# CAD PACKAGE FOR DESIGN OF GRID FLOORS

**A DISSERTATION**

*Submitted in partial fulfilment of the requirements for the award of the degree of*

MASTER OF ARCHITECTURE
(With Specialization in Building Science and Technology)



*By*

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# CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in this dissertation entitled "**CAD PACKAGE FOR DESIGN OF GRID FLOORS**" in partial fulfilment of the requirement for the award of the degree of **MASTER OF ARCHITECTURE** with specialization in BUILDING SCIENCE AND TECHNOLOGY, submitted in the Department of Civil Engineering, University of Roorkee, Roorkee, is an authentic record of my own work carried out for an effective period of about six months, from September 2000 to February 2001, under the supervision of **Dr. N.M. Bhandari**, Professor, Department of Civil Engineering, University of Roorkee, Roorkee.

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

Place: Roorkee

Dated: 28th February, 2001

Gopesh Sharma.

**(Gopesh Sharma)**

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

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Gopesh Sharma

Gopesh Sharma

# ABSTRACT

A grid floor can be described as a set of coplanar interconnecting ribs spaced at a particular distance in two directions. Grid floors are suitable for large spans (10-20m) and are usually provided in the buildings where top flat surface of the floor is an essential requirement. Grid floor can be supported on walls or columns as per the functional and structural demands.

To analyze and design the grid floors manually is a tedious and time-consuming affair as a number of structural members (ribs/slab) are interconnected. It is also cumbersome to repeat the same design using different parameters. Hence there is a great need to automate the process of designing. A CAD package for design of grid floors has been developed in this dissertation to full fill this demand.

The package is written in Fortran and C language. The software is interactive and user friendly. The input is minimal. It can be easily modified and expanded for future works. The package can help designers to make more rational and efficient design. Some test problems have also been illustrated to check the efficacy of the software.

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# INTRODUCTION

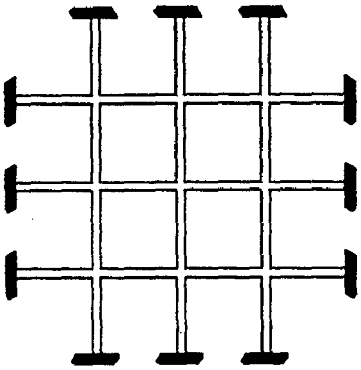
## 1.1 General

The primary requirement of most of the constructions taking place is for the purpose of shelter or for accommodating one or more activities of mankind. As the industrial revolution took place and technology progressed, the requirement for large column free spaces also increased. Usually the primary aim of the structural systems used for covering the space in the interior of the buildings/offices can be summarized as:

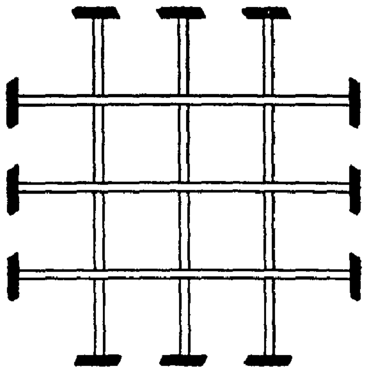
1) To have a flat soffit and flat upper floor.
2) The structural depth of the system should not be excessive so as to achieve good headroom within a reasonable storey height. Though this may not be an essential requirement in very large halls, auditorium, airport lounges, etc. where both head room and structural depth may be secondary as compared to the span achieved. This objective can be achieved in number of ways and each method has its own merits and demerits. Grid floor is one such structural arrangement of orthogonally placed beams cast monolithically with R.C. slab topping which can generally satisfy such requirements especially the first one. Grid floor are generally used in interior of buildings.

## 1.2 DEFINITION

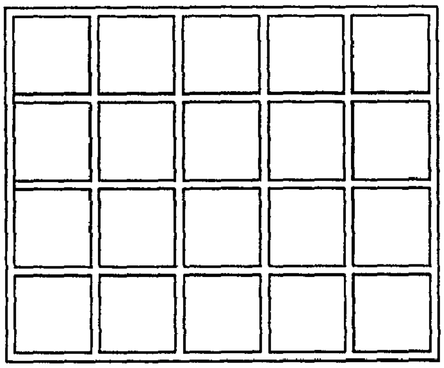
A grid structure can be defined as an assembly of coplanar intersecting beams on a square or rectangular matrix. Grid floor should not be confused with



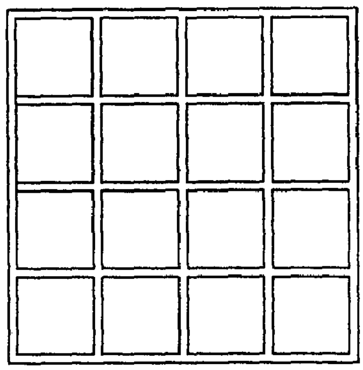
Grillage (Torsion Elements)



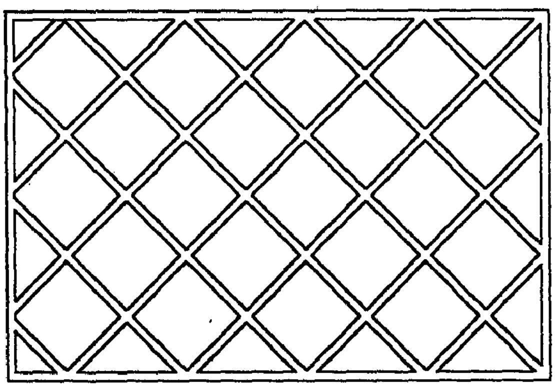
Grillage (No Torsion Element)



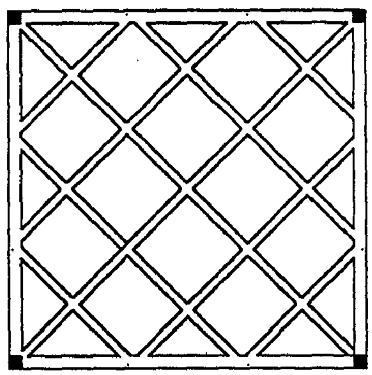
Rectangular Grid



Square Grid



Rectangular Area



Square Area

Diagrids

Fig . 1.1 VARIOUS TYPES OF GRID FLOOR SYSTEMS waffle slabs, which they resemble greatly. Waffle slabs are slabs from which some structurally dispensable material has been removed to reduce the self-weight of the concrete. For a Waffle slab to respond to loading in a manner similar to a solid slab, rather than a series of intersecting beams as in grid floor, the ribs should be closely spaced. Recommendations for such spacing of ribs have been given in IS 456-2000 (clause-30.5).

A grid having no torsional element is called a grillage as shown in Fig. –1.1

When the beams are placed at an angle to the span with equal or unequal inclination in two directions as shown in Fig. -1.1. Then such a structural arrangement is called diagrid. In grid and diagrids all the intersecting beams have rigid connection.

Grids are economical as compared to many structural systems where beams and slab are the primary structural elements. Grids are most efficient when used in square or nearly square floor plans. In rectangular plans the bending moment is primarily in short span and beams in long spans are not efficiently utilized. Structural efficiency is also attempted to be maximized by optimum spacing and cross-sections of the constituting ribs and beams.

The efficiency of grid floor is due to the fact that any super-imposed concentrated loads are distributed to all its members avoiding high local stresses and a fairly even distribution of stress occurs.

## 1.3 STRUCTURAL BEHAVIOUR

Grids can be termed as the mass bearing system in which all the three dimensions are involved in the load transfer mechanism. The typical three-dimensional mechanism is that of resisting of twisting moment. In grid the beams are rigidly connected to each other at the joints so that the whole of the system by its

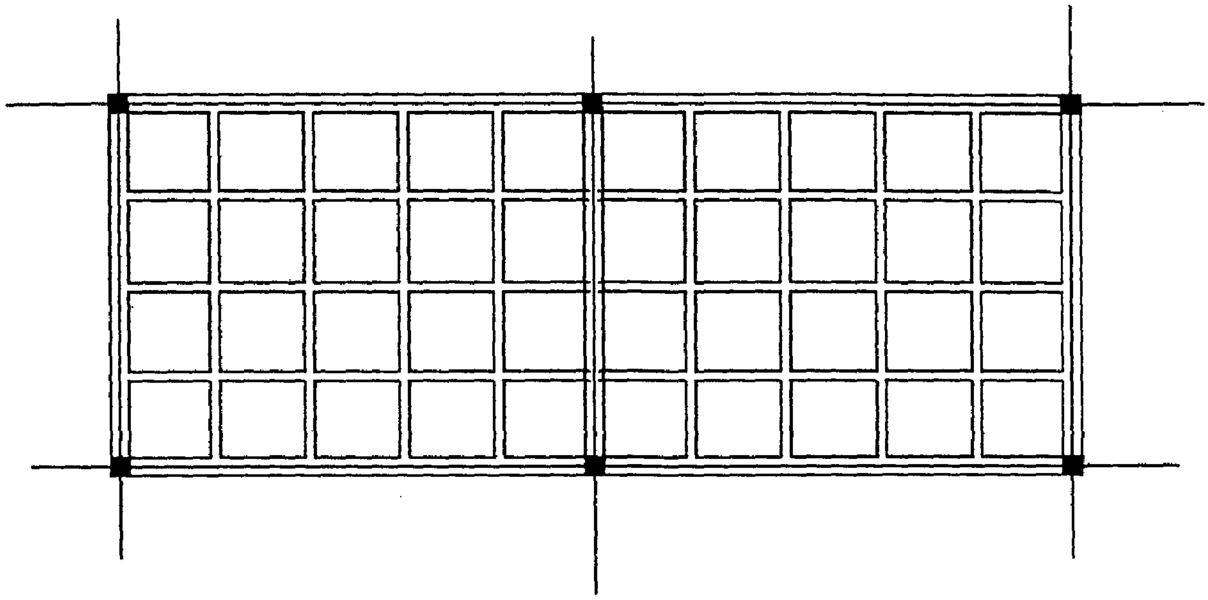


Fig. 1.2 Continuous Grid Floors

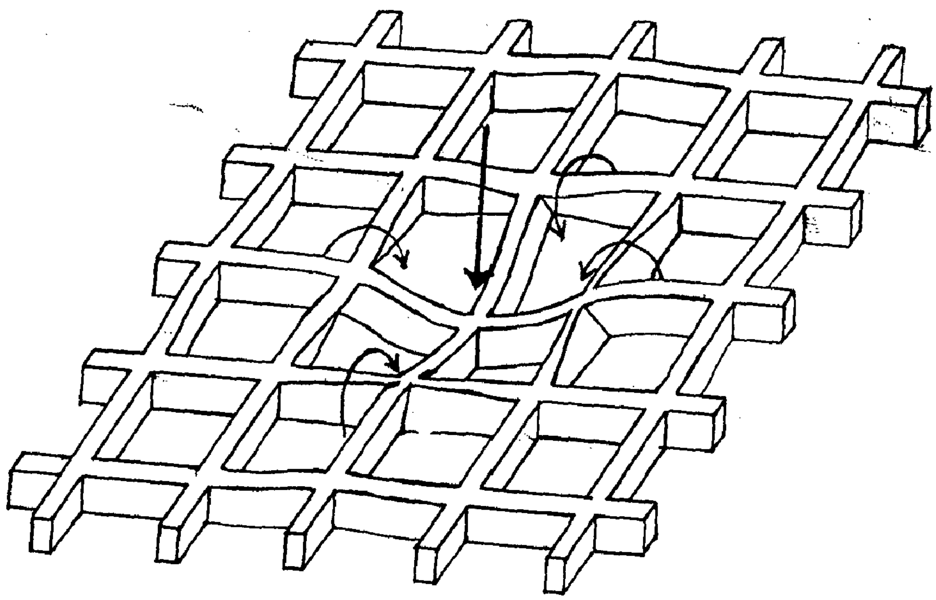


Fig. 1.3 Structural behaviour of Grid Floors

stiffness resists the deformation caused by loading. If a concentrated load (say load of contributing floor area) is applied at the junction of two beams, the junction shall deflect downwards, while forcing the neighboring beams to deform as well,as shown in the Fig – 1.3. In this way load is more evenly distributed to a number of beams. Hence a large part of bearing system resists deformation and solely bending and shear alone do not transfer loads. They are also transferred by twisting, since the load which presses downward at the junction of two beams tends to curve the beams which in turn tends to twist the beams perpendicular to it. The rotation in one beam is equal to the angle of twist in the beam perpendicular to it. Hence there are three unknown displacements at each joint i.e. vertical deflection and rotations about two perpendicular axis.

There are three internal forces i.e. vertical shear force, bending moment and torsional moment at each end of a member. Hence grid structures are statically indeterminate to a high degree of indeterminacy.

## 1.4 NEED OF PRESENT WORK

Since the grids are an efficient system for covering large span. There is a need to automate the process of analysis and design for the same, which will save a lot of hard work and time. Hence in this dissertation a C.A.D. package has been developed for the analysis and design of R. C. grid floors.

## 1.5 OUTLINE OF THE DISSERTATION

Chapter one deals with the introduction of the dissertation and general definitions. Commonly used analysis techniques have been outlined in the chapter two. Chapter three describes the analysis and design principles used in the C.A.D. package. C.A.D. package has been explained in the chapter four. Validation of the program and the methodology used has been presented in chapter five .Few test examples have been solved in chapter six . Conclusions has been presented in chapter seven.

# REVIEW OF LITERATURE

## 2.1 GENERAL

In the early days the structural action of the grid floor was not understood and analysis was done on the basis of grillage action assuming no torional rigidity of the beams. Since 1938, when Hetenyi published his work, much work has been done to develop rational theoretical analysis considering the internal forces.

The method of harmonic analysis by Hendry and Jaeger is based on many assumptions and yields accurate result only for certain type of grid structure. The behavior of grid structure is represented by a set of simultaneous ordinary differential equations. If loading is represented in terms of series function, these differential equations reduce to ordinary linear simultaneous algebraic equations. Methods given by Hetenyi , Hendry and Jaeger give good result only when the transverse beams are very close to each other. These methods do not lend themselves to generalization for an unspecified load positions. With the advent of computers. these methods, now a days have become obsolete.

Analysis of grid structure by equivalent plate method is limited in scope and can give only an overall picture of the stress distribution. Timoshenko realised the mathematical similarity between a plate and a grid, and Genjor developed this concept to grid of zero torsional stiffness. Massonet extended Guyon's work and generalized the method by inducting the effect of torsion. He also gave interpolation formula for the distribution coefficients for any value of torsional parameters. National

Building organization has published a handbook, which provides a method of analysis, which has been translated from original Italian work by G.C. Mathur.

All these methods are useful, but have very limited applicability and it is necessary to use numerical techniques for more general problems. In general these techniques use some approximation to convert the solution procedure to one of assembling and solving a set of linear simultaneous equations. Two such numerical methods have been considered, finite elements and **grillage analogy**.

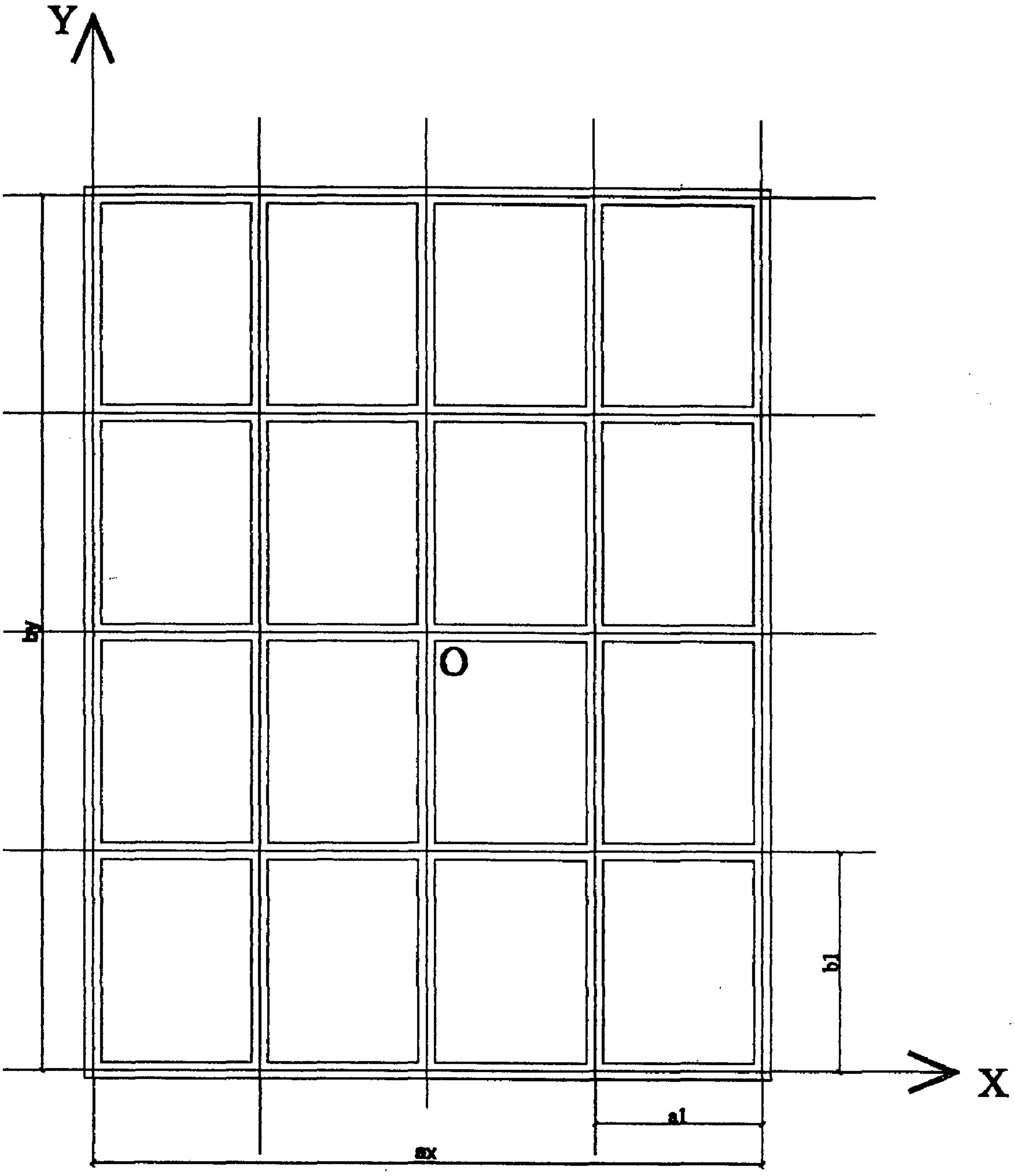
The following methods have been discussed here

1. Orthotropic plate theory
2. N.B.O. Method
3. F.E. Method
4. Matrix Displacement Method

### 2.1 Orthotropic Plate Theory

A reinforced concrete grid floor with ribs at close interval in two mutually perpendicular directions connected by slab in between the ribs can be considered as an orthotropic plate freely supported on four sides. An orthotropic plate has been defined as one, which has different specified elastic properties in two directions. The method of analysis is based on the classical Poisson-Kirchoff assumptions, which are specified as below (14)

i. The material of plate is perfectly elastic and homogeneous.
ii. The thickness of plate is uniform and small in comparison to the other dimensions of the plate.
iii. Deflections of plate are small in comparison with its thickness.
iv. The deflections of the plate are such that there is no normal strain in planes tangent to middle surface.
v. The direction of external load is perpendicular to the plane of the plate.



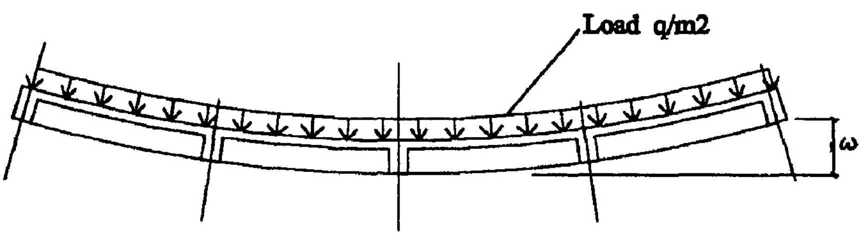




Fig. 2.1 Deflection chracteristics of grid floors

vi. Points of the plate lying initially on a normal to the plate middle-plane remain on the normal to the middle surface of the plate even after bending.

vii. Normal stresses in the direction transverse to the plane of the plate are negligible.

Timoshenko's analysis may be used to evaluate the moments and shears in the grid, which depend upon the deflection surface.

The vertical deflection 'ω' at any point of the grid shown in Fig –2.1 is expressed as, (8)

$$\omega = \frac{16q}{\pi^6}\left[\frac{\sin\left(\frac{\pi x}{a_x}\right)\sin\left(\frac{\pi y}{b_y}\right)}{\frac{D_x}{a_x^4}+\frac{2H}{a_x^2 b_x^2}+\frac{D_y}{b_y^4}}\right]$$

Where q = total uniformly distributed load per unit area

$a_x$, $b_y$ = Length of the plate in x and y direction respectively

$D_x$, $D_y$ = Flexural rigidity per unit length of plate along x and y directions

$C_x$, $C_y$ = Torsional rigidity per unit length of plate along x and y directions.

If $a_1$ and $b_1$ are the spacing of the ribs in x and y direction respectively, then we have the relations,

$$D_x = (EI_1/b_1) \qquad C_x = (C_1/b_1)$$
$$D_y = (EI_2/a_1) \qquad C_x = (C_2/a_1)$$

Where $EI_1$, $EI_2$, $C_1$ and $C_2$ are the flexural and torsional rigidities of the effective section in x and y directions. The moments and shears are computed using following equation:

$$M_x = -D_x\left(\frac{\partial^2\omega}{\partial x^2}\right) \qquad M_y = -D_y\left(\frac{\partial^2\omega}{\partial y^2}\right)$$

$$T_{xy} = -\frac{C_1}{b_1}\left(\frac{\partial^2\omega}{\partial x.\partial y}\right) \qquad T_{yx} = -\frac{C_2}{a_1}\left(\frac{\partial^2\omega}{\partial x.\partial y}\right)$$

$$Q_x = -\frac{\partial}{\partial x}\left[D_x\left(\frac{\partial^2\omega}{\partial x^2}\right)+\frac{C_2}{a_1}\left(\frac{\partial^2\omega}{\partial x.\partial y}\right)\right]$$

$$Q_y = -\frac{\partial}{\partial x}\left[D_y\left(\frac{\partial^2\omega}{\partial y^2}\right)+\frac{C_1}{b_1}\left(\frac{\partial^2\omega}{\partial x.\partial y}\right)\right]$$

Maximum bending moments develop at the center of the span while maximum torsional moments are generated at the corners of the grid and maximum shear forces develop at mid points of longer side supports. This method can be easily used for design and analysis of simply supported grid floors (provided rib spacing is close).

### 2.3 N.B.O. Method:

This method is reported in N.B.O. handbook. This book provides readymade tables to design intergrid and diagrid systems for covering rectangular openings. These have been compiled for various ratios of size of rectangular openings to meet the demand of almost all cases that are likely to occur in practice and are a valuable aid to designers. These tables give the coefficient of deflection function, which obviate time consuming calculations.

Equating the deflections and rotations of the ribs, with the deflection and rotation of the slab, we arrive at an intergrid differential system, which fully expresses the problem. The method followed for the solution of intergrid differential equations in which the elastic problem is expressed, consists in their transformation into linear algebraic equations by substituting the function of the influence of the slab by its development in series of its own characteristic functions, neglecting the higher order terms.

The following six cases have been covered in N.B.O. method

i. Rectangular plates with two diagonal ribs not monolithic with slab.

ii. Rectangular plate with two diagonal monolithic ribs.

iii. Rectangular plates with non-monolithic ribs in two directions at $45^0$ to sides.

iv. Rectangular plates with a double system of monolithic ribs inclined at $45^0$ to the side.

v. Rectangular plates with a double system of non-monolithic ribs and parallel to the sides.

vi. Rectangular plates with monolithic ribs parallel to the sides.

The tables provided in this handbook give influence coefficients '$\alpha$11', '$\alpha$13', '$\alpha$31' for various combinations of beams, In either direction with a maximum of 10 beams in each direction. These coefficients are dependent upon two quantities $\lambda$ and $\mu$ defined as below

$$\lambda = \frac{De}{N} \qquad and \qquad \mu = \frac{B}{N}$$

Where,

| | | |
|---|---|---|
| D | = | Flexural rigidity of the slab $= \dfrac{Et^2}{12(1-\upsilon)^2}$ |
| N | = | Flexural rigidity of the beam |
| B | = | Torsional rigidity of beam |
| e | = | spacing of ribs. |

These table provides the values of the reciprocal of the influence coefficients for $\lambda$ = 0/1/2/3/4, $\mu$ =0/1. For the intermediate values of $\lambda$ and $\mu$ linear interpolation has to be resorted to.

## 2.4 Finite Element Method

In the finite elements methods the deflected surface is represented approximately by piece wise continuous, algebraic interpolation functions. An approximation to the total potential energy of the plate and loading is formulated in terms of displacement variables at a grid of stations covering the plate. A variation principle is than invoked to obtain a set of linear simultaneous equations. The method is capable of handling orthotropy, varying depth, edge beams, holes, irregularly positioned supports, complex plan forms and practical boundary conditions without much difficulty .It is the most powerful and versatile of the numerical techniques available for the analysis and sometimes the only valid technique of analysis.

## 2.5 Structural Idealization

A mathematical model of the structure is formulated in which it is represented as an assemblage of discrete two and three-dimensional members. The elements are connected at nodal points which posses an appropriate degrees of freedom. Each element has finite dimension and properties. In order to perform the subsequent analysis it is necessary to establish the force-displacement relationships of each element. The determination of these relationships forms the second stage of the process.

a. structural analysis of the element assemblage

The usual requirements to be satisfied are as following:

(i) Equilibrium of the internally and externally applied forces at each node of the element.

(ii) Geometric fit or compatibility of element deformations in such a way that they meet at the nodal points in the loaded configuration.

(iii) The internal force-development relationship must be established with each element, as dictated by the existing geometry and material property.

In practical application to engineering problems, this method has a number of limitations as indicated below:

(i) Cumbersome to use, because it needs a time consuming and lengthy data preparation and a lot of time is required for interpretation of the results.

(ii) Expensive, as regards to computer time.

(iii) If the choice of element is incorrect, then the results may be far more inaccurate than those predicted by simpler methods such as grillage method.

(iv) Use of fewer elements within a moment field that changes sign can result in wrong prediction of point of contraflexure.

**2.6 Matrix Displacement Method:**

This method has been used for the analysis of the grid floors in the present study. It has been described in some detail in next chapter.

**2.7 Concluding Remarks**

Choice of analytical method is influenced by suitability and cost .Of all the methods described above the first two i.e. orthotropic plate theory and N.B.O. method can be used in design office without the help of computers.

When complexity and general nature of problem indicates use of numerical methods, the similarity of the structure to a grillage or an orhtotropic plate can be deciding factor. With the increasing efficiency of computers the cost of computing power may not be important. Often the cost of data preparation and processing of results are dominant factors. Generally however the finite element needs considerably more central processor time and length data preparation. The grid floor can easily be simulated as grillage. This method provides satisfactorily detailed results. On all these accounts same method has been used for development of C.A.D.package.

# METHODOLOGY USED FOR DESIGN AND ANALYSIS

## 3.1. CONCEPT OF GRID ANALOGY

Analysis of grillage of beams by stiffness method is a straightforward and economical process. To predict the behavior of a grid floor by grillage analysis it is first necessary to specify the properties and layout of component beams. The accuracy of the solution is largely dependent upon the aptness of this structural modeling.

### 3.1.1 STRUCTURAL IDEALISATION

A grillage member can represent a part of a top flange a single rib with part of a top flange or part of the top flange with several ribs. When a grillage member represents a single rib with a part of a top flange (as in grid floor) it is placed at the center -line of the web (1).

Hence an equivalent grid of beams represents a grid floor. The loads are distributed between longitudinal beams by bending and twisting of transverse beams. The stiffness method is developed on the basis of writing joint equilibrium equations in terms of stiffness coefficients and unknown joint displacements. The dispersed bending and torsion stiffness in every region of the grid are assumed to be concentrated in the nearest equivalent grid members for the purpose of analysis. The longitudinal stiffness of the slab is concentrated in the longitudinal beams while the transverse stiffnesses are concentrated in the transverse beams. The beam stiffness should be such that when prototype slab and equivalent grid are subjected to identical loads, the two structures should deflect identically and the moments, shear forces and torsion in any grid beam should be equal to the resultant of the stresses on the cross-section of the part of the slab the beams represent.

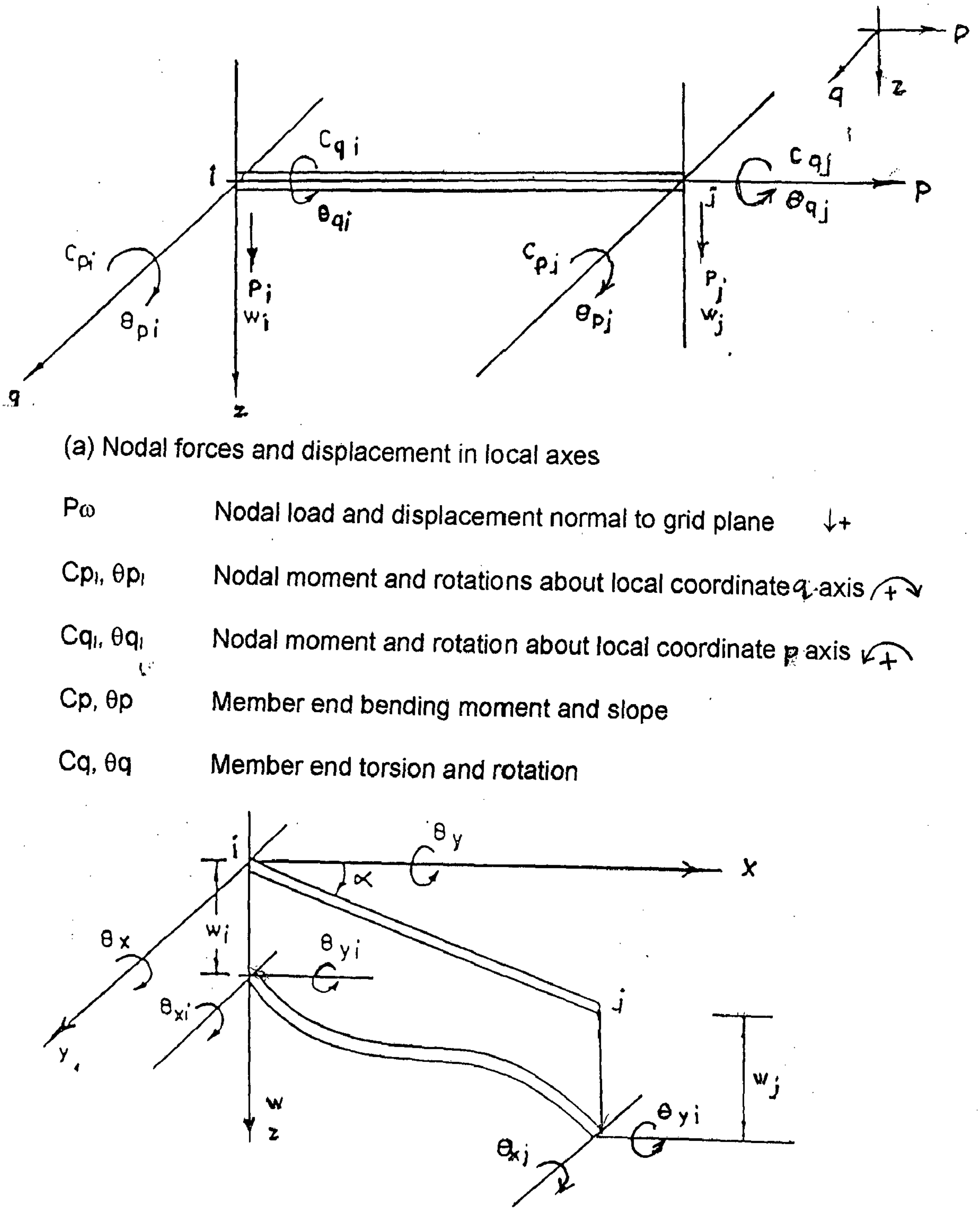
Both force and displacements can be applied at joints of a qrillage. To ensure that the support conditions are represented reasonably at least one grillage member should go to each discrete bearing. It is also recommended that the ratios of any two lengths of the members meeting at a joint should not exceed about five. (ill conditioning of stiffness equations can occur when members of widely differing stiffnesses meet at a joint).

Solution of the stiffness equations provides the joint deflections and rotations. Bending moments are usually calculated from the concentrated bending and torsional couples at end of each grillage members. Loads are applied as concentrated forces and couples at joints. The consistent joint forces from members are equal, but opposite in sign, to its fixed end moments and shear forces.

### 3.1.2 MATHEMATICAL MODEL

A grid or grillage analysis is usually conducted using straight members of constant cross-section (prismatic members), although no special difficulties are presented by the use of curved or varying cross-section members. The stiffness properties of straight member of constant cross section are defined in terms of four parameters these are its flexural modulus EI, its torsional modulus GJ, its length L and its inclination to the co-ordinate axis system. The method of calculating these parameters has been given in appendix A-I.

The deformations considered for element stiffnesses are two orthogonal rotations in the horizontal plane and a vertical deflection at each of the nodes. Nodal displacements in the horizontal plane and rotation along the vertical axis are not considered keeping in view that they do not significantly contribute to the structural behavior and hence are ignored.




(a) Nodal forces and displacement in local axes

| | |
|---|---|
| Pω | Nodal load and displacement normal to grid plane ↓+ |
| $C_{pi}$, $\theta_{pi}$ | Nodal moment and rotations about local coordinate q-axis |
| $C_{qi}$, $\theta_{qi}$ | Nodal moment and rotation about local coordinate p axis |
| Cp, θp | Member end bending moment and slope |
| Cq, θq | Member end torsion and rotation |



(b) Grid member in Global axes

Fig. 3.1 Typical Element of Grid Framework

The two dimensional grid representations of the actual three-dimensional grid floors have been found to serve the purpose satisfactory. A typical element with the associated element forces is designated with reference to the element and global coordinate axes as shown in Fig 3.1.

The stiffness matrix for joint forces and displacements referred to the member axis system isgiven by:

$$\begin{Bmatrix} p_i \\ C_{pi} \\ C_{qi} \\ P_j \\ c_{pj} \\ c_{qj} \end{Bmatrix} = \begin{bmatrix} \frac{12EI}{L^3} & \frac{6EI}{L^2} & 0 & \frac{-12EI}{L^3} & \frac{6EI}{L^2} & 0 \\ \frac{6EI}{L^2} & \frac{4EI}{L} & 0 & \frac{-6EI}{L^2} & \frac{2EI}{L} & 0 \\ 0 & 0 & \frac{GJ}{L} & 0 & 0 & \frac{-GJ}{L} \\ \frac{-12EI}{L^3} & \frac{-6EI}{L^2} & 0 & \frac{12EI}{L^3} & \frac{-6EI}{L^2} & \\ \frac{6EI}{L^2} & \frac{2EI}{L} & 0 & \frac{-6EI}{L^2} & \frac{4EI}{L} & 0 \\ 0 & 0 & \frac{-GJ}{L} & 0 & 0 & \frac{GJ}{L} \end{bmatrix} \begin{Bmatrix} w_i \\ \theta pi \\ \theta qi \\ w_j \\ \theta p_j \\ \theta q_j \end{Bmatrix} \quad (3.1)$$

Equation 3.1 can alternatively be written in the following matrix notations form

$$\{ p \}_e = [ K ]_e \{ D \}_e \quad (3.2)$$

The above equation is the stiffness matrix equation for an element with nodes i and j of the discretized structure referred to the local coordinate axes. $[ K ]_e$ is called the stiffness matrix of the element. Stiffness of all the elements constituting the descretized structure must properly be superimposed to obtain the overall stiffness matrix of the complete structure. Stiffnesses of the elements, therefore, must be expressed in terms of a global coordinate system.

Linear displacement of a co-ordinate system does not introduce any change in the element stiffnessess. However, the rotation of a coordinate system does effect it. The transformation matrix relating forces and displacements in the local and global co-ordinate system shown in Fig.- 3.1b is given by equation 3.3.

$$\begin{Bmatrix} P_i \\ C_{pi} \\ C_{qi} \end{Bmatrix} = \begin{bmatrix} 1 & 0 & 0 \\ 0 & \cos\alpha & \sin\alpha \\ 0 & -\sin\alpha & \cos\alpha \end{bmatrix} \begin{Bmatrix} P_i \\ C_{xi} \\ C_{yi} \end{Bmatrix} \qquad (3.3)$$

Equation 3.3 may be written as

$$\{P_i\}_e = [R]\{P_i\}_g \qquad (3.4)$$

Where, [ R ] is the rotation transformation matrix composed of direction cosines of the element. When forces at both the nodes of an element are considered simultaneously, the relationships can be expressed as given below:

$$\begin{Bmatrix} P_i \\ P_j \end{Bmatrix}_e = \begin{bmatrix} R & O \\ O & R \end{bmatrix} \begin{Bmatrix} P_i \\ P_j \end{Bmatrix}_g \qquad (3.5)$$

or simply as $\{P\}_e = [T]\ \{P\}_g$ (3.6)

where in $[T] = \begin{bmatrix} R & o \\ o & R \end{bmatrix}$ (3.7)

Similarly, the deformation transformation matrix equation can be written as

$$\{D\}_e = [T]\ \{D\}_g \qquad (3.8)$$

Stiffnessess equations for structural system in the local and global co-ordinate system respectively are given as

$$\{P\}_e = [K]_e\ \{D\}_e \qquad (3.9)$$

$$\{P\}_g = [K]_g\ \{D\}_g \qquad (3.10)$$

Substitution of the expressions from equations (3.6) and (3.8) into equation (3.2) results in

$$[T]\ \{P\}_g = [K]_e\ [T]\ \{D\}_g \qquad (3.11)$$

with multiplication by $[T]^{-1}$ on both sides equation (3.11) becomes

$$\{P\}_g = [T]^{-1}\ [k]_e\ [T]\ \{D\}_g \qquad (3.12)$$

Comparing equation (3.12) with (3.11), it will be concluded

$$[\,K\,]_g \quad = \quad [\,T\,]^{-1} \quad [\,K\,]_e \quad [\,T\,] \qquad (3.13)$$

Thus, through equation(3.13), the element stiffness matrix in local co-ordinate system is expressed in global co-ordinate system. It may be noted that inverse of [ T ] is equal to the transpose of [ T ]. This characteristic of [ T ] allows the general stiffness matrix of equation (3.13) to be obtained without the necessity of inverting a large [ T ] which is generally quite tedious.

After effecting the operation involved in equation (3.13), final expression for element stiffness in global co-ordinate system is obtained as given below:

$$\begin{Bmatrix} P_i \\ Cx_i \\ Cy_i \\ P_j \\ Cx_j \\ Cy_i \end{Bmatrix} = \begin{bmatrix} S_{11} & S_{12} & S_{13} & -S_{11} & S_{12} & -S_{13} \\ S_{12} & S_{22} & S_{23} & -S_{12} & S_{25} & S_{26} \\ S_{13} & S_{23} & S_{33} & -S_{13} & S_{26} & S_{36} \\ -S_{11} & -S_{12} & -S_{13} & S_{11} & -S_{12} & -S_{13} \\ S_{12} & S_{25} & S_{26} & -S_{12} & S_{22} & S_{23} \\ S_{13} & S_{26} & S_{36} & -S_{13} & S_{23} & S_{33} \end{bmatrix} \begin{Bmatrix} w_i \\ \theta x_i \\ \theta y_i \\ w_j \\ \theta x_j \\ \theta y_j \end{Bmatrix}$$

where,

$S_{11} \quad = \quad 12\ EI/L^3$

$S_{12} \quad = \quad 6\ EI \cos\alpha/L^2$

$S_{13} \quad = \quad 6\ EI \sin\alpha/L^2$

$S_{22} \quad = \quad (GJ \sin^2\alpha + 4EI \cos^2\alpha)/L$

$S_{23} \quad = \quad (4\ EI - GJ) \sin\alpha \cos\alpha/L$

$S_{25} \quad = \quad (2\ EI \cos^2\alpha - GJ \sin^2\alpha)/L$

$S_{26} \quad = \quad (GJ + 2\ EI) \sin\alpha \cos\alpha/L$

$S_{33} \quad = \quad (4\ EI \sin^2\alpha + GJ \cos^2\alpha)/L$

$S_{36} \quad = \quad (4\ EI \sin^2\alpha - GJ \cos^2\alpha)/L$

After forming element stiffness matrix in the global co-ordinate system, all the elements constituting the disceretized structure are appropriately assembled. It should be noted that all the elements entering any particular node must have the same nodal displacements. Thus at any node the corresponding stiffnesses which in effect ensure the common nodal displacements should be superimposed. Thus nodal stiffness matrix of the discretized structure is obtained.

Applied loads are transformed into a system of equivalent nodal forces , which could be single direct load or a combination of direct load and/or bending moment and/or torsion. After obtaining nodal stiffness matrix and equivalent forces acting at the nodes of the discretized structure, the governing stiffness equation

$$\{P\}_g = [K]_g \{d\}_g \qquad (3.15)$$

can be solved for nodal displacements subjected to the support conditions. These displacements, now, from the basis for computing the direct forces, bending moments and torsion at any node of the discretized member in the structure.

**3.2 Limit State Design of RC Section**

**3.2.1 Limit State of Collapse: Flexure Assumption**

Limit state design of a reinforced concrete section for limit state of collapse in bending is based on the following assumptions

a. Plan sections normal to the axis remain plan after bending.

b. The maximum strain in concrete at the outermost compression fibre is taken as 0.35% in bending regardless of strength of concrete.

c. The relationship between the compressive stress-strain distributions in concrete is assumed to be parabolic. The maximum compressive stress is equal to 0.67 $f_{ck}$/1.5 (1.5 being the partial safety factor) or 0.446 $f_{ck}$.

d. The tensile strength of concrete is ignored.

e. The stress in reinforcement is derived from the representative stress-strain curves for the type of steel used.

f. The maximum strain in tension reinforcement in the section at failure should not be less than the following.

$$\varepsilon_s \geq \frac{\sigma_Y}{1.15E_s} + 0.002$$

Where $\sigma_Y$ = Characteristic strength of steel

$E_S$ = modulus of elasticity of steel

$\varepsilon_S$ = Strain in steel at failure

### 3.2.2 Moment of Resistance for Rectangular Sections

The value of moment of resistance for under–reinforced, balanced and over reinforced concrete sections are summarized as below. Over reinforced sections are avoided.

a) For under reinforced sections $x_u/d < x_u\ max/d$.

Where $x_u$ = Depth of neutral axis.

$x_u\ max$ = Limiting value of depth of neutral axis.

For equilibrium, force of compression should be equal to force of tension

$$0.36\ \sigma_{ck} . b . x = 0.87\ \sigma y\ A_{st}$$

$$x_u = \frac{0.87\sigma_y A_{st}}{0.36\sigma_{ck} b}$$

Where $A_{st}$ = Area of tension steel

Lever arm z = d – 0.42

$$= d - 0.42\left[\frac{0.87\ \sigma_y A_{st}}{0.36\sigma_{ck} b}\right]$$

$$= \quad d - \frac{\sigma_y A_{st}}{\sigma_{ck} b}$$

Moment of Resistance

$M_u \quad = \quad 0.87\ \sigma_y\ Ast..Z$

(b) For balanced section moment of resistance is calculated by following equation

$X_u \quad = \quad X_{u\,max}$

$Z \quad = \quad d - 0.42\ X_{u\,max}$

$M_u \quad = \quad 0.36\ \sigma_{ck}\ b.X_{u\,max}\ (d - 0.42\ X_{u\,max})$

Or $M_u \quad = \quad 0.87\ \sigma_y\ A_{st}\ (d - 0.42\ X_{u\,max})$

### 3.2.3 Moment of Resistance for Flanged Selections

(a) When $X_u \leq Df$ (i.e. N.A. lies in the flange)

Moment of Resistance may be calculated from equation given in earlier selection

(b) When $X_u > Df$ (i.e. N.A. lies out side the flange)

Limiting values of the moment of resistance of the section may be obtained by the following equations

(1) When the $Df \leq \frac{3Xu}{7}$

$$Mu = 0.36\ \frac{X_{u\max}}{d}\left(1 - \frac{0.42\ X_{u\max}}{d}\right)\sigma_{ck}\ b_w\ d^2 + 0.45\,\sigma_{ck}\left(b_f - b_w\right).D_f\left(d - \frac{D_f}{2}\right)$$

Where

$M_u \quad = \quad$ Ultimate moment of Resistance

$X_{u\,max} \quad = \quad$ Limiting value of $X_u$

$X_u \quad = \quad$ Depth of Neutral axis

| | | |
|---|---|---|
| d | = | Effective depth |
| $\sigma ck$ | = | Characteristic strength of concrete |
| $b_w$ | = | Width of web |
| $b_f$ | = | Width of flange/compression face |
| $D_f$ | = | Depth of flange |

(a) When $D_f > \frac{3Xu}{7}$

Moment of resistance may be calculated by the following equation

$$M_u = 0.36\ \frac{Xu\max}{d}\left(\frac{1-0.42X_{u\max}}{d}\right)f_{ck}b_w d^2+0.45f_{ck}(b_f-b_w)y_f\left(\frac{d-y_f}{2}\right)$$

Where $Y_f$ = $(0.15\ X_u + 0.65\ D_f)$ but not greater $D_f$

When Xu,max > Xu > Df , the moment of resistence may be calculated by equations given earlier , When $D_f/X_u$ exceeds 0.43 or otherwise as the case is applicable but in both cases substitute $X_{u\ max}$ by $X_u$.

### 3.2.4 Tension Reinforcement

Section has to be redesigned if the max. area of tension R/F exceeds the 0.04 bD (i.e. 4% of gross cross-section).

Minimum area of tension reinforcement is given by

$$A_{st} = \frac{0.85\,bd}{\sigma y}$$

Where,

| | | |
|---|---|---|
| d | = | effective depth |
| D | = | overall depth |
| b | = | width of section (or width of web for flanged sections) |

### 3.2.5 Limit state of collapse : Shear

The nominal shear stress $\tau_v$ in beams of uniform depth shall be obtained by the following equation.

$$\tau_v = \frac{V_u}{bd}$$

Where,

$V_u$ = Shear force due to design loads

b = Breadth of the member for flanged section which shall be taken as the breadth of web, (bw)

d = effective depth

**Shear Reinforcement**

(a) No shear reinforcement :

The code required that shear reinforcement need not be provided in following cases :

(1) when $\tau_v$ is less than 0.5 times the shear capacity $\tau_c$ as given in code

(2) In members of minor structural importance such as lintels

(b) Minimum shear reinforcement :

It is needed when $\tau_v \leq \tau_c$ and is given by

$$A_o = \frac{0.4 b S_v}{0.87\, \sigma y}$$

$A_o$ = Total cross sectional area of stirrup legs effective in shear

b = breadth of the beam or breadth of the web of a flanged beam

$\sigma_y$ = characteristics strength of shear reinforcement (shall not be taken greater than 415 N/mm$^2$

$S_v$ = Spacing of shear reinforcement

(a) Shear reinforcement is needed when $\tau_v > \tau_c$. Under this case shear stirrups may be provided in the form of either vertical stirrups or inclined stirrups or a combination of both. The spacing of shear stirrups is calculated as follows :

Shear reinforcement is designed to carry a shear $V_{us} = V_u - \tau_c bd$. The strength of shear R/F shall be calculated as below :

For vertical stirrups

$$V_{us} = \frac{0.87 \sigma y A_{sv} d}{S_v}$$

(d) Under no circumstances shear stress $\tau_v$ shall exceed $\tau_{cmax}$ (max shear stress) given in I.S. 456.

### 3.2.6 Limit state of collapse : Torsion

I.S. 456-2000 prescribes that where the torsional stiffness of the members is taken into account in the analysis, the members shall be designed for the torsion.

Equivalent shear: ($V_e$) ∴ it shall be calculated as follows.

$V_e$ = $V_u + 1.6\, T_u/b$

$V_e$ = Equivalent shear

$V_u$ = Shear

$T_u$ = Torsional moment

b = Breadth of beams

b = bw (in case of flanged section)

Equivalent nominal stress : It shall be calculated as follows

$$\tau_{ve} = \frac{V_e}{bd}$$

If $\tau_{ve} > \tau_{c\,max}$ (max shear stress) It is a case of shear failure and section should be redesigned.

$\tau_{ve} \leq \tau_c$ (design shear strength of conc.) minimum shear reinforcement shall be designed as per formula given below :

$$\frac{A_{sv}}{b.S_v} \geq \frac{0.4}{fy}$$

If $\tau_{ve} > \tau_c$ then both longitudinal and shear reinforcement is designed as follows :

**Longitudinal reinforcement :**

Longitudinal reinforcement is designed for an equivalent moment given by

$M_{e1} = M_u + M_t$

Where, $M_t = T_u \left(\frac{1+D/b}{1.7}\right)$

$M_u$ = Bending moment at cross section

$T_u$ = Torsional moment

If $M_t > M_u$

Additional longitudinal reinforcement if provided on the compression face which is designed to resist bending moment given by

$M_{e2} = M_t - M_u$

**Transverse reinforcement**

For two legged closed loops enclosing the corner longitudinal bars. The area of cross-section of steel bars is calculated as follows :

$$A_{sv} = \frac{T_u S_u}{b_1 d_1 (0.87\,\sigma y)} + \frac{V_u S_v}{2.3 d_1 (0.87\,\sigma y)}$$

But the total transverse reinforcement should not be less than as given below

$$A_{sv} \geq \frac{(\tau_{ve} - \tau_c) b S_v}{0.87 fy}$$

Where,

Tu = torsional moment

Vu = shear force

Sv = spacing of shear reinforcement

b1 = centre to center distance between corner bars in the direction of width

d1 = centre to center distance between corner bars in the direction of depth

b = breadth of member

fy = characteristic strength of steel

$\tau$ve= equivalent shear strength.

$\tau$c = shear strength of steel

**3.2.7 Check for Deflection**

As per clause 23.2 of IS: 456-2000, the deflection of a structural or part of there of, shall not adversely affect the appearance or efficiency of the structure or finishes or partitions. The deflection shall generally be limited to the following.

(a) The final deflection due to all loads including the effect of temperature, shrinkage and creep measured from as cast level of the supports of floors, roofs and other horizontal members, should not normally exceed span/250.

The deflection including effect of shrinkage, creep and temperature occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm whichever is less.

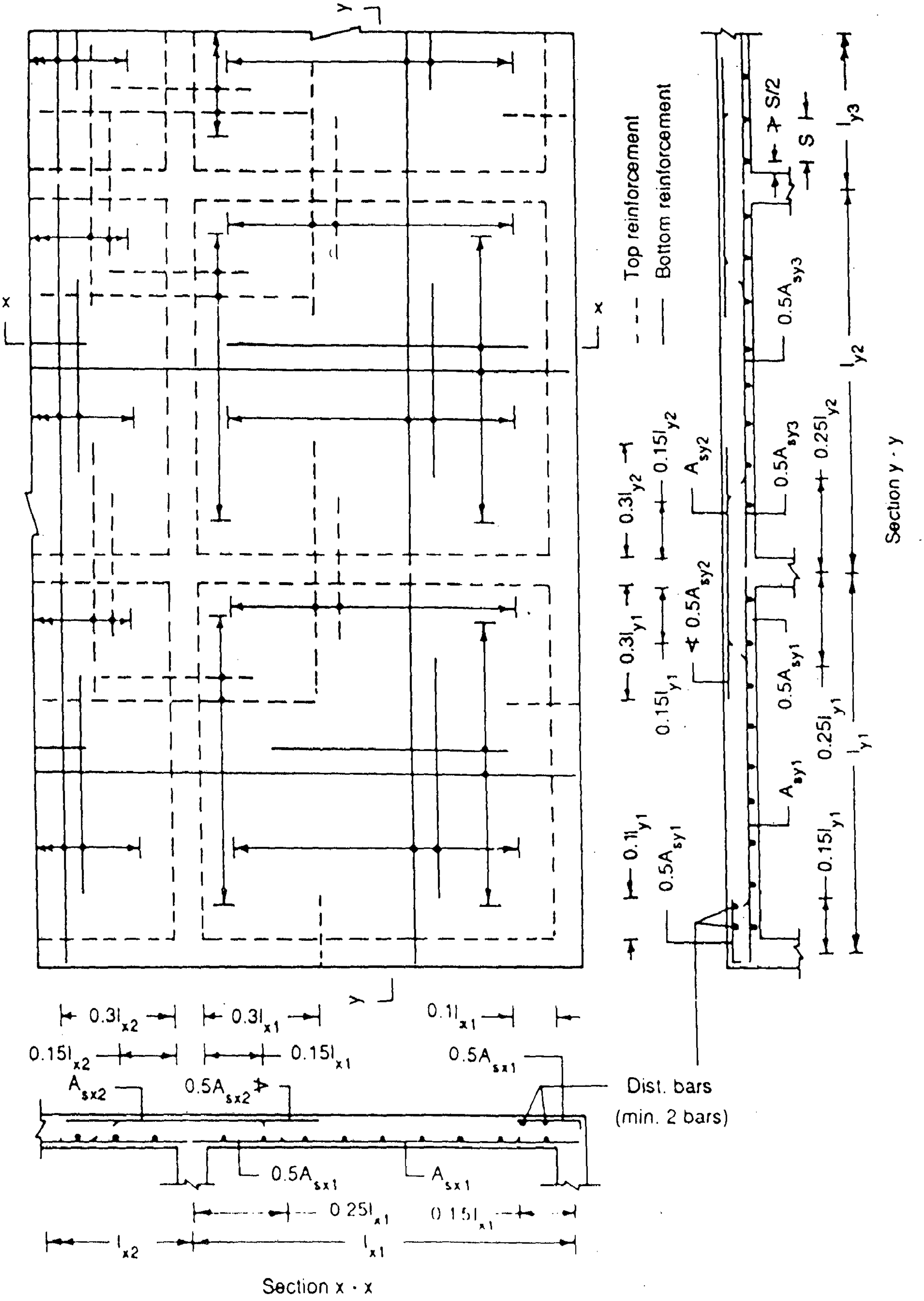
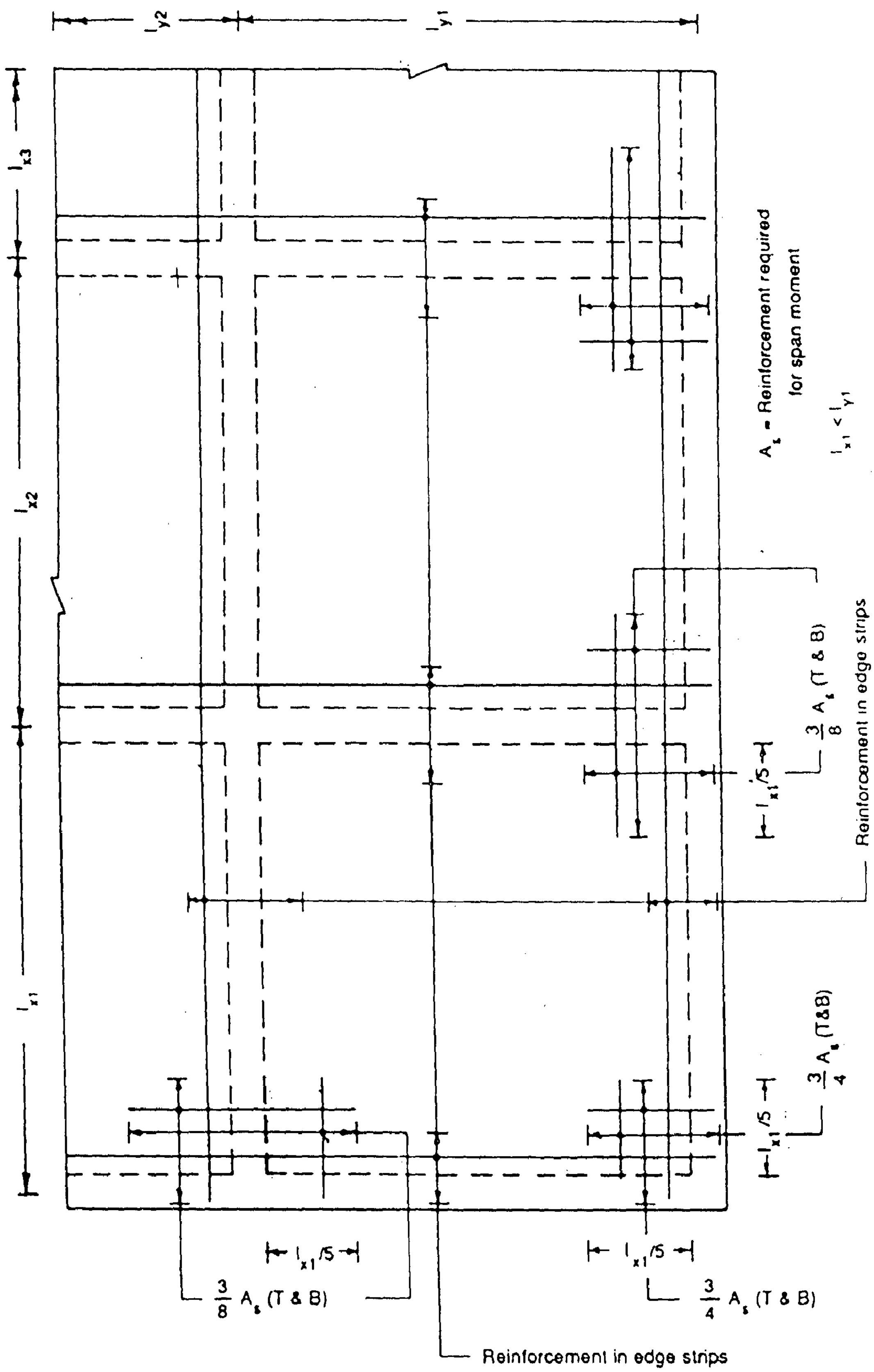



FIG. 3.2 Reinforcement detailing for middle strips with bent up bars




(a) Reinforcement details for edge strips and corners

Fig: 3·3 Reinforcement detailing for edge strips and at corners

# COMPUTER PROGRAM

## 4.1 GENERAL

A CAD package has been developed to assist the designer in the design of the grid floors. Though the process of analysing, design have been integrated and automated to a great extent. At the same time the package has been made interactive and flexible so that human intelligence and experience, which is an important aspect of any design exercise, has a role to play. Important parameter has been allowed to be changed by designer's discretion. The package can be divided into four parts. Datas both the input and outputs, are put in to separate files. So that results can be checked and printed also. The computer language used for programming is FORTRAN & C- language.

The program can be divided into following modules

a. Program for preparing input file.

b. Program for analysis of grid floor.

c. Program for sorting and interpreting the results of analysis.

d. Program for designing.

**4. Program for preparing input file.**

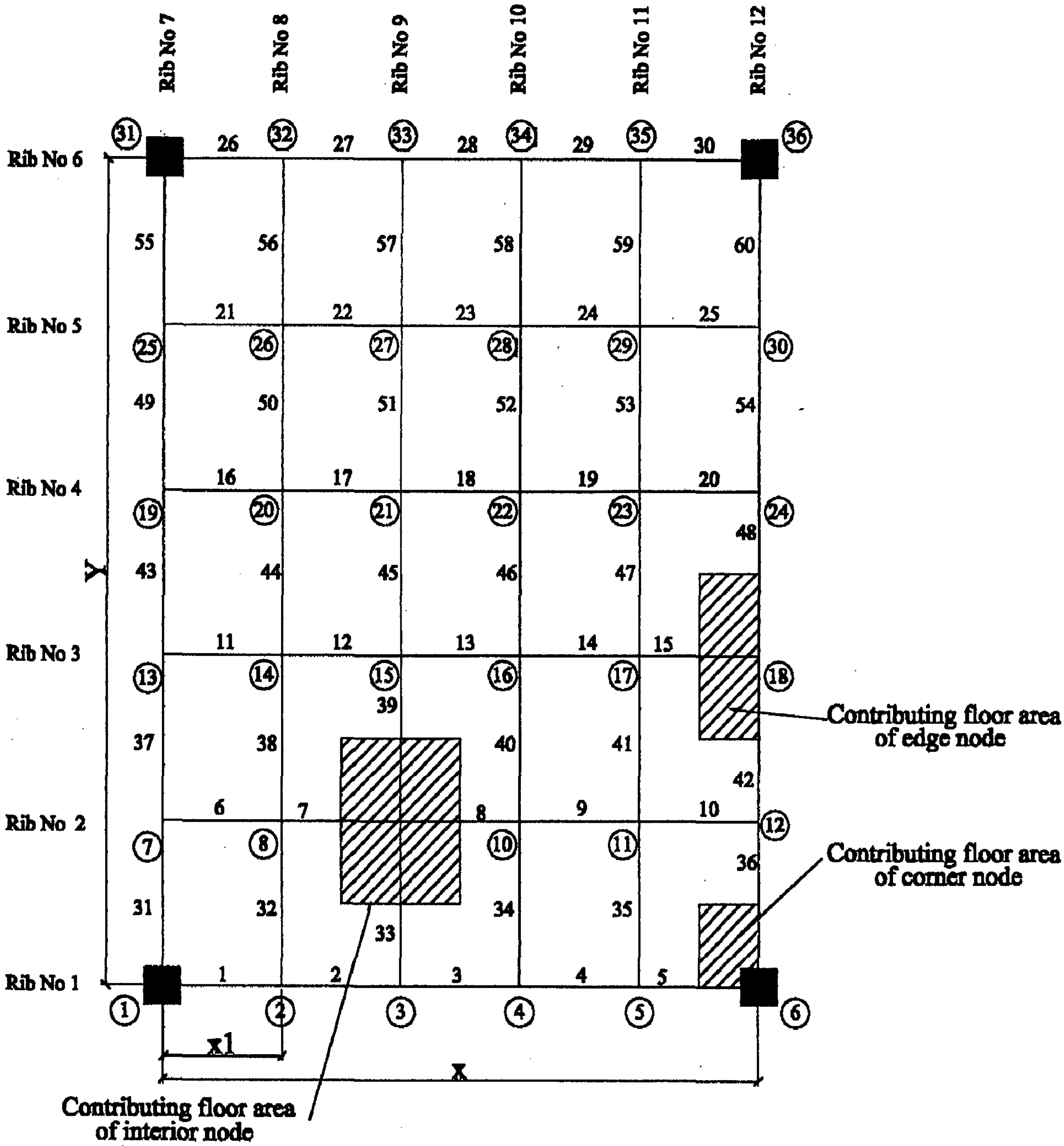
Matrix methods require a lengthy data preparation, which is time consuming. The inputs required are total no of members nodes, no of different member types, load cases, no of restrained joints & no of loaded joints etc. The analysis also requires member connectivity, their geometric properties (EI & GJ.) grouped according to member of the same properties. Detailed information about joint member & joint loads has to be provided. Restrained joint data in the form of node no. and type of restrain is also required. The program calculates all this according to i and j nodes of each member and its type. The program can prepare input file for following cases.

1. Simply supported isolated panels
2. Isolated grid floor panel supported on four corner columns
3. Interior panel of a continuous grid floor.
4. Edge panel -one short side discontinuous.
5. Edge panel -one long side discontinuous.
6. Corner panel with two adjacent sides discontinuous.
7. Continuous grid floors

We can further reduce the inaccuracies in the result due to the assumptions in the support condition by using the case 7 which can produce input file for all the possible cases.

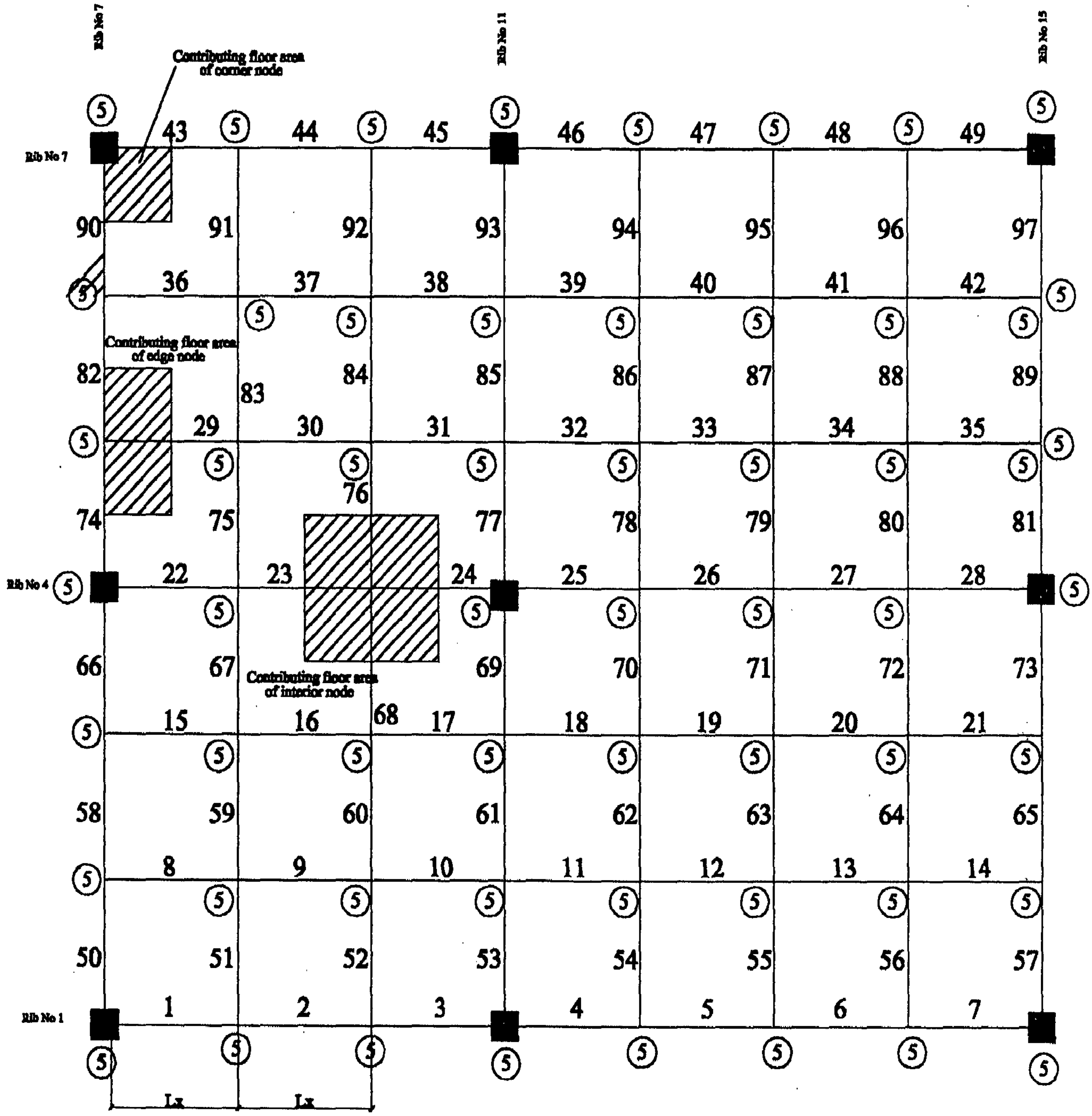
The program assumes that the depth of edge beams and ribs are equal. Basic dimensions e.g. depth of ribs, width of edge beam, and interior beams can be input, though the default procedure for dimensioning of section is also provided. The default value of width of ribs shall be calculated, as depth/3 and value shall lie. between 200 and 300 mm. The width of edge beams is assumed to be twice of interior beams. For start, program adopts a span/depth ratio of 20 for calculation of depth. Effective flange widths are automatically calculated according to the location of member. Number of ribs can be varied according to the entered spacing of rids. The program follows the order shown in Fig. – 4.1 & 4.2 while numbering the nodes, members and the ribs.

In addition to this the program calculates nodal load according to the contributing area of each node, which in turn depends upto the location of node, as shown in Fig. 4.1and 4.2. The shorter side of grid is put along the X-axis. This module prints the results in a file GRID. in which is the input file for the next module.




# Fig. 4.1 An Isolated Grid

## Method of Node, Member Numbering And Rib Numbering




# Fig . 4.2 Typical Continuous Grid

Method of Node , Member Numbering
And Rib Numbering

### 4.3 Program for Analysis

The grid floor is analyzed using a Fortran program available in a standard text (1). (1). This program has been partially modified to suit the present requirements.

The program activates five subroutines in turn.

INPUT reads data for a particular problem from input file (GRID . IN) prepared earlier and performs some minor calculations.

ASSMBL determines member stiffness matrices for each different member type and then assembles the overall stiffness matrix from them and details of member connectivity.

BCS performs the modifications to the stiffness equations to enforce satisfaction of specified boundary conditions. Three degrees of freedom has been considered (w, $\theta_x$ $\theta_y$).

BSOLV solves the stiffness equations. Half of the banded stiffness matrix is stored and solution is by Cholesky decomposition.

RESULTS prints the joint displacements, calculates the member end moments and shear forces and the values of joint reactions and then prints them in an output file, named GRID.OUT.

The output file also contains the data, which has been input so that designer can check for any error.

### 4.4 Program for sorting and interpreting the results of analyses module.

This program picks the results from the output file (Grid . Out) and arranges. them according to increasing member numbers. It also calculates the equivalent moment and equivalent shear for each member and puts them in tabular form. This has been done to assist the designer in the selection of critical sections. This module checks the grid floor for failure against max. equivalent moment ,max. equivalent shear and deflection. In case of failure in any of the three cases it prompts the user to reanalyze the grid according to revised section dimensions. The deflection is checked according to the relation:

max. deflection (long term) < span/250.

The other two are checked in the usual manner.

If the grids is safe in all the three cases the program then the control is transferred to the design module. The results are printed in a file named Analysis .Dat.

## 4.5 Design Module

This section designs the section according to the rib number. When a particular rib no. (order shown in the Fig. - 4.1 & 4.2 ) is specified it picks up the values of maximum positive and maximum negative moments and design steel for them automatically. Then it asks the user if he wants to design another member in the rib, if answered no it asksthe user to design another rib. It is left on user's wisdom to choose the ribs & the members in it to be designed other than the max. positive & max. negative moment for the chosen rib. This process is iterative and continues till the user desires. It designs sections for combined effect of shear, torsion and bending moments and calculates longitudinal reinforcement and shear reinforcements. It can calculate the same for a section when torsion is zero and corresponding shear reinforcement .

This module also designs the reinforcement for the slab and writes the data to a file named Slab . Dat.

The module can design T-sections, singly, doubly R/F beams according to the magnitude and sign of moment. The results of the design are written in a file Design . dat. All the provision of IS 456-2000 has been incorporated in the program.

**4.6 Assumptions**

Actual behaviour of structures cannot be simulated accurately in to a mathematical model. To resolve this problem many assumptions are made. Assumptions also simplify the process of solution. Following assumptions have been made.

1. Rotation about X-axis and Y-axis & vertical deflection has been allowed for free edges.
2. To account for the partial fixity at a continuous edge the same has been assumed to be fully fixed.
3. At column location the node has been restrained only vertically and rotations about both the axis have been allowed.
4. For a simply supported case analyses are performed with vertical support at edge joints and with additional restrain to rotations about normal to the supported edge.
5. For continuous grids all nodes are free to rotate in both the axes. Only at column locations nodes are vertically restrained. Nodes at the edge columns locations has been restrained for rotations about normal to the axis of the edge
6. For grid cases 1-6 it has been assumed that in case of continuous edges grid floors of same dimensions extend in the opposite direction. So that stiffness of floors is same hence the same amount of fixity is provided to all other continuous edges.

## 4.7 Limitations:

1. It is not possible to analyse complex shapes of the grid floor other than rectangle or square plans.
2. Only those continuous grid floors can be analyzed at a time, which form a complete rectangle or square although for other cases we can analyze the grid floor in parts.
3. All the constituting ribs of a grid floor can only have same spacing in one direction (say x). Spacing can be changed for the orthogonal direction.
4. Dimensions of only the edge beams and the interior ribs can differ (i.e. All the interior ribs shall have same dimensions and all the edge beams shall also be of equal dimensions).

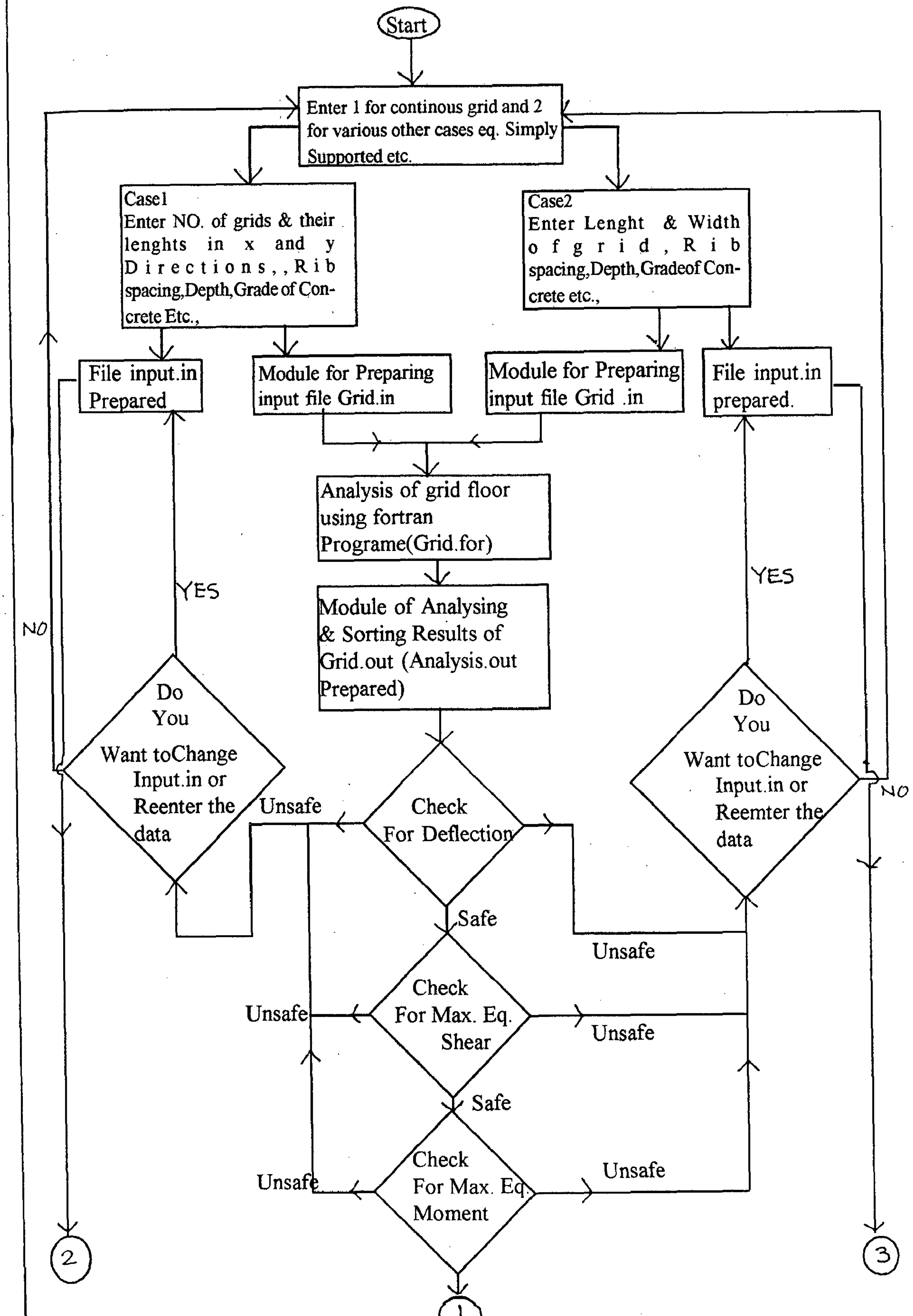
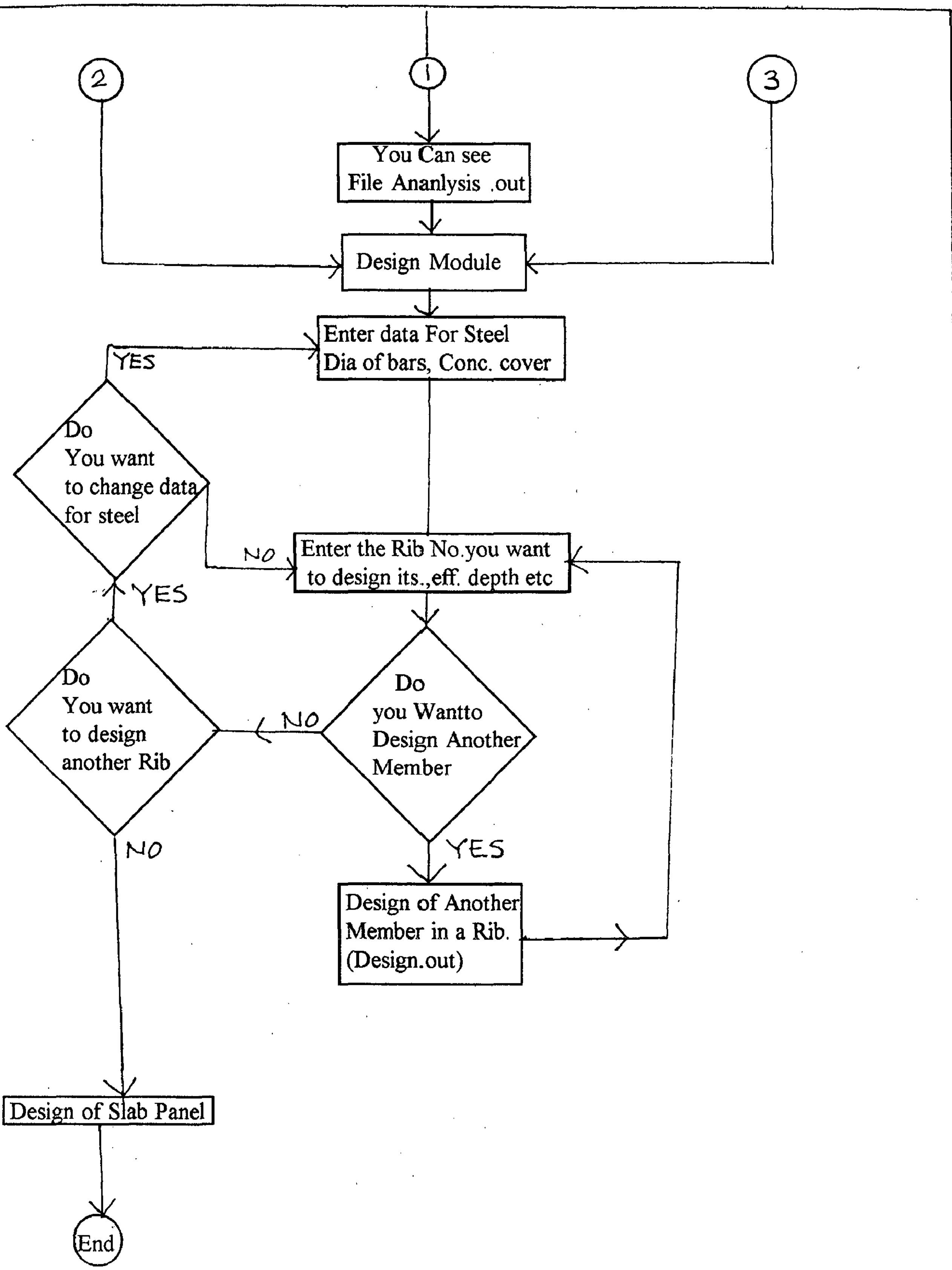



FIG. 4.3 FLOW CHART FOR CAD PACKAGE



2
1
3
You Can see File Ananlysis .out
Design Module
Enter data For Steel Dia of bars, Conc. cover
YES
Do You want to change data for steel
NO
Enter the Rib No.you want to design its.,eff. depth etc
YES
Do You want to design another Rib
NO
Do you Wantto Design Another Member
YES
Design of Another Member in a Rib. (Design.out)
NO
Design of Slab Panel
End

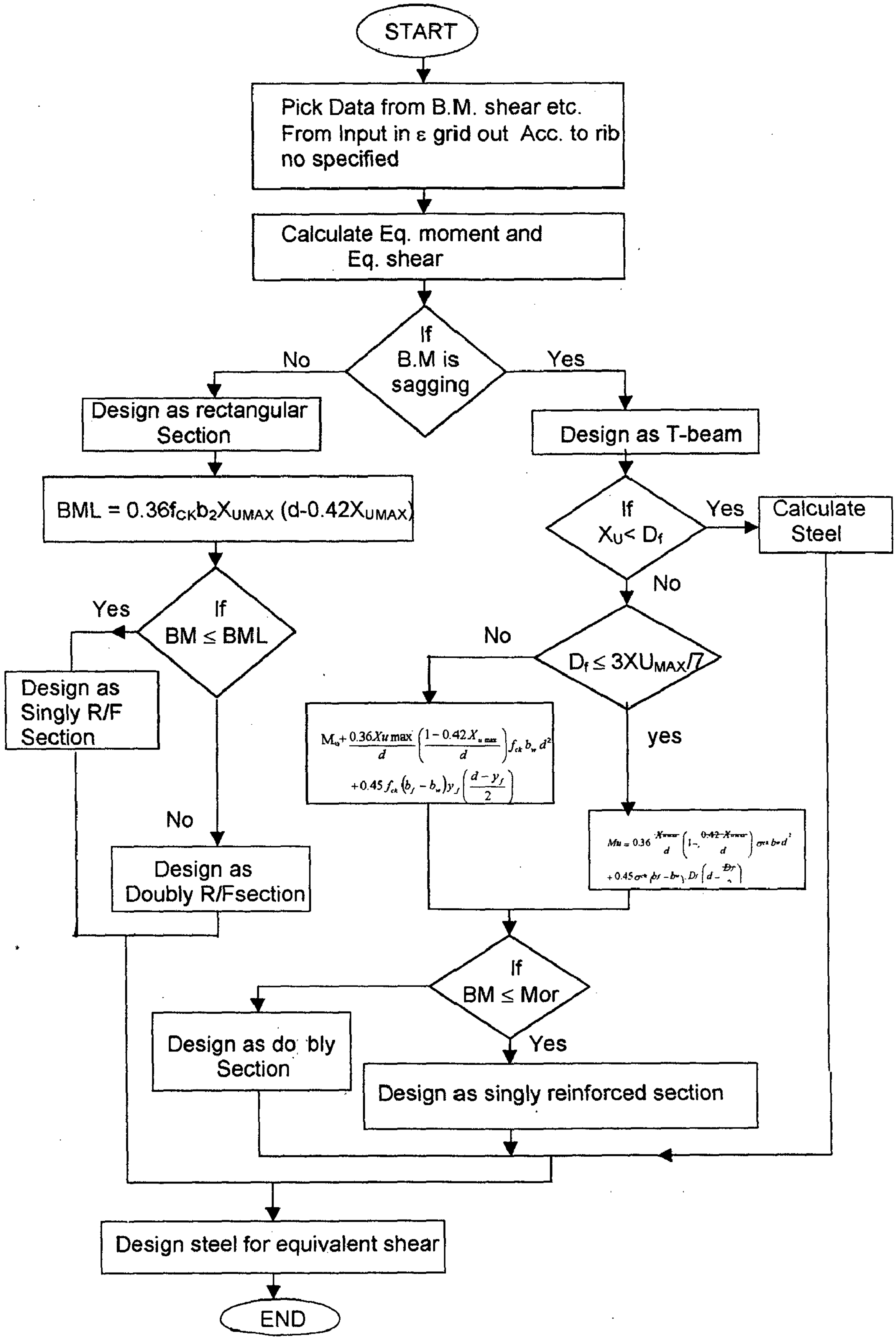



FIG. 4.4 FLOW CHART FOR DESIGN OF BEAMS

# VALIDATION OF PROGRAM

## 5.1 General

To check the accuracy of the program and the methodology used a problem from the standard text (8) has been picked up and analyzed using CAD package. Then the results have been compared with the text results.

## 5.2 Text Example

A reinforced concrete grid floor is to be designed to cover a floor area of size 12 m by 16 m. The spacing of the ribs in mutually perpendicular directions being 2 m c/c. Live load = 1.5 kN/m$^2$ . Adopt M-20 grade concrete and Fe –415 grade tor steel.

The grillage method gives the absolute value of moments at the both ends of a member and the plate theory gives the values in terms of per meter width of grid .Since the length of typical member for grillage method is two meters hence the values given by grillage method has been divided by two to arrive at moment per meter of width of grid.

**Table –1: Moments and Shear Forces Per Metre Width of Grid Using Plate Theory**

| Point | x (m) | y (m) | $M_X$ (kn-.m) | $M_y$ (kN-.m) | $M_{xy}$ (kN.m) | $M_{yx}$ (kN.m) | $Q_x$ (kN) | $Q_y$ (kN) |
|---|---|---|---|---|---|---|---|---|
| E | 6 | 8 | 108 | 61 | 0 | 0 | 0 | 0 |
| F | 6 | 12 | 77 | 43 | 0 | 0 | 0 | 9.4 |
| G | 6 | 16 | 0 | 0 | 0 | 0 | 0 | 13.4 |
| H | 0 | 8 | 0 | 0 | 0 | 0 | 10.1 | 0 |
| I | 2 | 8 | 54 | 30.5 | 0 | 0 | 8.74 | 0 |
| J | 4 | 8 | 94 | 53 | 0 | 0 | 5.05 | 0 |
| K | 0 | 12 | 0 | 0 | 3.74 | 3.74 | 7.14 | 0 |
| D | 0 | 16 | 0 | 0 | 5.30 | 5.30 | 0 | 0 |

**Table – 2 : Moments and Shear Forces Per Meter Width of Grid Using Grillage Method of Analysis**

| Point | Node No. | X (m) | y (m) | Mx (km-m) | $M_y$ (km-m) | Mxy (km-m) | Myx (km-m) | Qx (Km) | Qy (km) |
|---|---|---|---|---|---|---|---|---|---|
| E | 32 | 6 | 8 | 101.5 | 52.0 | 0 | 0 | 5.7 | 0.73 |
| F | 46 | 6 | 12 | 72..6 | 47.6 | 1.151 | 0 | 3.75 | 8.73 |
| U | 60 | 6 | 16 | 0 | 2.78 | 1.4 | 0 | 0 | 17.9 |
| H | 29 | 0 | 8 | 1.9 | 0 | 0 | 0.95 | 29.9 | 0 |
| I | 30 | 2 | 8 | 57.91 | 25.91 | 0 | 1.01 | 17.54 | 0 |
| J | 31 | 4 | 8 | 90.9 | 45 | 0 | 0.55 | 17.54 | 0.6 |
| K | 43 | 0 | 12 | 1.69 | 0 | 4.35 | 4.53 | 22.6 | 0 |
| D | 57 | 8 | 16 | 5.6 | 5.50 | 5.45 | 5.6 | 4.16 | 4.09 |
| L | 50 | 14 | 0 | 1.04 | 0 | 6.09 | 4.1 | 13.1 | 0 |

## Design

The standard text reports the design of two central ribs, which are as follows.

For central rib in X-axis

Area of steel calculated = 1700 $mm^2$

Area of steel provided = 1964 $mm^2$

Assuming two bars to be bent up near the supports provided nominal shear R/F using 6 mm. Dia. two legged stirrups at a spacing of 290 m.

For central rib inY-axis. (long span)

Provided 4 bars of 20 mm dia. (Ast 1256 $mm^2$) and 6 mm dia. 2 legged stirrups at 280 mm C/C near the supports.

For slab.

Provide mesh R/F consisting of 6 mm $\phi$ bars at 200 mm c/c provided both ways for positive and negative moments in the slab.

With the help of package the design of central ribs slab is as follows.

Central rib in x-axix :

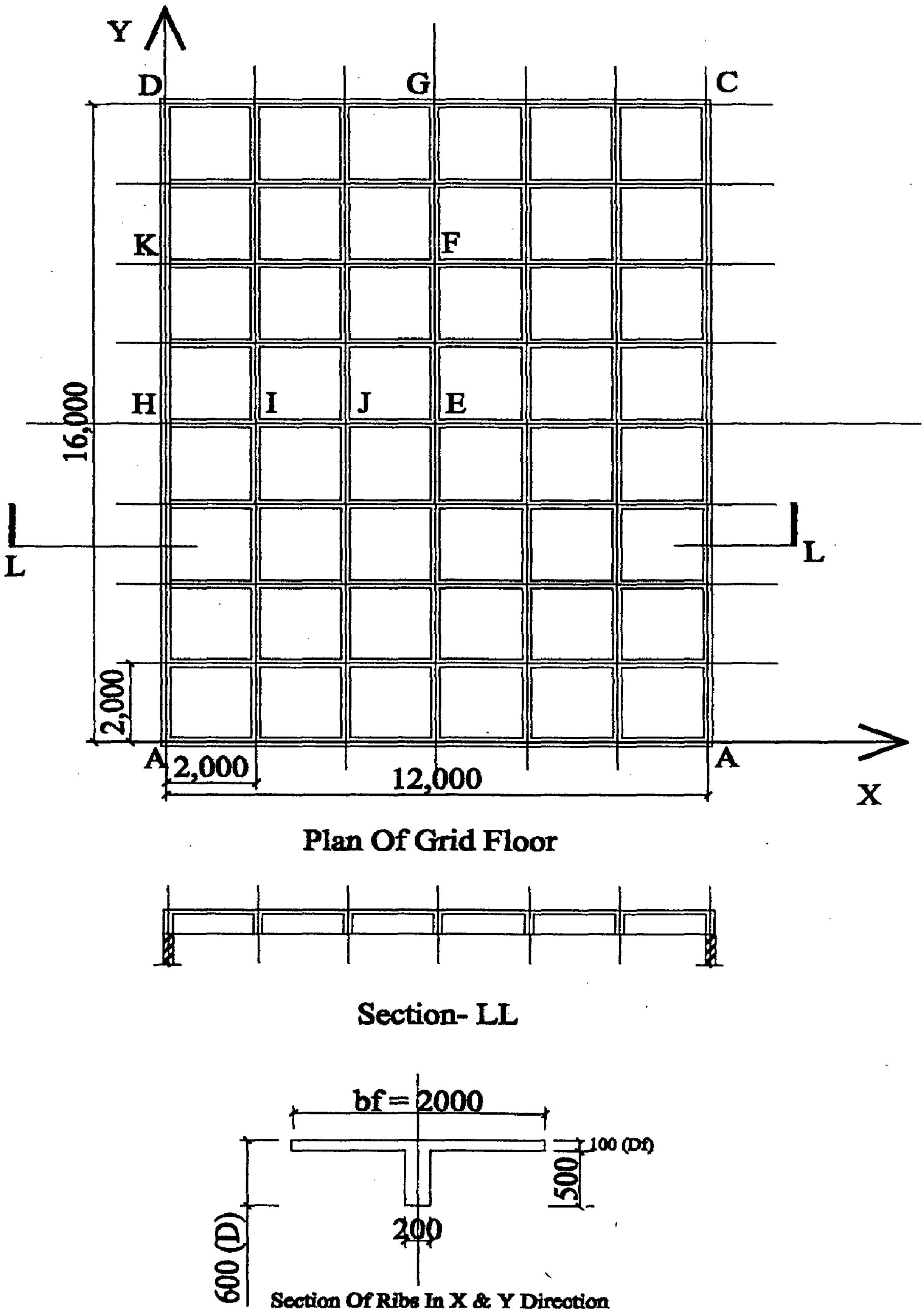
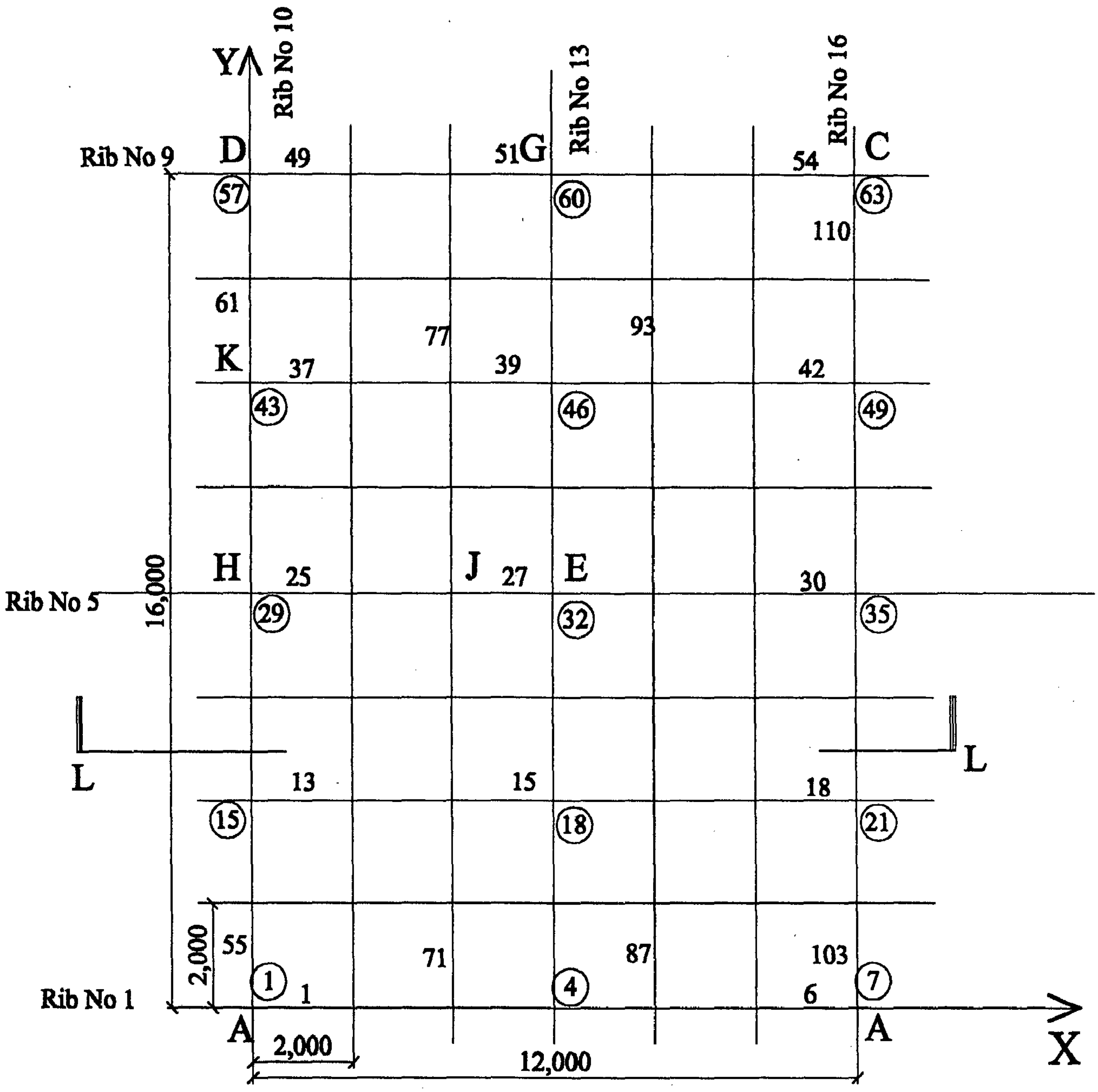



Fig. 5.1 Grid Floor (Text Example)




Plan Of Grid Floor

Fig. 5.2 Grid Floor (Text Example)

Method of Node , Member Numbering
And Rib Numbering

Area of steel calculated = 1610 $mm^2$

Area of steel provided = 1964 $mm^2$

For central rib in y-axis

Area of steel calculate = 1140 $mm^2$

Area of steel provided = 1256 $mm^2$

The results follow the difference in the design moments.

**5.3 Conclusions:**

**Table - 3: Comparison of Results**

| Method | Moments per meter width (km-m) | | Torsion (km-m) | | Shear (km) | |
|---|---|---|---|---|---|---|
| | Mx | My | Mxy | Myx | Qx | Qy |
| Plate Theory | 108 | 61 | 5.30 | 5.3 | 10.1 | 13,4 |
| Grillage Method | 101.5 | 52 | 6.09 | 5.6 | 29.88 | 17.9 |

Maximum deflection (short-term) as per plate theory = 0.015 m.

*Maximum deflection (short-term) as per grillage analogy = 0.0161 mm*

The plate theory over estimates the bending moments developed in x and y-directions to the extent of 6% and [illegible]%. The maximum shear is also under estimated by 66% and 25% in both x and y directions respectively. In addition to this plate theory dose not provide a complete picture of stresses in a member as can be observed from tables 1 and 2. Maximum torsion is also indicated in the member one rib away from the corner node.

Similarly the design of steel varies little according to the variation in the moments.

In can be said that though plate theory over estimates the maximum moments (varying from 6-14%) but it under-estimates the shear (3%-13.1%) and torsion (66%-25%) appreciably. It also does not give detailed information about stresses. More over it does not provide solution to the other general cases. Which normally occur., which is its greatest limitation. Hence the grillage analogy and the program gives the results to the desired accuracy and moreover used methodology is also general in nature and many type of problems can be solved easily by its use.

# TEST EXAMPLES

**6.1 General:** A CAD package can have various uses. It can be easily used to design the grid floor or to study their various aspects as no lengthy time is required to produce the detailed input files. The program is flexible in terms of the parameters to be input. is interactive, simple and easy to understand.

The program produces the results of the structural analysis and design and writes them in various different files. Which can be opened and correlated with each other to produce detailed drawings of the grid floor. Two typical grid floors have been analyzed and designed to explain the working of program. Some output files has also been displayed to show the results.

**6.2 Design Example 1**

A grid floor is to be designed for a span of 12 m x 12 m. The floor shall be supported on four corner columns. All the sides of the floor are discontinuous. Adopt M20 grade concrete and Fe-415 grade tor. steel. Live load is 3 kN/m$^2$. Rib spacing shall be kept 2 m in both the directions (Fig 6.1).

**Table – 4: Moments and Shears at Various Salient Points in the Grid**

| Point | Node No. | X (m) | Y (m) | Moment | | Torsion | | Shear | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Mx KN-m | My KN-m | Mxy KN-m | Myx KN-m | Qx KN | Qy KN |
| A | 1 | 0 | 0 | 78.5 | 78.5 | 78.5 | 78.6 | 218 | 218 |
| E | 4 | 6 | 0 | 617 | 43.7 | 21.8 | 0 | 33.7 | 42.4 |
| O | 25 | 6 | 6 | 231.8 | 231.8 | 0 | 0 | 12.5 | 12.5 |
| H | 9 | 2 | 2 | 189.5 | 189.5 | 12.60 | 12.6 | 83.9 | 83.9 |
| Max. Deflection = 18.1 mm which is safe. | | | | | | | | | |

Max. shear and max. torsion is found (in the outer edge beams) at the corner nodes and hence maximum equivalent shear is (427 kN) also at the same point.

Max. equivalent moment of (647 KN-m) is found at the mid points of edge beams.

Due to the symmetry of the grid floor the ribs placed equidistant from the central ribs are similarly stressed.

Detailed output has been attached in the appendix App1 and App2.

**Table – 5: Selection of ribs for design (Prob. no.1)**

| Rib No. | Max +Ve moment | Max – Ve moment |
|---|---|---|
| 1, 7, 8, 14 | 617 KNm | 78.5 |
| 2, 6, 9, 13 | 349 | 0 |
| 10, 12, 3, 5 | 264 | 0 |
| 4, 11 | 231.8 | 0 |

**Design :**

It can be seen from the above table that central ribs 4,11 and the rib Nos. 10, 12, 3, 5 does not have much variation in moment and hence steel may be designed for only one ribs of higher moment say rib No.10.

Edge ribs and rib No. 2 or 6, 9, 13 should be designed separately for the sake of economy. Since moments differ by 43%.

Designs of the three typical ribs have been done. Typical output of the program describing the steel required has been shown.

A slab panel of dimensions 2m x 2m is to be designed for the present problem. The details of steel are described in the output file (Slab.Dat) in appendix App3.

Dimensioning and detailing of steel has been separately drawn and shown in figures Fig 6. 6. and 6. ..

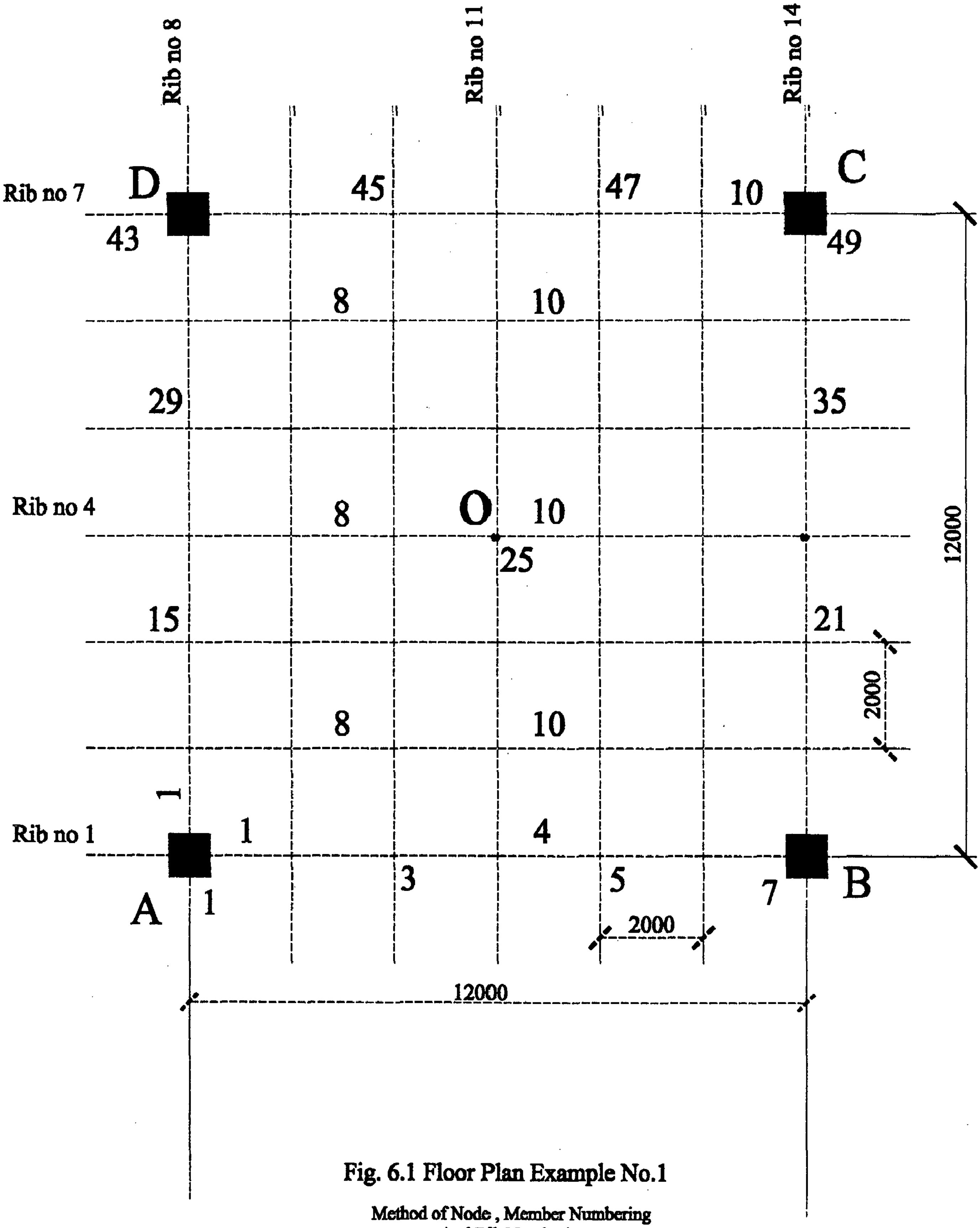
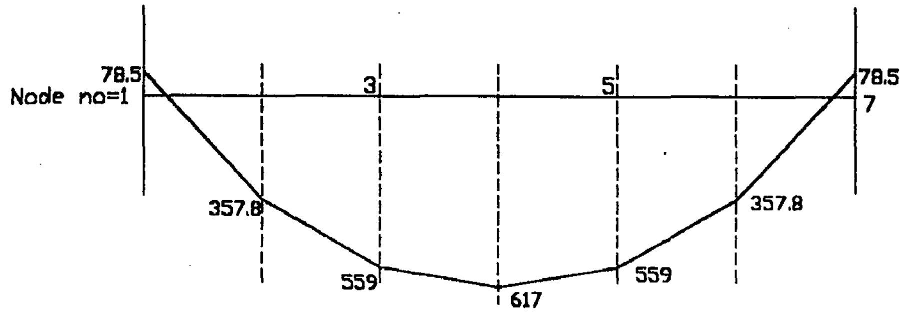
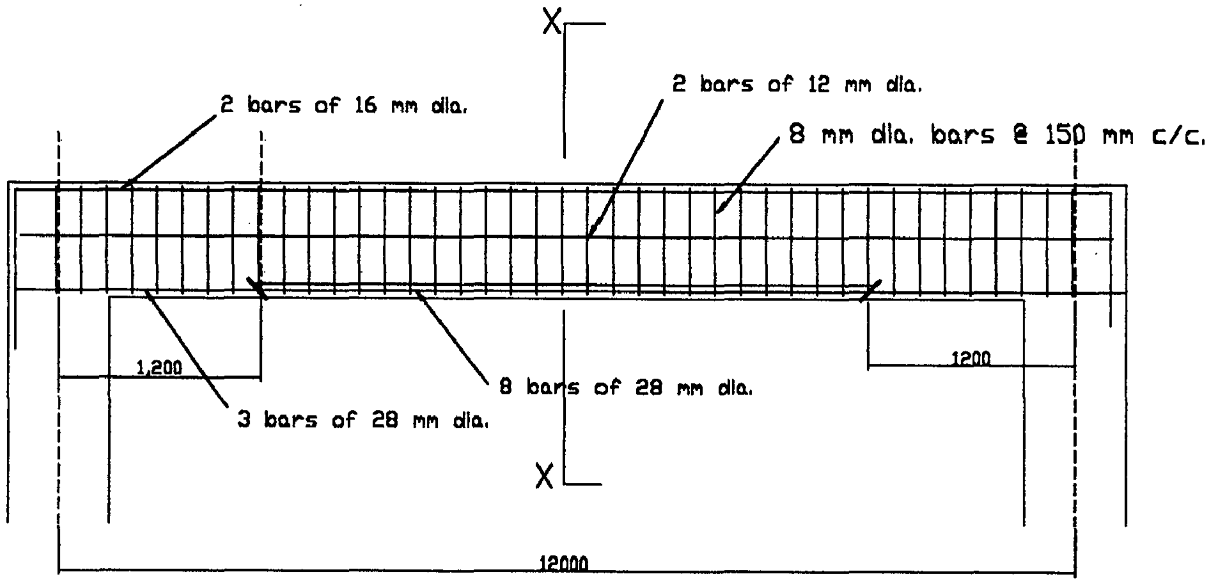


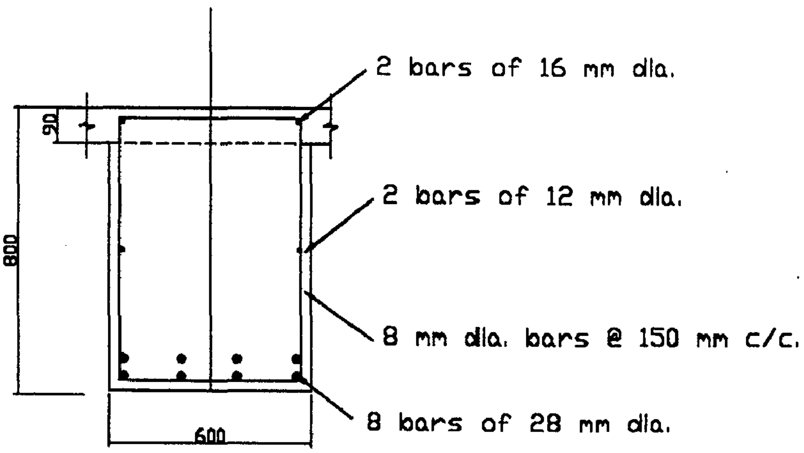

**Fig. 6.1 Floor Plan Example No.1**

**Method of Node , Member Numbering**
**And Rib Numbering**



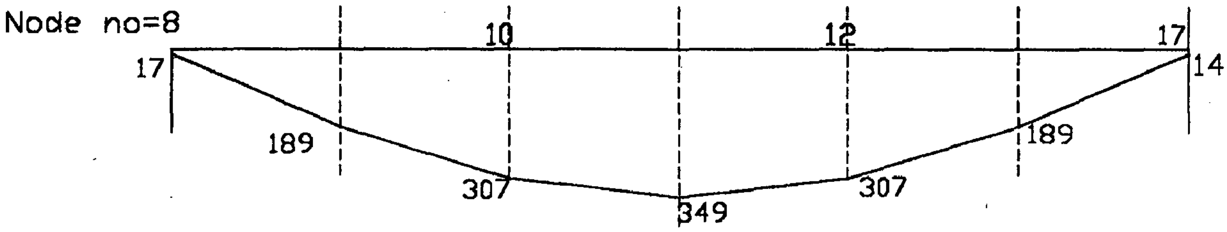

B.M.D. for rib no 1 (Edge beam)



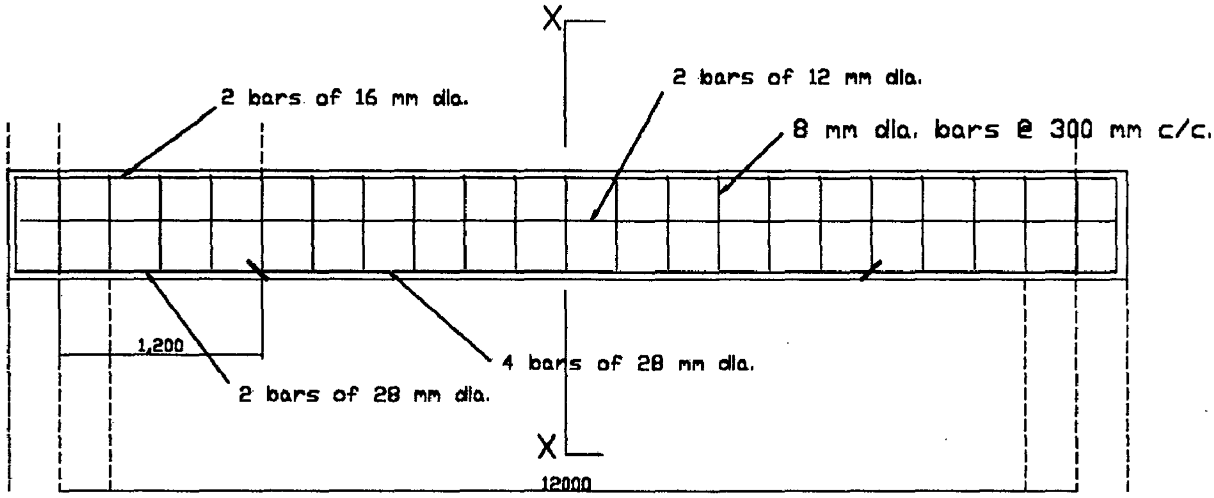



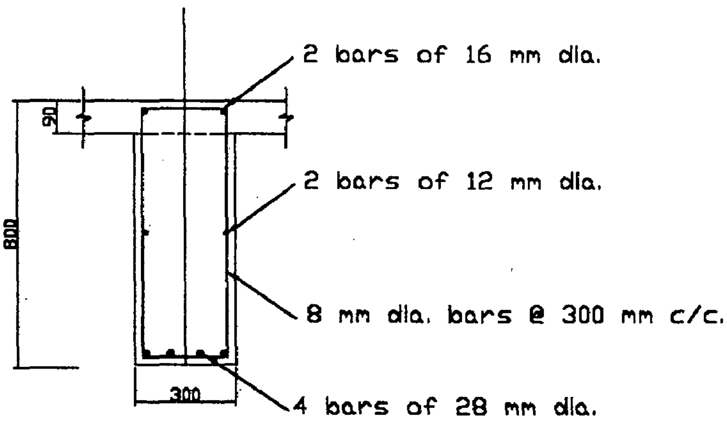

Section At X-X

Fig 6.2 Sections for Rib no 1( Outer Edge beam)



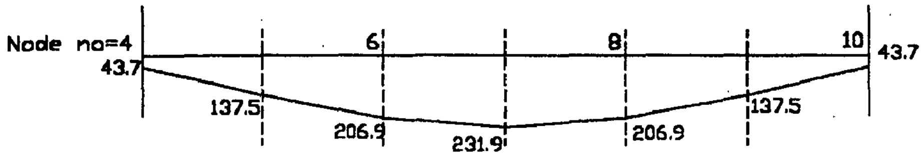

B.M.D. for rib no 2



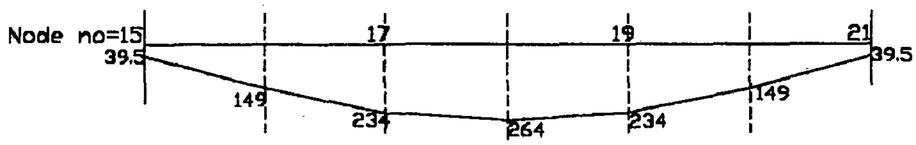




Section At X-X

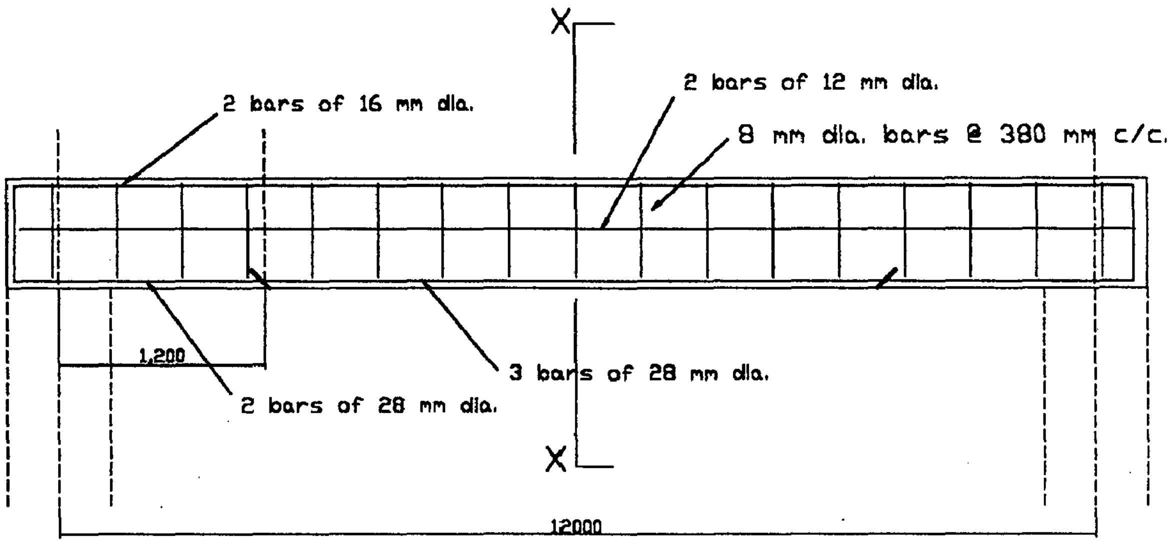
Fig. 6.3 Sections for Rib no 2



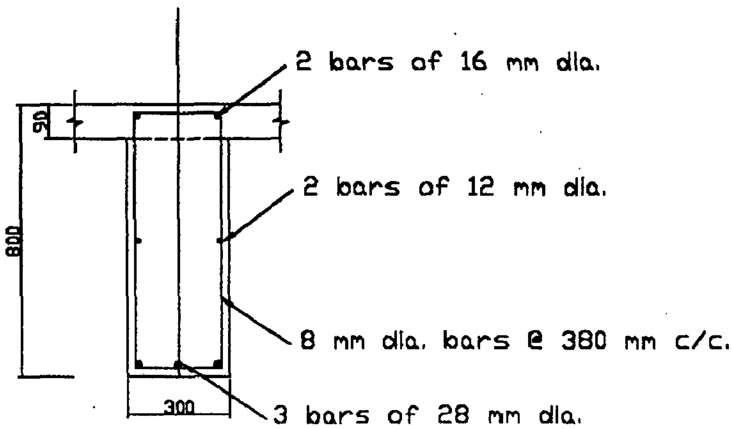

B.M.Dia. for rib no 4 (central rib)




B.M.D. for rib no 3




Longitudinal Section of rib no 3




Section At X-X

Fig. 6.4 Sections for Rib no 3

G10283

### 6.3 Design Example 2

This problem describes the design of a continuous grid floor. In total nine panels of continuous grid floor are to be designed as shown in the Fig. 6.5. The size of individual panels is 12m x 12m and columns are placed at the corners of each panel. Rib spacing in all the panel is 2 m which is same in x and y direction. M-20 grade concrete and Fe-415 grade tor steel has been adopted in the present example. Live load has been assumed to be 3 $kN/m^2$.

**Results of Analysis**

**Table - 6 : Moments and Shears at Various Salient Points in the Grid**

| Point | Mode No. | X (m) | Y (m) | Moment | | Torsion | | Shear | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Mx (kM-m) | My (kM-m) | Mxy (kM-m) | Myx (kM-m) | Qx KM | Qy kM |
| A | 1 | 0 | 0 | 16.2 | 16.2 | 16.2 | 16.2 | 93.4 | 93.4 |
| | 7 | 12 | 0 | 90.6 | 38.8 | 27.6 | 11.5 | 113 | 172 |
| | 10 | 18 | 0 | 179.6 | 20.2 | 10.1 | 0 | 26.0 | 34.2 |
| | 67 | 18 | 6 | 10.1 | 129 | 0 | 0 | 4.2 | 32 |
| | 181 | 18 | 18 | 9.4 | 9.4 | 0 | 0 | 8.8 | 8.8 |
| | 124 | 18 | 6 | 228.0 | 68.1 | 0 | 0 | 0 | 25.38 |
| | 121 | 12 | 12 | 784 | 784.6 | 2.9 | 2.9 | 389 | 389 |
| | 118 | 6 | 12 | 386.5 | 35 | 4.4 | 1.9 | 5.12 | 19.2 |
| | 61 | 6 | 6 | 101 | 101 | 0 | 0 | 20.7 | 20.7 |

Maximum equivalent shear and maximum equivalent moment is found at the nodes (121, 127, 235, 241) of interior columns.

**Table –7 : Selection of Ribs for design (Prob .no. 2)**

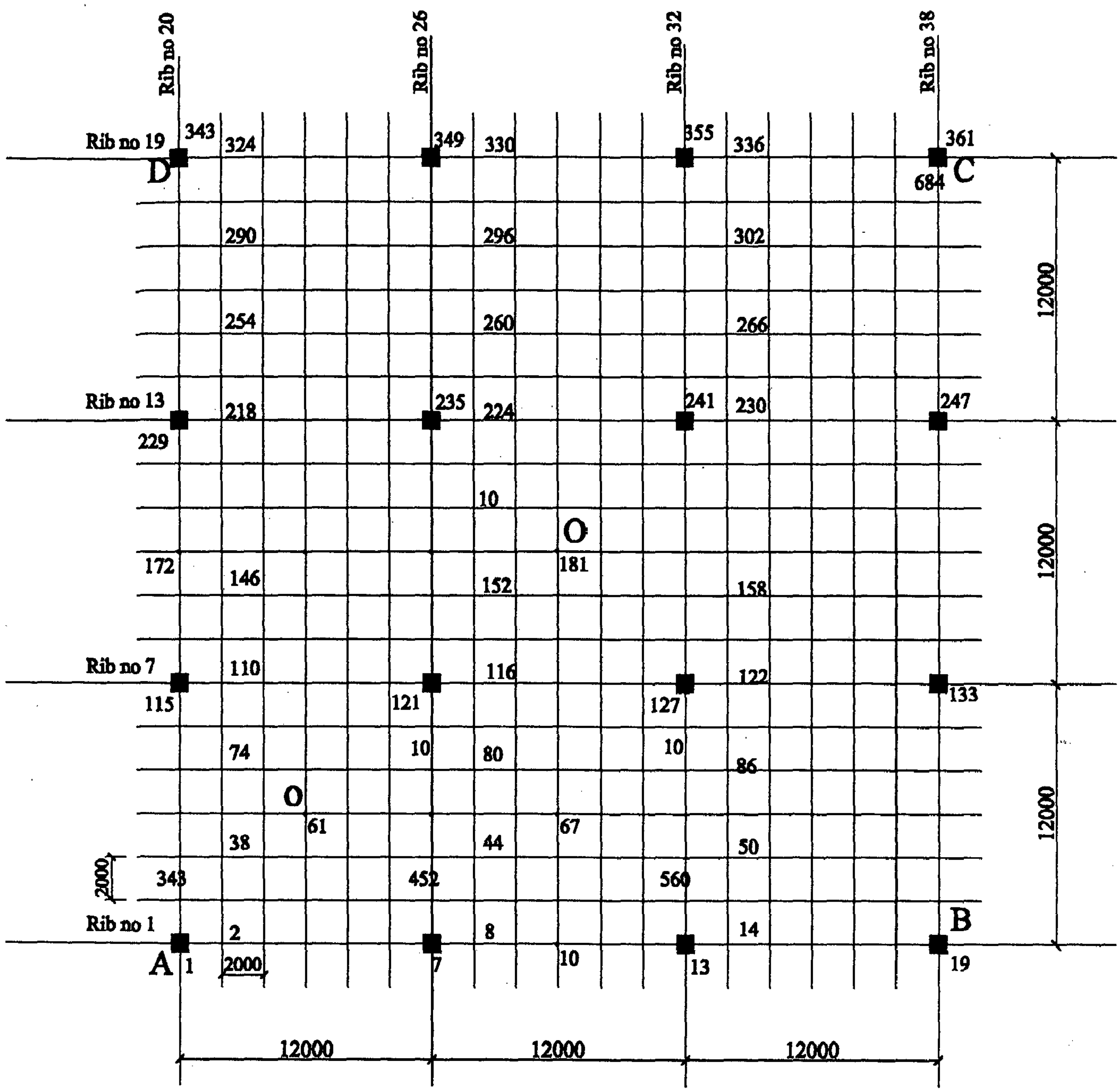
| Rib No. | Max +Ve moment | | | Max – Ve moment |
|---|---|---|---|---|
| | Ist span | IInd span | III span | |
| 1, 19, 20, 38 | 242 | 179.6 | 242 | 507.5 |
| 2, 18, 21, 37 | 121 | 56 | 121 | 129 |
| 7 (Edge beams), 13, 26, 32 | 399 | 228 | 399 | 784.6 |
| 68, 12, 14, 25.27, 29, 31 | 215 | 111 | 215 | 275 |
| 10 Central, 29 Beam | 139 | 9.4 | 139 | 88.78 |
| 9, 11, 28, 30 | 156 | 28.2 | 156 | 111 |
| 4, 16, 23, 34 | 101 | 10.1 | 101 | 69.5 |
| 3, 5, 15, 17, 22, 24, 33, 35 | 91.2 | 7.2 | 91.2 | 65.9 |

Design of following ribs can be clubbed together,

1. All outer edge beams e.g. 1, 19, 20, 38.
2. All intermediate edge beams e.g. 7, 13, 26, 32.
3. Three central ribs in each grid panel in both directions can be designed together.
4. Ribs adjacent to outer edge ribs and adjacent to the intermediate edge ribs should be designed separately.

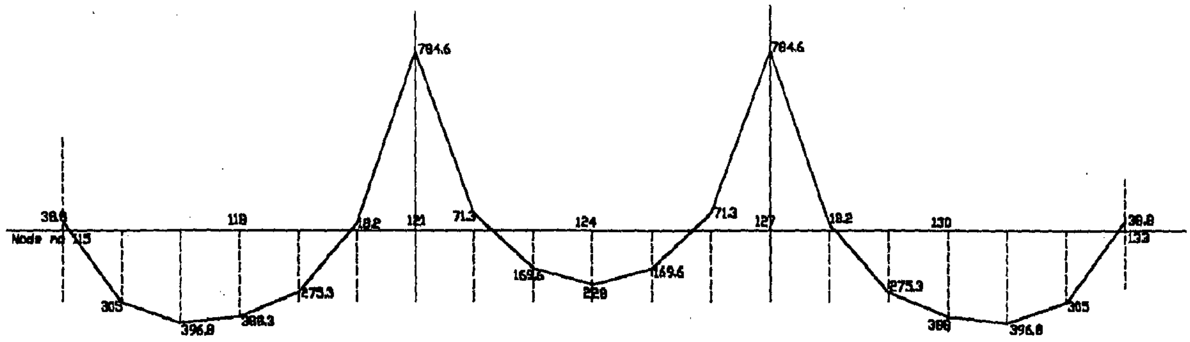
Only three ribs have been designed here namely rib No. 1 (Outer edge beam), rib No (7) (Intermediate edge beam) and rib No.10 the (central rib). There bending moment diagrams with the sectional drawing is shown in the fig.6.6,6.7 and 6.8 The detailed output of the steel design at various sections has been shown in appendix app4.

As the size of the slab panel has not changed so the design similar to that provided in the example 1 should be adopted.

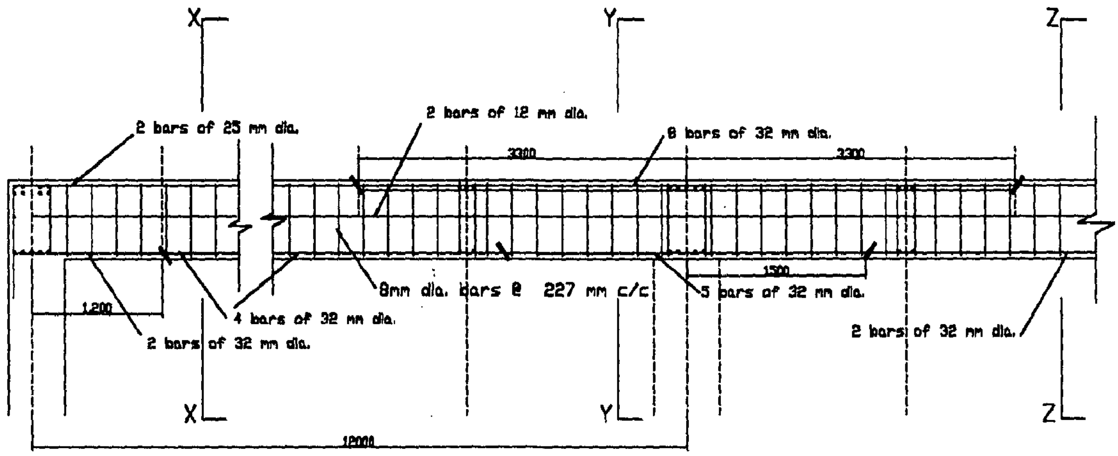



# Fig .6.5 Floor Plan ExampleNo.2

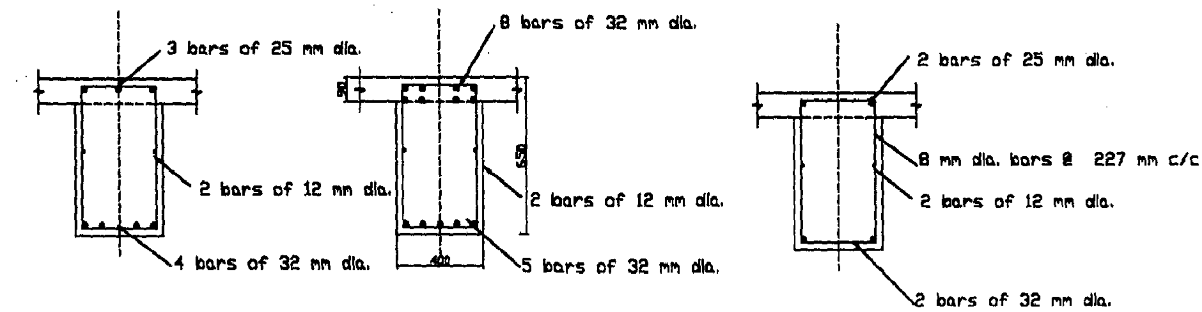
**Method of Node , Member Numbering And Rib Numbering**




B.M.D. for rib no 7 (Intermediate Edge Beam)

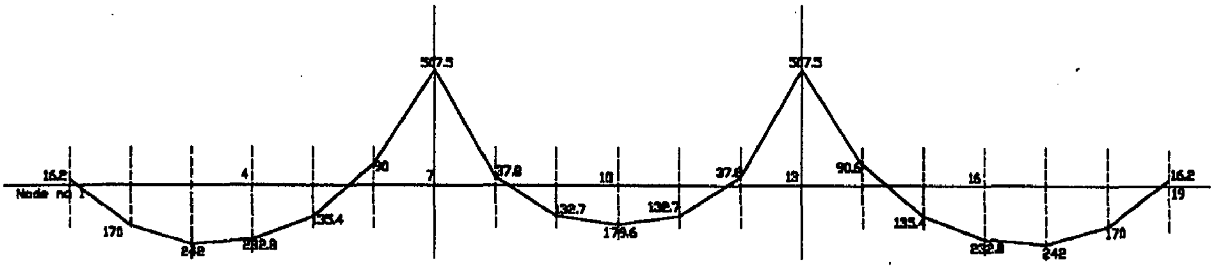



Longitudinal Section of The Rib

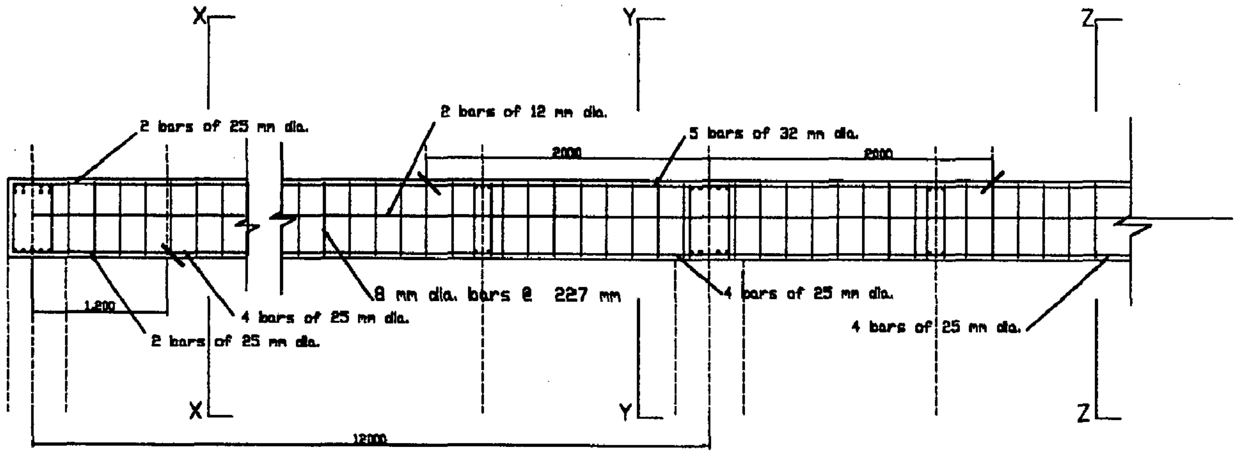



Section At X-X Section At Y-Y Section At Z-Z

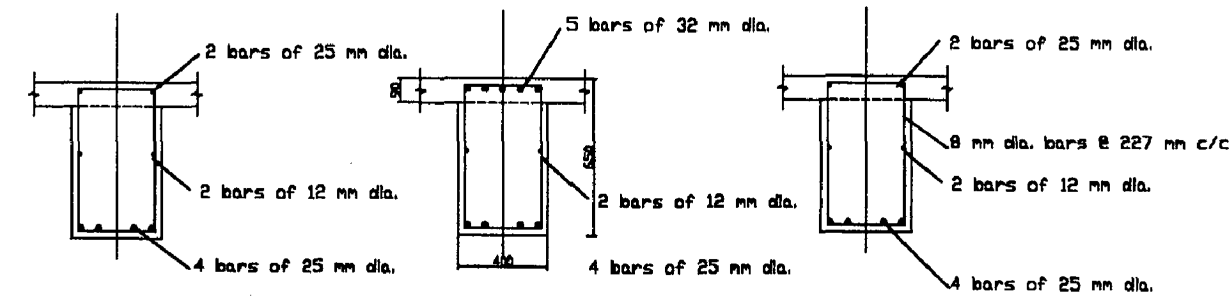
Fig 6.7 Sections for Rib no 7 C Grid (Intermediate Edge Beam)




B.M.D. for rib no 1 (Outer edge beam)




Longitudinal Section of The Rib

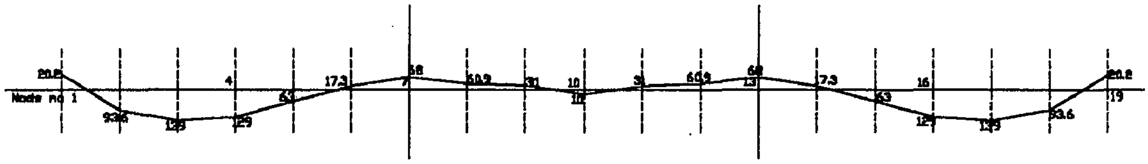



Section At X-X

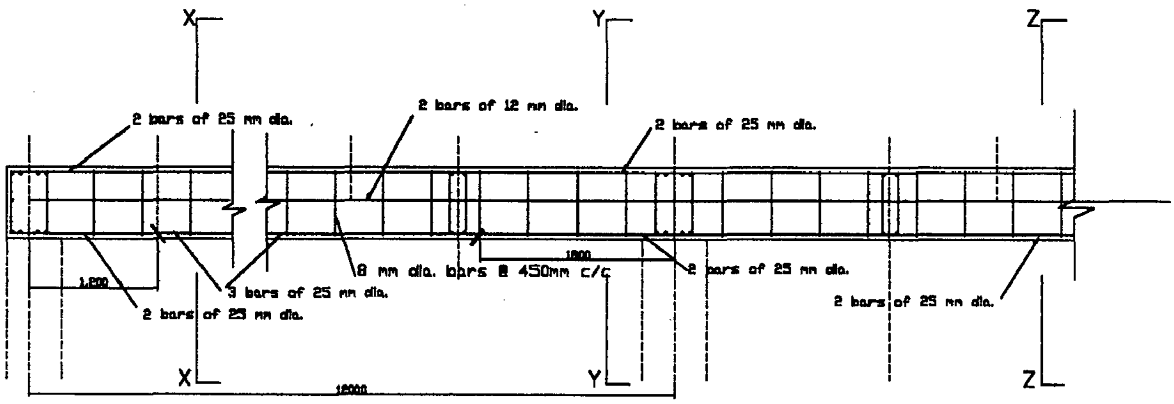
Section At Y-Y

Section At Z-Z

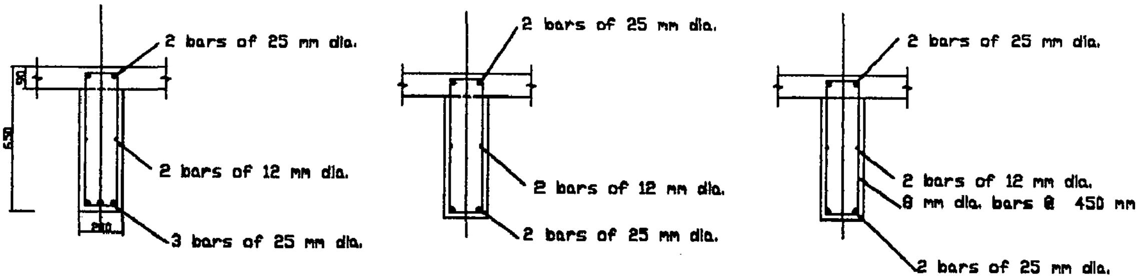
Fig. 6.6 Sections for Rib no 1 C.Grid (Outer Edge Beam)




B.M.D. for rib no 10 (Central Rib)




Longitudinal Section of The Rib




Section At X-X

Section At Y-Y

Section At Z-Z

Fig. 6.8 Sections for Rib no 10 C Grid (central rib)

# CONCLUDING REMARKS

With the easy availability of computing power at the desktops, its application has increased manifold in the field of civil engineering. Now a days computer aided design of structural systems has become commonplace. The C.A.D. package for design of grid floor is a small step in that direction. The package is flexible, easy to understand and use as well. All the latest relevant recommendations of the Indian Standards have been incorporated in the package.

Based upon the work presented following conclusions can be drawn.

The structural behavior of grid floors as observed is as follows:

1. Due to the symmetry the stresses in the ribs equidistant from the central ribs are same.
2. In case of isolated grids the outer edge beams are heavily stressed. The maximum bending moments develop at the center of spans while maximum torsional moments are generated at the corners of the grid and maximum shear forces develop at the mid points of the longer side supports.
3. In case of continuous grid floors the maximum stresses are developed in the intermediate edge beams. Values of maximum equivalent shear and maximum equivalent moments are found at the interior column locations. The central ribs, in outer grids are more stressed as compared to the central rib in the central grid.
4. The shear and torsional moments are negligible in the central ribs and its value increases as we move away from the central rib. The values of bending moments are also very small in the central ribs.

The features of the package can be summarized as:

1. Program can analyze various types of problems, which are general in nature. It is easy to modify it and increase its versatility.

2. As the detailed results of the analysis are available to the design module a rib can be designed in great detail without much effort. A designer can easily economize the grid floor by designing for variation of moments in the grid.

**7.1 Scope of Future Work**

1) If possible programming should be windows based for which languages, namely, visual $C^{++}$, visual basic etc. should be used. This eliminates many drawbacks associated with the D.O.S. environment e.g. memory requirements etc. Most professional packages are windows based.

2) Packages can be made to produce bending moment diagrams for each rib or a grid floor as a whole, that will give the designer a very lucid idea of situation of stresses in the grid floor.

3) A professional C.A.D. package for grid floor can be designed to produce the drawings of the reinforcement to be provided.

4) Further C.A.D. packages for other type of grid floors can be developed such as;

   a) Grid in grid floor for large spans.

   b) Prestessed and partially prestessed grid floors.

   c) Skewed grid floor i.e. diagrids.

5) Automatic optimization of a grid floor in terms of most economical rib spacing, depth & thickness of ribs, edge beams, and slab can also be made in to a feature of C.A.D. package.

6) The facility to estimate the quantities of steel and concrete can be a part of the package.

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The sectional properties EI and GJ can be calculated as follows;

Moment of inertia (I):

Moment of inertia of beams are calculated in the usual manner considering. The composite cross section of a T-beam.

Torsional Constant (J):

The procedure is as follows;

i) Rectangular Section:

Torsional constant for rectangle shown in Fig A-1 . with sides a and b is given by $J = Kba^3$ where a is the shorter side. K is the Timoshinko torsion coefficient, is a function of aspect ratio b/a .& is obtained from Fig A-1 (a) . Alternatively K can be calculated by the formula given below

$$k = \frac{1}{3}\left[1 - \frac{0.63}{b/a}\right]$$

ii) T-Section

Here the torsional constant is obtained by conceptually dividing the T-section in to rectangles and summing the values of j for the individual elements.

iii) Modulus of elasticity (E):

Short term modulus of elasticity of concrete is given under (as per I.S. 456-2000)

$$Ec = 5000\sqrt{f_{ck}}$$

$f_{ck}$ = Characteristic cube strength of concrete (in $N/mm^2$).

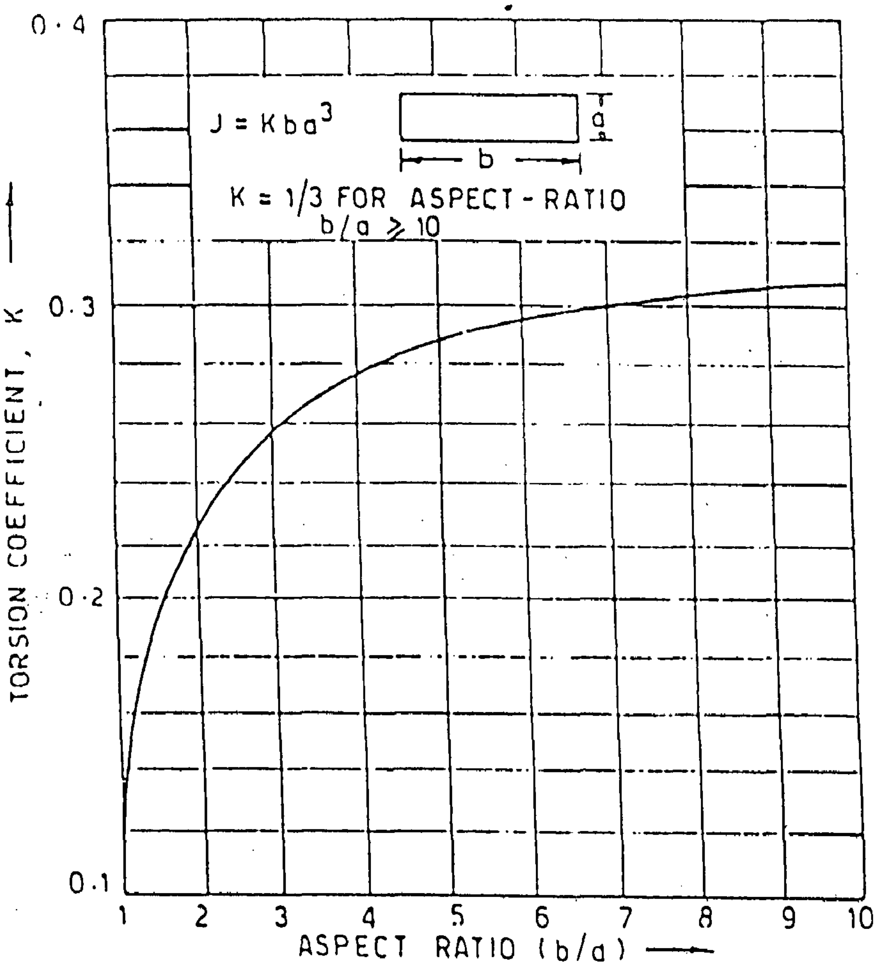
iv) Shear Modulus (G):

$$G = \frac{E}{2\ (1+v)}$$

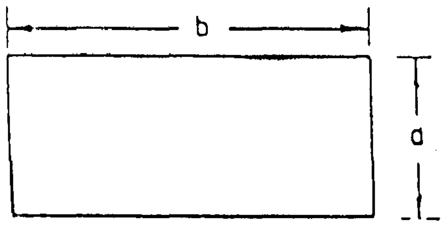
Poisson ratio $\gamma$ may be taken as 0.15.

EI and GJ then can be calculated by multiplying the respective quantities

| b/a | 1 | 1.5 | 2 | 2.5 | 3 | 4 | 6 | 10 | ∞ |
|---|---|---|---|---|---|---|---|---|---|
| K | 0.141 | 0.196 | 0.229 | 0.249 | 0.263 | 0.281 | 0.299 | 0.312 | 0.333 = 1/3 |

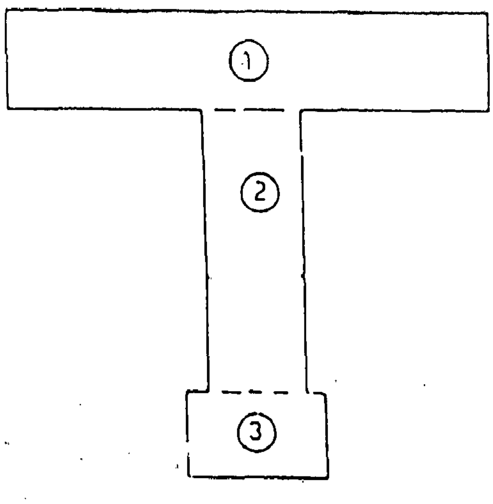



(a) K VS b/a




(b) RECTANGULAR SECTION

$J = Kba^3$




(c) T-SECTION

TORSIONAL CONSTANT COMPUTATION

## APPENDIX APP1

Choose and enter the member no you want to design from the table no-1 where the shear torsion and moments are given after closing the editor window. Equivalent moments have been calculated in table no-2 as an aid for_choosing the section for_designing. Maximum and minmum Equivalent moments have been extracted for_the same reason.

CHECK FOR DEFLECTION
The maximum deflecion =18.088 MM
The node of maximum deflection is=25

The grid is safe in deflection

TABLE 1

| i no | j | shear | torsion | moment1 | moment2 |
|---|---|---|---|---|---|
| member no=1 | | | | | |
| 1 | 2 | -218161 | 78552656 | -78537264 | -357784192 |
| member no=2 | | | | | |
| 2 | 3 | -109307 | 61426760 | 340583712 | -559196672 |
| member no=3 | | | | | |
| 3 | 4 | -33667 | 21872500 | 549765376 | -617098368 |
| member no=4 | | | | | |
| 4 | 5 | 33670 | -21872440 | 617097984 | -549757184 |
| member no=5 | | | | | |
| 5 | 6 | 109309 | -61428432 | 559190272 | -340572992 |
| member no=6 | | | | | |
| 6 | 7 | 218165 | -78553936 | 357774912 | 78553936 |
| member no=7 | | | | | |
| 8 | 9 | -83923 | 17201050 | 17124810 | -184970400 |
| member no=8 | | | | | |
| 9 | 10 | -58990 | 12660190 | 189511600 | -307490208 |
| member no=9 | | | | | |
| 10 | 11 | -19733 | 4483119 | 309637792 | -349104384 |
| member no=10 | | | | | |
| 11 | 12 | 19741 | -4483228 | 349104096 | -309622912 |
| member no=11 | | | | | |
| 12 | 13 | 58990 | -12660530 | 307481408 | -189501408 |
| member no=12 | | | | | |
| 13 | 14 | 83921 | -17201320 | 184965296 | -17123780 |
| member no=13 | | | | | |
| 15 | 16 | -50712 | 9431393 | 39549432 | -140973104 |
| member no=14 | | | | | |

16 17 -40095 7283903 149150800 -229340192
member no=15
17 18 -15161 2694195 233930400 -264252000
member no=16
18 19 15168 -2694614 264253904 -233917200
member no=17
19 20 40093 -7283965 229328400 -149143296
member no=18
20 21 50705 -9431363 140965200 -39556672

member no=19
22 23 -42409 44 43737380 -128555104
member no=20
23 24 -32008 -162 137521600 -201537296
member no=21
24 25 -12467 65 206927392 -231860496
member no=22
25 26 12469 -350 231858704 -206921408
member no=23
26 27 32010 105 201534304 -137514800
member no=24
27 28 42401 312 128548600 -43747008

member no=25
29 30 -50711 -9431675 39548208 -140970304
member no=26
30 31 -40098 -7284146 149146208 -229341792
member no=27
31 32 -15162 -2694629 233929408 -264253696
member no=28
32 33 15163 2694722 264252000 -233926592
member no=29
33 34 40095 7284329 229335600 -149146000
member no=30
34 35 50707 9431752 140967808 -39555020
member no=31
36 37 -83925 -17201470 17122550 -184970896
member no=32
37 38 -58987 -12660440 189514304 -307488096
member no=33
38 39 -19735 -4483030 309638304 -349107392
member no=34
39 40 19737 4483445 349108416 -309635392
member no=35

40 41 58992 12660760 307487104 -189504304
member no=36
41 42 83921 17201470 184966000 -17125500
member no=37
43 44 -218164 -78554096 -78537960 -357788896
member no=38
44 45 -109310 -61427552 340584512 -559204480
member no=39
45 46 -33669 -21871570 549770304 -617108992
member no=40
46 47 33673 21873960 617109632 -549763520
member no=41
47 48 109311 61429008 559196608 -340574400
member no=42
48 49 218165 78553856 357776384 78552976

member no=43
1 8 -218170 -78537640 -78552736 -357786304
member no=44
8 15 -109313 -61415420 340585312 -559210496
member no=45
15 22 -33668 -21868830 549779776 -617115904
member no=46
22 29 33669 21868800 617116032 -549778816
member no=47
29 36 109314 61417560 559209920 -340581696
member no=48
36 43 218171 78539856 357785600 78555136

member no=49
2 9 -83922 -17201270 17125560 -184968800
member no=50
9 16 -58993 -12660550 189508608 -307493696
member no=51
16 23 -19733 -4483296 309639904 -349106816
member no=52
23 30 19735 4483471 349104288 -309633408
member no=53
30 37 58987 12660930 307484608 -189511504
member no=54
37 44 83921 17201550 184967504 -17126610

member no=55

3 10 -50709 -9431408 39553848 -140971392
member no=56
10 17 -40096 -7284021 149147696 -229339504
member no=57
17 24 -15160 -2694357 233928704 -264247696
member no=58
24 31 15161 2694671 264245600 -233923696
member no=59
31 38 40093 7284229 229333200 -149147696
member no=60
38 45 50707 9431417 140970704 -39556032

member no=61
4 11 -42403 -73 43745500 -128551696
member no=62
11 18 -32006 176 137520000 -201532096
member no=63
18 25 -12471 215 206922400 -231864496
member no=64
25 32 12462 61 231845408 -206922000
member no=65
32 39 32005 -87 201533200 -137524000
member no=66
39 46 42402 -279 128553000 -43748352

member no=67
5 12 -50705 9431663 39556420 -140966096
member no=68
12 19 -40095 7284189 149144896 -229335504
member no=69
19 26 -15165 2694636 233924496 -264254800
member no=70
26 33 15163 -2694694 264251296 -233926304
member no=71
33 40 40093 -7284523 229336800 -149151200
member no=72
40 47 50710 -9431856 140974496 -39554840

member no=73
6 13 -83922 17201820 17124670 -184967600
member no=74
13 20 -58993 12660740 189507696 -307493888
member no=75

| | | | | | | |
|---|---|---|---|---|---|---|
| 20 | 27 | -19735 | 4483142 | 309639616 | -349110016 |
member no=76

| | | | | | | |
|---|---|---|---|---|---|---|
| 27 | 34 | 19734 | -4483646 | 349110112 | -309641216 |

member no=77

| | | | | | | |
|---|---|---|---|---|---|---|
| 34 | 41 | 58990 | -12661010 | 307493184 | -189513104 |

member no=78

| | | | | | | |
|---|---|---|---|---|---|---|
| 41 | 48 | 83923 | -17201880 | 184971696 | -17125050 |

member no=79

| | | | | | | |
|---|---|---|---|---|---|---|
| 7 | 14 | -218161 | 78553640 | -78553344 | -357767616 |

member no=80

| | | | | | | |
|---|---|---|---|---|---|---|
| 14 | 21 | -109309 | 61427792 | 340565312 | -559182976 |

member no=81

| | | | | | | |
|---|---|---|---|---|---|---|
| 21 | 28 | -33665 | 21871890 | 549753920 | -617083776 |

member no=82

| | | | | | | |
|---|---|---|---|---|---|---|
| 28 | 35 | 33669 | -21874360 | 617085184 | -549747776 |

member no=83

| | | | | | | |
|---|---|---|---|---|---|---|
| 35 | 42 | 109305 | -61429360 | 559178816 | -340568800 |

member no=84

| | | | | | | |
|---|---|---|---|---|---|---|
| 42 | 49 | 218162 | -78553776 | 357769408 | 78554176 |

TABLE 2

| member no | i node | j node | Eqmoment | Eqshear |
|---|---|---|---|---|
| 1 | 1 | 2 | 465601568 | 427635 |
| 2 | 2 | 3 | 643507904 | 273112 |
| 3 | 3 | 4 | 647119424 | 91994 |
| 4 | 4 | 5 | 647118976 | 91997 |
| 5 | 5 | 6 | 643503808 | 273118 |
| 6 | 6 | 7 | 465594048 | 427642 |
| 7 | 8 | 9 | 222070704 | 175662 |
| 8 | 9 | 10 | 334796512 | 126511 |
| 9 | 10 | 11 | 358773856 | 43643 |
| 10 | 11 | 12 | 358773792 | 43652 |
| 11 | 12 | 13 | 334788448 | 126513 |
| 12 | 13 | 14 | 222066176 | 175661 |
| 13 | 15 | 16 | 161315328 | 101013 |
| 14 | 16 | 17 | 245050576 | 78942 |
| 15 | 17 | 18 | 270063008 | 29530 |
| 16 | 18 | 19 | 270065824 | 29539 |
| 17 | 19 | 20 | 245038912 | 78941 |
| 18 | 20 | 21 | 161307360 | 101006 |
| 19 | 22 | 23 | 128555200 | 42409 |
| 20 | 23 | 24 | 201537648 | 32009 |

| | | | | |
|---|---|---|---|---|
| 21 | 24 | 25 | 231860640 | 12467 |
| 22 | 25 | 26 | 231859456 | 12471 |
| 23 | 26 | 27 | 201534528 | 32011 |
| 24 | 27 | 28 | 128549272 | 42403 |
| 25 | 29 | 30 | 161313136 | 101013 |
| 26 | 30 | 31 | 245052688 | 78947 |
| 27 | 31 | 32 | 270065632 | 29533 |
| 28 | 32 | 33 | 270064160 | 29535 |
| 29 | 33 | 34 | 245046896 | 78945 |
| 30 | 34 | 35 | 161310800 | 101010 |
| 31 | 36 | 37 | 222072112 | 175666 |
| 32 | 37 | 38 | 334794912 | 126509 |
| 33 | 38 | 39 | 358776672 | 43644 |
| 34 | 39 | 40 | 358778592 | 43649 |
| 35 | 40 | 41 | 334794624 | 126516 |
| 36 | 41 | 42 | 222067216 | 175662 |
| 37 | 43 | 44 | 465608256 | 427642 |
| 38 | 44 | 45 | 643516800 | 273117 |
| 39 | 45 | 46 | 647128768 | 91993 |
| 40 | 46 | 47 | 647132736 | 92004 |
| 41 | 47 | 48 | 643510912 | 273122 |
| 42 | 48 | 49 | 465595392 | 427642 |
| 43 | 1 | 8 | 465583072 | 427604 |
| 44 | 8 | 15 | 643506176 | 273087 |
| 45 | 15 | 22 | 647131968 | 91985 |
| 46 | 22 | 29 | 647132032 | 91986 |
| 47 | 29 | 36 | 643508544 | 273094 |
| 48 | 36 | 43 | 465585408 | 427611 |
| 49 | 2 | 9 | 222069584 | 175662 |
| 50 | 9 | 16 | 334800768 | 126516 |
| 51 | 16 | 23 | 358776672 | 43644 |
| 52 | 23 | 30 | 358774528 | 43647 |
| 53 | 30 | 37 | 334792512 | 126512 |
| 54 | 37 | 44 | 222068880 | 175663 |
| 55 | 3 | 10 | 161313648 | 101010 |
| 56 | 10 | 17 | 245050144 | 78944 |
| 57 | 17 | 24 | 270059040 | 29530 |
| 58 | 24 | 31 | 270057632 | 29533 |
| 59 | 31 | 38 | 245044288 | 78942 |
| 60 | 38 | 45 | 161312976 | 101008 |
| 61 | 4 | 11 | 128551856 | 42403 |
| 62 | 11 | 18 | 201532480 | 32007 |
| 63 | 18 | 25 | 231864960 | 12472 |
| 64 | 25 | 32 | 231845536 | 12462 |
| 65 | 32 | 39 | 201533392 | 32005 |

| | | | | |
|---|---|---|---|---|
| 66 | 39 | 46 | 128553600 | 42403 |
| 67 | 5 | 12 | 161308896 | 101007 |
| 68 | 12 | 19 | 245046496 | 78944 |
| 69 | 19 | 26 | 270066752 | 29536 |
| 70 | 26 | 33 | 270063392 | 29535 |
| 71 | 33 | 40 | 245048512 | 78944 |
| 72 | 40 | 47 | 161317712 | 101013 |
| 73 | 6 | 13 | 222069568 | 175665 |
| 74 | 13 | 20 | 334801376 | 126517 |
| 75 | 20 | 27 | 358779552 | 43645 |
| 76 | 27 | 34 | 358780736 | 43647 |
| 77 | 34 | 41 | 334801248 | 126515 |
| 78 | 41 | 48 | 222073792 | 175666 |
| 79 | 7 | 14 | 465586336 | 427637 |
| 80 | 14 | 21 | 643495616 | 273116 |
| 81 | 21 | 28 | 647104000 | 91990 |
| 82 | 28 | 35 | 647108800 | 92001 |
| 83 | 35 | 42 | 643493632 | 273117 |
| 84 | 42 | 49 | 465588320 | 427639 |

CHECK FOR MAXIMUM EQUIVALENT SHEAR
The maximum equivalent shear is = 427642 Nmm2
The member of maximum equivalent shear is= 6
Grid is safe in combined action of shear & torsion

The maximum equivalent moment is = 647132736 Nmm2
The member of maximum equivalent moment is= 40

The minimum equivalent moment is = 28549272.0 Nmm2
The member of minimum equivalent moment is = 24

CHECK FOR MAXIMUM EQUIVALENT MOMENT
Open the Result.dat file and find the results

## APPENDIX APP2

***** DESIGN OF VARIOUS RIBS *****

Following are the dimensions of the sections
Gross depth of internal ribs = 800 mm
Width of internal ribs = 300 mm
Gross depth of edge ribs = 800 mm
Width of edge ribs = 600 mm

CHECK FOR MAXIMUM EQUIVALENT MOMENT
The member of max eq.moment has following properties
Member no= 3 inode= 3 j node= 4
Shear = 33673 N
Torsion = 21873960 N-mm
Moment at end one= 617109632 N-mm
Moment at end two=-549763520 N-mm
The eff. depth is= 750 mm
Design for j node moment

LONGITUDINAL REINFORCEMENT

Equivalent design moment for tension face = 970699072 N-mm
Section is singly reinforced (for T-section)
Area of actual calculated steel is 4531.7 mm2
Area of tension steel is less than Max (4 percent of gross X-section) area of steel. Hence O.K.
Provide 8 - 28 mm dia bars in tension
Total area of tension steel provided = 4926 mm2
Percentage area of tension steel provided = 1.0909 mm2

SHEAR REINFORCEMENT

Value of design shear st (from table) = 0.638
Value of MAX shear st (table) = 2.800
Provide min shear reinforcement
Provide 8 mm - 2 legged MS stirrups @ 151 mm c/c

SIDEFACE REINFORCEMENT.

Since the depth exceeds 450 mm.
Provide 0.1% (of the web area) steel along the verical sides
Provide 2 - 12 mm dia bars on side faces
The bars shall be equidistant from both the ends

*** DESIGN OF RIB NO = 1 ***

The max positive moment in the rib is = 78537264 N-mm
The max negative moment in the rib is =507587040 N-mm

The points of contraflexure in the rib lies at the following distance from the left side of the rib=476 mm ,11524 mm

Member no 3 of the rib no 1 has already been designed

The member you have chosen has following properties
Member no= 1 i node= 1 j node= 2
Shear = -218161 N
Torsion = 78552656 N-mm
Moment at i node = -78537264 N-mm
Moment at j node =-357784192 N-mm
The eff. depth is= 750 mm
Design for i node moment

LONGITUDINAL REINFORCEMENT

Equivalent design moment for tension face = 279531968 N-mm
Section is singly reinforced (rectangular section)
Area of calculated steel is = 1086.8 mm2
Area of tension steel is less than Max (4 percent of gross X-section) area of steel. Hence O.K.
Provide 4 - 20 mm dia bars in tension
Total area of tension steel provided = 1257 mm2
Percentage area of tension steel provided = 0.279 mm2

SHEAR REINFORCEMENT

Value of design shear st (from table) = 0.374
Value of MAX shear st (table) = 2.800
Provide foolowing shear reinforcement
Provide 8 mm - 2 legged MS stirrups @ 151 mm c/c

SIDEFACE REINFORCEMENT.

Since the depth exceeds 450 mm.
Provide 0.1% (of the web area) steel along the verical sides
Provide 2 - 12 mm dia bars on side faces
The bars shall be equidistant from both the ends

*** DESIGN OF RIB NO = 2 ***

The max positive moment in the rib is = 349104384 N-mm
The max negative moment in the rib is = 0 N-mm
There is no point of contraflexure in the rib

The member you have chosen has following properties
Member no= 9 i node= 10 j node= 11
Shear = -19733 N
Torsion = 4483119 N-mm
Moment at i node = 309637792 N-mm
Moment at j node =-349104384 N-mm

The eff. depth is= 750 mm
Design for j node moment

LONGITUDINAL REINFORCEMENT

Equivalent design moment for tension face = 538160768 N-mm.
NA lies in the web
Area of tension steel is less than Max (4 percent of gross X-section) area of steel. Hence O.K.
Area of steel is 2045.3 mm2
Provide 4 - 28 mm dia. bars in tension
Total area of tension steel provided = 2462.8 mm2
Percentage area of tension steel provided = 0.977511 mm2

SHEAR REINFORCEMENT

Value of design shear st (from table) = 0.614
Value of MAX shear st (table) = 2.800
Provide min shear reinforcement
Provide 8 mm - 2 legged MS stirrups @ 302 mm c/c

SIDEFACE REINFORCEMENT.

Since the depth exceeds 450 mm.
Provide 0.1% (of the web area) steel along the verical sides
Provide 2 - 12 mm dia bars on side faces
The bars shall be equidistant from both the ends

*** DESIGN OF RIB NO = 3 ***

The max positive moment in the rib is =264252000 N-mm N-mm
The max negative moment in the rib is =0 N-mm
There is no point of contraflexure in the rib

The member you have chosen has following properties
Member no= 15 i node= 17 j node= 18
Shear = -15161 N
Torsion = 2694195 N-mm
Moment at i node = 233930400 N-mm
Moment at j node =-264252000 N-mm
The eff. depth is= 750 mm
Design for j node moment

LONGITUDINAL REINFORCEMENT

Equivalent design moment for tension face = 405094496 N-mm
NA lies in the web
Area of tension steel is less than Max (4 percent of gross X-section) area of steel. Hence O.K.
Area of steel is 1528.3 mm2
Provide 3 - 28 mm dia bars in tension
Total area of tension steel provided = 1847.1 mm2

Percentage area of tension steel provided = 0.698 mm2

SHEAR REINFORCEMENT

Value of design shear st (from table) = 0.543
Value of MAX shear st (table) = 2.800
Provide min shear reinforcement
Provide 8 mm - 2 legged MS stirrups @ 302 mm c/c

SIDEFACE REINFORCEMENT.

Since the depth exceeds 450 mm.
Provide 0.1% (of the web area) steel along the verical sides
Provide 2 - 12 mm dia bars on side faces
The bars shall be equidistant from both the ends

**APPENDIX --- App3**

***** Design of Slab Panels ****

Minimum positive R/F is being provided in the short span

The area of minimum positive R/F for the short span is= 151 mm2

Minimum positive R/F is being provided in the long span

The area of minimum positive R/F for the long span is= 151 mm2

Following design of steel shall be followed in all the slab panels

The gross depth of slab shall be = 90 mm

These reinforcements shall be provided in the middle strip in two directions

The length of middle strip in long direction =1500 mm

The length of middle strip in short direction =1500 mm

The area of reinforcements provided shall be as follows

Area of top Reinforcement for short span = 151 mm2

Area of top Reinforcement for long span = 151 mm2

Area of bottom Reinforcement for short span = 151 mm2

Area of bottom Reinforcement for long span = 151 mm2

Adopting 8 - mm dia bars the spacing of the R/F shall be as follows

Top Reinforcement for short span = 195 mm, long span = 195 mm

Bottom Reinforcement for short span= 195 mm, long span = 195 mm

The length of -ve R/F for continuous edges shall be as follows

Top R/F for short span shall extend into slab for a distance of = 200 mm

Atleast 50 per of top R/F for short span shall extend into slab

for a distance of = 600 mm

Top R/F for long span shall extend into slab for a distance of = 200 mm

Atleast 50 per of top R/F for long span shall extend into slab

for a distance of = 600 mm

Following reinforcements shall be provided in the edge strips

in the two directions

Adopting 8 mm dia bars the spacing of the R/F shall be = 333 mm

These reinforcements shall be provided at the top at discontinuous edges

Following reinforcements shall be provided at the top for short edge

Adopting 8 mm dia bars the spacing of the R/F shall be 500 mm

The length of such R/F shall be 200 mm

Following reinforcements shall be provided at the top for long edge

Adopting 8 mm dia bars the spacing of the R/F shall be 500 mm

The length of such R/F shall be 200 mm

## DESIGN FOR TORSION STEEL

These bars are for the corner where two discontinuous edges meet

These bars shall be provided in four layers

The total no of such bars shall be (for each layer) = 2

Use 8 mm dia.bars @ 200 mm c/c along short span

The length of bars shall be 400 mm along short span

Use 8 mm dia.bars @ 200 mm c/c along long span

The length of bars shall be 400 mm along long span

These bars are for the corner where one discontinuous edge and one continuous edge meet

The total no of such bars shall be (for each layer) 2

Use 8 mm dia.bars @ 400 c/c mm along short span

The length of bars shall be 400 mm along short span

Use 8 mm dia.bars @ 400 mm c/c along long span

The length of bars shall be 400 mm along long span

## CHECK FOR SHEAR

The slab is safe in shear

**APPENDIX App4**

***** DESIGN OF VARIOUS RIBS *****

Following are the dimensions of the sections
Gross depth of internal ribs = 650 mm
Width of internal ribs = 200 mm
Gross depth of edge ribs = 650 mm
Width of edge ribs = 400 mm
Gross depth of inter. col. to col ribs = 650 mm
Width of inter. col. to col. ribs = 400 mm

CHECK FOR MAXIMUM EQUIVALENT MOMENT
The member of max eq.moment has following properties
Member no=564 i node=108 j node=127
Shear = 389265 N
Torsion = 2905132 N-mm
Moment at i node = -6090498 N-mm
Moment at j node = 784620352 N-mm
The eff. depth is= 600 mm
Design for j node moment
The bending moment is hogging

LONGITUDINAL REINFORCEMENT

Equivalent design moment for tension face = 1183659392 N-mm
Section is doubly reinforced
Total area of calculated compression steel= 4147 mm2
Total area of calculated tension steel = 6360 mm2
Area of tension steel is less than Max (4 percent of gross X-section) area of steel. Hence O.K.
Provide 13 - 25 mm dia bars in tension
Total area of tension steel provided = 6381 mm2
Percentage area of tension steel provided = 2.658822 mm2
Provide 9 - 25 mm dia bars in compression
Total area of provided compression steel = 4418 mm2

SHEAR REINFORCEMENT

Value of design shear st (from table)= 0.82
Value of MAX shear st (table) = 2.80
Provide following shear reinforcement
Use 8 mm dia.2 legged vertical stirrups @ 87 mm c/c

SIDEFACE REINFORCEMENT.

Since the depth exceeds 450 mm.
Provide 0.1% (of the web area) steel along the verical sides
Provide 2 - 12 mm dia bars on side faces

The bars shall be equidistant from both the ends

*** DESIGN OF RIB NO = 1 ***

The max positive moment in the rib is =242066448 N-mm
The max negative moment in the rib is =507587040 N-mm
The points of contraflexure in the rib lies at the following distance from the left side of the rib=530 mm ,9357 mm , 14580 mm , 21420 mm ,26643 mm 35470 mm

Design for max. positive moment in the rib

Member no= 2 i node= 2 j node= 3
Shear = -37437 N
Torsion = 7761762 N-mm
Moment at i node = 167193104 N-mm
Moment at j node =-242066448 N-mm
The eff. depth is= 600 mm
Design for j node moment

LONGITUDINAL REINFORCEMENT

Equivalent design moment for tension face = 381077280 N-mm
NA lies in the web
Area of tension steel is less than Max (4 percent of gross X-section) area of steel. Hence O.K.
Area of steel is 1854.2 mm2
Provide 4 - 25 mm dia bars in tension
Total area of tension steel provided = 1964 mm2
Percentage area of tension steel provided = 0.818 mm2

SHEAR REINFORCEMENT

Value of design shear st (from table) = 0.576
Value of MAX shear st (table) = 2.800
Provide following shear reinforcement
Provide 8 mm - 2 legged MS stirrups @ 227 mm c/c

SIDEFACE REINFORCEMENT.

Since the depth exceeds 450 mm.
Provide 0.1% (of the web area) steel along the verical sides
Provide 2 - 12 mm dia bars on side faces
The bars shall be equidistant from both the ends

Design for max. negative moment in the rib

Member no= 6 i node= 6 j node= 7
Shear = 211915 N
Torsion = -13873494 N-mm

Moment at i node = -83757608 N-mm
Moment at j node = 507587040 N-mm
The eff. depth is= 600 mm
Design for j node moment

LONGITUDINAL REINFORCEMENT

Equivalent design moment for tension face = 729247104 N-mm

Section is doubly reinforced

Total area of calculated compression steel= 1750 mm2
Total area of calculated tension steel = 4012 mm2
Area of tension steel is less than Max (4 percent of gross X-section) area of steel. Hence O.K.
Provide 5 - 32 mm dia bars in tension
Total area of tension steel provided = 4201 mm2
Percentage area of tension steel provided = 1.84 mm2
Provide 4 - 25 mm dia bars in compression
Total area of provided compression steel = 1963 mm2
-ve steel shall extend 750 mm from the point of contraflexure

SHEAR REINFORCEMENT

Value of design shear st (from table)= 0.764
Value of MAX shear st (table) = 2.800
Area of R/F satisfies the min. R/F requirement
Henceis O.K.
Use 8 mm dia.2 legged vertical stirrups @ 257 mm c/c

SIDEFACE REINFORCEMENT.

Since the depth exceeds 450 mm.
Provide 0.1% (of the web area) steel along the verical sides
Provide 2 - 12 mm dia bars on side faces
The bars shall be equidistant from both the ends

Design for various other sections in the rib

Total no of members designed so far = 3
The member you have chosen has following properties
Member no= 9 i node= 9 j node= 10
Shear = -26021 N
Torsion = 10132608 N-mm
Moment at i node = 127642280 N-mm
Moment at j node =-179684208 N-mm
The eff. depth is= 600 mm
Design for j node moment
The bending moment is sagging

LONGITUDINAL REINFORCEMENT

Equivalent design moment for tension face = 292995200 N-mm
NA lies in the web
Area of tension steel is less than Max (4 percent of gross X-section)

area of steel. Hence O.K.
Area of steel is 1407.3 mm2
Provide 3 - 25 mm dia bars in tension
Total area of tension steel provided = 1473 mm2
Percentage area of tension steel provided= 0.613 mm2

SHEAR REINFORCEMENT

Value of design shear st (from table)= 0.516
Value of MAX shear st (table) = 2.800
Provide following shear reinforcement
Provide 8 mm - 2 legged MS stirrups @ 227 mm c/c

SIDEFACE REINFORCEMENT.

Since the depth exceeds 450 mm.
Provide 0.1% (of the web area) steel along the verical sides
Provide 2 - 12 mm dia bars on side faces
The bars shall be equidistant from both the ends

*** DESIGN OF RIB NO = 7 ***

The max positive moment in the rib is = 396833568 N-mm
The max negative moment in the rib is = 784131456 N-mm
The points of contraflexure in the rib lies at the following distance from the left side of the rib=470 mm ,9650 mm , 14880 mm , 21120 mm ,26350 mm 35530 mm

Design for max. positive moment in the rib

Member no=111 i node=117 j node=118
Shear = 5128 N
Torsion = 4479932 N-mm
Moment at i node = 396833568 N-mm
Moment at j node =-386577920 N-mm
The eff. depth is= 600 mm
Design for j node moment

LONGITUDINAL REINFORCEMENT

Equivalent design moment for tension face = 605626688 N-mm
Section is balanced,NA lies in the web
Area of tension steel is less than Max (4 percent of gross X-section) area of steel. Hence O.K.
Area of steel is = 2945.7 mm2
Provide 4 - 32 mm dia bars in tension
Total area of tension steel provided = 3363 mm2
Percentage area of tension steel provided = 1.43 mm2

SHEAR REINFORCEMENT

Value of design shear st (from table)= 0.706

Value of MAX shear st (table) = 2.80
Provide following shear reinforcement
Provide 8 mm - 2 legged MS stirrups @ 227 mm c/c

SIDEFACE REINFORCEMENT.

Since the depth exceeds 450 mm.
Provide 0.1% (of the web area) steel along the verical sides
Provide 2 - 12 mm dia bars on side faces
The bars shall be equidistant from both the end

Design for max. negative moment in the rib

Member no=115 i node=121 j node=122
Shear = -362284 N
Torsion = 2434746 N-mm
Moment at i node =-784131456 N-mm
Moment at j node = 59562444 N-mm
The eff. depth is= 600 mm
Design for j node moment

LONGITUDINAL REINFORCEMENT

Equivalent design moment for tension face = 1181836416 N-mm

Section is doubly reinforced

Total area of calculated compression steel = 4137 mm2
Total area of calculated tension steel = 6350 mm2
Area of tension steel is less than Max (4 percent of gross X-section) area of steel. Hence O.K.
Provide 8 - 32 mm dia bars in tension
Total area of tension steel provided = 6722 mm2
Percentage area of tension steel provided = 2.65
Provide 5 - 32 mm dia bars in compression
Total area of provided compression steel = 4201 mm2
-ve steel shall extend 750 mm from the point of contraflexure

SHEAR REINFORCEMENT

Value of design shear st (from table)= 0.82
Value of MAX shear st (table) = 2.80
Provide following shear reinforcement
Provide 8 mm - 2 legged MS stirrups @ 227 mm c/c

SIDEFACE REINFORCEMENT.

Since the depth exceeds 450 mm.
Provide 0.1% (of the web area) steel along the verical sides
Provide 2 - 12 mm dia bars on side faces
The bars shall be equidistant from both the ends

Design for various other sections in the rib

Total no of members designed so far = 3

The member you have chosen has following properties
Member no=117 i node=123 j node=124
Shear = -31950 N
Torsion = 624193 N-mm
Moment at i node = 164149376 N-mm
Moment at j node =-228049120 N-mm
The eff. depth is= 600 mm
Design for j node moment
The bending moment is sagging

LONGITUDINAL REINFORCEMENT

Equivalent design moment for tension face = 343519392 N-mm
NA lies in the web
Area of tension steel is less than Max (4 percent of gross X-section) area of steel. Hence O.K.
Area of steel is 1631.8 mm2
Provide 2 - 32 mm dia bars in tension
Total area of tension steel provided = 1681 mm2
Percentage area of tension steel provided = 0.818229 mm2

SHEAR REINFORCEMENT

Value of design shear st (from table) = 0.57
Value of MAX shear st (table) = 2.80
Provide following shear reinforcement
Provide 8 mm - 2 legged MS stirrups @ 227 mm c/c

SIDEFACE REINFORCEMENT.

Since the.depth exceeds 450 mm.
Provide 0.1% (of the web area) steel along the verical sides
Provide 2 - 12 mm dia bars on side faces
The bars shall be equidistant from both the ends

*** DESIGN OF RIB NO = 10 ***

The max positive moment in the rib is = 139672432 N-mm
The max negative moment in the rib is = 68115776 N-mm
The points of contraflexure in the rib lies at the following distance from the left side of the rib = 9530 mm , 17780 mm , 18220 mm ,26470 mm

Design for max. positive moment in the rib

Member no=165 i node=174 j node=175
Shear = 5298 N
Torsion = 16 N-mm
Moment at i node = 139672432 N-mm
Moment at j node =-129075752 N-mm
The eff. depth is= 600 mm

LONGITUDINAL REINFORCEMENT

Equivalent design moment for tension face= 209508704
NA lies in the web
Area of tension steel is less than Max (4 percent of gross X-section) area of steel. Hence O.K.
Area of steel is 983.9 mm2
Provide 3 - 25 mm dia bars in tension
Total area of tension steel provided= 1473 mm2
Percentage area of tension steel provided= 1.227344 mm2

SHEAR REINFORCEMENT

Value of design shear st (from table)= 0.665469
Value of MAX shear st (table)= 2.800000
Provide min shear reinforcement
Provide 8 mm - 2 legged MS stirrups @ 450 mm c/c

SIDEFACE REINFORCEMENT.

Since the depth exceeds 450 mm.
Provide 0.1% (of the web area) steel along the verical sides
Provide 2 - 12 mm dia bars on side faces
The bars shall be equidistant from both the ends

Design for max. negative moment in the rib

Member no=168 i node=177 j node=178
Shear = 25382 N
Torsion = -53 N-mm
Moment at i node = -17352252 N-mm
Moment at j node = 68115776 N-mm
The eff. depth is= 600 mm
Design for j node moment

LONGITUDINAL REINFORCEMENT

Equivalent design moment for tension face= 102173464
Section is singly reinforced (rectangular section)
Area of calculated steel is 518.1 mm2
Area of Min.tension steel is 245.8 mm2
Area of Max.tension steel is 5200.0 mm2
Area of tension steel is less than Max (4 percent of gross X-section) area of steel. Hence O.K.
Provide 2 - 25 mm dia bars in tension
Total area of tension steel provided = 982 mm2
Percentage area of tension steel provided = 0.818 mm2

SHEAR REINFORCEMENT

Value of design shear st (from table)= 0.57
Value of MAX shear st (table) = 2.80
Provide following shear reinforcement
Provide 8 mm - 2 legged MS stirrups @ 450 mm c/c

SIDEFACE REINFORCEMENT.

Since the depth exceeds 450 mm.

Provide 0.1% (of the web area) steel along the verical sides
Provide 2 - 12 mm dia bars on side faces
The bars shall be equidistant from both the ends

Design for various other sections in the rib

Total no of members designed so far = 3
The member you have chosen has following properties
Member no=171 i node=180 j node=181
Shear = -8891 N
Torsion = -34 N-mm
Moment at i node = 27244562 N-mm
Moment at j node = -9462448 N-mm
The eff. depth is= 600 mm
Design for j node moment

LONGITUDINAL REINFORCEMENT

Equivalent design moment for tension face = 14193544 N-mm
NA lies in the web
Area of tension steel is less than Max (4 percent of gross X-section) area of steel. Hence O.K.
Area of steel is 245.8 mm2
Provide 1 - 25 mm dia bars in tension
Total area of provided(min) tension steel= 491 mm2
Percentage area of tension steel provided= 0.409115 mm2

SHEAR REINFORCEMENT

Value of design shear st (from table) = 0.435
Value of MAX shear st (table) = 2.800
Provide following shear reinforcement
Provide 8 mm - 2 legged MS stirrups @ 450 mm c/c

SIDEFACE REINFORCEMENT.

Since the depth exceeds 450 mm.
Provide 0.1% (of the web area) steel along the verical sides
Provide 2 - 12 mm dia bars on side faces
The bars shall be equidistant from both the ends