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Design and Analysis of Diagrid Structures Without Interior Column

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Abstract:- In recent years diagrid structures have received increasing attention among both designers and researchers of tall buildings for creating one-of-a-kind signature structure. This paper presents an advancement to diagrid structures. The idea to develop and modify diagrid structure arised from the model of a foldable book stand along with arch action. The structure is designed as an office building of dimension 20x15x12 m. This structure is supporting only on 4 diagonal composite columns. All the other interior columns were eliminated and thus the space area has increased. We put forward a unique structure which is composed of both steel (beam), RCC (slab) and composite components (columns) hence contributing the advantages of all the three in one structure. ETABS (2017) software is used for analyzing and designing the structure. Satisfactory result has been obtained.

I. INTRODUCTION

In modern age, the decrease of available free land and increase of land prices along with the wide spread of urban area has made architect and engineers to develop the cities vertically. For vertical growth the only option is to construct the buildings as high as possible. It is a task of a structural designer to make the desired building stand and stable throughout its life.

Anciently tall buildings were constructed as concrete structure. Later, the constructions using steel has become more common, the high rise building thus also be arise as steel structures. All other materials talk about high strength is still less than that of structural steel even when enhanced by steel reinforcing. In fact the increase in the standard strength of steel used in buildings today compared to ten years ago is greater than the total strength of competing “high strength” materials. Structural steel allows the project architect a greater degree of expression and creativity in their design than most our construction materials. Structural steel section can be bend and rolled to create non linear members to further enhance the aesthetic appeal of the structure.

A steel concrete composite column is a compression member, comprising either a concrete encased hot- rolled steel section or a concrete filled tabular section of hot rolled steel. Hence we are preferred to design this office building in RCC-steel-composite combination. Now a days the building is created with steel in the form of a triangle with diagonal support beams. These structures are called Diagrid. It provides lot of strength to structure. Shear and moment is present in the diagonals. Recently diagrid structural system is adopted in tall buildings due to its structural efficiency and flexibility in architectural planning. Compared to closely spaced vertical columns in framed tube, diagrid structure consists of inclined columns on the exterior surface of building.

II. AIM

As diagrid structure are efficient in providing solution both in terms of strength and stiffness. Therefore nowadays widespread application of Diagrid is used in high rise building and skyscrapers, particularly when complex geometrics and curved shapes are involved. As height of building rises, not only D.L and L.L are predominant forces but along with it W.L and seismic forces equally hold a share with it. In order to provide resistance against these forces conventional design approach might be sufficient to counteract these loads but may lead to uneconomical design, lesser F.O.S, greater stability requirement and aesthetic part may not be up to the mark. Diagrid takes in to account above mentioned limitation which conventional building faces and proves to be one of the solution for getting optimum structure skyscrapers, particularly when complex geometrics and curved shapes are involved.

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- Increasing usable space area- eliminating interior vertical columns.
- Decreases cost of construction-eliminating interior vertical columns
- Load is transferred through the four diagonal columns only.
- To increase the aesthetic view

III. SCOPE

Lot of development are occurring in construction field, that gives arise to different shapes of building. Increasing price value of available free land and also the scarcity of available free land makes architect and engineers to develop the city vertically which gives arise to high rise building. The structure presented in this paper is more stable as compared to conventional structure. It also gives more usable space area as well as more aesthetic view. This structure is also an earthquake resistant structure.

- Increase usable space area.
- More stable structure.
- Earthquake resistant building.

IV. METHODOLOGY

A. Idea Development For Model

The idea to develop and modify diagrid structures arise from the model of a foldable book stand. In foldable book stand the load is uniformly distributed by arch action.

B. Site Selection

For an economic and successful project the site selected should have the following requirements.

1. Transportation facilities
2. Communication facilities
3. Availability of enough land
4. Water supply and electricity
5. Availability of construction materials

Since the proposed site has almost all the above requirements we selected so we select the same site for project.

C. Modeling

The modeling of our structure is done by ETABS 2016 software.

D. Loading

The loads considered for the analysis of structure were ,

- Dead load
- Live load
- Seismic load

E. Analysis

The structure is analyzed using ETABS 2016 software. The innovative and revolutionary new ETABS 2016 is the ultimate integrated software package for the structural analysis and design of buildings. Incorporating 40 years of continuous research and development, this latest ETABS 2016 offers unmatched 3D object based modeling and visualization tools, blazingly fast linear and non linear analytical power, sophisticated and comprehensive design capabilities for a wide range of materials and insight full graphic displace, reports and schematic drawings that allows users to quickly and easily decipher and understand analysis and design results.

F. Result

The project is analyzed successfully and satisfactory result is obtained.

G. Future Scope

The building can be constructed as earthquake resistant by providing different types of base isolation methods. The usable space area of the structure can be increased.

H. Report Preparation

Results from ETABS and manual design where prepared.

V. IDEA BEHIND THE PROJECT

A. Foldable Book Stand

The idea behind the project is based on the principle of load distribution in a foldable book stand. The foldable book stand is in the shape of 'X'. In this the load is transferred from the upper half portion to the middle and from the middle the load is transferred through the remaining half portion by arch action.

In this the load is transferred from the upper half portion to the middle and from the middle the load is transferred through the remaining half portion by arch action.

It has a larger potential to withstand the lateral loads to a great extent than the conventional means of loading. It is almost stable when lateral and vertical loads were acted upon it.

Fig 1:- Foldable book stand

B. Arch Action

An arch is a soft compression form. It can span a large area by resolving forces into compressive stresses and, in turn eliminating tensile stresses. This is sometimes referred to as arch action. As the forces in the arch are carried to the ground, the arch will push outward at the base, called thrust. Arch action work by transferring the weight of the whole structure and its loads partially into a horizontal thrust restrained by the abutments at either side. Instead of pushing straight down, the load of an arch shaped structure is carried outward along the curve of the arch to the supports at each end. The weight is transferred to the supports at either end. These supports, called the abutments, carry the load and keep the ends of the structure from spreading out.

An arch works excellently in compression. A structural arch can carry much more load than a flat beam or plank. The forces exerted by an arch are tangential to the ends of the arch, and are called thrust. Thus, a flatter arch will create more force or thrust that is transferred to the ground safely. Overall, an arch will handle compressive loads better than a straight structural member because of the way it safely transfers the loads applied to it more efficiently to the ground. For example, if you had two identical planks of wood and formed one into an arch, the flat plank would break into pieces long before the arch form when identical loads were applied to both.

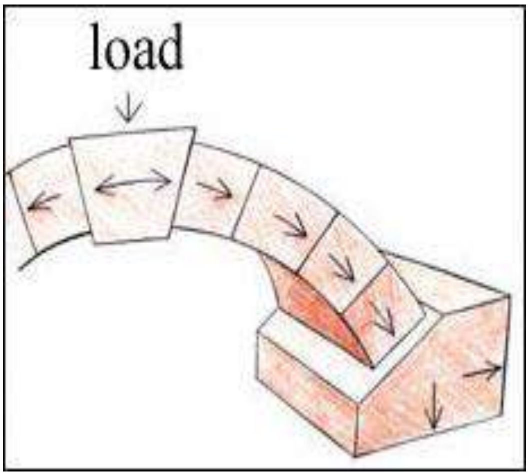

Fig 2:- Load transfer

C. Diagrid Structure

Diagrid structure has the qualities of aesthetic expression, 0structural efficiency and most importantly geometric versatility. The diagonal members provide both gravity and lateral load resistance.

Recent trend shows that the diagrid structural system is becoming popular in the design of tall buildings due to its inheritent structural and architectural advantages. Diagrid an exterior structural system in which all perimeter vertical columns are eliminated and consists of only inclined column on the façade of the building shear and overturning moments developed are resisted by axial action of these diagonals compared to bending of vertical columns in fed tube structure.

The diagrid system are the evolution of braced tube structures. The major difference between a braced tube building and a diagrid building is that there are no vertical columns present in the perimeter of diagrid building. Diagrid structures do not need high shear rigidity cores because shear can be carried by the diagrid located on the perimeter. The configuration and efficiency of a diagrid system reduce the number of structural element required on the façade of the building , therefore less destruction to the outside view.

The structural efficiency of diagrid system also help in avoiding interior and corner columns, therefore allowing significant flexibility with the floor plan .Perimeter diagrid system saves approximately 20% structural weight when compared to a conventional moment frame structure .As in the diagrid , diagonals carry both shear and moment thus the optimal angle of diagonal is highly dependent upon the building height. Usually adopted 60-70 degree.

1. Diagrid Components

- Nodes
- Diagonal members
- Ring beams
- Tie beams
- Core
- Floor slab

Fig 3:- Diagrid structure

The diagrid structural system can be defined as a diagonal members formed as a framework made by the intersection of different materials like metals, concrete or wooden beams which is used in the construction of buildings and roofs.

Diagrid structures of the steel members are efficient in providing solution both in term of strength and stiffness. But nowadays a widespread application of diagrid is used in the large span and high rise buildings, particularly when they are complex geometries and curved shapes.

The materials used in the construction of diagrid are

D. Steel Diagrid Structural System

The most commonly and popularly used material in the construction of diagrids is steel. The sections commonly used are rectangular HSS, rounded HSS and wide flanges. The weight and size of the sections are made so as to resist the high bending loads. They can be quickly erected and the cost of labor for the installation is low.

Fig 4:- Steel Diagrid structure

E. Concrete Diagrid Structural System

The most commonly used diagrid material is concrete. The concrete diagrids are used in both type, precast and cast in-situ. As the precast concrete sections are flexible, it allows them to fit perfectly in the structure geometry. It also protects from fire damages. But the precast concrete constitutes more to the dead load of the structure.

Fig 5:- Concrete Diagrid structure

F. Timber Diagrid Structural System

The least used material in the construction of diagrid is timber. This material has more disadvantages. The only advantage of this material that the section of timber are easily available in any shape and size. The installation cost is low. The major disadvantages are that timber has lesser material strength. Durability and weathering of timber are the major issues that makes for the disadvantages of timber as a diagrid construction material.

Fig 6:- Timber Diagrid structure

VI. ADVANTAGES AND DISADVANTAGES OF DIAGRID STRUCTURAL SYSTEM

A. Advantages of Diagrids

The advantages of the diagrid in the construction of the structure majorly improves the aesthetic view of the building. The use of diagrid reduces the steel up to 20% compared to brace frame structure. It doesn't need technical labor as the construction technology is simple.

The diagrid makes the maximum use if the structural material is used. When glass material is used with the diagrid, it allows generous amount of light inside the structure. These structures have majorly column free exterior and interior, free and clear, unique floor plans can be implemented.

B. Disadvantages of Diagrids

The major disadvantages of diagrid system are that it is still not completely explored. This construction needs a skilled labor and the present crew has no idea or the experience in installing diagrids. As the diagrid completely takes over the aesthetic appearance of the building, the design is limited only to diagrid. The common language of floor to floor design is effected as a single diagrid stacks over 2 to 6 floors in it. Only high rise building can install diagrids. If diagrids are not properly designed or installed, it effects the economy and safety of the structure.

C. Base Isolation

We are suggesting the base isolation technique to improve the building as an earthquake resistant structure. Base isolation, also known as seismic base isolation or base isolation system, is one of the most popular means of protecting a structure against earthquake forces. It is a collection of structural elements which should substantially decouple a superstructure from its substructure resting on a shaking ground thus protecting a building or non-building structure's integrity.

Base isolation is one of the most powerful tools of earthquake engineering pertaining to the passive structural vibration control technologies. It is meant to enable a building or non-building structure to survive a potentially devastating seismic impact through a proper initial design or subsequent modifications. In some cases, application of base isolation can raise both a structure's seismic performance and its seismic sustainability considerably. Contrary to popular belief base isolation does not make a building earthquake proof.

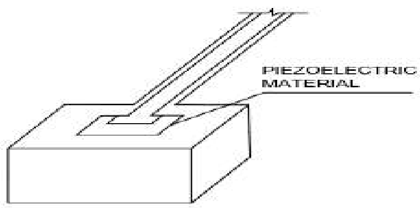

Fig 7:- Piezoelectric isolation for foundation

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➢ *Principle of Base Isolation*

The basic principle behind base isolation is that the response of the structure or a building is modified such that the ground below is capable of moving without transmitting minimal or no motion to the structure above. A complete separation is possible only in an ideal system. In a real world scenario, it is necessary to have a vertical support to transfer the vertical loads to the base.

The relative displacement of ground and the structure is zero for a perfectly rigid, zero period structure, since the acceleration_induced in the structure is same as that of ground motion. Whereas in an ideal flexible structure, there is no acceleration induced in the structure, thus relative displacement of the structure will be equal to the ground displacement.

No Structure is perfectly rigid or flexible, therefore, the response of the structure will be between the two explained. Maximum acceleration and displacements are a function of earthquake for periods between zero to infinity. During earthquakes there will be a range of periods at which acceleration in the building will be amplified beyond maximum ground acceleration, though relative displacements may not exceed peak ground displacements. Base isolation is the ideal method to cater this, by reducing the transfer of motion, the displacement of building is controlled.

The main principle of base isolation is to try and isolate the structure from the ground movement so you could just about put it on ball bearings if you like and ground could move underneath it and the buildings stays still.

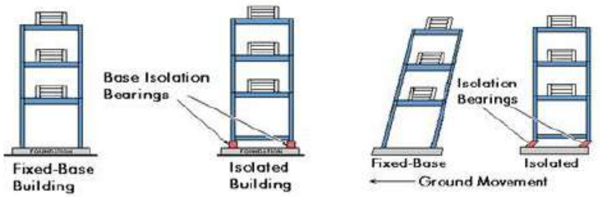

Fig 8:- Base isolation in a structure

➢ *Types of Base Isolation*

- *Elastomeric Bearings*

The base isolation system that has been adopted most widely in recent years is typified by the use of elastomeric bearings, where the elastomeric is made of either natural rubber or neoprene. In this approach, the building or structure is decoupled from the horizontal components of the earthquake ground motion by interposing a layer with low horizontal stiffness between the structure and the foundation. Rubber bearings are most commonly used for this purpose; a typical laminated rubber bearing A rubber bearing typically consists of alternating laminations of thin rubber layers and steel plates, bonded together to provide vertical rigidity and horizontal flexibility. These bearings are widely used for the support of bridges.

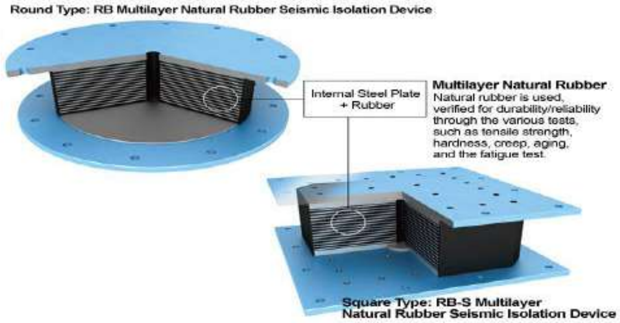

Fig 9:- Elastomeric bearing

- *Sliding Systems*

Sliding isolator work on principle of friction this approach is based on the premise that the lower the friction coefficient the less the shear transmitted. In sliding isolator, two pure flat stainless steel plates or spherical surface and articulated friction slider slide over each other during earthquake excitation for initiation of sliding the intensity of existing force must be more than frictional force of isolator. Hence during earthquake excitation the frequency of which is not harmonic, the isolator displacement is of stick-slip nature.

Fig 10:- Sliding system

- *Spherical Sliding Base Isolation System*

Another type of base isolator is the spherical sliding base isolation system in which the building is supported by bearing pads that have a curved surface and low friction. During an earthquake, the building is free to slide both vertically and horizontally.

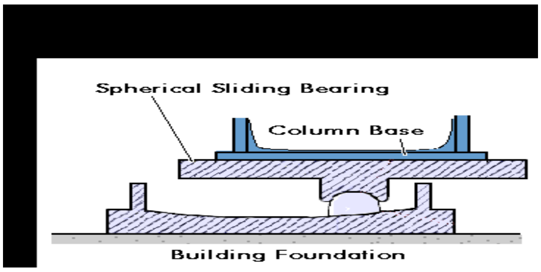

Fig 11:- Spherical sliding base isolation system

- *Friction Pendulum Bearing*

Friction pendulum is a specially designed base isolators. This base isolator work on the principle of simple pendulum by increasing natural time period of oscillation.A similar system is the Friction Pendulum Bearing (FPB), another name of Friction Pendulum System (FPS). It is based on three aspects: an articulated friction slider, a spherical concave sliding surface, and an enclosing cylinder for lateral displacement restraint.

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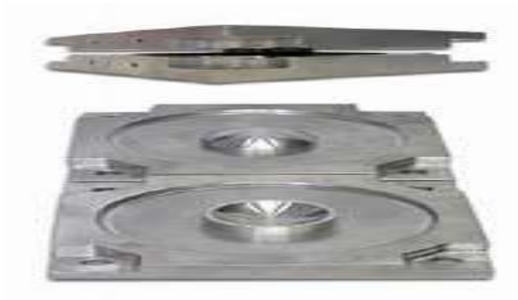

Fig 12:- Friction pendulum

Fig 13:- Friction pendulum applied on a structure

VII. SITE SELECTION

A. Soil Investigation

10 numbers 100mm diameter boreholes are drilled up to 60m depth using heavy duty calyx drilling rig with direct mud circulation. Standard Penetration Test (SPT) were done at 2m interval as per. IS 2131 – 1963. The SPT value viz (N-Values) are recorded in the bore log charts and accompanying this report. The samples of soil recovered using the SPT split spoon sampler were classified and tested in the laboratory. These laboratory test results are noted in this report.

Table 1. Laboratory Test Results

B.H.No	Chainage	Depth (m)	Side
B H-I	390	60m from ground level	Valiyazheekkal side
B H -II	433	60.30m from ground level	Valiyazheekkal side
B H - III	508	60m from ground level	Valiyazheekkal side
B H - IV	553	60m from ground level	Valiyazheekkal side
B H - V	580	60m from bed level of stream	Under water
B H - VI	890	60m from bed level of stream	Under water
B H - VII	937	60m from ground level	Azheekkal side
B H -VIII	983	60m from ground level	Azheekkal side
B H - IX	1037	60.10m from ground level	Azheekkal side
B H – X	1088	60.10m from ground level	Azheekkal side

The strata for about first 18m depth is generally fine sand mixed with silt and clay with very low N value. For 18m to 30m depth soil is medium sand mixed with little clay with N value around 50. Below 30m depth is soil is lateritic clay or clayed sand with N value more than 50.

B. Recommendations

As had rock in not met with in the boring, DMC Pile foundation is recommended as follows with depth about 50 m for office building at the site.

The capacity of pile is recommended as follows:

Table 2:- Recommended capacity of Pile

Diameter of pile in cm	Recommended Safe Load in tone
80	140
90	180
100	230
120	350

Foundation has to be constructed as per relevant IS codes and to be certified by a qualified Engineer. Pile load test is recommended as per the IS Code. Any difference in soil profile found during execution may be referred back to the consultant or designer for revision in design that may be necessary.

VIII. BRIEF DESCRIPTION OF SOFTWARES USED

Softwares used were ,

- ETABS 2016
- AUTOCADD 2017

A. ETABS 2016

ETABS is a special-purpose computer program developed specifically for building structures. It provides the Structural Engineer with all the tools necessary to create, modify, analyze, design, and optimize building models. These features are fully integrated in a single, Windows-based, graphical user interface that is unmatched in terms of ease-of-use, productivity, and capability. ETABS is for linear, non-linear static and dynamic analysis, and the design of building systems. The need of special software was there for analysis and design of building and structure. From the analytical standpoint, multi-storied buildings constitute a very special class of structures and therefore deserve special treatment. This resulted in the development of the TABS series of computer programmers. The system built around a physical object based graphical user interface, powered by targeted new special algorithms for analysis and design, with interfaces for drafting and manufacturing, is redefining standards of integration, productivity and technical innovation.

The integrated model can include moment resisting frames, braced frames, staggered truss systems, frames with reduced beam sections or side plates, rigid and flexible floors, sloped roofs, ramps and parking structures, mezzanine floors, multiple tower buildings, stepped diaphragm systems with complex concrete and composite or steel joist floor framing systems. Solutions to complex problems such as panel zone deformation, diaphragm shear stress, and construction sequence loading are made simpler with this software. ETABS can solve a simple 2D frame structure or can perform dynamic analysis of a complex high-rise that utilizes non-linear dampers for inter-storey drift control.

➢ *Use of ETABS*

ETABS can be effectively used in the analysis and design of building structures which might consists of structural members like beams, columns, slabs, shear walls etc. With ETABS we can easily apply various construction materials to structural members like concrete, structural steel, reinforced concrete etc. ETABS automatically generates the self weight and the resultant gravity and lateral loads. The need for special purpose programmers are in more demand among structural engineers as they put non-linear dynamic analysis into practice and use the greater computer power available today to create large analytical models.

➢ *Characteristics and Advantages of ETABS*

- Fully integrated program that allows model creation, modification, execution of analysis, design optimization, and results review from within a single interface.
- ETABS uses a feature of similar stories where various properties and loads might be applied by selecting the option of similar stories
- Easy options and commands are there in ETABS like copy, paste, mirror, merge etc.
- ETABS is very precise, the snapping of the ends and joints allows the dimensions to be accurate and least chance of error is there.
- Applying of loads and creation of objects is very easy
- A lot of views (top, ends etc) are there.
- Integration with other soft wares like SAP2000 or SAFE etc
- Supports various codes like American building code, Euro code, British codes, Indian building codes, Pakistan building codes.
- Model and geometry of model can easily to be exported to. dxf files.
- A wide variety of automated templates allow a quick start for almost any building.
- Object based physical member modelling allows working with large members that do not need to be broken up at each joint.
- Fully integrated section designer allows definition of complex sections.
- Fully interactive steel, concrete, composite beam member design
- Onscreen result display.
- Animated display of deformed shapes, mode shapes, stress contours and time history results
- Import and export models in commonly used formats
- Context sensitive online help

B. AUTOCADD 2017

AutoCAD is software application for 2D and 3D computer-aided design (CAD) and drafting available since 1982 as a desktop application and since 2010 as a mobile web- and cloud-based app, currently marketed as AutoCAD 360. The native file format of AutoCAD is .dwg. This and, to a lesser extent, its interchange file format, DXF has become the standards for interchange of CAD data. AutoCAD has included support for dwg, a format developed and promoted by Autodesk, for publishing CAD data.

IX. MODELLING

A. Beam Model

The steel beam used is of I section having the size,

Beam, B1

Section Dimensions

Total Depth	= 700mm
Top Range Width	= 500mm
Top Range Thickness	= 25mm
Web Thickness	= 25mm
Bottom Flange Width	= 500mm
Bottom Flange Thickness	= 25mm

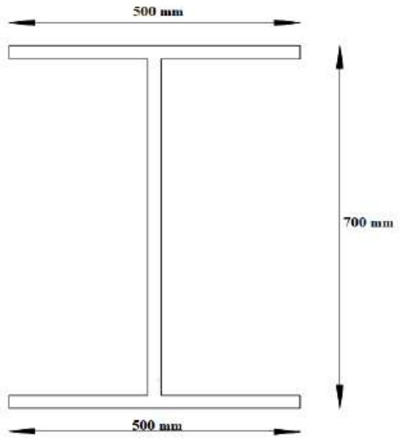

Fig 14:- Beam B1

Beam, B2

Section Dimensions

Total Depth	= 450mm
Top Range Width	= 250mm
Top Range Thickness	= 25mm
Web Thickness	= 21mm
Bottom Flange Width	= 250mm
Bottom Flange Thickness	= 25mm

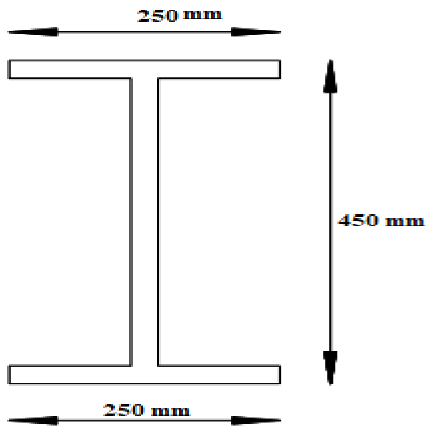

Fig 15:- Beam B2

B. Composite Column

The column used is composite

Fig 16:- Composite Column

C. ETABS Model

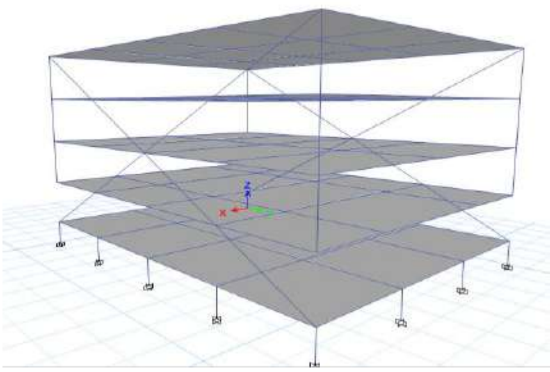

Fig 17:- Model prepared in ETABS 2016

X. LOADS ON THE BUILDING

A. Types of Loads

The various loads considered for analysis where:

1. Dead Loads

These include the self weights of the structure and the permanent loads on it. The dimensions of the cross section are to be assumed initially which enable to estimate the dead load from the known unit weights of the structure. The values of the unit weights of the materials are specified in IS 875(Part-1): 1987.

Dead Load (D.L) = Wall thickness *height * unit weight of the brick + Floor finish.

= 0.2 * 3 * 18 + 1
= 11.8 kN

2. Live Loads

These are also known as imposed loads and consist of all loads other than the dead loads of the structure. The values of the imposed load depend on the functional requirement of the structure. The standard values are stipulated in IS 875(Part- 2): 1987.

Live Load = 5 kN / m^2

3. Seismic Loads

Earthquake generate waves which move from the origin of its location with velocities depending on the intensity and magnitude of the earthquake. The impact of earthquake on structures depends on the stiffness of the structure, stiffness of the soil media, height and location of the structure, etc. The earthquake forces are prescribed in IS 1893: 2002 (PART- 1).

Since the building is located in Kerala it is included in Zone III and the seismic base shear calculation and its distribution was done as per IS 1893: 2002 (PART- 1)

The base shear or total design lateral force along any principal direction shall be determined by the following expression.

$V_b = A_h * W$

Where,

V_b= design base shear`

A_h = Design horizontal seismic coefficient based on fundamental natural period, T_a and type of soil.

W = Seismic weight of the building

The design horizontal seismic coefficient, $A_h = ZIS_a/ 2Rg$

Where,

Z = Zone factor given in table 2, for the Maximum Considered Earthquake (MCE) and service life of structure in a zone. The factor 2 in the denominator is used so as tro reduce the MCE zone factor to the factor for Design Basis Earthquake (DBE).

I = Importance factor, depending upon the functional use of structures, characterized by hazardous consequences of failure, post- earthquake functional needs, historical values or economic importance (Table 6 of IS 1893(PART-1) : 2002).

R = Response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformation. However, the ratio (I/R) shall not be greater than 1.0. The value of building are given table 7 of IS 1893(PART-1): 2002.

S_a/ g = Average response acceleration coefficient.

Seismic coefficient
Seismic zone factor, Z = 0.16
Importance factor, I = 1.2
Time period, T = 0.027

B. Design of Slab

Design of slab can be done by manually

1. Design of Two Way Slab

- SLAB (ABCD) 5 X 5 m

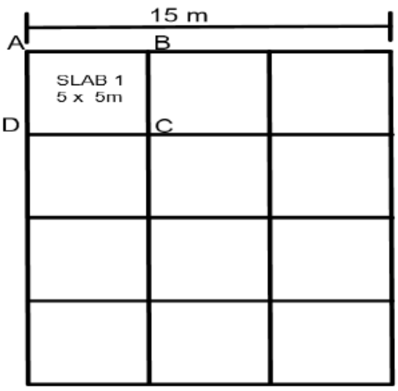

Fig 18:- Slab 5 x 5m

a) Material constants

Use M_{30} grade concrete and HYSD steel bars of grade Fe_{415}.
For M_{30}Concrete, f_{ck} = 30 N/mm^2
For Fe_{415}Steel, f_y = 415 N/mm^2

b) Type of slab

Centre to centre distance of longer span, L_y = 5 m
Centre to centre distance of shorter span, L_x = 5 m

Table 3. Reinforcement details in 2 way slab (5x5)

Location	A_{st}(required)	Spacingof 10mmϕ bars	A_{st}(provided)
1)short span	240 mm^2	160 mm	461.19 mm^2
2)long span	240 mm^2	140 mm	493.17 mm^2

$$\frac{L_y}{L_x} = \frac{5}{5} = 1 < 2$$

∴Two way slab

Type of slab: one edge discontinuous

c) Preliminary dimensioning

As per IS 456:2000, Clause 24.1,
Provide depth D= 174 mm
Clear cover = 20mm
Provide 10mm dia bar
Effective depth = 174+20+5 = 200 mm
Effective depth along shorter direction, d_x = 5174mm

d) Effective span

As per IS 456:2000, Clause 22(a)
Effective span along short and long spans are computed as:
L_{ex} =clear span +effective depth =5+0.174 = 5.174 m
L_{ey} =clear span +effective depth =5+0.174 = 5.174 m

e) Load calculation

Dead load = 5kN/m^2
Floor finish = 1 kN/m^2
As per IS:875(Part 2)-1987 Table-1
Live load = 5kN/m^2

Total service load = 11kN/m^2
Design ultimate load ,W_u =1.5× 11= 16.5 kN/m^2

f) Ultimate design moment

Refer table 26 of IS 456:2000 and read out the moment coefficients for

$$\frac{L_y}{L_x} = \frac{5..174}{5.174} = 1$$

Short span moment coefficients:

a) moment coefficient =α_x =0.062

Long span moment coefficients:

b) moment coefficient =α_y =0.062

$M_{ux} = \alpha_x \times w_u \times L_{ex}{}^2$ =0.062× 16.5 × 5.174 ^{2} = 27.38kNm
$M_{uy} = \alpha_y \times w_u \times L_{ex}{}^2$ =0.062× 16.5 × 5.174^2 = 27.38kNm

g) Check for depth

$(M_u)_{lim}$= 0.133× f_{ck} × b × d ^{2}

$$d_{required} = \sqrt{\frac{(M_u)_{lim}}{0.133 \times f_{ck} \times b}}$$

$$= \sqrt{\frac{27.38 \times 10^6}{0.133 \times 30 \times 1000}}$$

= 82.83 mm

$d_{required} < d_{provided}$(200- mm)

Hence the effective depth selected is sufficient to resist the design ultimate moment.

h) Reinforcements along short and long span directions

The area of reinforcement is calculated using the relation:

$$M_u = 0.87 \times f_y \times A_{st} \times d\left(1 - \frac{A_{st} \times f_y}{b \times d \times f_{ck}}\right)$$

Spacing of the selected bars are computed using the relation:

$$\text{Spacing} = S = \frac{\text{Area of one bar}}{\text{total area}} \times 1000$$

i) Check for spacing

As per IS 456:2000 clause 26.3.3(b)

$$\text{Maximum spacing} = \left\{ \begin{array}{c} 3d \\ \text{or} \\ 300 \text{ mm} \end{array} \right\} \text{whichever is less}$$

ISSN No:-2456-2165

$$= \left\{ \begin{matrix} 3 \times 112 = 336 \text{ mm} \\ \text{or} \\ 300 \text{ m} \end{matrix} \right\} \text{whichever is less}$$

= 300 mm

$spacing_{prov} < spacing_{max}$

$\therefore$ safe.

i) Check for area of steel

As per IS 456:2000 clause 26.5.2.1

$(A_{st})_{min} = 0.12\%$ of cross – sectional area

$= \frac{0.12 \times 1000 \times 200}{100}$

= 240 mm^2

$(A_{st})_{prov} > (A_{st})_{min}$

$\therefore$ safe.

k) Check for deflection **:**

$(A_{st})_{provided}$ = 493.17 mm²

$(A_{st})_{required}$ = 240 mm²

$f_s = \frac{0.58 \times fy \times (A_{st})_{required}}{(A_{st})_{provided}}$

$= \frac{0.58 \times 415 \times 240}{493.17}$

=117.13

$P_t = \frac{100 \times 493.17}{1000 \times 200} = 0.24$

As per IS 456: 2000, fig 4 , page 38

Modification factor = 1.7

As per IS 456:2000, clause 23.2.1

$(\frac{L}{d})_{basic} = 26$

$(\frac{L}{d})_{max} = 26 \times 1.7 = 44.2$

$(\frac{L}{d})_{provided} = \frac{5174}{200} = 25.87 < (\frac{L}{d})_{max}$

So deflection is safe with provided depth.

l) Check for shear

$V_u = \frac{W_u \times L_e}{2}$

$= \frac{16.5 \times 5.174}{2}$

=42.68 kN

As per IS 456:2000 clause 40.1

$\tau_v = \frac{V_u}{b \times d}$

$= \frac{42.68 \times 10^3}{1000 \times 174}$

= 0.24 N/mm^2

$p_t = \frac{100 \times A_{st}}{b \times d}$

$= \frac{100 \times 240}{1000 \times 174} = 0.146$

As per IS 456:2000 clause 40.2

Design shear strength of concrete = $k \times \tau_c$

$= 1.3 \times 0.28$

= 0.364 N/mm ^{2}

As per IS 456:2000, Table 20

Maximum shear stress,$(\tau_c)_{max}$ = .28 N/mm^2

$\tau_v < \tau_c < (\tau_c)_{max}$

$\therefore$ shear reinforcement is not provided.

m) Check for cracking

As per IS 456:2000, clause 43.1:

1. Steel provided is more than 0.12 percents
2. Spacing of main steel< $300mm$
3. Diameter of reinforcement< $\frac{D}{8} = \frac{160}{8} = 20$ mm

Hence cracks will be within the permissible limits.

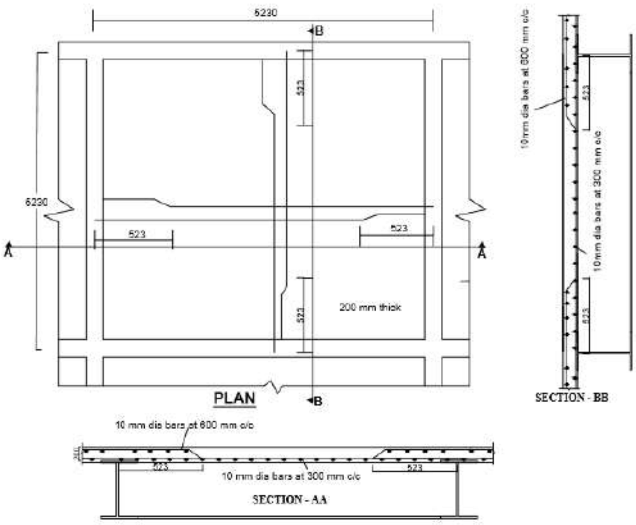

Fig 19:- Slab (ABCD) 5 x 5m

- SLAB (EFGH) 3.75 x 5

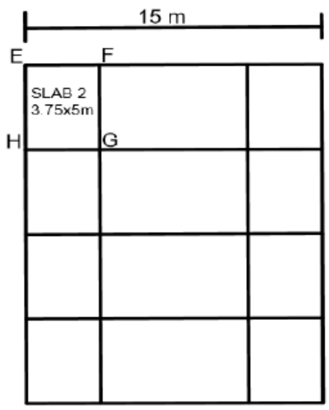

Fig 20:- Slab 3.75x5

a) Material constants

Use M_{30} grade concrete and HYSD steel bars of grade Fe_{415}.

For M_{30}Concrete, f_{ck} = 30 N/mm^2

For Fe_{415}Steel, f_y = 415 N/mm^2

b)Type of slab

Centre to centre distance of longer span, L_y = 5 m

Centre to centre distance of shorter span, L_x = 3.75 m

$\frac{L_y}{L_x} = \frac{3.75}{5} = 1.33 < 2$

$\therefore$Two way slab

Type of slab: one edge discontinuous

c) Preliminary dimensioning

As per IS 456:2000,Clause 24.1,

Provide depth D= 135 mm

Clear cover = 20mm

Provide 10mm dia bar

Effective depth = 135+20+5 = 160 mm
Effective depth along shorter direction, d_x = 3865mm

d)Effective span
As per IS 456:2000, Clause 22(a)
Effective span along short and long spans are computed as:
L_{ex} =clear span +effective depth =3.75+0.135 = 3.865 m
L_{ey} =clear span +effective depth =5+0.135 = 5.135 m

e) Load calculation
Dead load = $5 kN/m^2$
Floor finish = $1 kN/m^2$
As per IS:875(Part 2)-1987 Table-1
Live load = $5kN/m^2$
Total service load = $11kN/m^2$
Design ultimate load,W_u =1.5× 11= 16.5 kN/m^2

f) Ultimate design moment
Refer table 26 of IS 456:2000 and read out the moment coefficients for

$$\frac{L_y}{L_x} = \frac{5.135}{3.865} = 1.321$$

Table 5. Reinforcement details in two way slab

Location	A_{st}(required)	Spacing of 10mmϕ bars	A_{st}(provided)
1)short span	192 mm^2	140 mm	516 mm^2
2)long span	192 mm^2	125 mm	566.41mm^2

Short span moment coefficients:
c) moment coefficient =α_x =0.093
Long span moment coefficients:
d) moment coefficient =α_y =0.055

$M_{ux} = \alpha_x \times w_u \times L_{ex}^2$ =0.093× 16.5 × 3.885 ^{2}
= 23.16kNm
$M_{uy} = \alpha_y \times w_u \times L_{ex}^2$ =0.055× 16.5 × 3.885^2
= 23.929kNm

g)Check for depth
$(M_u)_{lim}$= 0.133× $f_{ck} \times b \times d^2$

$$d_{required} = \sqrt{\frac{(M_u)_{lim}}{0.133 \times f_{ck} \times b}}$$

$$= \sqrt{\frac{23.16 \times 10^6}{0.133 \times 30 \times 1000}}$$

= 91.6 mm
$d_{required} < d_{provided}$ (160- mm)
Hence the effective depth selected is sufficient to resist the design ultimate moment.

h)Reinforcements along short and long span directions

The area of reinforcement is calculated using the relation:

$$M_u = 0.87 \times f_y \times A_{st} \times d\left(1 - \frac{A_{st} \times f_y}{b \times d \times f_{ck}}\right)$$

Spacing of the selected bars are computed using the relation:

$$\text{Spacing} = S = \frac{\text{Area of one bar}}{\text{total area}} \times 1000$$

i) Check for spacing
As per IS 456:2000 clause 26.3.3(b)

Maximum spacing = { 3d or 300 mm } whichever is less
= { 3 × 112 = 336 mm or 300 m } whichever is less
= 300 mm
$spacing_{prov} < spacing_{max}$
∴ safe.

j)Check for area of steel
As per IS 456:2000 clause 26.5.2.1

$(A_{st})_{min}$ = 0.12% of cross – sectional area

$$= \frac{0.12 \times 1000 \times 160}{100}$$

= 192 mm^2

$(A_{st})_{prov} > (A_{st})_{min}$
∴ safe

k) Check for deflection:
$(A_{st})_{provided}$ = 566.41 mm²
$(A_{st})_{required}$ = 192 mm²

$$f_s = \frac{0.58 \times fy \times (A_{st})_{required}}{(A_{st})_{provided}}$$

$$= \frac{0.58 \times 415 \times 192}{566.41}$$

=81.59

$$P_t = \frac{100 \times 566.41}{1000 \times 160} = 0.35$$

As per IS 456:2000, fig 4 , page 38
Modification factor = 1.7
As per IS 456:2000, clause 23.2.1
$(\frac{L}{d})_{basic}$ = 26
$(\frac{L}{d})_{max}$ =26× 1.7 = 44.2
$(\frac{L}{d})_{provided} = \frac{5135}{160}$ = 32.09< $(\frac{L}{d})_{max}$
So deflection is safe with provided depth.

l) Check for shear

$$V_u = \frac{W_u \times L_e}{2}$$

$$= \frac{16.5 \times 3.885}{2}$$

=32.05 kN
As per IS 456:2000 clause 40.1

$$\tau_v = \frac{V_u}{b \times d}$$

$$= \frac{32.05 \times 10^3}{1000 \times 135}$$

= 0.23 N/mm²

$$p_t = \frac{100 \times A_{st}}{b \times d}$$

$=\frac{100\times192}{1000\times135}=0.153$
As per IS 456:2000 clause 40.2
Design shear strength of concrete = $k \times \tau_c$
$= 1.3 \times 0.28$
$= 0.364\ N/mm^2$
As per IS 456:2000, Table 20
Maximum shear stress, $(\tau_c)_{max}$ $= .28\ N/mm^2$
$\tau_v < \tau_c < (\tau_c)_{max}$
∴ shear reinforcement is not provided.

m) Check for cracking
As per IS 456:2000, clause 43.1:

4. Steel provided is more than 0.12 percents
5. Spacing of main steel $< 300mm$
6. Diameter of reinforcement $< \frac{D}{8} = \frac{160}{8} = 20$ mm

Hence cracks will be within the permissible limits.

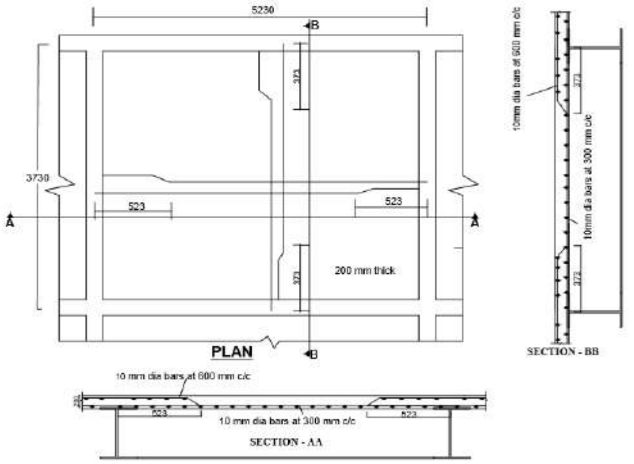

Fig 21:- Slab (EFGH) 3.75 x 5

- SLAB (IJKL) 7.5 X 5

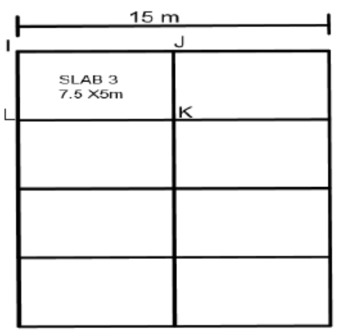

Fig 22:- Slab 7.5 x 5

a)Material constants
Use M_{30} grade concrete and HYSD steel bars of grade Fe_{415}.
For M_{30} Concrete, f_{ck} = 30 N/mm^2
For Fe_{415} Steel, f_y = 415 N/mm^2
b)Type of slab
Centre to centre distance of longer span, $L_y = 7.5$ m
Centre to centre distance of shorter span, $L_x = 5$ m
$\frac{L_y}{L_x} = \frac{7}{5} = 1.4 < 2$
∴Two way slab
Type of slab: one edge discontinuous

c)Preliminary dimensioning
As per IS 456:2000, Clause 24.1,
Provide depth D= 175 mm
Clear cover = 20mm
Provide 10mm dia bar
Effective depth = 175+20+5 = 200 mm
Effective depth along shorter direction, d_x = 5200mm
d)Effective span
As per IS 456:2000, Clause 22(a)
Effective span along short and long spans are computed as:
L_{ex} =clear span +effective depth =5+0.2 = 3.865 m
L_{ey} =clear span +effective depth =7.5+0.2 = 7.7 m
e) Load calculation
Dead load $= 5kN/m^2$
Floor finish $= 1\ kN/m^2$
As per IS:875 (Part 2)-1987 Table-1
Live load $= 5kN/m^2$
Total service load $= 11kN/m^2$
Design ultimate load, W_u =1.5× 11= 16.5 kN/m^2

f) Ultimate design moment
Refer table 26 of IS 456:2000 and read out the moment coefficients for
$\frac{L_y}{L_x} = \frac{7.7}{5.2} = 1.48$
Short span moment coefficients:
moment coefficient $=\alpha_x$ =0.099
Long span moment coefficients:
moment coefficient $=\alpha_y$ =0.051
$M_{ux} = \alpha_x \times w_u \times L_{ex}{}^2$ =0.099× 16.5×5.2^2 = 44.169kNm
$M_{uy} = \alpha_y \times w_u \times L_{ex}{}^2$ =0.051× 16.5×5.2^2 = 22.75kNm

g)Check for depth
$(M_u)_{lim} = 0.133 \times f_{ck} \times b \times d^2$
$d_{required} = \sqrt{\frac{(M_u)_{lim}}{0.133 \times f_{ck} \times b}}$
$= \sqrt{\frac{44.169\times10^6}{0.133\times30\times1000}}$
= 105.21 mm
$d_{required} < d_{provided}$ (200- mm)
Hence the effective depth selected is sufficient to resist the design ultimate moment.

h)Reinforcements along short and long span directions
The area of reinforcement is calculated using the relation:
$M_u = 0.87 \times f_y \times A_{st} \times d\,(1 - \frac{A_{st}\times f_y}{b\times d\times f_{ck}})$
Spacing of the selected bars are computed using the relation; Spacing = S = $\frac{\text{Area of one bar}}{\text{total area}} \times 1000$
i) Check for spacing
As per IS 456:2000 clause 26.3.3(b)
Maximum spacing $= \{\, 3d$ or 300 mm $\}$ whichever is less

$$= \left\{ \begin{array}{c} 3 \times 112 = 336 \text{ mm} \\ \text{or} \\ 300 \text{ m} \end{array} \right\} \text{whichever is less}$$
= 300 mm

\

$spacing_{prov} < spacing_{max}$

$\therefore$ safe.

j)Check for area of steel

As per IS 456:2000 clause 26.5.2.1

Table 4. Reinforcement details in two way slab

Location	A_{st}(required)	Spacing of 10mmϕ bars	A_{st}(provided)
1)short span	240 mm²	100 mm	769.19 mm²
2)long span	240 mm²	160 mm	402.22 mm²

$(A_{st})_{min}$ = 0.12% of cross – sectional area

$$= \frac{0.12 \times 1000 \times 160}{100}$$
= 192 mm²

$(A_{st})_{prov} > (A_{st})_{min}$

$\therefore$ safe.

k) Check for deflection :

$(A_{st})_{provided}$ = 769.19 mm²

$(A_{st})_{required}$ = 240 mm²

$$f_s = \frac{0.58 \times fy \times (A_{st})_{required}}{(A_{st})_{provided}}$$
$$= \frac{0.58 \times 415 \times 240}{769.19}$$
=75.10

$$P_t = \frac{100 \times 769.19}{1000 \times 200} = 0.38$$

As per IS 456:2000 , fig 4 , page 38

Modification factor = 1.7

As per IS 456:2000 , clause 23.2.1

$(\frac{L}{d})_{basic}$ = 26

$$(\frac{L}{d})_{max} = 26 \times 1.7 = 44.2$$

$$(\frac{L}{d})_{provided} = \frac{7700}{200} = 38.5 < (\frac{L}{d})_{max}$$

So deflection is safe with provided depth.

l) Check for shear

$$V_u = \frac{W_u \times L_e}{2}$$
$$= \frac{16.5 \times 5200}{2}$$
=42.9 kN

As per IS 456:2000 clause 40.1

$$\tau_v = \frac{V_u}{b \times d}$$
$$= \frac{42.9 \times 10^3}{1000 \times 175}$$
= 0.24 N/mm²

$$p_t = \frac{100 \times A_{st}}{b \times d}$$
$$= \frac{100 \times 240}{1000 \times 175} = 0.137$$

As per IS 456:2000 clause 40.2

Design shear strength of concrete = $k \times \tau_c$

= 1.3×0.28

= 0.364 N/mm²

As per IS 456:2000, Table 20

Maximum shear stress,$(\tau_c)_{max}$ = .28 N/mm²

$\tau_v < \tau_c < (\tau_c)_{max}$

$\therefore$ shear reinforcement is not provided.

m) Check for cracking

As per IS 456:2000, clause 43.1:

7. Steel provided is more than 0.12 percents
8. Spacing of main steel< 300mm
9. Diameter of reinforcement< $\frac{D}{8} = \frac{160}{8}$ = 20 mm

Hence cracks will be within the permissible limits.

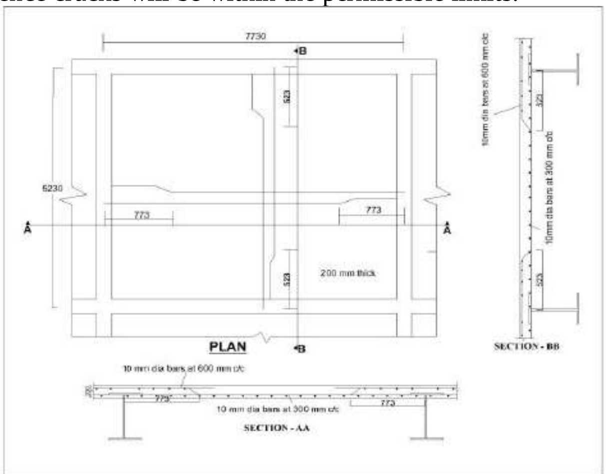

Fig 23:- Slab (IJKL) 7.5 x 5

B. Design of Composite Column

Details

Column dimension =500 x1000 x 25000

Concrete grade =M30

Steel section =ISLB 600

Reinforcement steel area =Fe 345 0.5% of gross concrete area

Cover from flange =40mm

Height of column =25000mm

1. List of Material Properties

a)Structural steel

Steel section ISLB 600

Nominal yield strength fy =250 N/mm²

Modulus of elasticity Ea =200 KN/mm

b) Concrete

Concrete grade =M 30

Characteristic strength Fck =30N/mm²

Section modulus of elasticity for short term loading, Ecm =28500 N/mm²

c) Reinforcing steel

Steel grade =Fe 200

Characteristic strength Fck =345 N/mm²

4. Partial safety factors
γa =1.15
γ^c =1.5
γ^s =1.15

2. List Section Properties Of The Given Section

Steel section

Aa =126.69 cm² =12669 mm²
h =600mm
tw =10.5mm
Iax =728.676 *10^6 mm^4
Iay =18.219 * 10^2 mm^4

a).Reinforcing steel

Area reinforcement =0.5% gross concrete area
=0.5/100*(48.7331) =2436.655mm²
Provide 5 bars , bar size 25,maximum size 150, As =2455 mm²

b).Concrete

Ac = A gross –An-As
=500 * 1000 -12669-2455
=484876 mm²

c) Design Check

1)Plastic resistance of the section

$Pp = A_a fy/\gamma a + \alpha c Ac Fck/\gamma c + As Fsk/\gamma s$
= [12669 * 250 /1.15 + 0.85 *48487 *30 /1.5 + 2455 *415 /1.15] /1000
=11882.95 KN

2) Calculation of effective elastic flexural stiffness of the section

About the major axis

(EI)ex =EaIax + 0.8 EcdIcx + EsIsx
Iax =728.676 *10^6 mm^4
Isx = Ah²
=2455 *[1000/2-40-12]²
=492.728 * 10^6 mm^4
Ics =500 *1000³/12 –(492.728 *10^6 +728.676 *10^6)
=40445.26 *10^6
(EI)ex =2 *10^5 *728.676 *10^6 +0.8 *21111 *40445.26*10^6 +2*10^5*492.728*10^2
=927.35*10^12 N/mm²

(3)Non dimensional slenderness

λ =(P pu/Pcr)½

Value of Ppu($\gamma a = \gamma c = \gamma s = 1$)

Ppu =Aafy+αcAcFck + As Fsk
=12669 *250 +0.85 *30 *484876 +345 *2455
=16378.563 KN

(Pcr) = π^2 (EI)ex/ l²
=π^2 (927.3510 *10^12)/(25000)²
=14644.166 KN

λ = (163.785/146.441)^(1/2)
= 1.057

4)Resistant of the composite column under axial compression

Buckling resistance of the section should satisfy the following condition

Pb < χ Pp

Where

Pb =buckling load
χ =reduction factor for column buckling
Pp =plastic resistance of the section
=11882.95 KN

χ values:

About major axis

αx =0.4

x χ =1/{ϕx +(ϕx ²- λx²)½

ϕx =0.5 [1+αx (λx-0.2)+ λx²
=0.5 [1 +0.4 (1.057-0.2) +1.057²]
=1.230

xχ =1/{1.23+[1.23²-(1.057)²]^(1/2)}
=0.537

(Pb)x =Xx Pp
=0.53 *1182.95
=6391.92 KN

3. Design of Foundation

Pile foundatio1n is adopted for foundation and following are the manual design of pile cap and pile.

a) Design Of Pile Cap

Details

Column dimension =500 x 1000 mm
Load = 3900kN
Concrete grade = M20
Steel = Fe 415
Depth of pile cap from punching shear conservation = 3900*1000/ (4*500*1.2)
= 1625
= 2000 mm

Depth from bending moment consideration

Load on column = 3900 kN
Weight of pile cap = 5% of column head
= 5/100 * 3900
= 195 kN
Total load = 3900 + 195
= 4095 kN
Load per pile = 4095/6
= 682.5 kN

Maximum BM for the pile cap
= 2* 682.5* 1000 [1000 – 250]
= 1023750000
= 1*10ˆ9Nmm

Adopting e = 7 N/mm^2
t = 230N/mm^2
m = 18.66

0.913bd^2 = 0.913* 1500* d^2
1*10^9 = 1369.5 d^2
d = 854.51 mm

Provide 80mm effective cover

Effective depth = 2000-80
= 1920mm

Ast = 1*10ˆ9/ (230* 0.90* 1920)
= 2516.10 mm^2

Provide 20 mm bars

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Number of bars= Ast /(∏/4* d^2)
= 2516.10/(∏/4*22^2)
= 6.6 ~ 7
Provide 7 bars of 25 mm diameter
Provide transverse reinforcement of 10 bars of 16mm diameter

b) Design Of Pile

Safe direct compressive stress in concrete = 5 N/ mm^2
Safe direct compressive stress in steel = 190N/ mm^2
Safe compressive stress in concrete in bending = 7 N/ mm^2
Safe tensile stress in steel in bending = 230 N/ mm^2
Modular ratio, = 280/ (3*7)
= 13.33
Load on each pile =3900/6
= 650kN
Rate of length of pile to least lateral dimension of pile
= l / D
= 10000/500
= 20
This is greater than 12, hence pile will be treated as long column
Reduction coefficient, C_r = 1.25 – l/48D
=1.25-10000/(48*500)
= 0.833
Safe direct compressive stress in concrete = 0.833*5
= 4.166N/ mm^2
Safe direct compressive stress in steel = 0.833* 190
= 158.27N/ mm^2
Safe compressive load on column = Safe load on concrete + safe load on steel
(10000-A_{sc}) * 3.125+ A_{sc}* 158.27 = 650 *10^3
155. 145 A_{sc} = 618750
A_{sc} = 3988.20
= 4000 mm^2
Provide 25mm diameter bars
Number of bars = 4000/(∏/4* 25^2)
= 8
Provide 8 bars of 25 mm diameter
Lateral ties
Let 8mm diameter ties be provided
Volume = 0.2% of volume of piles
Provide 8mm diameter bars at 150 spacing.

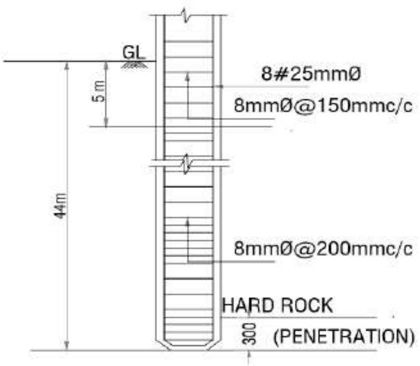

Fig 24:- Longitudinal section

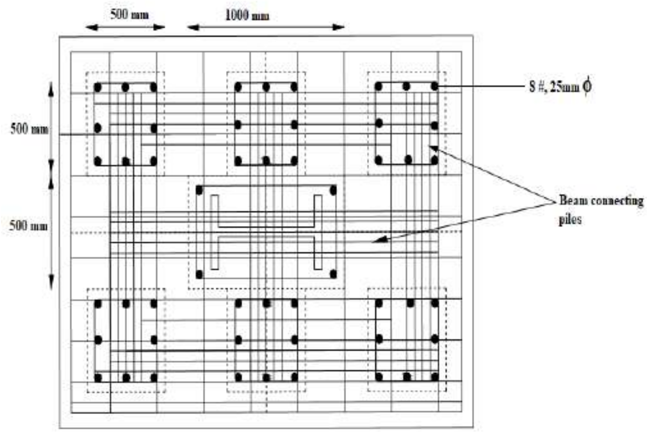

Fig 25:- Reinforcement details

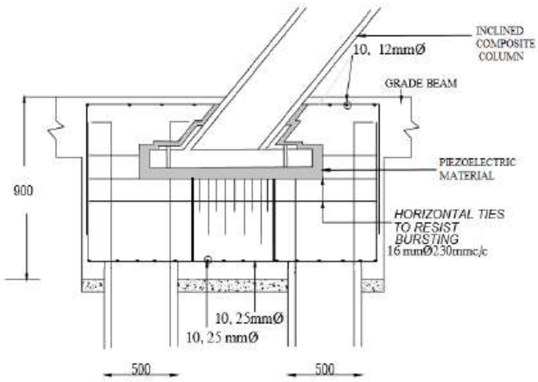

Fig 26:- Column foundation details

XI. PROJECT ANALYSIS

A. General

Structural analysis, which is an integral part of any engineering project, is the process of predicting the performance of a given structure under a prescribed loading condition. The performance characteristics usually of interest in structural design are:

1. Stress or stress resultant (axial forces, shears and bending moments)
2. Deflections
3. Support reactions

Thus the analysis of a structure typically involves the determination of these quantities caused by the given loads and / or the external effects. Since the building frame is three dimensional frames i.e. a space frame, manual analysis is tedious and time consuming. Hence the structure is analyzed with ETABS 2015. In order to analyze in ETABS 2015,I have to first generate the model geometry, specify member properties, specify geometric constants and specify supports and loads.

B. Generating Model Geometry

The structure geometry consists of joint members, their coordinates, member numbers, the member

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connectivity information, plate element numbers, etc. At first fix the position of beams and columns. Then the joint coordinates were fixed. Beam centre lines were taken for fixing joint coordinates. Then the members were connected along the joint coordinates using the member incidence command.

C. Specifying Member Property

The next task is to assign cross section properties for the beams and columns the member properties were given as Indian. The width ZD and depth YD were given for the sections. The support conditions were given to the structure.

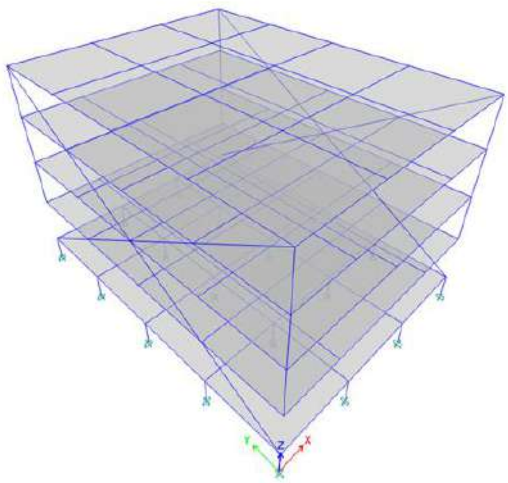

Fig 27:- ETABS 2016 model without load analysis

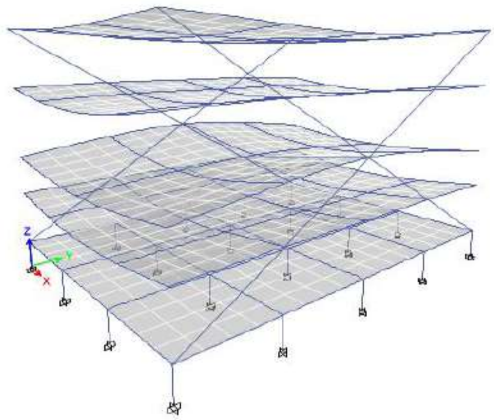

Fig 28:- Deflection of structure without tie member

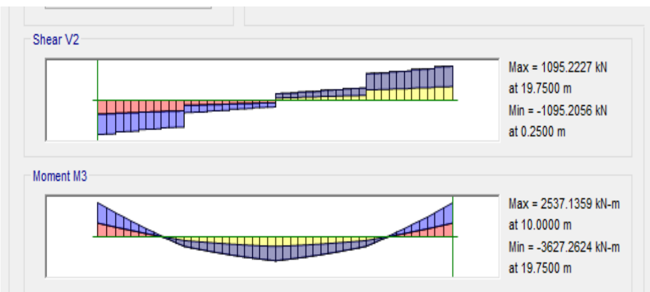

Fig 29:- Bending moment and shear force diagram of a beam

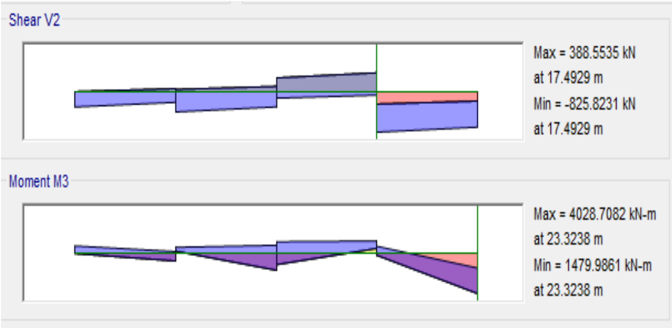

Fig 30:- Bending moment and shear force diagram of a column

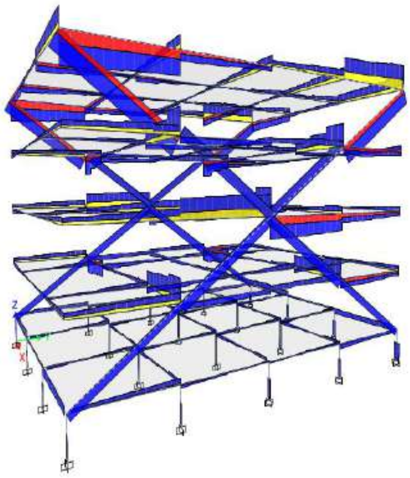

Fig 31:- Bending moment of the structure without tie member

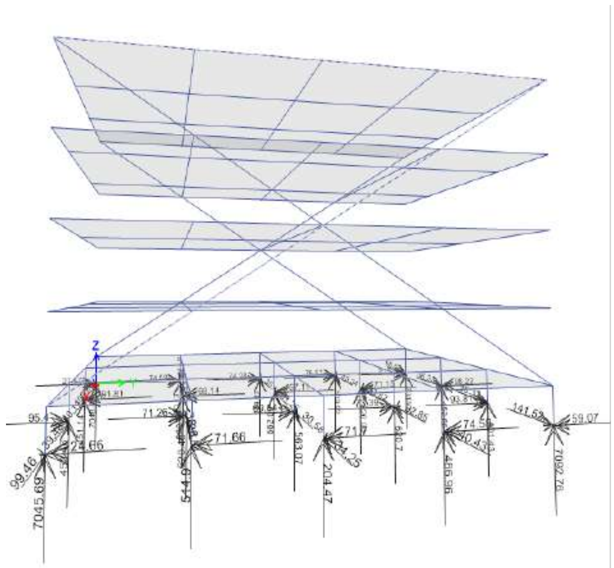

Fig 32:- Shear force diagram of structure without tie member

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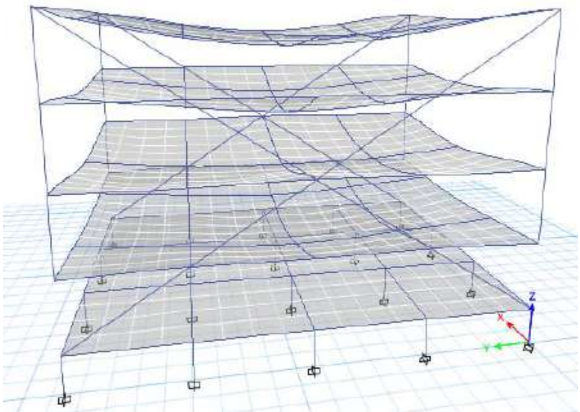

Fig 33:- Deflection of structure with tie member

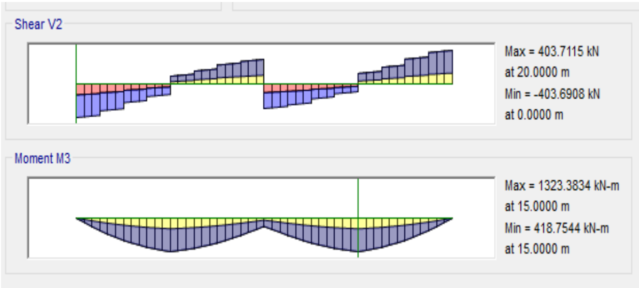

Fig 34:- BMD and SFD of beam with tie member

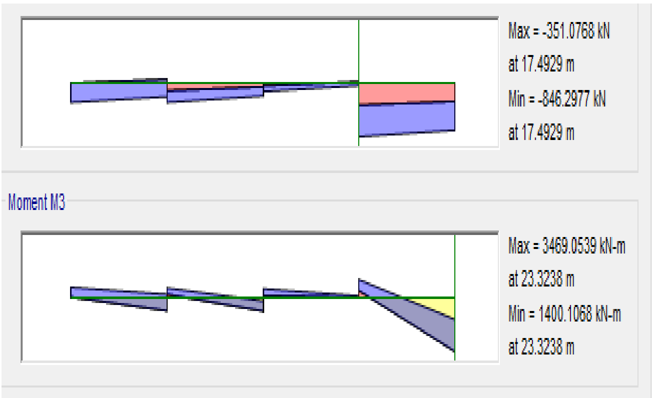

Fig 35:- Bending moment and shear force diagram of column

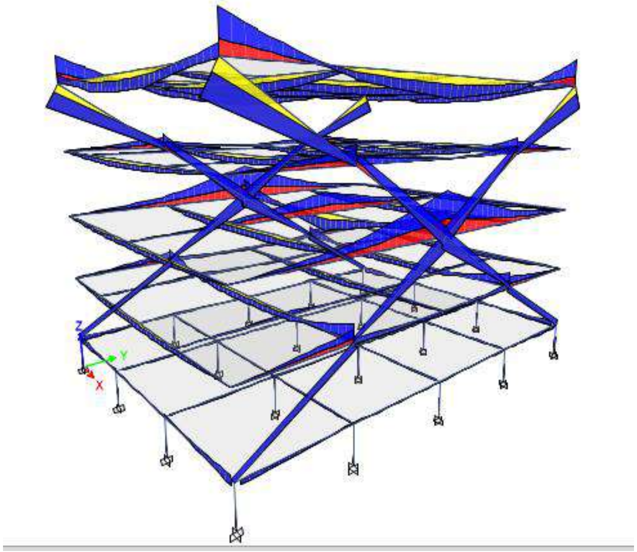

Fig 36:- Moment diagram

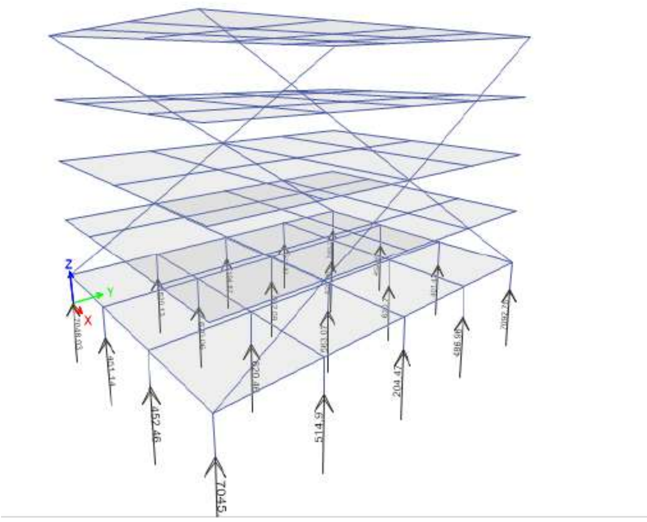

Fig 37:- Shear force diagram

XII. RESULT

A. General

The table shown below represents final result of the design. It gives the bending moment, shear force and torque of the corresponding column. In the table, V_2 is the shear force in X direction , V_3 is the shear force in Y axis. T is the torque, M_2 is the moment in X direction and M_3 is the moment in Y direction. Thus

Maximum shear force in X direction = 872.877kN
Maximum value of torque = 0.2556kNm
Maximum value of moment in X direction = 411.933kN
Maximum value of moment in Y direction = 1375.341kNm

Table 6. Result

Story	Column	Load Case/Combo	P	V2	V3
Story5	C1	ENVELOP Max	1188.3504	237.8861	211.2661
Story5	C1	ENVELOP Max	1193.8275	237.8861	211.2661
Story5	C1	ENVELOP Max	1199.3046	237.8861	211.2661
Story5	C1	DStlS26 Max	745.4264	-636.904	128.6414

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Story5	C1	DStlS26 Max	751.9989	-636.904	128.6414
Story5	C1	DStlS26 Max	758.5714	-636.904	128.6414
Story5	C4	ENVELOP Max	1185.3199	872.877	211.3146
Story5	C4	ENVELOP Max	1190.797	872.877	211.3146
Story5	C4	ENVELOP Max	1196.2741	872.877	211.3146
Story5	C4	DStlS26 Max	594.1869	423.6031	136.0442
Story5	C4	DStlS26 Max	600.7594	423.6031	136.0442
Story5	C4	DStlS26 Max	607.3319	423.6031	136.0442
Story5	C17	ENVELOP Max	1185.3995	239.0486	-82.4147
Story5	C17	ENVELOP Max	1190.8766	239.0486	-82.4147
Story5	C17	ENVELOP Max	1196.3537	239.0486	-82.4147
Story5	C17	DStlS26 Max	743.7299	638.2411	128.2813
Story5	C17	DStlS26 Max	750.3024	638.2411	128.2813
Story5	C17	DStlS26 Max	756.8749	638.2411	128.2813
Story5	C20	ENVELOP Max	1188.1523	870.3756	82.4004
Story5	C20	ENVELOP Max	1193.6294	870.3756	-82.4004
Story5	C20	ENVELOP Max	1199.1065	870.3756	-82.4004
Story5	C20	DStlS26 Max	595.7779	422.1766	-135.6368
Story5	C20	DStlS26 Max	602.3504	422.1766	-135.6368
Story5	C20	DStlS26 Max	608.9229	422.1766	-135.6368
Story4	C1	ENVELOP Max	660.8267	27.3341	37.0502
Story4	C1	ENVELOP Max	666.3038	27.3341	37.0502
Story4	C1	ENVELOP Max	671.7809	27.3341	37.0502
Story4	C1	DStlS26 Max	391.7544	-319.2522	14.565
Story4	C1	DStlS26 Max	398.3269	-319.2522	14.565
Story4	C1	DStlS26 Max	404.8995	-319.2522	14.565
Story4	C4	ENVELOP Max	658.2779	389.2886	36.8319
Story4	C4	ENVELOP Max	663.755	389.2886	36.8319
Story4	C4	ENVELOP Max	669.2321	389.2886	36.8319
Story4	C4	DStlS26 Max	293.9504	35.3209	20.6885
Story4	C4	DStlS26 Max	300.5229	35.3209	20.6885
Story4	C4	DStlS26 Max	307.0955	35.3209	20.6885
Story4	C17	ENVELOP Max	658.1791	27.1971	-0.095
Story4	C17	ENVELOP Max	663.6562	27.1971	-0.095
Story4	C17	ENVELOP Max	669.1333	27.1971	-0.095
Story4	C17	DStlS26 Max	390.3271	-319.3103	-12.0142
Story4	C17	DStlS26 Max	396.8996	-319.3103	-12.0142
Story4	C17	DStlS26 Max	403.4721	-319.3103	-12.0142
Story4	C20	ENVELOP Max	660.9897	388.9729	-0.2484
Story4	C20	ENVELOP Max	666.4668	388.9729	-0.2484
Story4	C20	ENVELOP Max	671.9439	388.9729	-0.2484
Story4	C20	DStlS26 Max	295.4647	35.105	-18.3506
Story4	C20	DStlS26 Max	302.0372	35.105	-18.3506
Story4	C20	DStlS26 Max	308.6097	35.105	-18.3506
Story3	C1	ENVELOP Max	84.8871	-37.2492	-11.8763

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Story3	C1	ENVELOP Max	88.1734	-37.2492	-11.8763
Story3	C1	ENVELOP Max	91.4596	-37.2492	-11.8763
Story3	C1	DStlS26 Max	-19.225	473.7658	112.7677
Story3	C1	DStlS26 Max	-12.6525	473.7658	112.7677
Story3	C1	DStlS26 Max	-6.08	473.7658	112.7677
Story3	C4	ENVELOP Max	84.4258	593.9313	-12.24
Story3	C4	ENVELOP Max	87.712	593.9313	-12.24
Story3	C4	ENVELOP Max	90.9983	593.9313	-12.24
Story3	C4	DStlS26 Max	-50.0597	120.7635	-90.3551
Story3	C4	DStlS26 Max	-43.4872	120.7635	-90.3551
Story3	C4	DStlS26 Max	-36.9146	120.7635	-90.3551
Story3	C17	ENVELOP Max	83.9227	-38.466	176.3051
Story3	C17	ENVELOP Max	87.209	-38.466	176.3051
Story3	C17	ENVELOP Max	90.4953	-38.466	176.3051
Story3	C17	DStlS26 Max	-20.546	-474.966	119.0203
Story3	C17	DStlS26 Max	-13.9735	-474.966	119.0203
Story3	C17	DStlS26 Max	-7.401	-474.966	119.0203
Story3	C20	ENVELOP Max	85.1496	591.762	174.9119
Story3	C20	ENVELOP Max	88.4359	591.762	174.9119
Story3	C20	ENVELOP Max	91.7222	591.762	174.9119
Story3	C20	DStlS26 Max	-48.5829	119.5958	94.7381
Story3	C20	DStlS26 Max	-42.0104	119.5958	94.7381
Story3	C20	DStlS26 Max	-35.4379	119.5958	94.7381
Story1	C1	ENVELOP Max	-2518.4235	67.308	176.2713
Story1	C1	ENVELOP Max	-2516.9839	67.308	176.2713
Story1	C1	DStlS26 Max	-4732.7565	67.774	86.0992
Story1	C2	ENVELOP Max	-26.5369	136.3009	59.4957
Story1	C2	ENVELOP Max	-23.6577	136.3009	59.4957
Story1	C2	DStlS26 Max	-98.4441	-70.1349	22.1942
Story1	C3	ENVELOP Max	-31.0361	86.4715	58.5227
Story1	C3	ENVELOP Max	-29.5965	86.4715	58.5227
Story1	C3	DStlS26 Max	-330.4877	113.1384	21.4564
Story1	C4	DStlS26 Max	-4034.4517	126.8906	121.7657
Story1	C4	DStlS26 Max	-4030.1329	126.8906	121.7657
Story1	C4	DStlS26 Max	-4025.8142	126.8906	121.7657
Story1	C5	DStlS26 Max	-357.5776	-92.587	-40.9083
Story1	C5	DStlS26 Max	-351.8193	-92.587	-40.9083
Story1	C6	ENVELOP Max	-226.1844	86.9751	38.5226
Story1	C6	DStlS26 Max	-366.856	-72.3769	-5.2579
Story1	C6	DStlS26 Max	-361.4576	-72.3769	-5.2579
Story1	C6	DStlS26 Max	-356.0592	-72.3769	-5.2579
Story1	C7	ENVELOP Max	-228.9402	77.7738	37.3452
Story1	C7	ENVELOP Max	-226.241	77.7738	37.3452
Story1	C7	DStlS26 Max	-373.5625	-84.2966	-5.7899
Story1	C8	ENVELOP Max	-171.0882	95.4844	10.6155

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Story1	C8	ENVELOP Max	-169.6486	95.4844	10.6155
Story1	C8	ENVELOP Max	-168.209	95.4844	10.6155
Story1	C9	ENVELOP Max	-75.8363	56.0182	47.1991
Story1	C9	ENVELOP Max	-74.3967	56.0182	47.1991
Story1	C9	ENVELOP Max	-72.9572	56.0182	47.1991
Story1	C9	DStlS26 Max	-138.6363	-94.1685	0.6764
Story1	C10	ENVELOP Max	-214.7609	81.5968	40.5175
Story1	C10	ENVELOP Max	-212.0617	81.5968	40.5175
Story1	C10	ENVELOP Max	-209.3625	81.5968	40.5175
Story1	C10	DStlS26 Max	-336.6228	-75.0864	-1.3034
Story1	C10	DStlS26 Max	-331.2244	-75.0864	-1.3034
Story1	C11	ENVELOP Max	-215.0483	75.1509	39.2267
Story1	C11	DStlS26 Max	-349.0634	-84.0769	-0.8577
Story1	C11	DStlS26 Max	-343.665	-84.0769	-0.8577
Story1	C12	ENVELOP Max	-79.249	92.598	42.7308
Story1	C12	ENVELOP Max	-76.3699	92.598	42.7308
Story1	C12	DStlS26 Max	-120.4116	-48.9113	1.5983
Story1	C12	DStlS26 Max	-114.6533	-48.9113	1.5983
Story1	C13	ENVELOP Max	-180.4404	56.9727	74.0363
Story1	C13	ENVELOP Max	-179.0008	56.9727	74.0363
Story1	C13	ENVELOP Max	-177.5612	56.9727	74.0363
Story1	C13	DStlS26 Max	-358.295	-90.5922	41.0646
Story1	C13	DStlS26 Max	-355.4158	-90.5922	41.0646
Story1	C13	DStlS26 Max	-352.5367	90.5922	41.0646
Story1	C14	ENVELOP Max	-231.1958	84.3923	44.7284
Story1	C14	ENVELOP Max	-228.4966	84.3923	44.7284
Story1	C14	DStlS26 Max	-355.4644	-70.0946	2.4875
Story1	C15	ENVELOP Max	-231.3136	75.4487	43.2888
Story1	C15	ENVELOP Max	-228.6143	75.4487	43.2888
Story1	C15	ENVELOP Max	-225.9151	75.4487	43.2888
Story1	C16	ENVELOP Max	-177.4483	92.9999	70.0549
Story1	C16	DStlS26 Max	-298.8954	-45.5681	37.2119
Story1	C16	DStlS26 Max	-296.0162	-45.5681	37.2119
Story1	C17	ENVELOP Max	-2522.9831	71.3558	39.0273
Story1	C17	ENVELOP Max	-2521.5435	71.3558	39.0273
Story1	C17	DStlS26 Max	-4733.8459	71.264	-81.8021
Story1	C18	ENVELOP Max	-19.8158	130.9898	23.1276
Story1	C18	ENVELOP Max	-18.3762	130.9898	23.1276
Story1	C18	ENVELOP Max	-16.9367	130.9898	23.1276
Story1	C19	ENVELOP Max	-20.8687	81.3104	21.9757
Story1	C19	ENVELOP Max	-19.4291	81.3104	21.9757
Story1	C19	DStlS26 Max	-377.3562	-121.2026	-22.7239
Story1	C20	ENVELOP Max	-2517.4028	74.1356	36.8912
Story1	C20	DStlS26 Max	-3983.8923	72.9754	-67.9812
Story1	C20	DStlS26 Max	-3978.134	72.9754	-67.9812

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Story	Column	Load Case/Combo	T	M2	M3
Story5	C1	ENVELOP Max	0.0033	162.8857	-114.9681
Story5	C1	ENVELOP Max	0.0033	30.8732	494.1207
Story5	C1	ENVELOP Max	0.0033	128.59	1370.7983
Story5	C1	DStlS26 Max	0.0031	98.2129	-259.6602
Story5	C1	DStlS26 Max	0.0031	-49.7231	483.9872
Story5	C1	DStlS26 Max	0.0031	-197.2117	1228.7433
Story5	C4	ENVELOP Max	0.0033	163.0012	761.6884
Story5	C4	ENVELOP Max	0.0033	-30.8492	232.6793
Story5	C4	ENVELOP Max	0.0033	-128.5888	-42.0337
Story5	C4	DStlS26 Max	0.0031	109.7239	652.9941
Story5	C4	DStlS26 Max	0.0031	-46.7259	177.0599
Story5	C4	DStlS26 Max	0.0031	-202.7269	-297.7679
Story5	C17	ENVELOP Max	0.0033	-60.9655	-114.7141
Story5	C17	ENVELOP Max	0.0033	80.0046	496.298
Story5	C17	ENVELOP Max	0.0033	323.0202	1375.341
Story5	C17	DStlS26 Max	0.0031	-97.8384	-259.2955
Story5	C17	DStlS26 Max	0.0031	49.6858	485.8693
Story5	C17	DStlS26 Max	0.0031	197.6592	1232.1636
Story5	C20	ENVELOP Max	0.0033	-60.9263	761.8305
Story5	C20	ENVELOP Max	0.0033	80.0623	233.7203
Story5	C20	ENVELOP Max	0.0033	323.0358	-39.97
Story5	C20	DStlS26 Max	0.0031	-109.2289	653.069
Story5	C20	DStlS26 Max	0.0031	46.7548	178.7534
Story5	C20	DStlS26 Max	0.0031	203.1859	-294.4325
Story4	C1	ENVELOP Max	0.0041	112.1718	-212.9542
Story4	C1	ENVELOP Max	0.0041	72.4189	72.4166
Story4	C1	ENVELOP Max	0.0041	32.6659	418.2847
Story4	C1	DStlS26 Max	0.0039	55.5798	-336.6591
Story4	C1	DStlS26 Max	0.0039	38.8301	30.4809
Story4	C1	DStlS26 Max	0.0039	23.9483	401.0534
Story4	C4	ENVELOP Max	0.0041	112.0978	642.5889
Story4	C4	ENVELOP Max	0.0041	72.6167	336.2645
Story4	C4	ENVELOP Max	0.0041	33.136	275.0907
Story4	C4	DStlS26 Max	0.0039	63.9246	334.1492
Story4	C4	DStlS26 Max	0.0039	40.1329	293.5302
Story4	C4	DStlS26 Max	0.0039	18.2059	256.3418
Story4	C17	ENVELOP Max	0.0041	-18.537	-212.1662
Story4	C17	ENVELOP Max	0.0041	-18.4277	72.8123
Story4	C17	ENVELOP Max	0.0041	-9.1841	419.1368
Story4	C17	DStlS26 Max	0.0039	-51.7519	-335.9976
Story4	C17	DStlS26 Max	0.0039	-37.9356	31.2092
Story4	C17	DStlS26 Max	0.0039	-22.25	401.8078

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Story4	C20	ENVELOP Max	0.0041	-18.5958	643.8582
Story4	C20	ENVELOP Max	0.0041	-18.3101	337.4053
Story4	C20	ENVELOP Max	0.0041	-9.0807	275.6244
Story4	C20	DStlS26 Max	0.0039	-60.1169	334.6777
Story4	C20	DStlS26 Max	0.0039	-39.0137	294.3069
Story4	C20	DStlS26 Max	0.0039	-16.047	257.3279
Story3	C1	ENVELOP Max	0.0019	-64.463	-76.5049
Story3	C1	ENVELOP Max	0.0019	-50.8052	-32.7127
Story3	C1	ENVELOP Max	0.0019	15.5135	450.7773
Story3	C1	DStlS26 Max	0.0016	-270.1385	-729.4353
Story3	C1	DStlS26 Max	0.0016	-140.4556	-183.649
Story3	C1	DStlS26 Max	0.0016	-8.2488	362.1373
Story3	C4	ENVELOP Max	0.0019	-65.1415	919.502
Story3	C4	ENVELOP Max	0.0019	-51.0655	237.2489
Story3	C4	ENVELOP Max	0.0019	15.9844	-2.4774
Story3	C4	DStlS26 Max	0.0016	-226.8499	206.6682
Story3	C4	DStlS26 Max	0.0016	-122.9415	68.7501
Story3	C4	DStlS26 Max	0.0016	-16.5026	-69.168
Story3	C17	ENVELOP Max	0.0019	411.9332	-77.9913
Story3	C17	ENVELOP Max	0.0019	222.1196	-32.7658
Story3	C17	ENVELOP Max	0.0019	48.5703	452.5228
Story3	C17	DStlS26 Max	0.0016	281.7033	-731.0247
Story3	C17	DStlS26 Max	0.0016	144.83	-183.8241
Story3	C17	DStlS26 Max	0.0016	10.4814	363.3764
Story3	C20	ENVELOP Max	0.0019	409.2431	916.7126
Story3	C20	ENVELOP Max	0.0019	220.6277	236.9783
Story3	C20	ENVELOP Max	0.0019	49.3207	-1.7398
Story3	C20	DStlS26 Max	0.0016	234.8545	205.1772
Story3	C20	DStlS26 Max	0.0016	125.9056	68.632
Story3	C20	DStlS26 Max	0.0016	19.482	-67.9133
Story1	C1	ENVELOP Max	0.1227	93.0095	5.771
Story1	C1	ENVELOP Max	0.1227	22.501	24.3989
Story1	C1	DStlS26 Max	0.117	4.3721	-21.163
Story1	C2	ENVELOP Max	0.1227	39.4861	88.4177
Story1	C2	ENVELOP Max	0.1227	-4.6422	3.6517
Story1	C2	DStlS26 Max	0.117	-8.2242	0.4889
Story1	C3	ENVELOP Max	0.1227	38.4725	66.1545
Story1	C3	ENVELOP Max	0.1227	15.2298	31.5659
Story1	C3	DStlS26 Max	0.117	-7.7976	15.707
Story1	C4	DStlS26 Max	0.2439	55.9172	-33.76
Story1	C4	DStlS26 Max	0.2439	7.2109	-84.5163
Story1	C4	DStlS26 Max	0.2439	-39.2379	-135.2725
Story1	C5	DStlS26 Max	0.117	-18.1891	-66.6235
Story1	C5	DStlS26 Max	0.117	14.5375	7.4461
Story1	C6	ENVELOP Max	0.1227	38.0062	56.7678
Story1	C6	DStlS26 Max	0.117	-2.7893	-57.5532

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Story1	C6	DStlS26 Max	0.117	1.1541	-3.2705
Story1	C6	DStlS26 Max	0.117	7.2385	54.4598
Story1	C7	ENVELOP Max	0.1227	2.4291	3.6966
Story1	C7	ENVELOP Max	0.1227	37.3933	67.1724
Story1	C7	DStlS26 Max	0.117	7.563	66.9882
Story1	C8	ENVELOP Max	0.1227	17.8375	68.7238
Story1	C8	ENVELOP Max	0.1227	13.5913	30.5301
Story1	C8	ENVELOP Max	0.1227	17.2033	-4.4215
Story1	C9	ENVELOP Max	0.1227	35.3598	50.0492
Story1	C9	ENVELOP Max	0.1227	16.4801	27.6419
Story1	C9	ENVELOP Max	0.1227	2.5103	13.7306
Story1	C9	DStlS26 Max	0.117	0.4741	-67.7274
Story1	C10	ENVELOP Max	0.1227	31.3743	61.3216
Story1	C10	ENVELOP Max	0.1227	0.9862	0.6685
Story1	C10	ENVELOP Max	0.1227	32.3355	55.5361
Story1	C10	DStlS26 Max	0.117	-1.0145	-59.1633
Story1	C10	DStlS26 Max	0.117	-0.037	-2.8486
Story1	C11	ENVELOP Max	0.1227	30.3839	58.6359
Story1	C11	DStlS26 Max	0.117	-0.647	-63.1983
Story1	C11	DStlS26 Max	0.117	-0.0038	-0.1406
Story1	C12	ENVELOP Max	0.1227	32.2508	66.2589
Story1	C12	ENVELOP Max	0.1227	2.886	-4.5546
Story1	C12	DStlS26 Max	0.117	1.5632	47.416
Story1	C12	DStlS26 Max	0.117	0.6071	-8.2869
Story1	C13	ENVELOP Max	0.1227	47.1265	50.8636
Story1	C13	ENVELOPMax	0.1227	17.5119	28.0746
Story1	C13	ENVELOP Max	0.1227	-6.2179	13.731
Story1	C13	DStlS26 Max	0.117	18.6003	-65.0144
Story1	C13	DStlS26 Max	0.117	2.1745	-28.7775
Story1	C13	DStlS26 Max	0.117	-14.2514	7.4594
Story1	C14	ENVELOP Max	0.1227	32.9328	62.9088
Story1	C14	ENVELOP Max	0.1227	-0.0517	0.3656
Story1	C14	DStlS26 Max	0.117	-0.9015	53.7155
Story1	C15	ENVELOP Max	0.1227	31.9364	59.1556
Story1	C15	ENVELOP Max	0.1227	-0.078	3.6921
Story1	C15	ENVELOP Max	0.1227	29.0369	66.642
Story1	C16	ENVELOP Max	0.1227	44.0241	66.7719
Story1	C16	DStlS26 Max	0.117	2.6618	-26.5804
Story1	C16	DStlS26 Max	0.117	-12.223	-8.3531
Story1	C17	ENVELOP Max	0.1227	31.6922	9.0235
Story1	C17	ENVELOP Max	0.1227	16.0813	22.5174
Story1	C17	DStlS26 Max	0.117	30.5987	-47.9919
Story1	C18	ENVELOP Max	0.1227	23.5697	84.1544
Story1	C18	ENVELOP Max	0.1227	14.3187	31.7585
Story1	C18	ENVELOP Max	0.1227	13.8031	3.0003

Story1	C19	ENVELOP Max	0.1227	22.6417	62.1725
Story1	C19	ENVELOP Max	0.1227	13.854	29.6483
Story1	C19	DStlS26 Max	0.117	-10.4606	-77.6444
Story1	C20	ENVELOP Max	0.1227	47.9776	47.8202
Story1	C20	DStlS26 Max	0.117	-29.6641	9.5027
Story1	C20	DStlS26 Max	0.117	26.3823	-48.5926

XIII. CONCLUSION

A. General

The idea from the foldable book stand resulted to arise a modified form of diagrid structure. In diagrid structure, only the façade columns were eliminated. But the succeeding interior columns will act as an obstruction to the open space area .This paper presented the safe design of a four storied column less office building. The building is designed in such a way that the entire building load is carried by four inclined composite columns provided on the periphery. It has a convenient shape to transfer the load safely to base through arch action. For the safe design, we have modeled our structure in Auto CAD 2017 and analyzed using software ETABS 2017. And also the manual calculation were done. Dead load. Live load, seismic load were considered while designing. Pile foundation is adopted by considering the site of erecting.

At present we are not designed it as an earthquake resistant, only suggestions were made to make the building as an earthquake resistant. For this, numerous base isolation techniques were described in this paper.

By analyzing the model, we acquired a conclusion that two opposite natured moment were acting on the middle floor and top most roof and the deflection on these two layer is also in opposite direction. We put forward some suggestions for the efficient working of the model when excessive deflection were occurred. Vierendeel girder and tie member are one of a kind. A tie member can be used so as to counteract the opposite deflections. This tension member hooked from the four corners of middle floor can be used to cross the top roof diagonally an then tied to the middle floor. Tie member shouldn't carry any vertical load on slab or walls but take axial compression load.

Now, we have visualized our project as three dimensional object based modeling with the help of ETABS 2016 and our structure was analyzed successfully and satisfactory result was obtained.

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