{S}{a'b'}$

$$= \frac{14,010}{0.87 \times 46.40 \times 220.4}$$

$= 1.58$ kg/cm^2

less than 5 kg/cm^2

oooooooooooo

ANALYSIS AND DESIGN OF SLABS

Introduction:

In this auditorium, according to the boundary conditions of the slab, the floors are designed as fixed on four edges and continuous and also as three side fixed and one side discontinuous. The main alteration in the design of the trapezoidal slabs is the conversion of trapezoidal area into equivalent rectangular area.

Yield line theory, which is very advanced is used for analysing and designing of slabs since now a days knowledge of it has been very much improved. In spite of conventional design, the Yield line theory is used in the design of both the floor andthe ramp slabs. Here only 4 critical slab and one balcony slabs are designed.

Reinforced concrete slabs when supported along the edges or columns at a number of points, when loaded bend in all planes and the bending in one plane will affect the bending in all other planes. The elastic analysis in such slabs is usually based on the theory of elasticity an applicable to thin plates made of homogeneous and isotropic material. These assumptions are not wholly justified, as reinforced concrete is not isotropic and in many cases the reinforcement in two mutually perpendicular directions is not the same. For slabs of irregular shapes elastic analysis will be very complicated. Maximum load that a slab can take without being structurally damaged is much greater than obtained by elastic theory.

The above can be explained by the fact that at thepoint of maximum bending, plastic deformationstakes place and there is redistribution ofmoments for further loading, thus more and more portion of the slab will have plastic deformation before coppapse will take place. Near failure, therefore plastic d deformations should be taken in to account. Such method which utilises plastic deformations is known as 'Yield line theory'.

Yield line theory is applicable to slabs in which the percentage of steel along any given line is constant. That means same mesh isused throughtout the slab. If the ultimate flexural strength along two perpendicular directionsin the slab is the same, the slab is called an isotropically reinforced slab, if the ultimate strength isdifferent in two directions at right angles the slab is called an orthotropically reinforced slab.

Concept.

Consider an isotropically reinforced square slab under uniformly distributed load, simply supported on all four edges, as shown in fig. . As the load is increased the slab behaves elastically up to the stage at which tensile reinforcement in the Central part of the slab, the area of maximum bending moment, begins to yield. This causes small cracks on the underside of ~~the slab, pro~~

the slab, probably ofthe form shown in fig.

If the slab is under reinforcedy with respect to failure and in practice most of the slab sections are, then the cracked section will continue to deform without any appreciable increase in moment. As a result, increase in the slab load will cause the steel in the adjacent sections to yield also, and the cracks, or yield lines, will extend progressively untill they reach the boundary of the slab as shown in fig. ( ). At this stage since the yield line can propagate no further, and the resistance moments along the yield lines are utmost at their ultimate values, the slab is carrying the maximum load possible . Any further increase in load will cause excessive deflection and crushing of the concrete at some section ofthe yield, liner leading to the over loadingofthe remainder of the slab and complete collapse. Thus the condition of the slab when the yield liner have just reached the boundry may be regarded as the collapse condition of the slab.

From the previous section it is clear that a full yield line pattern cannot develop unless the momentrotation characteristocs of the slab are such that a long have the form shown in fig. For this to be the case effective the effective proportion of steel, $q_e = \frac{A_t \sigma_{sy}}{bd \; \sigma_{cu}}$ should be less than 0.333. In practice, rearly all slabs have effective proportions ofsteel very much less than this, and it can generally be assumed that the moment rotation curve will have the necessary characteristics.

Once a yield line pattern at collapse has been assumed, the collapse load may be determined in terms of the ultimate bending strength of the slab, either bystudying the vook done for a given virtual deflection or by considering the equilibirium of the individual slab portions under the forces acting on them at collapse. The rules governing thepossible yield line patterms may be summarised as follows in order to help in seléctiǿng a geometrically possible pattern.

1) A yield line, or a yield line produced, must pass through the point of intersection of the axes of rotation of the two slab parts connected by the yield line.

2) A yield liner are straight. They cannot change direction unless joined by another yield line.

3) Liner supports form axês of rotation, and point, i.e. column supports form "pivot points" (i.e.) axes of rotation can pass over than at any angle.

<u>Yield Moments:</u>

When a yield line is at right angles to the direction of the reinforcement the yield line ultimate moment is given by the general equation for the ultimate moment of an under-reinfored flexure member (i.e.)

$$m = M_u = A_t \sigma_{sy} \; d \left\{ \frac{(1 - 0.91) A_t \sigma_{sy}}{bd \; \sigma_{cu}} \right\}$$

Notations for analysis.

$\overline{\sigma_{sy}}$	= Yield strength ofsteel.
$\overline{\sigma_{cu}}$	= Ultimate cube strength ofconcrete in compression.
A_t	= Area of steel in tension
d	= Effective depth of slab
b	= Breadth of slab.
m	= Ultimate mometn of positive yield line per unit length.
m^1	= Ultimate moment ofnegative yield line per unit length.
Ψ	= Ratio of moments in xand y directions.

Calculation of Ultimate Moment : (Pattern 1)

In the yield line pattern assumed the angle ϕ is unknown and the correct yield line pattern will be the one corresponding to the value of ϕ which gives the lowest value of the collapse load. It is sometimes helpful to think of the external work done by the uniformly distributed ultimate load as the volume of the deflected shape multiplied by the load per unit area. Hence in this case for a deflection of unity at yield line of,

Volume under segment (1) = Volume under segement (3)

$$= \frac{1}{2} l_y^2 \ (\alpha - \tan \phi + 1/12 l_y^2 \tan\phi$$

$$= 1/4 \ l_y^2 \ (\alpha - 2/3 \tan \phi)$$

Volume under segment (2) = Volume under segment (4)

$= 1/12\ l_y^2\ \tan \phi$

Hence, because of continuity, collapse will not takes place untill the four negative yield lines at the support have formed. The negative steel in the top of the slab at the supports is assumed to be such that the ultimate moment ofyield lines parallel to the long sides is m^1 and parallel to the short sides um^1.

Internal work done on yield lines ae, bf, cf, and de:

in rotation x direction $= 4\ \mu m\ \frac{1}{2}\ l_y\ \frac{1}{\frac{1}{2}\ l_y\ \tan \phi}$

$= \frac{4\ \mu m}{\tan \phi}$

in rotation in y direction $= 4m\ \frac{1}{2}\ l_y\ \tan \phi\ \frac{2}{l_y}$

$= 4m\ \tan \phi$

Internal work done on yield linef is

$= m\ l_y(\ \alpha - \tan \phi\ )\ \frac{4}{l_y}$

$= 4m\ (\ \alpha - \tan \phi\ )$

Hence total internal work done by all the positibe yield lines $= \frac{4\ \mu m}{\tan \phi} + 4m\ \tan \phi\ + 4m\ (\alpha - \tan \phi)$

$= 4m\ (\ \alpha + \frac{\mu}{\tan \phi})$

Work done by -ve yield lines:

Internal work done on yield line ab $= $ Work done on cd.

$$= m^1 \alpha l_y \frac{2}{l_y} = 2 \alpha m^1$$

Internal work done on yield line bc = work done on da

$$= \mu m^1 l_y \frac{2}{l_y \tan \phi}$$

$$= 2. \frac{\mu m^1}{\tan \phi}$$

Hence total work done on negative yield lines $= 4m^1 (\alpha + \frac{\mu}{\tan \phi})$

Therefore total work done by negative and + ve yield lines $= 4 (m + m^1) (\alpha + \frac{\mu}{\tan \phi})$

o Equality the external and internal work done

0 $w \frac{1}{6} l_y^2 (\alpha - 1/3 \tan \phi) = 4 (m + m^1) (\alpha + \frac{\mu}{\tan \phi})$

Therefore $\frac{w l_y^2}{24(m+m^1)} = \frac{(\alpha + \mu / \tan \phi)}{(3 \alpha - \tan \phi)}$ ....(1)

In order to find the minimum value of the collapse load, or, for design the maximum value of the ultimate moment for a given ultimate load, it is necessary to differentiate. The right handsize of this equation with respect to tan ø andequate the result to zero.

Length / Breadth = 5 / 4.4 is less than 2

.'. The slab has to design as two way continous slab.

Min. ower all thickness

d = L / 35 = 500 / 35 = 14.2 Say 15 cm.

Hence self weight of slab = 0.24 x 15 x 100 = 360 kg/m^2

Ultimate load =,1.5 x 360 + 2.2 x 500

= 1640 kg/m^2

For slab dimanisch given = L/B = 5 / 4.4 = 1.136

Say 1.14

$$\tan\phi = \sqrt{(3\mu + \frac{\mu^2}{\alpha^2})} - \frac{\mu}{1.4}$$

Assmpeu = 1

$$\tan\phi = \sqrt{(3 + \frac{1}{1.14^2})} - \frac{1}{1.14}$$

$$= 1.064$$

m = m'

$$w = \frac{48\,m}{l^2} \times \tan^2\phi$$

$$m = \frac{w\,l^2}{48\,\tan^2\phi}$$

$$= \frac{1640 \times 4.4 \times 4.4 \times 100}{48 \times 1.064 \times 1.064} = 58428.49 \text{ kgcm. / m}$$

Using 10mm ϕ^r and 200 mix and high stensile bars.

Effect depth = 15 - 1.3 - 0.5 = 13.2 cm

$58429 = A_t \times 4250 \times 13.2 \left(1 - \frac{0.91 \times 4250 \; A_t}{100 \times 13.2 \times 200}\right)$

$58429 = 56100 \; A_t - \frac{56100 \; A_t^2 \times 0.91 \times 4250}{100 \times 13.2 \times 200}$

$A_t = 1.09 \; cm^2$

Minimum A_t = 0.12% of the Cross sectional area.

Depth of slab= 15 cm.

$\therefore \; A_t = \frac{0.12}{100} \times 100 \times 15 = 1.8 \; cm^2$

Provide 10mm ϕ^r rods.

Spacing $= \frac{100}{1.8 / \frac{\pi}{4} \times 1^2}$

Provide 4 rods of spacing 25 cm C/c both the direction

Slab Design ..(2)

Given data

Live load		$= 500 \; kg / m^2$
Length	L	= 5m.
Beradth	B	= 4.4 m.

L / B = 5 / 4.4 is less than 2

$\therefore$ The slab is designed as two way slab.

Overall depth L/ 35 = 500 / 35 = 14.2 Say 15 cm.

Self weight $= 0.25 \times 15 \times 100 = 360 \; kg/m^2$

Ultimate load $= 1.5 \times 360 + 2.2 \times 500$
$= 1640 \; kg/m^2$

$= L/B = 5 / 4.4 = 1.136$ Say 1.14

$$\tan \phi = \sqrt{3u + \frac{u^2}{2}} - \frac{u}{1.14}$$

Where u = 1 for two way continous slab.

$$\tan \phi = \sqrt{3 + \frac{1}{1.14^2}} - \frac{1}{1.14}$$

$$= 1.064$$

for two way slab continous on four sides.

$m = m'$

$$w = \frac{48\,m}{l^2} \times \tan^2 \phi$$

$$\therefore \quad m = \frac{1640 \times 4.4 \times 4.4 \times 100}{48 \times 1.064 \times 1.064} = 58429 \text{ kgcm/m.}$$

Using 10mm ϕ and 200 mix and deormed bar.

Effective depth = 15 - 1.3 - 0.5 = 13.2 cm

$$589429 = A_t \times 4250 \times 13.2 \left(1 - \frac{0.91 \times 4250}{100 \times 13.2 \times 200} A_t \right)$$

$$822\,A_t^2 = 56100\,A_t - 58429$$

Solving this we get

$$A_t = 1.06 \text{ cm}^2$$

Depth = 15 cm.

$$\therefore A_t = \frac{0.12}{100} \times 100 \times 15 = 1.8 \text{ cm}^2$$

Provide 4 rods of spacing 25cm C/c both the direction.

<u>Design of slab ..(3)</u>

Continous on four sides

$= 7.486\ m + 4.3\ m'$

Assuming top and bottom reinforcement as same.

Total internal work done $= 11.78\ m.$

Total external work done $= \frac{w\ L^2}{6}\ (3 - 2x)$

$$= \frac{1604 \times 1.47 \times 5 \times 5}{6}\ (3 - 2 \times 0.617)$$

$$= 17350$$

Therefore $m = \frac{17350}{11.78} = 1472.8$ kgm/m length

$$147280 = A_t \times 12.2 \times 4250\ \left(1 - \frac{A_t \times 0.91 \times 4250}{200 \times 100 \times 12.2}\right)$$

$$= 51850\ A_t - 822\ A_t^2$$

$$\therefore\ A_t = 3.03\ cm^2$$

Depth $= 10$ cm.

$A_t = \frac{0.12}{100} \times 100 \times 10 = 1.2\ cm^2$

Provide 6mm ø rods spacing 25 cm C/C.

Design of slab ..(5)

Given data

Live load $= 500\ kg/m^2$

Length L = Breadth B = 4 m.

L / 35 = 10 cm.

Self weight $= 0.24 \times 10 \times 100 = 240\ kg/m^2$

Ultimate load $= 1.5 \times 240 + 2.2 \times 500$

$= 1460\ kg/m^2$

Assume = 1 ; u = 1

$\tan \phi = \sqrt{3 + 1} - 1$

$= 1$

m = m'

$w = \frac{48\ m}{l^2} \times \tan^2\phi$

$m = \frac{1460 \times 16 \times 100}{48 \times 1 \times 1} = 48666.6$ kgcm./m.

Effective depth = 8.8 cm.

$48666.6 = A_t \times 4250 \times 8.8 \left(1 - \frac{0.91 \times 4250\ A_t}{100 \times 8.8 \times 200} \right)$

$= A_t\ 37400 - A_t^2\ 822$

$\therefore\ A_t = 1.33\ cm^2$

Provide 8mm ø rods R.T.S @ 20 cm C/c.

Design of Balcony slab

Live load = 500 kg /m^2

Length L = 23m.

Breadth B = 4 m.

L / B = 23 / 4 is greater than 2

Minimum overall depth

400/3 = 11.33 Say 10cm.

Self weight = 0.24 x 10 x 100 = 240 kg

Ultimate load= 1.5 x 240 + 2.2 500 = 1460 kg.

Ultimate Bending Moment = $wl^2 / 16 = \frac{1460 \times 16}{16} = 1460$ kgm.

DESIGN OF SEPTIC TANK

Capacity of theatre = 1500

The design capacity ismuch due to the following reasons.

i) Performance are conducted only for a short time in a day.

ii) No. of persons using w.c. is very less.

Let us design septic tank for 500 persons.

i) Quantity of sewage:

The tank is to be designed on the percapita basis of 100 litres/ person.

Effective capacity	= 2 x 500 x 100	= 100000 litres
Sludge capacity	= 2 x 500 x 10	= 10000 litres
	Total capacity	= 110000
		= 110 m^3

Let depth of tank to be 3.5m including sludge depth.

Area of tank = $\frac{110}{3.5}$ = 31.4 m^2

Let L: B = 3:1

Length and tank	= 9.25 m.
Breadth of tank	= 3.5 m.
Depth	= 3.5 m.

Area of tank = 32.4 m^2

Volume of tank = 113.4 m^3 Hence O.k.

Size: = 9.25 x 3.5 x 3.5 m

DESIGN OF RAMP

Introduction

To make an easy way for the elevated places, ramps on elevators are constructed. To economise the building cost and due to the lack of power, in this auditorium ramp is provided. For the ramps, the recommended slope is 1:7 to 1:10. In this auditorium, the slope of 1:10 is provided to make an easy ascending. Here the ramp is treated as a separate structure and designed. To encount the peak croud at the end of the film.show, the ramp passage width is provided as 2m., which is quite large.

For ramp also, yield line theory is used for design. Here the slab is designed as two edge fixed, continuous and the other two sides as free.

The rectangular slab is supported by two cantilever beams from appropriate points of the respective columns. For all the columns in the ramp, the size of the column is 40 x 40 cms. The live load is taken as 500kg/cm^2, according to croud condition, as per N.B. Code of India 1970, section 4, load table-1.

DESIGN OF RAMP SLAB.

Span = 4m.

Assuming 15cm thick slab.

Loads.

Live load = 500 kg/m^2

Self weight of slab is 2400 x 1.5 x 1 = 360 kg.

Celeing finishing = 30 kg/m^2

Floor finishing = 50 kg.

Total load = 440 kg/m^2

Ultimate load = 1.5 x 440 + 500 x 2.2

= 660 + 1100 = 1760 kg/m

Ultimate moment = $\frac{1760 \times 4 \times 4}{12}$

= 2347 kg/m.

K = $\frac{2347}{13.2^2}$ = 14.57 kg.

100 p_t = 0.595

M_{200} Mix and high tensile steel is used.

A_{st} = 0.595 x 13.2 = 7.85 cm^2

Provide 10mmdia rods at 10cm c/c.

Provide the same reinforcement for sharter span.

Design of Cantilever Beam.

Size of beam = 30 x 25

Self wt. of the beam = 1.9 x 0.30 x 0.25 x 2400

= 342 kg/m^2

Ultimate moment = 513 kg.m.

Ultimate load = 1760 kg/m^2

Ultimate load for slab = 3344 + 513

= 3857 kg/m^2

Ultimate moment = $\frac{3857 \times 1.9}{2}$ = 3665 kg.m.

X = $\frac{3665}{30 \times 22 \times 22}$ = 25.25

100 P_t = 0.734

M_{200} Mix andhig tensile steel is used.

A_{st} = 0.734 x 30 x 22 = 4.85 cm^2

Provide 12mm dia rods 5 nos. at bottam and 2nos. crank.

Check for shear.

Total load = 342 + 1760 = 2102kg.

Shear force = 1051 kg.

Shear stress = $\frac{1051}{0.87 \times 22 \times 30}$

= 1.83 is less than 7 kg/cm^2 Hence Safe.

Check for Bend

Check for Bond.

Shear force = 1051 kg.

Bond stress $= \frac{1051}{0.87 \times 22 \times 5 \times 3.8}$

$= 2.89$ kg/cm^2 is less than 10 kg/cm^2

Provide nominal stirups 6mm M.S at 15 cm C/c.

Columns and footing

The column and footing for this is taken as to same as 40 x 40 cm size and 3.2 m x 3.2m squre footing as designed as for column supporting prestressed concrete beam for balcony.

oooooooo

RETAINING WALL

Introduction:

As underground car parking is provided, the pressure on the supporting wall is very large. So, it is necessary to construct a retaining wall.

DESIGN:

Height of stem = 5.5m

Bearing capacity = 10 t/m^2

Soil weight = 1900 kg/m^3

Angle of repose = 30°

Top Width

This may be kept at 20cm.

Bottom Width

This will be determined from bending moment consideration.

Height of stem = 5.50 metres.

Consider one metre run of the wall.

Maximum bending moment for the stem per metre length

$$= C_p \frac{wh^3}{6} = \frac{1}{3} \times 1900 \times \frac{5.5^3}{6} \text{ kg.m.}$$

$$= 17570 \text{ kg.m} = 1757000 \text{ kg.cm.}$$

Adopting C = 50 kg/cm^2

$t = 1400$ kg./cm^2

and $m = 18$, and equating the moment of resistance to the maximum ~~bending~~ bending moment, we have,

$$8.50 \times 100\, d^2 = 1{,}757{,}000 \text{ kg.cm.}$$

$$\therefore \quad d = 45.5 \text{ cm.}$$

Providing an effective cover of 4cm., theoverall depth

$= 45.5 + 4 = 49.5$ cm., Say 50cm.

The base slab also will be made 50cm. thick

$\therefore$ Total height of wall $= 5.50 + 0.50 = 6.00$m.

Width of base.

$$b = 0.5\,H \text{ to } 0.6\,H$$

$$0.5\,H = 0.5 \times 6 = 3.00 \text{ metres}$$

$$0.6\,H = 0.6 \times 6 = 3.60 \text{ metres}$$

Let us provide a base width of 3.50 metres.

Toe Projection.

This may be about one-third of the base width $= \frac{3.5}{3} = 1.17$m.

Let us provide a toe projection of 1 m.

The proposed overall dimensions for the wall are shown in Fig.

STABILITY CALCULATIONS FOR ONE METRE RUN OF WALL

Load due to		Magnitude of load (kg)	Distance from a(metres)	Moment about a (kg.m.)
W_1	$0.2 \times 5.5 \times 2400$	2640	2.1	5544
	$\frac{0.3}{2} \times 5.5 \times 2400$	1980	2.3	4554
W_2	$3.4 \times 0.5 \times 2400$	4200	1.75	7350
W_b	$2 \times 5.5\ 1900$	20900	1.00	20900
Moment of earth pressure $= C_p \frac{wH^3}{6} = \frac{1}{3} \times \frac{1900 \times 6^3}{6}$				22800
Total		29720		61148

∴ Distance of the point of application of the resultant from the end 'a'

$$= Z = \frac{61148}{29720} = 2.06 \text{ m.}$$

∴ Eccentricity $= e = Z - \frac{b}{2}$

$$= 2.06 - \frac{3.5}{6} \qquad = 0.31\text{m.}$$

But, $\frac{b}{6} = \frac{3.5}{6} = 0.58\text{m.}$

∴ e is less than $\frac{b}{6}$

∴ the maximum and minimum pressures at the base are given by

$$P = \frac{29720}{3.5} \left(1 \pm \frac{6 \times 0.31}{3.5} \right) kg./m^2$$

$$\therefore \quad P_{max} = 13000 \ kg./m^2$$

$$P_{min} = 3980 \ kg./m^2$$

DESIGN OF THE STEM

Maximum bending moment for the stem per metre length

= 1,757,000 kg-cm

$$\therefore \quad A_t = \frac{1,757,000}{1400 \times 0.87 \times 45} \ cm^2$$

$$= 31.3 \ cm^2$$

$$\therefore \text{Spacing of 20mm. bars} = \frac{3.14 \times 100}{31.3} = 10cm.$$

Distribution Steel

This will be provided at 0.15 percent of the gross area

$$= \frac{0.15}{100} (100 \times 50) \ cm^2$$

$$= 7.5 \ cm^2$$

$$\therefore \text{Spacing of 10mm. bars} = \frac{0.79 \times 100}{7.5} = 10.5 \ cm.$$

Say 10 cm.

If the distribution steel is provided near both the faces, then the spacing of 10mm. bars will be at 20cm. centres near both the faces.

Design of the toe slab

For one metre length of the wall, the bending momentcalculations for the toe slab are shown in the following table.

Load due to	Magnitude of load (kg)	Distance from c (metre)	Moment about c (kg.m.)
Upward pressure c dif			
10420 x 1	10420	0.5	5210
Upward pressure c fj			
½ x 1 x 2580	1290	$\frac{2}{3}$	860
			6070
Deduct for self weight			
1 x 1 x 0.5 x 2400	1200	1/2	600
∴ Maximum bending moment for toe slab			5470

Hence, for one metre strip of the toe slab, the maximum bending moment = 5470 kg.m.

Effective cover to reinforcement of toe slab = 6 cm.

∴ Effective depth = 50 - 6 = 44 cm.

∴ $A_t = \frac{5470 \times 100}{1400 \times 0.87 \times 44} = 10.21 \text{ cm}^2$

∴ Spacing of 16mm. diameter bars $= \frac{2.01 \times 100}{10.21} = 20$cm centres

$\therefore$ Spacing of 20mm. bars $= \frac{3.14 \times 100}{22.2}$ = 14cm. centres

Check for sliding

Total horizontal earth pressure per metre length

$$= C_p \frac{WH^2}{2}$$

$$= \frac{1}{3} \times 1900 \times \frac{6^2}{2} = 11400 \text{ kg.}$$

Assuming $\mu = 0.6$.

Maximum possible friction

$$= 0.6 \times 29720 \text{ kg.}$$

$$= 17832 \text{ kg.}$$

Factor of safety against sliding

$$= \frac{17832}{11400}$$

$$= 1.56$$

oooooooooo

CHAPTER

ANALYSIS OF THE FRAMED STRUCTURE

INTRODUCTION :-

The multistorey framed structure considered for this project is a three storeyed structure. Multistoreyed frames are multiparalled net work of beamsand columns which are built monolithically and rigidly with each other at their junctions. All members of such frames are continuous at their ends. Framed structure distributes the load more uniformly to all the members because of continuity and eliminates excessive effects of localised loads.

Analysis of frame is to compute the moment, etc., in the beams and columns. Rigourously, the same should be analysed as a three dimensional complex hyper static structure. The analysis of such frames under different conditions of vertical and horizontal loads is very cumbersome and time comsuming. Hence the analysis is done by considering the framed structure as two plane frames vertical member and neglecting the interaction between them.

Either limit analysis or elastic analysis can be used for the two dimensional fram analysis I.S. Code does not provide anything regarding limit analysis since connected parameters are still under investigation. Hence elastic analysis is done using the 'Moment distribution' Method. The analysis for dead load is done for the entire same using single cycle moment distribution.

The analysis for live loads is done by choosing suitable substitute frames. As the height of the frame is lessthan twice the least lateral dimension ofthe building in plan; the effect of wind loads is neglected.

PRELIMINARY DIMENSION OF BEAMS

In making an elastic analysis of a structural frame work it is necessary to know the cross sectional dimensions of the members, so the moments of interia, stiffness and distribution facters can be calculated. So a preliminary estimate of members of sizes must be one of the first step in the analysis.

In this connection it is worth that structural frame analysis is concerned with relative stiffness only, not the absolute stiffness. To calculate the distribution factors, it is enough if relative stiffness is known andtherefore moment of inertia has been calculated for beam and column considering the entire concrete neglecting steel as specified in table 64 of Reinforced concrete designer's Hand book by Reynolds.

In buildingframes, beam sizes are usually governed by relative months and shears at supports where their effective section is rectangular. The total load coming on the beam is

(i) It's self weight (ii) Load from slab as per I.S. 456.

(iii) Loads from walls supported by the beam.

To arrive the preliminary dimensions, the following points are to be kept in mind.

(i) Depth of beam - L/15 to L/20 or based on approximate B.M.

(ii) Width of beam ½ to 2/3 of depth (20 to 300 cm)

CALCULATION OF MOMENT OF INERTIA

Beams at the centre of its span will have either tee-section or ELL- section. But at the support every beams will have only rectangular section. The moment of inertia of beams are calculated by assuming that the beams would have either tee section of ELL section through out its length.

For Tee beams and ELL Beams, the amount of inertia is calculated using the table 64 given in Reinforced concrete design Hand Book Reynolds. The table is given separatedly.

For Column the moment of inertia is calculated using the formula $1/12\ bd^3$ calculations for the same are shown.

The corresponding stiffness factors 'K' in I/L (Where I = moment of inertia, L = Span) are also shown is the tables below. The stiffness factor is used to calculate the distribution factor.

Span = 500 cm

B is least of

i) 1/6 x span = 1/6 x 500 = 83cm

ii) b_r + 0.5 clear distance between ribs

= 25 + 0.5 x 400 = 225 cm.

iii) $b_r + 4d_s$ = 25 + 4 x 15 = 85 cm.

BEAM A'_2 B'_2

b_r / B = 25/83 = 0.3

d_s / D = 15 / 45 = 0.33

C = 0.134

Moment of inertia about xx = $c\, b_r\, D^3$

$= 0.134 \times 25 \times 45^3$

= 305268.75 cm^4

BEAM D'_6 E'_6

Moment of Inertia xy = $1/12\ bd^3$

$= 1/12 \times 310 \times 40^3$

= 2047083.3 cm^4

PRELIMINARY DIMENSIONS OF BEAMS

Design data : Mix used $= M_{200}$

Steel used = Tor steel

$\sigma_{st} = 2300\ kg/cm^2$

Notations used :-

b = breadth of beam

d = effective depth of beam

m = modular ratio

t = tensile stress in steel

n = depth of neutral axis

a = lever arm

L = span of beam.

MOMENT OF RESISTANCE OF BALANCED SECTION

Modular ratio, $m = \frac{2800}{3 \sigma_{cb}} = \frac{2800}{3 \times 70} = 13$

From the strain stress diagram, $\frac{c}{t/m} = \frac{n}{d - n}$

Therefore $= \frac{c\ (d-n)}{t/m} = \frac{70\ (d - n)}{2300/13}$

$= \frac{910(d-n)}{2300}$

$n + \frac{910n}{2300} = \frac{910d}{2300}$

$n = \frac{910d}{3210} = 0.283\ d$

Leave arm $= a = d - n/3 = d - \frac{0.283d}{3} = 0.906d.$

Moment of resistance of the section.

$$= \frac{\bar{c}}{2} \cdot b\,n \left(d - \frac{\bar{n}}{3} \right)$$

$$= \frac{\overline{70}}{2} \times b \times (0.286d) \left(\frac{d-0.0286d}{3} \right)$$

$$= 9.06\ bd^2$$

The rest of the prime Preliminary Designs of Beams are shown by means of framed sketches.

*****

PRELIMINARY DESIGN OF COLUMNS

The Columns are the common vertical members for two sets of mutually perpendicular plane frames. Hence it is subject to moments due to the plane frames which may be uniaxial or biaxial in addition to the axial load from the floors above. It is important to consider the above facts for fixing up the preliminary dimension of the Column.

Preliminary size of columns is mainly governed by the axial loads which is predominant at the lower floorsand moments are taken in to account for fixing up the sizes at top floors. The column dimension may be fixed depending on the loadingand spacing of the column. For fixing up the size, the direct load coming above a certain section of the column is computed from the loaded area and to account for moment, the axial load is multiplied by suitable factors depending on its position with reference to plans and elevation which is given below. (REYNOLD'S HAND BOOK & Page 228)

EQUIVALENT DIRECT LOAD ON COLUMN

Type of Column	Top	Top [illegible]	Lower
Intermediate	1	1.0	1.0
End	4.5	2.0	1.4
Corner	6.0	2.3	1.8

The cross section of the column can be obtained by dividing the load by direct stress. While fixing up the preliminary sizes it is important the greater dimension along the plane of greater moment.

PRELIMINARY DIMENSIONS OF COLUMN

Column A'$_6$ A'$_5$

Load from folded plates	= 69000 kg.
Load from Beam	= 270
Self weight of column	= 6927
	76197

Equivalent axial load	= 76197 x 18
	= 42331.66
Arearequired	= $\frac{42331.66}{50}$ = 846.33
	= 30 x 30 cm.

Column A'$_5$ A'$_4$

1	Load from upper column	= 76197 kg.
	Load from slab	= 5955
		82152
	Self weight of column 10%	= 8215.2
		90367.2

Equivalent axial load	= 90367.2 x 1.8
area required	= $\frac{162660.36}{50}$
Size	= 55 x 55 cm.

$A'_4 A'_3$

Load from top = 90367.2

Self wright of col.10% = 9036.72

99403.92

required area = $\frac{99403.32}{50}$

= 55 x 55 cm.

$A'_3 A'_2$

Load from top = 99403.92

Self weight = 9940.392

109344.312 kg.

Area required = $\frac{109344.31}{50}$ = 2186.9 cm^2

Size = 55 x 55 cm.

Column $E'_6E'_5$

Load from top = 73680

Self from top = 7368

81048

Equivalent axial load = 81048 x 1.8

= $\frac{145886.4}{50}$ = 2917.79 cm^2

Size = 50 x 50 cm^2

Equivalent axial load = 1177.6 x 2

$= \frac{2355.2}{50} = 47.104$

Provide 15 x 15 cm.

<u>Col. J_3 J_2</u>

Load from top = 1070.6

Load from slab = 5996

7066.6

Self wt. = 706.6

7773.2

equivalent axial load = 7773.2 x 1.2

area $= \frac{9327.84}{50} = 310$

Size = 20 x 20

<u>Col. J_2 J_1</u>

Load from top = 7773.2

Load from slab = 12856.0

20629.2 kg.

Self wt. = 2062.92

22692.12 kg.

Area required $= \frac{22692.12}{50} = 453.84$

Size = 25 x 25 cm.

R.L. = I

Joint	A_4			B_4			
Member	$A_4 A_5$	$A_4 B_4$	$A_4 A_3$	$B_4 B_5$	$B_4 A_4$	$B_4 B_3$	$B_4 C_4$
D.F	0.375	0.298	0.331	0.275	0.22	0.224	0.262
F.E.M.	—	-59	—	—	59	—	—
Bal.	22.125	17.582	19.529	-16.225	-12.98	-13.216	-15.458

Three cycles over. Final results

TOTAL	24.921	-46.653	21.9367	-18.9057	53.6162	-15.3995	-18.0114

R.L.-I

Joint		A_3				B_3	
Member	$A_3 A_4$	$A_3 B_3$	$A_3 A_2$	$B_3 B_4$	$B_3 A_3$	$B_3 B_2$	$B_3 C_3$
D.F	0.02	0.964	0.02	0.009	0.449	0.008	0.534
F.E.M	—	-59	—	—	59	—	—
Bal.	1.18	56.876	1.18	-0.531	-26.491	-0.472	-31.506

Three cycles over. Final results

TOTAL	1.3726	-2.8306	1.3726	-0.8444	51.6959	-0.7506	-50.101

R.L.- I

Joint	C_3				D			
Member	C_3C_4	C_3D_3	C_3C_2	C_3B_3	D_3D_4	D_3C_3	D_3D_2	D_3E_3
D.F	0.0042	0.551	0.007	0.455	0.006	0.42	0.3	0.3
F.E.M.	—	-81	—	—	—	+81	—	—
Bal.	0.3402	43.01	0.567	36.855	-0.486	-34.02	-24.3	-24.3

Three cycles over. Final results

TOTAL	0.4305	-48.0843	0.7177	46.6467	-0.6161	62.0658	-32.103	-32.103

R.L.-I

Joint		C_4				D_4		
Member	$C_4 C_5$	$C_4 C_4$	$C_4 C_3$	$C_4 B_4$	$D_4 D_5$	$D_4 C_4$	$D_4 D_3$	$D_4 E_4$
D.F	0.253	0.281	0.225	0.241	0.263	0.292	0.234	0.240
F.M.M	—	-70	—	—	—	70	—	—
Bal.	17.71	19.67	15.75	16.87	-18.41	-20444	-17.01	-14.77

Three cycles over. Final results

TOTAL	20.653	-58.7106	18.3726	19.6751	-21.6364	54.668	-19.6474	-17.148[illegible]

R.L - I

Joint	I_2				J_2		
Member	$I_2 I_3$	$I_2 J_2$	$I_2 I_1$	$I_2 H_2$	$J_2 J_3$	$J_2 I_2$	$J_2 J_1$
D.F	0.05	0.45	0.06	0.44	0.08	0.8	0.12
FEM	—	-197	—	—	—	197	—
Bal.	9.85	88.65	11.82	86.68	-15.76	-157.6	-23.64

Three cycles over. Final results

Total	14.6765	-161.4495	16.548	129.1532	-20.7217	51.811	-31.0866

-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-X-

R.L.-II

Joint	A'_5			B'_5			
Member	$A'_5 A'_6$	$A'_5 B'_5$	$A'_5 A'_4$	$B'_5 B'_6$	$B'_5 A'_5$	$B'_5 C'_5$	$B'_5 B'_4$
D.F.	0.36	0.17	0.47	0.3	0.14	0.16	0.4
F.E.M	—	-59	—	—	59	--	--
Bal.	21.24	10.03	27.73	-27.7	-6.26	-9.44	-23.6

Three cycles ober. Final results

TOTAL	22.8564	-52.6889	29.84	-19.315	55.354	-10.2994	-26.7404

R.L.-II

Joint		A'_3				B'_3		
Member	$A'_3 A'_4$	$A'_3 B'_3$	$A'_3 A'_2$	$B'_3 B'_4$	$B'_3 B'_3$	$B'_3 B'_2$	$B'_3 C'_3$
D.F	0.57	0.004	0.43	0.57	0.0008	0.43	0.0009
F.E.M.	—	-59	—	—	59	—	—
Bal.	33.63	0.236	25.37	-33.63	-0.0472	-23.37	-0.0531

Three cycles over. Final results

TOTAL	33.765	-58.527	25.4715	-33.697	59.071	-23.421	-0.05322

RLL.-II

Joint		C_5'				D_5'		
Member	$C_5' C_4'$	$C_5' D_5'$	$C_5' B_5'$	$C_5' C_2'$	$D_5' D_{[illegible]}'$	$D_5' C_5'$	$D_5' E_5'$	$D_5' D_2'$
D.F	0.57	0.0006	0.0009	0.043	0.57	0.001	0.0008	0.43
F.E.M	—	-59	—	—	—	59	—	—
Bal.	33.63	0.04[illegible]2	.0.486	25.3[illegible]	-33.63	-0.0649	-0.0472	-25.37

Three cycles over. Final rsults

TOTAL	33.649	-58.9852	0.0486	25.384	-33.643	58.9587	-0.04722	-25.3801

DESIGN OF BEAMS

Introduction

In general the beam members of a multistoreyed frame are subjected to bending moment, axial force and shear force. But usually the effect of axial force will be very small compared with bending moment. So the beam members are designed for bending and shear.

They are designed by ultimate strength theory as per I.S:456 recommendations, and with the aid of charts and tables given by M/s. V.K. Shankar, R.Chandra, and S.Sarkar, the hand book for ultimate strength design of concrete members. The design ultimate bending moment have been faten from thesuitable substitute frames.

ASSUMPTIONS

1. Plane sections normal to the axis remain plane after bending.
2. Tensile strength of concrete is neglected.
3. At ultimate strength, stresses and strains are not proportional and the distribution ofcompressive stress in a section subject to bending is non-linear.
4. The maximum strain at the extreme edge fo the concrete compression zone at ultimate strength may be assumed equal to 0.003.

5. Maximum fibre stress in concrete does not exceed 0.68 c_{cu}.

DESIGN PROCEDURE (Using tables)

a) The value of K (not to exceed 0.185 c_{cu}) is obtained knowing thesize of the beams. The value of K isobtained from the equation

$$K = \frac{M_u}{Bd_1^2}$$

b) For the above 'K' value and known value of (c_{cu}) the value of P_t isread from the table.

FRAME R.L.- I

T A B L E

Beam No.	L.L. -ve B.M. at Centre	D.L. -ve B.M. at centre	Simpley suported D.L.B.M. at centre	S.S. L.L. B.M. at centre	Total D.L. B.M.at Centre	Total L.L. B.M.at centre	Design Ultimate moment 1.5 D.L B.M. + 2.2 L.L B.M.	Section Size (cm.)
1.	2.	3.	4.	5.	6.	7.	8.	9
A_2 B_2	-8057	-2567	4321	1968	1754	911	4635.2	25x30
B_2 C_2	-1702	-3398	7798	4367	4390	2665	12448	25x30
C_2 D_2, D_2E_2	-892	-3199	7334	1968	3235	976	8500	25x35
D_2 E_2, E_2F_2	-1081	-4489	7334	1968	2945	887	6369	25x35
G_2H_2, H_2I_2, I_2J_2	-1133	-6580	16002	4367	9422	3234	21247.8	25x30
E_3F_3, A_3B_3, D_3E_3	---	-3616	4138	---	522	---	783	30x30
B_3C_3, C_3D_3, G_3H_3, H_3I_3, I_3J_3	---	-3950	4238	---	188	---	282	30x30

Cont....

FRAME R.L. - II

TABLE

1.	2.	3.	4.	5.	6.	7.	8.	9.
$A'_3 B'_3$, $A'_4 B'_4$, $D'_4 E'_4$, $E'_3 F'_3$, $E'_4 F'_4$, $A'_6 B'_6$, $B'_3 C'_3$, $B'_4 C'_4$, $G'_3 H'_3$, $G'_4 H'_4$, $G'_6 H'_6$, $F'_5 G'_5$, $F'_4 G'_4$, $I'_3 J'_3$, $I'_4 J'_4$, $I'_6 J'_6$, $C'_3 D'_3$, $C'_4 D'_4$, $C'_6 D'_6$, $H'_3 I'_3$, $H'_4 I'_4$, $H'_6 I'_6$, $D'_3 E'_3$	—	11.25	—	—	11.25	—	—	30 x 20
All the beams provided for bracing purpose.								
$A'_2 B'_2$, $A'_5 B'_5$, $D'_2 E'_2$, $D'_5 E'_5$, $E'_2 F'_2$, $E'_5 F'_5$.	118	—	—	—	718.625	—	836.625	30 x 25
$B'_2 C'_2$, $G'_2 H'_2$, $G'_5 H'_5$, $B'_5 C'_5$	94	—	—	—	466.917	--	608	30 x 20

Cont......

DESIGN OF COLUMNS

INTRODUCTION :

Concrete compression members whose un supported length is more than three times the last dimension of the cross section are classified as columns. Here the columns are designed both for axial loads and bending moments using ultimate strength design.

Ultimate strength design of a rectangular sectionss subjected to axial compression and bending in one plane according to the code is based on the following assumptions.

a) The depth of rectangular stress block 'a' is to be 0.75 times the depth of the netural axis subject to the maximum value equal to the depth of the corss section.

b) If a is less than $0.5d$, the average compressive stress across the rectangular stress block is taken as $0.55\ c_{cu}$.

c) If a is greater than $0.5d_1$, the average compressive stress acrossthe rectangular stress block isreduced such that the resultant moment of compression about the centroid of tension reinforcement or the reinforcement in least compression always remains equal to the moment obtained for $a = 0.5d_1$.

d) Stress in steel should be equal to the appropriate strain in steel times E_s or c_{sy} which ever is less.

e) The maximum strain in concrete is assumed to be 0.003.

f) The capacity of the section for combined axial compression and bending moment is determined from the following conditions.

i) P_u = algebraic sum of the forces in concrete and steel.

ii) Moment due to external force P_u about any reference line = algebraic sum of the moments of forces in concrete and steel about the same reference line.

<u>DESIGN PROCEDURE :</u> (Using charts and tables)

I. <u>Column subjected to uni axial bending andaxial compression</u>

i) For the assumed column size the value of $\frac{M_u}{\sigma_{cu} bd^2}$

and $\frac{P_u}{\sigma_{cu} bd}$ are calculated.

ii) Using the chart with the appropriate cover ratio $\frac{d_1}{d}$,

obtain $\frac{r}{\sigma_{cu}}$ for the point locating $\frac{P_u}{\sigma_{cu}}$ and $\frac{M_u}{\sigma_{cu} bd^2}$

as obtained in step (i) For the values of cover ratios different from those given in the charts, the value of $\frac{r}{\sigma_{cu}}$ may be linearly interpolated.

iii) If the percentage of steel so obtained is not suitable, the size of the member should be charged and the process repeated.

<u>Design of column by ultimate strength design using Chandra and Chakr</u>

Total load on column	28,176
say	= 28.5 ton
Max. B.M.	= 17941 kg.m.
	= 1.8×10^6 kg cm.

$B = 40$ cm

$d = 35$ cm

$d_2 = 4$ cm

Cover ratio $d_1/d = 31/35 = 0.896$ say 0.90

$\sigma_{cu} = 200$ $\sigma_y = 2600$ kg/cm^2

$$\frac{P_u}{\sigma_{cu} bd} = \frac{28500}{200 \times 40 \times 35} = 0.136$$

$$\frac{M_u}{\sigma_{cu} bd^2} = \frac{1800000}{200 \times 40 \times 35^2} = 0.245$$

Referring chart No.10 in ultimate hand book by Chandra&Sarkar

$$\frac{P}{\sigma_{cu}} = 2.15 \times 10^{-4}$$

Area steel required $= 200 \times 2.15 \times 10^{-4} \times 30 \times 35$

$= 45.15$ cm^2

$A_{sc} = A_{st} = 22.575$ cm^2

Provide equal rods on both sides

Stirrups Adopt $= \frac{0.2 \times 35 \times 30}{30} = 7$ cm^2

Provide 10 mm ⌀ @ 10 cm c/c.

DESIGN OF FOOTINGS

Footings for column $A'_3A'_2$

Size of column	30 cm x 40 cm
Axial loaded	1,09,344 kg.
Self wt. of footings	1,0934.4
Total load	1,20,278.4 kg.

Assuming Bearings capacity of soil 20t / m^2

Area of footing $= \frac{1,20,278.4}{20,000} = 6.1\ m^2$

Let length of footing be L metres

Breadth B $= \frac{6.1}{L}$

Equating the projection beyond column tables

$$\left(\frac{6.1}{L} - 0.3\right) = (L - 0.4)$$

$$6.1 - .3L = L^2 - 0.4\ L$$

$$6.1 = L^2 - 0.L$$

$$L^2 - 0.L - 6.1 = 0$$

$$L = 2.5\ m$$

$$B = \frac{6.1}{2.5}\quad 2.45\ m.$$

Projection beyond column faces $= \frac{2.5 - 0.42}{2} = 1.05$

$$8.7\ bd^2 = 2410$$

$$d = \sqrt{\frac{2410}{8.7 \times 1.15}}$$

$$= 16\ cm.$$

Over all depth = 25cm

$$A_{st} = \frac{241000}{0.87 \times 1400 \times 16}$$

$$= 12.36$$

Provide 12mm Ø rods 10 nos. per meter on the bothe direction.

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ACOUSTICS OF AUDITORIA

Introduction:

In designing auditiorium and theatre, controlling the sound (usually known as the 'Principle of acoustics') is very important, and hence some of the fundamental principles of acoustics and the design of auditoria based on them is presented here.

Acoustics is a science which deals with production, transmission and absorption of sound in buildings. It also includes the sound control and dissipation ofexternal noise. Second is transmitted in the form of spherical waves which are a series of compressions and rarefactions. Created in the medium through which it travels. At normal temperature and pressure sound travels with a velocity of 340 m/sec.

Characteristics of Audiable sound:

Frequency:

The frequency is defined as the no. of vib/sec. Frequency of vibrations known as pitch of the sound. The highest audiable pitch is 20000 cycles/sec. and the lowest is 20 cycles/Sec.

Intensity:

Intensity of sound in specified direction is defined as the flow of sound energy/sec/unit area. If I_1 and I_2 are the intensities of two sources of sound then the ratio of other intensities can be expressed as $N = \log_{10} (I_1 / I_2)$ bels. In practise it is very convenient to express it in terms of deciblels (d_B) which are length of bels on the same log scale $N = 10 \log_{10} (I_1 / I_2) \, d_B$.

It has been found that sound travels much faster than in other medium than in air. Eg. the velocity propagation can be expressed in air, water, brick, concrete, glass, aluminium, steel are 340, 1420, 3900, 3300, 4100, 5100, 5200 m/sec. respectively.

The velocity propagation of sound in Various Media:

When the sound is produce either as a speech or music it is transmitted from the source in all directions. It travels through till it impinges on some surface and from there it will be neglecte partly, partly obsorbed and transmitted to the other side. The amount of reflection, absorption and transmission depends upon the characters was and wall surfaces. For example a hard and smooth wall reflects most of the sound so that little amount is transmitted and absorbed where as a pourous wall, the absorption and transmission is greater than reflection. The reflection soundhas certain virtues in acoustics such as enrichment of total quality and loudness if it is properly controlled otherwise it results in acoustic defects such as echoes, dead spots etc.

Echoes:

An echo is produced when a reflection sound wave coming from the same source reactes the ear just when a direct Second

$t = 0116v / axs$

Where t = Reverbaration time in seconds.

0.16 = A Constant for standard loudness.

v = Volume of the room in cum.

axs = Total absorption power is Sq.metre of all the surfaces and objects present in the room.

The factor axs is determind by adding absorption of the various surface in theroom.

Thus $a \times s = a_1 s_1 + a_2 s_2 + a_3 s_3 + . \; . \; . \; . \; .$

Where S_1 , S_2 , S_3 v$...... = Surface areas.

a_1 , a_2 , a_3 .... = Absorption Coefficient

reverbaration time varies from hall to hall according to the usage.

Auditorium used for	Acceptable Reverbaration time in seconds.
Speech	1 to 1.5
Music	2 to 2.5
Music & Speech	1.5 to 2
Cinema	0.6 to 1.2

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Acoustics on shell roofs:

Acousties of shell roofs doesnot usually, reveive the attention it deserves at the planning stage. The result is that expensive corraction measures are usually found necessary after the building is comisioned. Acoustic treatment of shell roofs is job for a

A C O U S T I C D E S I G N

To control noise, feed back etc. We have design the auditomium for acoustic conditions. For that many factor taken in account, which are mentioned above topics. The acoustic design procedure is given below.

FLOOR PLAN

For acoustical condition the walls are constructed as hexagon in which front side (screen) and back side walls are length other four sides have same length angle of contact of these each two are very large. For this same condition the floor area is taken into account.

GROUND FLOOR

Ground floor area = 2 (35 x 2x1/2) + (35 x 21)

= 685 m^2

Area provided for each pattrons are = 0.5 m^2

No. of seats available = 685 / 0.5 = 1370

Due to convenient arangement of seats and gang way only 1252 seats are provided. So the floor area is more than required.

BALCONY

Area of balcony $= \frac{1}{2} \times 8 (21 + 23)$

$= 176\ m^2$

Area provided for each pattrons $= 0.7\ m^2$

No. of seats available $= \frac{176}{0.7} = 251$

According to accomodation seats provide = 240

Hence the area is adequate.

SEATING ARRANGEMENTS

The seating arrangements are shown in detailed drawings. The seats are provided forthe distance between row to row of 100cm. from one seat to anotherand seat to seat is 50cm. The interior Gang way is given to a minimum width of 1.5m.

GROUND FLOOR

The groundfloor seating arangemests are done 4 group according to the weightage of classes. It consists of 3 columns and 4 rows. The first row group contians 7 rows. The second row group contains 8 rows. The third row group contians 8rows. The fourth row group contains 5 row. The total no. of seatings are shown in the table.

SOUND ASBSORBING AREA

The area is calculated and tabulated. The amin oretirea is the theatre may not be full for all the shows, so the optissmum strength is taken as 1000 patterns.

No.	Particulars	Area m^2	absorption Coeff.	Sound absoring area m^2
1.	Stage Curtian (Screen)	144	0.23	23.12
2.	Concrete floor	1014	0.03	30.42
3.	Acoustic plaster on folded roof	1100	[illegible]	330.00
4.	Acoustic plaster on side walls	568	0.30	170.40
5.	Plaster on walls	368	0.02	7.36
6.	Audience	1000Nos.	0.44	440.00
7.	Audience un occupied	500Nos.	0.40	201.60
8.	Chairs and upholster, seats	742	0.16	118.72
9.	Doors Contains (if oppend Coeff.=1)	32	0.23	7.36
		Total area		= 1358.06

Area provided to absorb the sound = $1358.06 m^2$

Area required to absorb the sound = $951.38 m^2$

So the are a provided is enough.

Anyhow the beside the main speaker the speachers are also provided so provision of glass-wool fibres on the top of balcony backwallis necessary to avoid reflection. The balcony rail is provided by means of steel angles. Front side of the pre-stressed beam provide gloss-wool fibres to prenvent reflection of soundwaves.

Hence the theatre is acoustically designed.

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ESTIMATION AND COSTING

The estimation and costing are done in detial for the Pre-Stressed Concrete folded plate.

Where other items are done in not so detailed manner.

S.No.	Qty.	Description of work	Rate	Amount Rs.nP.
(1)	(2)	(3)	(4)	(5)
1.	5544 m^3	Earth work excavation for foundation and under ground car parking.	$5.00/m^3$	27,720.00
2.	1584 m^3	Earth filling	$2.00/m^3$	3,168.00
3.	584.3m^3	Retaining walls Including .2% steel	$300.00/m^3$	1,75,297.50
4.	396 m^2	Under ground floor finishing	$40.00/m^2$	15,840.00
5.	180 m^3	Ground floor slab with 2% steel	$350.00/m^3$	63,000.00
6.	67.2m^3	Brick work forthe theatre walls, under supporting the slab excluding the gap for doors and ventilators	$240.00/m^3$	16,128.00
7.	180 m^3	First floor slab with 2% steel	$360.00/m^3$	64.800.00
8.	34.5m^3	Balcony slab	$380.00/m^3$	13,110.00
9.	284.92m^3	Pre-stressed concrete folded plate (31 mt span) including the pre-stressing charge	$500.00/m^3$	1,42,460.50
10.	1.5 t	Steel required for folded plate High tensile steel taking sp.wt.=7.85 gm/cu cm taking 0.7cm dia of 12 cables of 12 No.	6000.00/t	9,000.00
11.	0.75t	Steel required for folded plate of 38 mt span 0.7cm dia of 12 cables of 20 No.	6000.00/t	4.500.00

(1)	(2)	(3)	(4)	(5)
12.	$105.36 m^3$	Prestress concrete folded plate of 38 mt. span including prestress charge	$500.00/m^3$	51,680.00
13.	1.5tons	Ordinary - Non-prestressed steel required for both the plates	3600.00/t	5,400.00
14.	$41.4m^3$	P.C. Beams of span 23 mt. and 21 mt.	$450.00/m^3$	18,630.00
15.	$24 m^3$	Rcc slab of 80 mt. length of 0.15 mt. thickness.	$3600.00/m^3$	8,640.00
16.	2.6/t	Steel required for pre-stressing	6000.00/t	15,600.00
17.	0.5/tq	Non-pre-stressing steel required for articulation and shear.	4000.00/t	2,000.00
18.	$200m^3$	Size of column 40x40 50 numbers.	$360.00/m^3$	72,000.00
19.	$50.4m^3$	28 No. of 30 x 30 size column	$360.00/m^3$	18,144.00
20.	$43.75 m^3$	35 No. of 25 x 25 size column	$360.00/m^3$	15,750.00
21.	46 t.	The total steel required for the columns.	3600.00/t	1,66,600.00
22.	$29.115m^3$	Total volume for all footings.	$360.00/m^3$	10,481.00
23.	$873.45m^3$	Total volume of all the beams	$360.00/m^3$	3,14,442.00
24.	136 tons	Steel required	3600.00/ton	4,93,674.00
			Rs.	17,28,064.50

Total cost of building Rs.17,28,064.50

Total area of building $= 2704 m^2$

Plinth area estimation $= \frac{1728065.0}{2704} =$ Rs.$638/m^2$

BIBILIOGRAPHY

1. A.S.C.E. Manual for Sheel Roofs
   33 West 39th Street, New York -18.
2. CHAS.E. Reynolds : Hand Book for Structural Engineers.
3. SARKAR AND CHANDRA : Hand Book for Ultimate Strength Design.
4. P.D. ARTHUR and K. RAMAKRISHNAN : Ultimate Strength Design.
5. U.N.VAZIRANI and M.M. RATWANI : Concrete Structures.
6. R. RAMAMURTHAM : R.C. Structures.
7. B.C. PUNMIA : R.C. Structures.
8. Acoustic Design - I.S. 2526 : 1963
9. KINSLER and FREY Fundamentals of Acoustics.
10. DUTTA - Estimating and Costing.
11. THE TAMIL NADU CINEMA AUTOGRAPH RULES AND REGULATIONS 1957.
12. BURRIS, MEYER and GOIE - Theatres and Auditerium.
13. DYANARATHNAM - Pre-Stressed Concrete Structures
    (Shells and Folded Plates)

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