

DESIGN OF AN EARTHQUAKE RESISTANT BUILDING BY USING PERFORMANCE-BASED SEISMIC CONCEPT

Submitted in partial fulfillment of the requirements for the award of Bachelor of
Engineering degree in Civil Engineering

By

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**SCHOOL OF BUILDING AND ENVIRONMENT
DEPARTMENT OF CIVIL ENGINEERING**

SATHYABAMA

INSTITUTE OF SCIENCE AND TECHNOLOGY (DEEMED TO BE UNIVERSITY)

Accredited with Grade "A" by NAAC

JEPPIAAR NAGAR, RAJIV GANDHI SALAI, CHENNAI - 600 119

APRIL 2021



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DEPARTMENT OF CIVIL ENGINEERING

BONAFIDE CERTIFICATE

This is to certify that this Project Report is the Bonafide work of **J.JESTALT SRISANTH (Reg.No.37200026)** and **MARIA BEVEN (Reg.No.37200038)** who carried out the project entitled "**DESIGN OF AN EARTHQUAKE RESISTANT BUILDING BY USING PERFORMANCE-BASED SEISMIC CONCEPT**" under my supervision from December 2020 to April 2021.

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DECLARATION

We, J.JESTALT SRISANTH (Reg.No.37200026) and MARIA BEVEN (Reg.No.37200038) hereby declare that the Project Report entitled “**DESIGN OF AN EARTHQUAKE RESISTANT BUILDING BY USING PERFORMANCE-BASED SEISMIC CONCEPT**” done by us under the guidance of **Dr.V. Sampathkumar**, M.A, M.Sc., M.E, PhD, Diploma in IRT Delhi, is submitted in partial fulfillment of the requirements for the award of Bachelor of Engineering degree in Civil Engineering.

1.

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DATE:

PLACE:

**SIGNATURE OF THE
CANDIDATE**

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ABSTRACT

An earthquake is a natural phenomenon occurring with uncertainties which is a result of sudden release of energy in the earth's crust which leads to the formation of seismic waves. Preventive measures are incorporated during the designing process of the structure. An earthquake resistant building is a structure that is designed to resist the effect of earthquake and reduce the structural damage of the building. Hence, within the areas which are susceptible to seismic effects, construction and designing requires measures to make sure stability, serviceability, strength with considerable levels of safety. Earthquake constitutes one of the greatest hazards of life and property on the earth. Due to suddenness of their occurrence, they're least understood and most dreaded. This project presents the brief essentials of an earthquake resistant building by using performance based design to improve the seismic resistance of the building. From the last decade of the twentieth century there has been much progress in the seismic activity in India, in fact more than 50% area in the country is considered prone to damaging earthquakes. One of the main causes of earthquakes in these regions is due to the movement of the Indian plate towards the Eurasian plate at the rate of 50 mm per year. The study area chosen for this project is Admin block of Sathyabama and its main objectives is to design structure that can withstand seismic activities or conditions than their counterpart conventional structures so that loss of life is minimized by taking preventive measures during construction process and compare the seismic provision of a building to IS and ASCE codes. In conclusion, the Strong Column Weak Beam concept is considered for study and the results in longitudinal rebar area of the beams using different methods are compared with the existing structure.

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LIST OF ABBREVIATIONS AND SYMBOLS

SYMBOLS	DESCRIPTION
$\sum_{i=1}^n$	Sigma
ASCE	American society of civil engineering
DDBD	direct displacement based design
EL	Earthquake load
h_b	height of beam
H_e	Effective Height
H_i	height of the floor
IS	Indian standard
L_b	length of base
Mb	Body Wave Magnitude
MDOF	multi degree of freedom
m_e	equivalent mass
m_i	mass intensity
M_L	Local Magnitude
M_s	Surface wave magnitude
M_w	Magnitude of the earthquake
M_w	Seismic Moment magnitude
ϕ	diameter of steel reinforcement
SDF	Single Degree Freedom
SDOF	single degree of freedom
SOA	state of art
SOP	state of practice
T_c	corner period
T_e	Effective period of substitute structure
T_n	natural vibration period
δ_c	corner displacement
Δ_d	Design Storey Displacements
Δ_i	top lateral displacement
δ_i	Intermediate displacement
δ_{max}	maximum displacement

SYMBOLS

DESCRIPTION

ε	eccentricity
ε_y	equivalent damping ratio
ζ	damping ratio
θ_y	yield displacement
μ	Design Displacement Ductility

CHAPTER 1

INTRODUCTION

1.1 GENERAL

An earthquake is defined as a sudden violent shaking of the bottom surface causing great destruction, leading to movements within the earth's crust. According to geologists, an earthquake is described as a sudden slip on a fault, or by volcanic and magmatic activity. During the nineteenth century, there had been tremendous progress on the instrumental side for the measurement of earthquake data. This data is known as seismological data. After collecting abundant data from the recorded earthquakes; they were analyzed to map and understand the phenomena of earthquakes. These data were wont to resolve the earth's internal structure to an interesting degree, which helped towards the event of the many theories to elucidate the causes of earthquakes. With the help of collected seismological data, the rational design of structures to withstand earthquakes has been formatted, it has also revealed the uncertain nature of future earthquakes for which structures are to be designed. Therefore, probabilistic concepts in dealing with earthquakes and earthquake-resistant designs have unfolded. Even though their aims are different, both seismologists and earthquake engineers use seismological data for the understanding of an earthquake and its impacts. Seismologists focus their attention on the global issue of earthquakes and the propagation of seismic waves in geological materials including the prediction of earthquakes whereas, earthquake engineers are concerned mainly with the local effects of earthquakes which are capable of causing significant damage to the structures. They help in transforming seismological data into a more appropriate form for the prediction of structural damages, alternatively the safe design of structures. There are many topics in seismology that are of immediate engineering interest and their use for the seismic design of structures.

1.2 EARTH AND ITS INTERIORS

Geological investigations with seismological data revealed that the earth primarily consists of four distinct layers: the inner core, the outer core, the mantle, and the crust, as shown in figure 1.1.

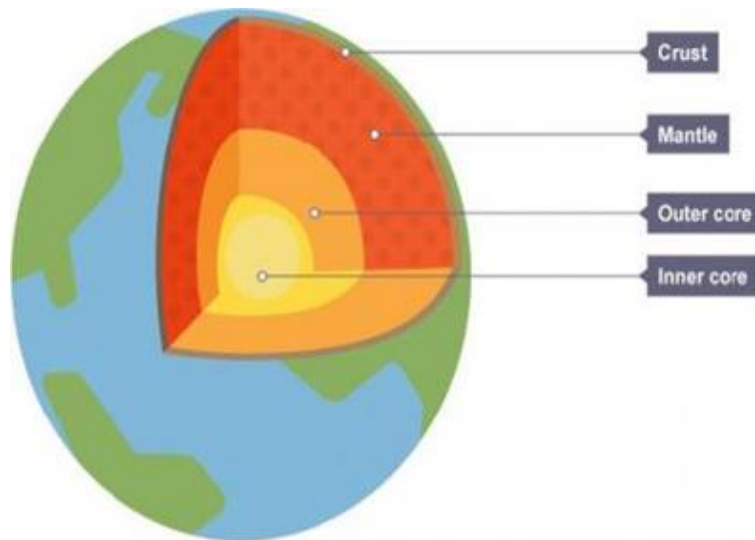


Figure 1.1: Earth and its interiors

The uppermost layer called the crust is the outermost solid shell of the planet, its thickness varies from 5 to 70 km. The discontinuity between the crust and the mantle was first discovered by Mohorovicic through observing a sharp change in the velocity of seismic waves passing the mantle to the crust. This discontinuity is thus known as Mohorovicic discontinuity ("M Discontinuity"). The average seismic wave velocity (P-Wave) within the crust ranges from 4-8 Km s⁻¹. The oceanic crust is relatively thin (5-15km) while the crust beneath the mountains is relatively thick. Based on this principle mantle is considered to consist of an upper layer that is fairly rigid. The upper layer along with the crust of thickness ~ 120 km is known as the lithosphere. Straightaway below this zone is called the asthenosphere, which extends for another 200 km. This zone is considered to be composed of molten rock and highly plastic ~ 2900 km. Because of this plastic character, the variation of seismic wave velocity is less, indicating that the mass is nearly homogeneous. The movement in the plates is different in both magnitude and direction. This provides the basis of the foundation of the theory of tectonic

earthquakes. Below the mantle is the outer core then the inner core in which it was observed that only P waves pass through the core region, while both P and S waves can pass through the mantle. The inner core is very dense and is assumed to consist of metals such as nickel and iron (thickness ~ 1290 km). Surrounding that is a layer of similar density (Outer core thickness ~ 2200 km) which is considered to be a liquid where S waves cannot pass through it.

1.3 PLATE TECTONICS

Plate tectonics could also be a theory describing the large-scale motion of the plates within the earth's lithosphere. The drift apart of plates is due to the flow of hot mantle upwards to the surface of the earth. The continental motions are related to a spread of circulation patterns. There are seven such major tectonics plates.

They move in several directions and at varying speeds. In general, there are three sorts of inter-plate interactions giving rise to 3 sorts of boundaries: convergent, divergent, and transform boundaries.

Earthquakes that are formed due to the faults at the plate boundaries are known as inter-plate earthquakes. Many earthquakes also occur within the plate away from the faults; these types of earthquakes are known as intraplate earthquakes, in which a sudden release of energy takes place due to the mutual slip of the rock beds, resulting in the formation of earthquake faults. In these faults, two types of slippages are observed: dip-slip, and strike-slip. Dip-slip takes place in the vertical direction whereas, strike-slip takes place in the horizontal direction, as shown in figure 1.2.

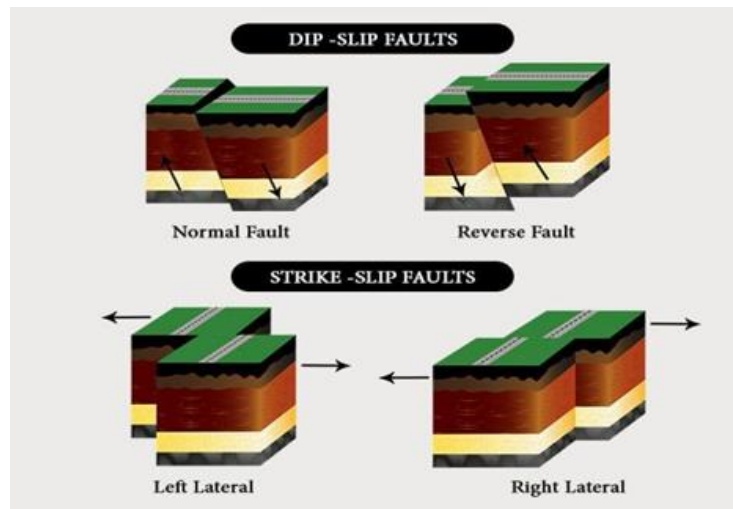


Figure 1.2: Type of Earthquake faults

1.4 CAUSES OF EARTHQUAKES

The movement of tectonic plates associated with one another, both in direction and magnitude results in the buildup of strain at the plate boundaries. When the strain energy reaches its limiting value along a weak region or at plate boundaries, a sudden movement (slip) occurs releasing the accumulated strain energy. These elastic potential energy waves are called seismic waves, which transmit energy from one point of the world to a different through different layers and eventually carry the energy to the surface causing destruction. Within the earth, the elastic waves propagate through an isotropic and homogeneous medium, which are known as body waves. On the surface, this energy waves propagate as surface waves.

The body waves are of two types: P- waves and S- waves.

- P Wave – Primary wave or Longitudinal waves, they can pass through the central core and mantle, they are waves in which the direction of particle motion is in the same or opposite direction to that wave propagation.
- S wave – Secondary wave or Transverse wave, can pass only through the mantle, waves in which the direction of particle motion is at right angles to the direction of wave propagation.

Surface waves - Waves that propagate on the earth's surface. Classifications are:

- L wave– Also known as love waves, particle motion takes place in the horizontal plane only and it is transverse/perpendicular to the direction of propagation.
- R wave- also known as Rayleigh waves, the particle motion is always in a vertical plane and traces an elliptical path, which is retrograde to the direction of wave propagation.

L waves travel faster than R waves.

Apart from tectonic earthquakes, volcanic activities, a sudden collapse of the reservoir are the additional causes of earthquakes.

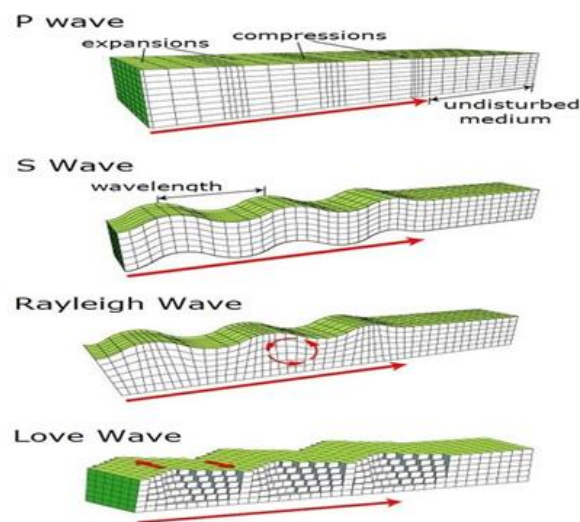


Figure 1.3: Type of seismic waves

1.5 EARTHQUAKE PARAMETERS

Seismic Intensity parameters refer to the quantities by which the size and energy of an earthquake is described. There are many earthquake measurement parameters, some are indirectly derived from the measured one. The most important parameters are the Magnitude and Intensity of an earthquake. Various types of waves propagating scales have been proposed after the use of Richter's magnitude scale. They are:

- Local Magnitude(M_L)

The local magnitude is defined as the logarithm of the maximum amplitude that is obtained from the record of a seismic event using a Wood -Anderson torsional seismograph located 100Km from the epicenter of the earthquake. Seismograph must have a natural period of 0.8s.

- Body Wave Magnitude(M_b)

A Long period-instrument is used to determine the body wave magnitude for periods from 5-15s, mainly used for the largest body waves LP, PP.

- Surface wave magnitude (M_s)

Surface wave magnitude is based on Rayleigh waves that travel primarily along the uppermost layer of the earth with a period of 20 seconds.

- Seismic Moment magnitude (M_w)

Seismographs are used to measure Seismic Moment, using long-period waves. Up to a value of six, the Magnitude of $M = M_L = M_s = M_b = M_w$ will be the same. However, beyond the value of 6, it is better to specify the type of magnitude.

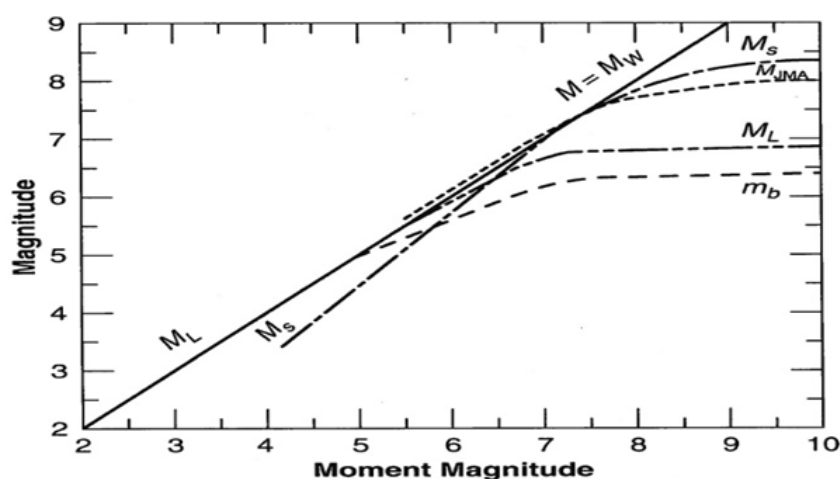


Figure 1.4: Earthquake Moment magnitudes

1.6 SEISMIC INPUT FOR STRUCTURES

Seismic inputs are the earthquake data required to perform seismic analysis. These data are presented in two different ways, deterministic and probabilistic forms. Seismic inputs in the deterministic form are used for deterministic analysis and design of structures whereas, the probabilistic form is used for various vibration analysis of structures for earthquake forces, seismic risk analysis of buildings or structures, and estimated damage of structures due to future earthquakes. Seismic inputs for structural analysis are provided in terms of time domain or in the frequency domain. They include magnitude, intensity, peak ground acceleration/velocity/displacement, duration, and predominant ground frequency.

1.7 TIME HISTORY RECORDS

Time history record is the most common way to describe a ground motion. The motion parameters adopted in this methodology are acceleration, velocity, displacement, or all three combined. The measured time histories of records include errors resulting from many sources, such as noises at high and low frequencies, baseline error, and instrumental error. These errors are withdrawn from data before they are used. The measured data are in an analog form which is digitalized before they are used as seismic inputs. At any measuring station, the ground motions are recorded in three orthogonal directions; two of them are in a horizontal direction and the third in a vertical direction. For structural analysis, these three components of ground motions are transformed into principal directions. The components of ground motion in the principal directions are uncorrelated. Based on this concept, three principle components of ground motions are artificially generated in places where representative time histories of the ground motions are not available. Three components of ground motion for the EL Centro earthquake are shown in figure 1.5.

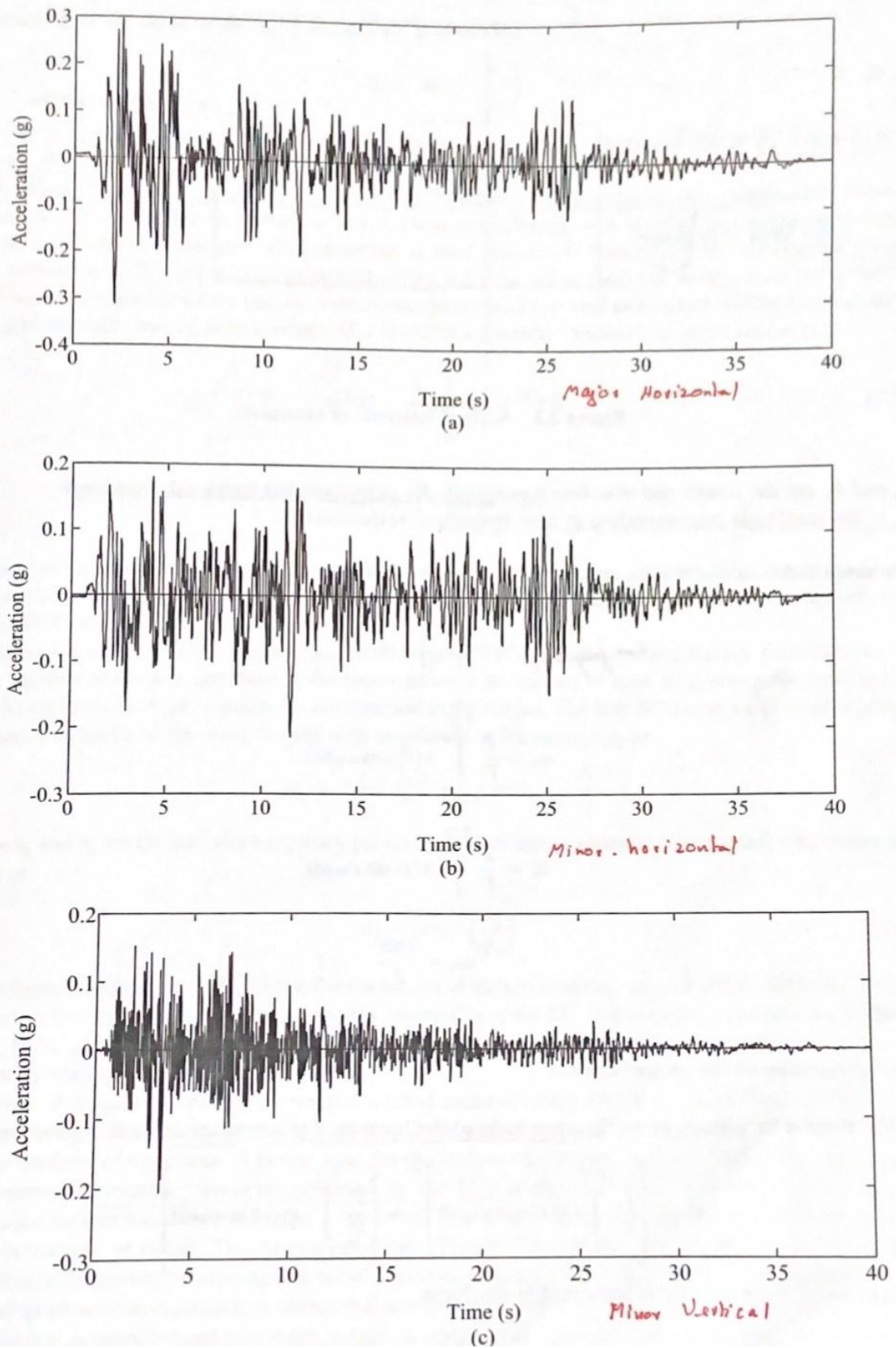


Figure 1.5: Components of ground motion for the EL Centro earthquake

1.8 RESPONSE SPECTRUM OF AN EARTHQUAKE

A plot of the peak value of a response quantity as a function of the natural vibration period (T_n) of the system, that are forced into motion by the same base vibration is called the response spectrum of that quantity. Each plot is for Single Degree of Freedom (SDOF) systems having a fixed damping ratio (ζ) and several such plots for different values of ζ are included to cover the range of damping values which is experienced in actual structures. Engineers prefer to use natural period instead of natural frequency because the period of vibration is a more familiar concept. Several response spectra are defined for denoting the ground motions, such as displacement response spectrum, pseudo velocity response spectrum, absolute acceleration response spectrum, and energy spectrum. A response spectrum is visualized as a graphical representation of a series of long cantilever pendulums with increasing natural periods subjected to a common lateral seismic motion of the base.

CHAPTER-2

REVIEW OF LITERATURE AND OBJECTIVES

2.1 GENERAL

The main purpose of literature review is to give an idea about the research work conducted in the world. This forms the basis on which one can carry out the work and techniques that can be used for designing a building. The following literature review elaborates the research studies based on seismic resistance buildings in IS provisions and ASCE provisions.

2.2 REVIEW OF LITERATURE

- (1) **T. Paulay** et.al. (1992), focuses on the teaching of structural engineering, design of structures and research to the behaviour of reinforced concrete and masonry buildings. This book helped us to gain an understanding of seismic design principles and its practice. This book presented us with basic concepts of seismic design, causes and effects of earthquakes as well as structural response, seismic actions to be considered in design. Numerous illustrated applications and examples of deterministic and probabilistic method of seismic design has been introduced and helped us gain perspective of DDBD.
- (2) **Joseph E. Bowles** et.al. (1997) Fifth edition, this book provided us with the state of art (SOA) and state of practice (SOP) methods in foundation engineering. Design methods tend to vary between geographic regions and “design absolutes” is not possible under construction procedure, the book helped us gain computer method for the modelling of soil mass in SAP 2000 and taught us soil structure interaction modelling procedure.
- (3) **M.J.N Priestley** et.al. (2008), have studied the seismic attack perceived in terms of simple mass-proportional lateral forces, resisted by elastic structural action. In the 1940's and 1950's the influence of structural period in modifying the intensity of the inertia forces started to be included into structural design, but structural analysis was completely based on elastic structural response. It was concluded that strength was important to reduce

top lateral displacements or strains which can be directly related to damage potential. This realization has led to the development of large number of different seismic design philosophies based on deformation capacity. They are termed as performance based design philosophies, this book gave us the complete format of Design of Direct Displacement method (DDBD).

- (4) **Charles E. Reynolds** et.al. (2009) Eleventh edition, we have taken this handbook for the developments in design and practice. The Euro-code revision codes have done significant changes in the combination of Earthquake load (EL), and the characteristics yield strength of reinforcement has been aided in this handbook for the manual calculations of DDBD method.
- (5) **Andrew W. Charleson** et.al. (2010), taught us the concept about the prevention of collapses and failures of buildings during an earthquake. The book lectures the material integrate concept of earthquake design with existing concepts of architectural design and provide strategy to position earthquake design concepts to existing college courses of engineering and architecture students. The National Programme on Earthquake Engineering Education and the National Information Centre of Earthquake Engineering place their appreciation for the contribution of Andrew W. Charleson and C.V.R Murty for their milestone completion for the emphasis of deformed shapes of buildings to both faculty and students.
- (6) **T.K Datta** et.al. (2010), in this book for structural engineering we got to know broadly about the details of seismology (including ground effects), seismic analysis and seismic design. This book attempts to provide teaching of seismic analysis of structures, and also offers comprehensive treatment of all aspects of seismic analysis of structures. We learned about the seismic input for structures and response analysis for specified ground motions which deals with single degree and multi degrees of freedom, it taught us the seismic reliability analysis of structures and the performance

of model response spectrum which has been used in SAP 2000 for our project.

- (7) **Prof. R.N. Iyengar** et.al.,(2010) have investigated about the Earthquake Capacity of Buildings ,its Elastic Behavior , inelastic behavior as well as Mapping the seismic hazard. From this book we came to know about the tectonic setting and Seismogenic Zones.
- (8) **C.V.R Murty** et.al. (2015), explains the concept in behaviour of buildings during an earthquake. This book gave us knowledge about the basic concepts of earthquake resistant design of buildings. In this book the discussion is limited to normal buildings without any special devices like base isolation and other energy absorbing or dissipating devices and exaggerates the deformation shape to emphasis the deformation of the building and to develop the most needed intuition of structural behaviour of buildings during earthquakes and its consequences on earthquake-resistant design. The book mainly targets the audience of practicing seismic structural engineers and architects, in addition to students and teachers of engineering and architecture colleges to understand seismic behaviour, analysis and design of buildings.
- (9) **IS 1893: 2016 Part -1 General Provisions and Buildings**.Have introduced the concept performance based design, which focuses on the displacement based analysis and Static analysis method. By this code design of seismic force by static analysis method is carried out, with this method we have determined the shear force induced to the frames of each floors respectively using manual calculations, using tables and Indian code provisions.

2.3 OBJECTIVES

- ◆ To improve the seismic safety of an existing structure.
- ◆ Comparing longitudinal beam reinforcement of existing plan with proposed design by using static analysis, direct displacement based design, and modal response spectrum analysis.

CHAPTER-3

METHODOLOGY

3.1 METHODOLOGY

To achieve the objective of the study, the following methodology has been adopted in this project as shown in figure 3.1.

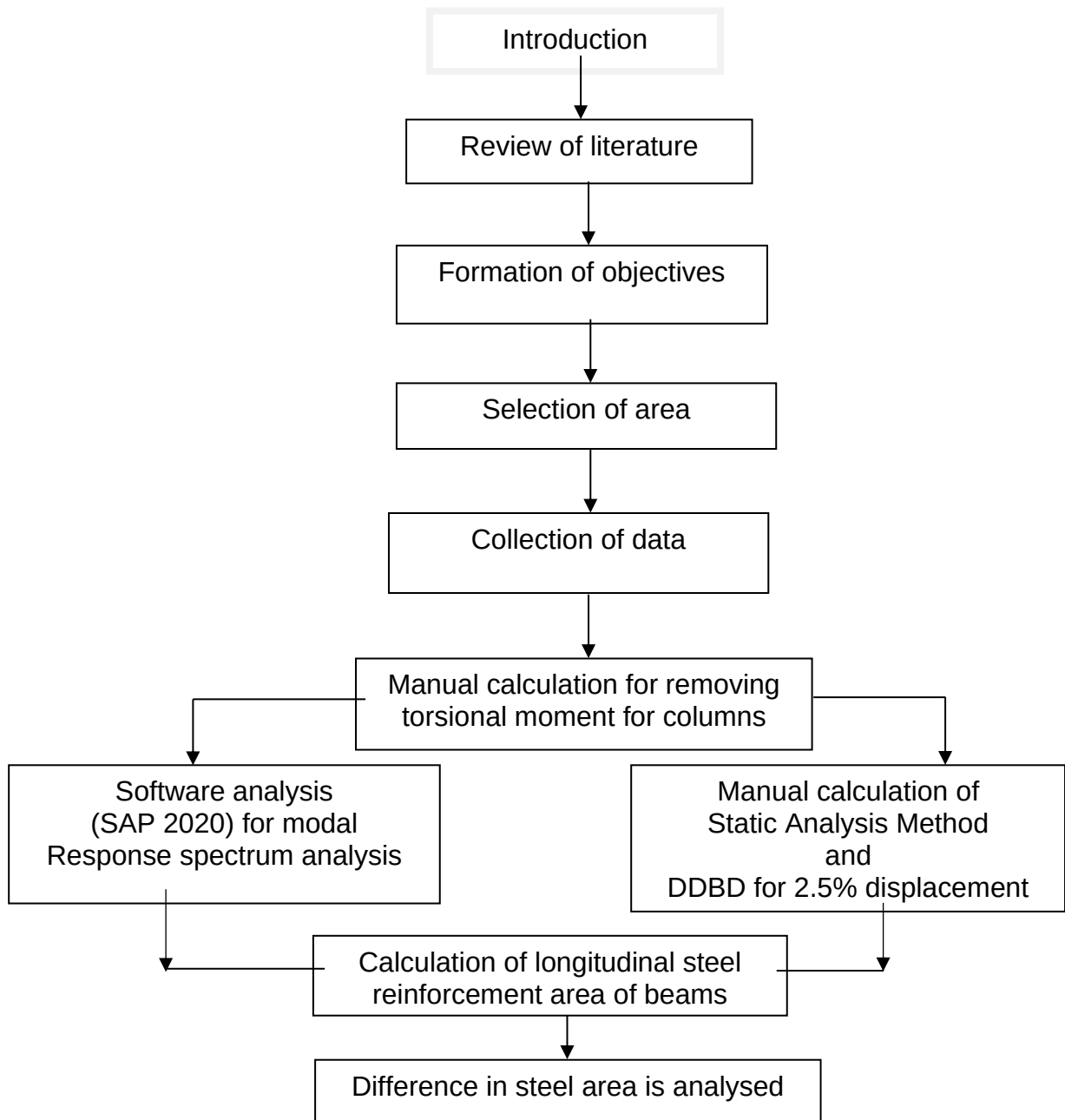


Figure 3.1 Methodology Chart

3.2 STUDY AREA AND DATA COLLECTION

Sathyabama is a university located in Sholinganallur, which is in the eastern part of Chennai. The admin block is located in the campus of four-hundred acres on Rajiv Gandhi salai. By the collection of architectural plan of the admin block (Figure 3.2) and with the help of our guide, we got the value of beam and column design. The dimensions of the main beams in X and Y directions are 9m of thickness 75cm, 3.7m of thickness 45cm, 6m of thickness 50cm respectively, whereas the dimension of column is 300 x 450 mm in all places. The foundation used in the admin block is double-under-reamed pile. The equivalent reinforcement steel of the beam used is 8 numbers of 25 mm ϕ bars in 9 m span beam and 5 numbers of 20 mm ϕ bars in 5.7 m beams.

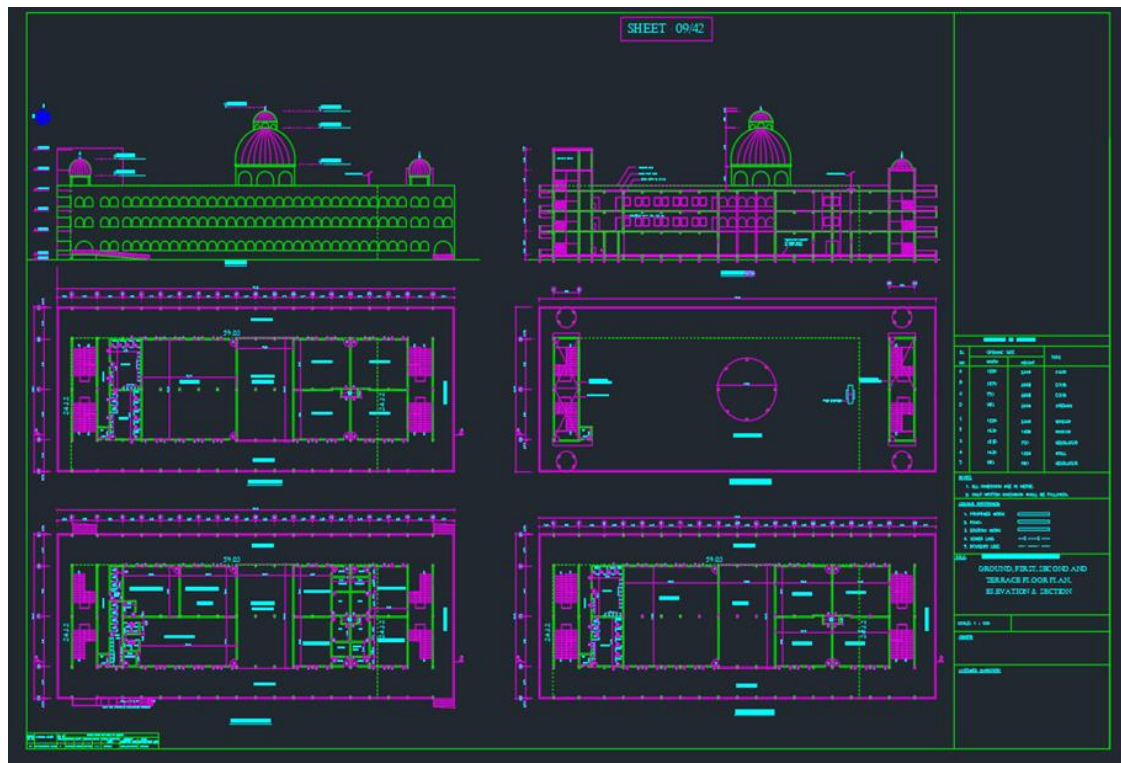


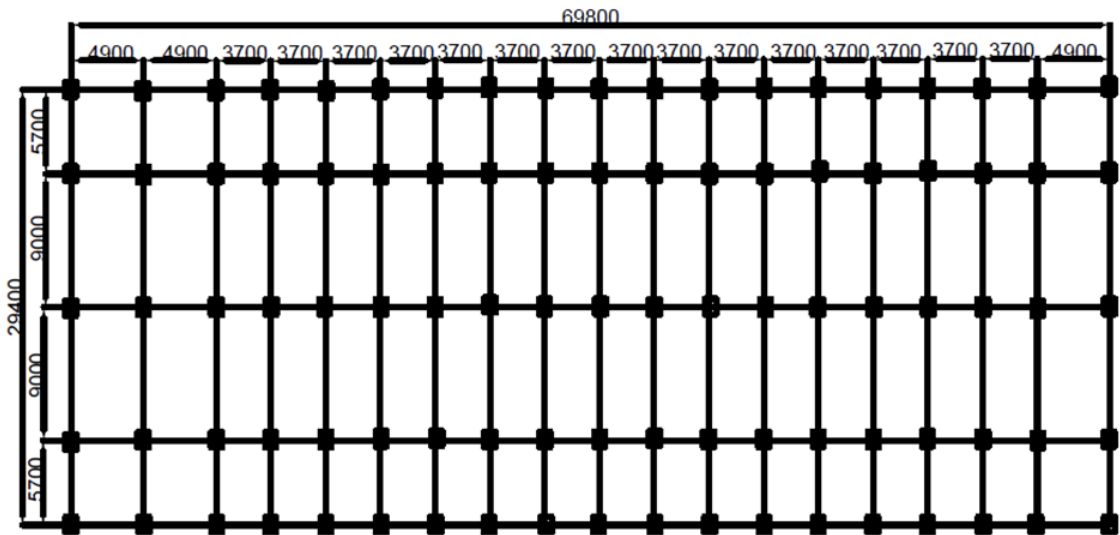
Figure 3.2 Existing Admin Block Plan of Sathyabama Institute of Science and Technology

The structural geometry of the existing admin block is tabulated in table 3.1

Table 3.1 Structural Geometry of the existing admin block plan of Sathyabama Institute of Science and Technology

X-Direction (m)	Y-Direction(m)	Z-Direction(m)
4.9,4.9, 3.7,3.7,3.7,3.7,3.7,3.7, 3.7,3.7,3.7, 3.7,3.7,3.7,3.7,3.7,3.7,4.9	5.7, 9, 9, 5.7	4.3, 3.7, 3.7

With the dimensions of the existing architectural plan, a column layout of the ground floor is taken and drawn in AutoCAD software as shown in figure 3.3.



Existing Key Plan(All Dimensions are in mm)

Figure 3.3 Existing Key Plan of Sathyabama Institute of Science and Technology

With the above collected details the stiffness centre, mass centre of both X and Y axis are calculated using the stiffness(k) formulas which is shown in the next chapter, eccentricity(ϵ) is also found out.

To tackle the torsion moment of the frame structure, a proposed design of beams and columns are introduced. With manual calculations the stiffness centre, mass centre of both X and Y axis are calculated again till the value of eccentricity reaches zero. Stating the torsional moment is nullified. A proposed key plan is done in AutoCAD and mentioned in figure 3.4.

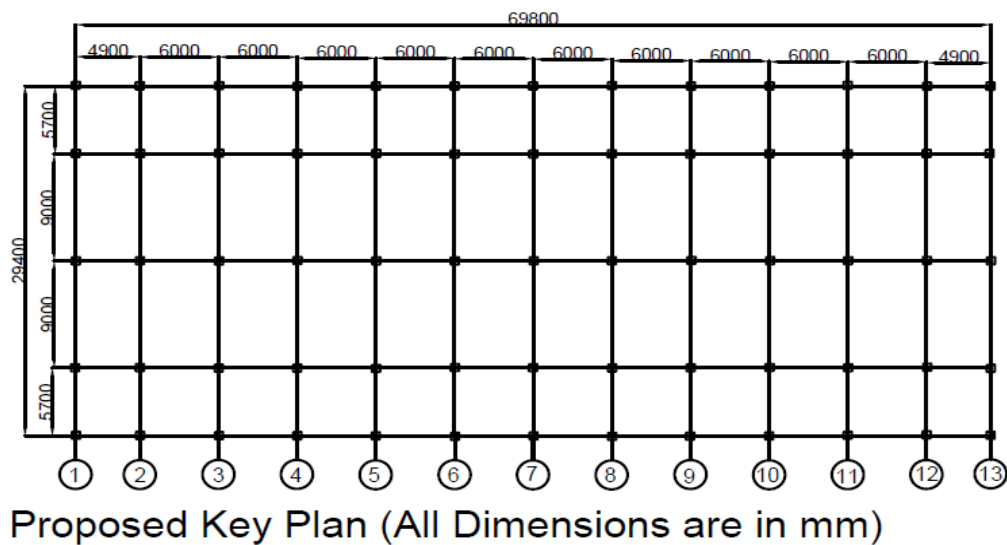


Figure 3.4 Proposed Admin Block Key Plan of SIST

The structural geometry of the proposed admin block is tabulated in table 3.2:

Table 3.2 Structural Geometry for the proposed Admin block plan of SIST

X-Direction (m)	Y-Direction(m)	Z-Direction(m)
4.9, 6, 6, 6, 6, 6, 6, 6, 6, 6, 6, 6, 4.9	5.7, 9, 9, 5.7	4.3, 3.7, 3.7

And the dimensions of the proposed column is 600 x 600. After the calculation of Mass of stiffness and mass centre, the eccentricity is made to be zero, which means tackling the torsional moment.

3.3 EXPLANATION ON MANUAL CALCULATIONS

Method 1 Static Analysis Method (IS 1893:2016) - From the above details, through IS provisions the seismic zone of the building is found out. With this the minimum design earthquake horizontal lateral force for which all the buildings should be designed is found out. (Clause 7.5.1) IS 1893 2016 part 1 as shown in the table 3.3

Table 3.3. Minimum design earthquake horizontal lateral force for which all buildings should be designed

Seismic Zone	ρ (%)
II	0.7
III	1.1
IV	1.6
V	2.4

The type of soil is considered as Type III in IS provisions (Clause 6.3.5.1) 1893: 2016 part-I as mentioned in the table 3.4

Table 3.4-Type of soils

Soil Type	Remarks
I- Rock or Hard Soils	Well graded gravel (GW) or well graded sand (SW) both with less than 5 percent passing 75 μ m sieve (Fines). Poorly graded sand (SP) or clayey sand (SC), all having N above 30. Stiff to hard clays having N above 16, where N is Standard Penetration Test value.

II- Medium or stiff soils	Poorly graded sands or poorly graded sands with gravel (SP) with little or no fines having N between 10 and 30. Stiff to medium stiff fine- grained soils, like silts of low compressibility(ML) or Clays of low Compressibility (CL) having N between 10 and 16
III- Soft Soils	All soft soils other than SP with $N < 10$. The various possible soils are Silts of intermediate compressibility(MI); Silts of High compressibility(MH) Clays of intermediate compressibility(CI); Clays of High compressibility(CH); Silts and Clays of Intermediate to high compressibility (MI-MH or CI-CH) Slit with clay of intermediate compressibility (MI-CI);and Slit with clay of high compressibility(MH-CH)
IV- Unstable, Collapsible, Liquefiable soils	Requires site specific study

After finding the seismic zone, seismic zone factor Z is found out by (Clause 6.4.2) Table 2 in IS 1893: 2016 Part I as mentioned in the table 3.5.

Table 3.5-Seismic Zone Factor (Z)

Seismic Zone Factor	II	III	IV	V
Z	0.10	0.16	0.24	0.36

Then importance factor I is found out from (Clause 6.4.2) Table 3 in IS 1893: 2016 Part I mentioned in the table 3.6.

Table 3.6 -Importance Factor (I)of various structures

SI No	Structure	I
1	Important service and community buildings or structures(for example, critical governance buildings, schools), signature structures, monument structures, lifeline and emergency structures (for Example, hospitals, telephone exchanges, television stations, radio stations, bus stations, metro rail structure and metro rail stations, railway stations, airports, water main lines and water tanks, food chain structures, fuel stations, electricity stations, fire stations, and bridges), and large community halls (for example, cinema halls, shopping malls, assembly halls and subway stations)	1.5
2	Residential or commercial buildings or structures with occupancy more than 200 persons	1.2
3	All other buildings or structures.	1.0

Response reduction factor R for different building systems is given in table 6 IS 1893: 2016 Part I. The value of R is used for design of buildings with lateral load resisting elements. The value R is taken as 5.0 as shown in Table 3.7.

Table 3.7- Response Reduction factor (R) for Building Systems

SL.NO	Lateral Load Resisting System	R
	Moment Frame systems	
1.	RC Buildings with Ordinary Moment Resisting Frame (OMRF)	3.0
2.	RC Buildings with Special Moment Resisting Frame (SMRF)	5.0
3.	Steel Buildings with Ordinary Moment Resisting Frame (OMRF)	4.0
4.	Steel Buildings with Special Moment Resisting Frame (SMRF)	5.0
	Braced Frame Systems	
5.	Steel Buildings with ordinary Braced frame with concentric Braces	4.0
6.	Steel Buildings with Special Braced frame with concentric Braces	4.5
7.	Steel Buildings with ordinary Braced frame with eccentric Braces	4.5
8.	Steel Buildings with Special Braced frame with eccentric Braces	
	Structural Wall Systems	
9.	Load Bearing Buildings with Masonry wall	
	● Unreinforced Masonry(designed as per IS 1905) without horizontal RC Seismic Bands	1.5
	● Unreinforced Masonry(designed as per IS 1905) with horizontal RC Seismic Bands	2.0
	● Unreinforced Masonry(designed as per IS 1905) with horizontal RC Seismic Bands and vertical reinforcing bars at corners of rooms and jambs of opening(with reinforcement as per IS 4326)	2.5

	● Reinforced Masonry	3.0
	● Confined Masonry	3.0
10.	Load Bearing Buildings with Ordinary RC Structural Walls	3.0
11.	Load Bearing Buildings with Ductile RC Structural Walls	4.0
	Dual Systems	
12.	RC Buildings with Ordinary RC structural wall with RC OMRF	3.0
13.	RC Buildings with Ordinary RC structural wall with RC SMRF	4.0
14.	RC Buildings with Ductile RC structural wall with RC OMRF	4.0
15.	RC Buildings with Ductile RC structural wall with RC SMRF	5.0

With these parameters and by using Design Seismic force by Static Analysis method, the base shear is tabulated and calculated out and distributed with height respectively, by using IS provisions. Also SAP modal analysis of un-cracked section is done and the beam longitudinal reinforcement is computed.

Method 2- Direct Displacement Based Design (DDBD)(ASCE STANDARD 7-10) - explains the complete procedure for the seismic design of buildings whose primary lateral force resisting systems is comprised of frames. The initial stage of the design process is the representation of the multi-degree-of-freedom (MDOF) structure by an equivalent single degree of freedom (SDOF) structure modelling the first inelastic mode of response (figure 3.5).

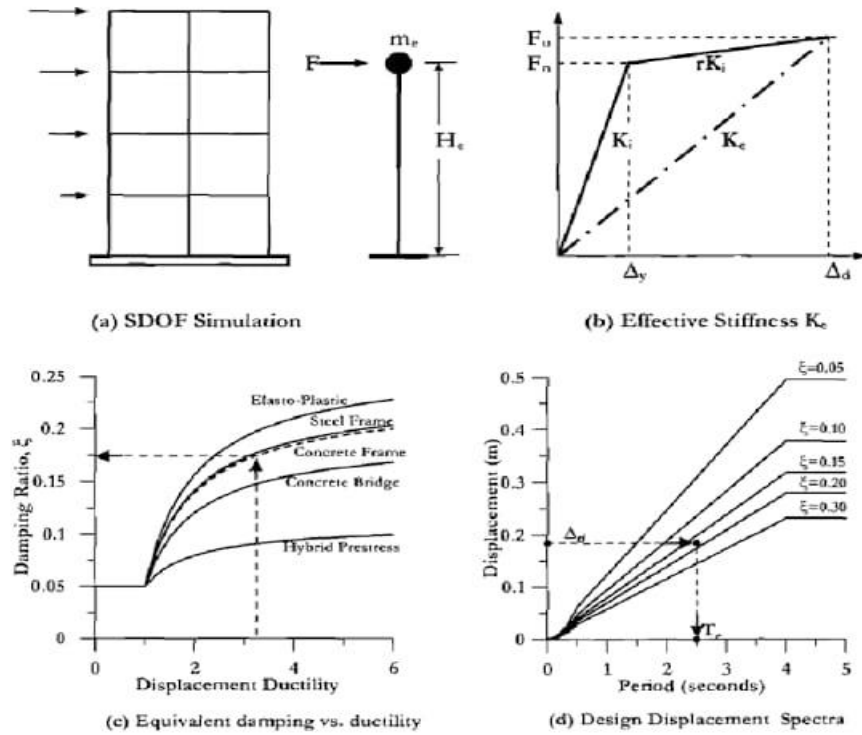


Figure 3.5- Fundamental Relationships of direct displacement based design

From the inelastic mode of response, following equations were developed

- (a) Design Storey Displacements: The design floor displacements of the frame are related to a normalized inelastic mode shape δ_i , where $i=1$ to n (the storey levels), and to the displacement Δ_c of the critical storey by the relationship

$$\Delta_i = \delta_i (\Delta_c / \delta_c) \Rightarrow \Delta_i = 0.025 \times H_i.$$

Where H_i is the height of the required floor level.

The normalized inelastic mode shape depends on the height, H_i and the roof height H_n to the relationship:

$$\text{For } n \leq 4: \quad \delta_i = H_i / H_n$$

$$\text{For } n > 4: \quad \delta_i = 4/3 H_i / H_n. (1 - H_i / 4H_n)$$

Equivalent SDOF Design Displacement (Δ_d): The equivalent design displacement is related to the storey displacements by the relationship:

$$\Delta_d = \frac{\sum_{i=1}^n m_i \Delta_i^2}{\sum_{i=1}^n m_i \Delta_i}$$

Where m = Mass in tonnes of the frame structure.

Where m_i is the mass at height H_i associated with displacement Δ_i .

Equivalent Mass (m_e): The equivalent SDOF mass m_e is given by:

$$m_e = \frac{\sum_{i=1}^n m_i \Delta_i}{\Delta_d}$$

Effective Height (H_e): The effective height H_e of the SDOF structure is given by:

$$H_e = \frac{\sum_{i=1}^n m_i \Delta_i H_i}{\sum_{i=1}^n m_i \Delta_i}$$

Design Displacement Ductility (μ): The SDOF design displacement ductility factor is related to the equivalent yield displacement (Δ_y) by:

$$\mu = \Delta_d / \Delta_y$$

The yield drift of a storey in a frame depended on geometry, and was independent of strength. The following expressions were developed:

For Reinforced concrete frames: $\theta_y = 0.5 \epsilon_y (L_b / h_b)$

For Steel Frames : $\theta_y = 0.65 \epsilon_y (L_b / h_b)$

Where L_b and h_b are the span between column centrelines, and overall beam depth respectively, and ϵ_y is the yield strength of the flexural reinforcement or structural steel. It is generally sufficiently accurate to assume a linear yield displacement profile for the purpose of estimating ductility demand, and hence the yield displacement is given by:

$$\Delta_y = \theta_y \cdot H_e$$

Equivalent Viscous Damping (ζ): The equivalent viscous damping of the substitute structure for frames can be conservatively related to the design displacement ductility demand as follows:

Reinforced concrete frames (ζ_{eq}) = $0.05 + 0.565 \cdot \left(\frac{\mu - 1}{\mu \pi} \right)$

Effective period of substitute structure (T_e): The effective period at peak displacement response is found from the displacement spectra set, entering with the design displacement and determining the period, T_e , corresponding to the calculated viscous damping.

Magnitude of the earthquake is considered as 7.5 M_W , and Corner period T_c is calculated by:

$$T_e = 1.0 + 2.5(M_W - 5.7) \text{ seconds}$$

Peak Response displacement δ_{max} is calculated in terms of mm

$$\delta_{max} = C_s \cdot 10^{(M_W - 3.2)} / r, \text{ (For 5\% damping)}$$

Considering the C_s value as 1.4 (Intermediate soil)

The spectral displacement reduction factor Equation (2.8)

$$R_e = \left(\frac{0.07}{0.02 + \zeta} \right)^\alpha, \text{ where } \alpha = 0.5 \text{ (seismic coefficient)}$$

Effective stiffness of substitute structure (K_e): The effective stiffness at maximum displacement response of the substitute structure, is given by:

$$K_e = 4\pi^2 m_e / T_e^2$$

Design Base Shear force (F): The design base shear force for the MDOF structure is found from the substitute structure:

$$F = V_{Base} = K_e \cdot \Delta_d$$

Storey Shear Force and Overturning Moments: Storey shear forces are found by summing the floor forces above the storey is considered. Storey overturning moments at the floor levels are also found from the lateral forces as

$$OTM_i = \sum_{j=i}^{12} F_j (H_j - H_i)$$

Column Base Moments: The total resisting moment provided at the column base will thus be

$$\Sigma M_{cj} = V_{Base} \times 0.65H_1 = 1190 \times 0.65 \times 4.5 = 3481 \text{ kNm}$$

$$V_{Base} = m_i \Delta_i$$

Beam Seismic Shears in an End bay corresponding to Design Lateral Forces: The outer bay shear force (V_{Bi})

$$\Sigma_{i=1}^n V_{Bi} = T = \frac{M_1}{2M_1 + 2M_2} \cdot (\Sigma_{i=1}^n F_i \cdot H_i - \Sigma_{j=1}^m M_{cj}) / L_1$$

The tensile force will be distributed to the beams of the end bay in proportion to the storey heights

$$V_{Bi} = T \cdot (V_{Si} / \Sigma_{i=1}^n V_{Si})$$

$$V_{Si} = F_i + F_{i+1}$$

The beam seismic moments in an end bay (M_{Bi}) ; design seismic moments for the end bay corresponding to the design lateral forces are calculated at the column phase with the column dimension (L_1) and the clear span between columns in the end bay (h_c). The average seismic moment will be

$$M_{Bi} = V_{Bi} \cdot (L_1 - H_c) / 2$$

After calculating and tabulating the moment of the frame structure, by finding stiffness (k) and using EC 2 Design chart for doubly reinforced rectangular beam -1 in REYNOLDS CONCRETE DESIGNERS HANDBOOK 11th edition; the value of μ is found out by $M_u / bd^2 f_{ck}$, with this value μ^l is assumed to be 0.21 and the area of steel (A_s) is found out. $A_s = 0.2 \times bd \times f_{ck} / f_y$. After measuring the area of steel, the percentage area of rebars is calculated and compared with Static analysis method.

CHAPTER 4

CALCULATION AND ANALYSIS

Stiffness centre is also termed as a centre of rigidity. According to IS 1893-2002, it is the point at which the restoring resultant force of the system acts.

4.1 STIFFNESS CENTRE OF EXISTING PLAN OF SATHYABAMA ADMIN BLOCK:

Y-axis (300 x 450) (mm)

I = Moment of Inertia

b = breadth of the column

d = depth of the column

$$I = bd^3/12 = (300 \times (450^3))/12 = 2.28 \times 10^{10} \text{mm}^4$$

$$\text{Stiffness center } (\bar{x}) = (A_1x_1 + A_2x_2 + A_3x_3 + \dots + A_nx_n) / (C \times A_n)$$

Where,

$$A = n \times I$$

n = Number of columns in the row (5)

C = no of sections (19)

x = Distance from origin to the point

$$\bar{x} = ((19 \times (2.28 \times 10^{10}) \times 0) + (19 \times (2.28 \times 10^{10}) \times 5.7) + (19 \times (2.28 \times 10^{10}) \times 14.7) + (19 \times (2.28 \times 10^{10}) \times 23.7) + (19 \times (2.28 \times 10^{10}) \times 29.4))$$

$$(19 \times (2.28 \times 10^{10}) \times 5)$$

$$\bar{x} = 14.7 \text{m}$$

$$\text{Mass center (Y-axis)} = L/2 = 29.4/2 = 14.7 \text{ m}$$

Span of the plan (L) = 29.4m

e (eccentricity)= mass center – stiffness center = 0

X-axis (450 x 300) (mm)

$$I = bd^3/12 = (450 \times (300^3))/12 = 1.01 \times 10^{10} \text{mm}^4$$

$$\text{Stiffness center } (\bar{x}) = (A_1x_1 + A_2x_2 + A_3x_3 + \dots + A_nx_n) / (C \times A_n)$$

Where,

$$A = n \times I$$

n= Number of columns in the row (5)

C = no of sections (19)

x= Distance from origin to the point

Stiffness center $\bar{x}$ =

$$\begin{aligned} & ((5 \times (1.01 \times 10^{10}) \times 0) + (5 \times (1.01 \times 10^{10}) \times 4.9) + (5 \times (1.01 \times 10^{10}) \times 9.8) + (5 \times (1.01 \times 10^{10}) \times 13.4) \\ & 6) + (5 \times (1.01 \times 10^{10}) \times 17.12) + (5 \times (1.01 \times 10^{10}) \times 20.8) + (5 \times (1.01 \times 10^{10}) \times 24.4) + (5 \times (1.01 \times 10^{10}) \times 28.1) \\ & + (5 \times (1.01 \times 10^{10}) \times 31.76) + (5 \times (1.01 \times 10^{10}) \times 35.4) + (5 \times (1.01 \times 10^{10}) \times 39) + (5 \times (1.01 \times 10^{10}) \times 42.74) \\ & + (5 \times (1.01 \times 10^{10}) \times 46.4) + (5 \times (1.01 \times 10^{10}) \times 50) + (5 \times (1.01 \times 10^{10}) \times 53.7) + (5 \times (1.01 \times 10^{10}) \times 57.4) \\ & + (5 \times (1.01 \times 10^{10}) \times 61.04) + (5 \times (1.01 \times 10^{10}) \times 64.7) + (5 \times (1.01 \times 10^{10}) \times 69.5)) \end{aligned}$$

$$(5 \times (1.01 \times 10^{10}) \times 19)$$

$$\bar{x} = 35.27 \text{m}$$

$$\text{Mass center (X-axis)} = 69.8/2 = 34.9 \text{m}$$

$$e (\text{eccentricity}) = 0.37$$

4.2 STIFFNESS CENTRE OF PROPOSED PLAN OF SATHYABAMA ADMIN BLOCK:

Y-axis (600 x 600)

$$I = bd^3/12 = (600 \times (600^3))/12 = 1.08 \times 10^{10} \text{mm}^4$$

$$\text{Stiffness center } (\bar{x}) = (A_1x_1 + A_2x_2 + A_3x_3 + \dots + A_nx_n) / (C \times A_n)$$

Where,

$$A = n \times l$$

n= Number of columns in the row

x= Distance from origin to the point

C = no of sections (19)

Stiffness center ($\bar{x}$) =

$$\frac{((13 \times 1.08 \times 10^{10}) \times 0) + ((13 \times 1.08 \times 10^{10}) \times 5.7) + ((13 \times 1.08 \times 10^{10}) \times 14.7) + ((13 \times 1.08 \times 10^{10}) \times 23.7) + ((13 \times 1.08 \times 10^{10}) \times 29.4))}{((13 \times 1.08 \times 10^{10}) \times 5)}$$

$$\bar{x} = 14.694 \approx 14.7\text{m}$$

e (eccentricity)=0

X-axis

$$I = \frac{bd^3}{12} = \frac{(600 \times (600^3))}{12} = 1.08 \times 10^{10} \text{mm}^4$$

Stiffness center ($\bar{x}$)

$$\frac{((5 \times (1.08 \times 10^{10}) \times 0) + (5 \times (1.08 \times 10^{10}) \times 4.9) + (5 \times (1.08 \times 10^{10}) \times 10.9) + (5 \times (1.08 \times 10^{10}) \times 16.9) + (5 \times (1.08 \times 10^{10}) \times 22.9) + (5 \times (1.08 \times 10^{10}) \times 28.9) + (5 \times (1.08 \times 10^{10}) \times 34.9) + (5 \times (1.08 \times 10^{10}) \times 40.9) + (5 \times (1.08 \times 10^{10}) \times 46.9) + (5 \times (1.08 \times 10^{10}) \times 52.9) + (5 \times (1.08 \times 10^{10}) \times 58.4) + (5 \times (1.08 \times 10^{10}) \times 64.9) + (5 \times (1.08 \times 10^{10}) \times 69.8))}{(5 \times (1.08 \times 10^{10}) \times 13)}$$

$$\bar{x} = 34.9\text{m}$$

$$\text{Mass Centre (X-axis)} = 69.8/2 = 34.9\text{m}$$

e (eccentricity) =0

Proposed design of Sathyabama admin block is free from torsional moment as the stiffness centre and mass centre lies on the same point.

4.3 LOAD CALCULATION:

The materials used are: M40 Grade concrete and HYSD 500 rebars

Proposed Plan:

Load Calculation (Ground and First floor)

Slab (Dead weight) = $29.4 \times 6 \times 3.75 = 661.5 \text{ KN}$

Floor finish = $29.4 \times 6 \times 2 = 330.75 \text{ KN}$

Parapet wall = $0.23 \times 1.2 \times 20 = 5.5 \text{ KN/m}$

Brickwork (partition) = $0.23 \times 3.1 \times 20 = 14.5 \text{ KN}$

Light partition = $0.12 \times 3.1 \times 20 = 7.5 \text{ KN}$

Beam

X-axis (350 x 900) = $29.4 \times 0.35 \times 0.9 \times 25 = 231.5 \text{ KN}$

Y-axis (300 x 600) = $29.4 \times 0.3 \times 0.6 \times 25 = 132.3 \text{ KN}$

Column (600 x 600) = $0.6 \times 0.6 \times 5 \times 25 \times 3.7 = 166.5 \text{ KN}$

Dead load of the framed structure = $5.75 \times 6 = 43 \text{ KN/m}$

Total Dead Load of the floor = 2155.6 KN

Total dead weight of the floor = 2155.6 KN

Load per $\text{m}^2 = (2155.6 / (29.4 \times 6)) = 12.2 \text{ KN/m}^2$

Load per m = $12.2 \times 6 = 73.2 \text{ KN/m} \approx 73 \text{ KN/m}$

Live load = $4 \text{ KN/m}^2 \times 6 = 24 \text{ KN/m}$

50% = 12 KN/m (IS Provision for live load)

Total Live Load = $29.4 \times 12 = 352.8 \text{ KN}$

Total weight (W_1) = $352.8 + 2155.6 = 2508.4 \text{ KN}$

$W_1 = W_2 = 2508.4 \text{ KN/m}$ say 2500 KN

Load Calculation (Roof Level)

Slab (Dead weight) = $29.4 \times 6 \times 3.75 = 661.5 \text{ KN}$

Floor finish (Dead weight) = $29.4 \times 6 \times 2 = 352.8 \text{ KN}$

Beam

X-axis = $29.4 \times 0.35 \times 0.9 \times 25 = 231.5 \text{ KN}$

Y-axis = $5 \times 6 \times 0.3 \times 0.6 \times 25 = 135 \text{ KN}$

Column (600 x 600) = $5 \times 0.6 \times 0.6 \times 25 \times 3.7 = 166.5 \text{ KN}$

50% of Live load = 83.275 KN

Total Dead load/m² = 1464.6 KN

Live load = $1.5 \text{ KN/m}^2 = 1.5 \times (6 / 2) = 4.5 \text{ KN/m}$

Total live load = $4.5 \times 29.4 = 132.3 \text{ KN/m}$

Total weight (W_2) = $132.3 + 1464.6 = 1596.3 \approx 1600 \text{ KN}$

Add extra weight for water tank and Dome = $(500 \text{ KN} + 1600 \text{ KN}) = 2100 \text{ KN}$

4.4 METHOD 1:

DESIGN OF SEISMIC FORCE BY STATIC ANALYSIS METHOD

DESIGN PARAMETERS

Seismic zone	-	III (By using IS 1893:2016 tables)
Zone factor (Z)	-	0.16 (By using IS 1893:2016 tables)
Importance factor (I)	-	1.5 (By using IS 1893:2016 tables)
Response reduction factor (R)	-	5 (By using IS 1893:2016 tables)

Seismic weight calculation using lumped Mass (W)

Floors:

$W_1 = W_2$ (Ground and first) = 2500KN

W_3 (Roof) = 2100 KN

Total Seismic weight of the structure $\Sigma W_i = W_1 + W_2 + W_3 = 7100 \text{ KN}$

Storey level is represented as i

Lateral load distribution with height by static method (table 4.1)

h_i = Height of the storey level (m)

W_i = lumped mass of the floor with respect to storey level

Fundamental translational period (T_n): (clause 7.6.2 of IS 1893 Part 1)

EL in X-Direction:

$$T_n = 0.09h / (\sqrt{d}) = 0.19 \text{ sec}$$

Where

d = Base dimension = 29.4m

h = Height of the building = 11.7m

The building is located in type III (Soft soil) from figure 2 of IS: 1893

For $T_n = 0.19 \text{ sec}$

$S_a/g = 2.5$

$A_h = (Z/2R) \times (S_a/g) = 0.06$ (horizontal force)

(Clause 6.4.2 of IS 1893 Part 1)

Design Base Shear

$$V_B = A_h \Sigma W$$

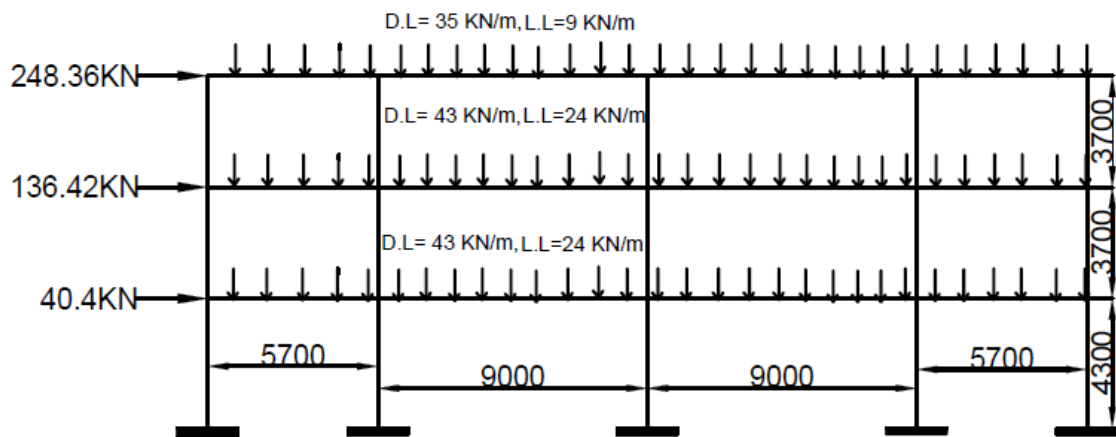
$$= 426 \text{ KN}$$

Lateral forces at i^{th} level for EL direction (KN) = $V_B \times (W_i h_i^2 / 1000) / \Sigma W_i h_i^2$

Table 4.1 lateral load distribution with height by static method

Storey level	$W_i(\text{KN})$	$h_i(\text{m})$	$W_i h_i^2 / 1000$	$(W_i h_i^2 / 1000) / \sum W_i h_i^2$	Lateral forces at i^{th} level for EL direction (KN)
3	2100	11.7	287.5	0.583	248.36
2	2500	8	160	0.32	136.32
1	2500	4.3	46.23	0.094	40.2
			$\Sigma = 493.73$	$\Sigma = 1$	$\Sigma = 425.3 \sim 426 \text{ KN}$

Frame configuration representing lateral force and loads are plotted in AutoCAD (figure 4.4)



Substitute Frame A6, B6, C6, D6, E6

Figure 4.1 Frame configuration representing Lateral force and loads

The above frame is analysed in SAP2000 software, from the software analysis the longitudinal beam reinforcement area of steel is found out. This area of steel is converted to number of 25 mm ϕ steel rebars.

The modal analysis result of SAP2000 is attached in figure (4.5)

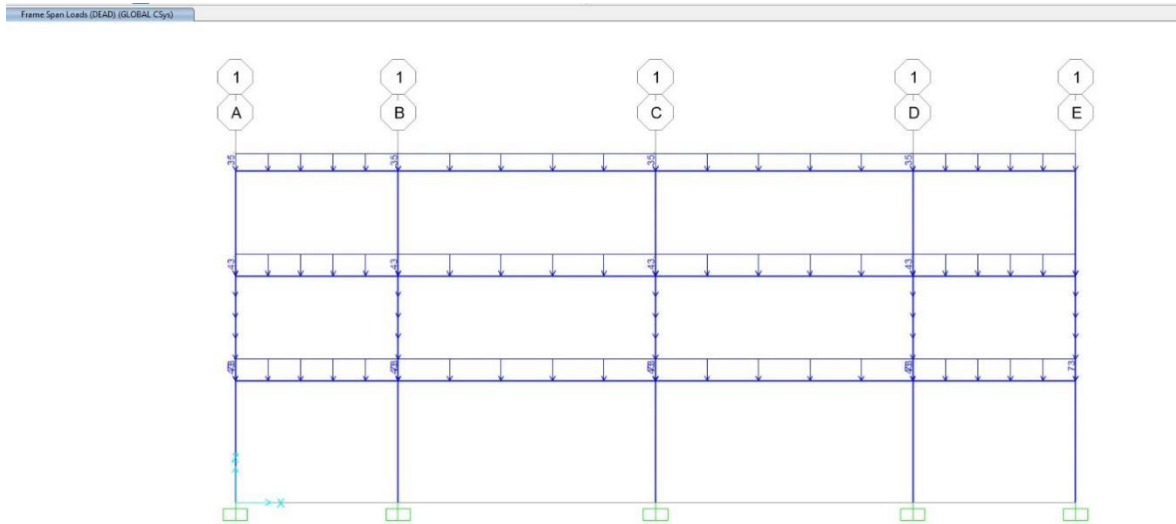


Figure 4.2 Assigned dead Load for the Frame

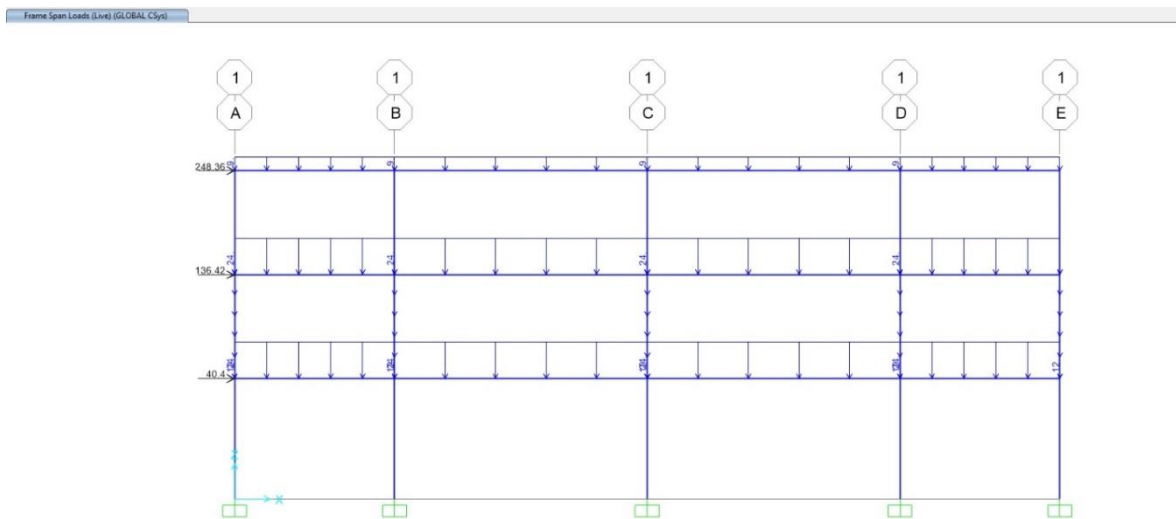


Figure 4.3 Assigned lateral force and live load for the Frame

After running analysis of the assigned loads, the deformed shape due to the combined effect of live, dead, lateral forces is shown in figure 4.7

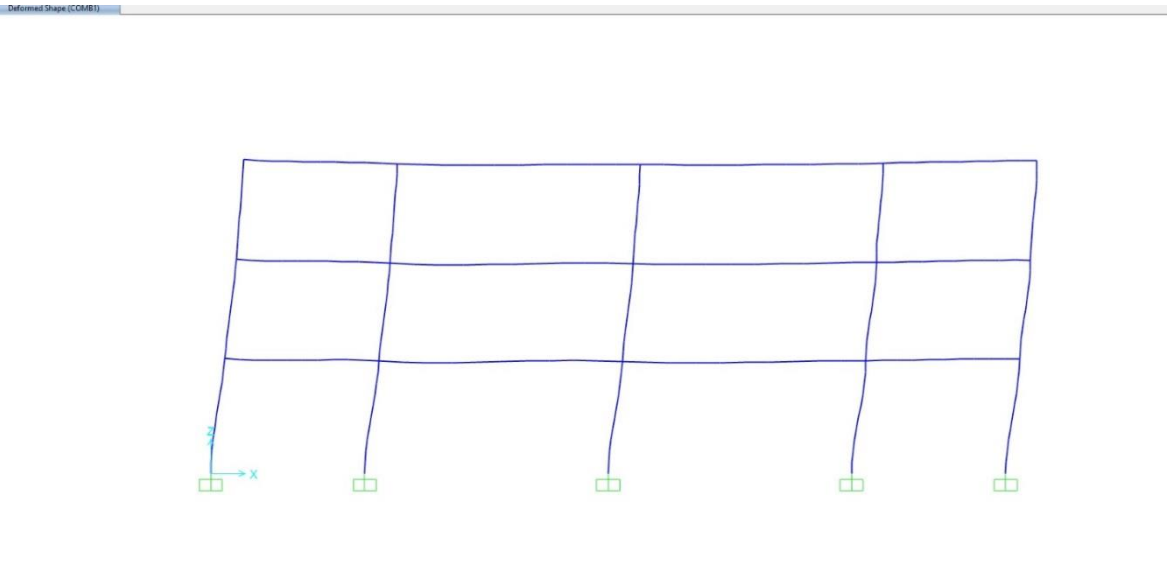


Figure 4.4 Resulted Deformed shape of the Frame

The concrete design output according to IS provision was done and the longitudinal reinforcement area of steel is found out in figure 4.8

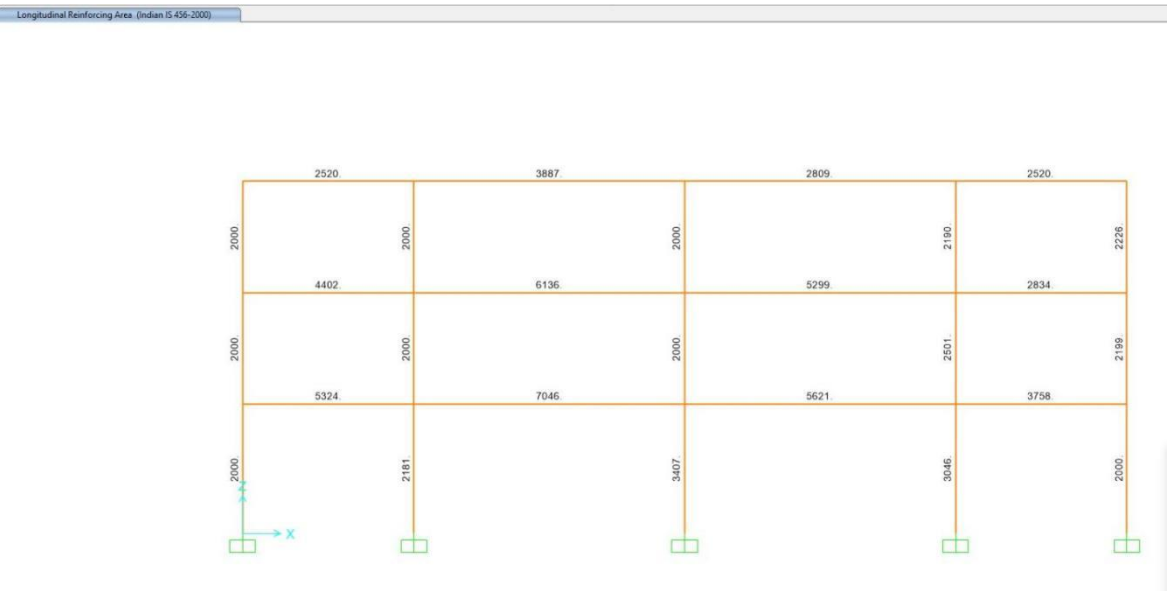


Figure 4.5 longitudinal reinforcement area of steel Frame

From the resulted output of steel reinforcement area, the reinforcement details of 9m and 5.7 m beam is taken and the number of 25 mm diameter of bars is calculated using the formula:

$$\text{No of steel bars} = A_{st} / 490.1$$

$$A_{st} = \text{area of steel } (\pi \cdot d^2 / 4 = 490.1 \text{ mm}^2),$$

$$d = 25 \text{ mm}$$

For 9 m length

$$A_{st} = 7046 \text{ mm}^2, 15 \text{ number of } \phi 25 \text{ mm bars.}$$

$$\text{Total Longitudinal steel area } A_s = N \times A_{st}$$

$$N = \text{Number of sections} = 13$$

$$A_s = 91598 \text{ mm}^2$$

For 5.7 m length

$$A_{st} = 5324 \text{ mm}^2, 11 \text{ number of } \phi 25 \text{ mm bars.}$$

$$A_s = 69212 \text{ mm}^2$$

4.5 Method 2 Direct Displacement Based Design

From the above calculated lumped mass (W), Calculation for design displacement, effective height is tabulate.

$$\text{Mass } (m_i) = W / 9.8 \text{ (conversion to tonnes)}$$

$$h_i = \text{height of the storey level}$$

Materials used

$$f_y = 500 \text{ N/mm (Fe 500 rebar)}$$

$$f_{ck} = 40 \text{ N/mm (M20 concrete)}$$

**Table-4.2.1- Lateral Load Distribution with height by Direct Displacement
Based Design Method**

Storey, i	Height (h _i)(m)	Mass, m _i (tonnes)	δ_i	Δ_i (m)	$m_i \Delta_i$	$m_i \Delta_i^2$	$m_i \Delta_i h_i$
3	11.7	214.3	1.0	0.29	62.15	18.02	727.6
2	8	255.1	0.68	0.2	51.02	10.20	408.2
1	4.3	255.1	0.37	0.1075	27.4	2.95	117.8
					$\Sigma=140.6$	$\Sigma=31.17$	$\Sigma=1253.6$

$$\Delta_i = 0.025 \times H_i.$$

$$\delta_i = \delta_i / \delta_{i-1}$$

Substitute structure design displacement:

$$\Delta_d = \frac{\sum_{i=1}^n (m_i \Delta_i^2)}{\sum_{i=1}^n (m_i \Delta_i)}$$

$$\Delta_d = 31.17 / 140.6 = 0.22 \text{ m}$$

$$\text{Max structural displacement for 2.5\%} = 0.025 \times 11.7 = 0.2925 \text{ m}$$

Substitute structure effective height

$$H_e = \frac{\sum_{i=1}^n (m_i \Delta_i h_i)}{\sum_{i=1}^n (m_i \Delta_i)}$$

$$H_e = 1253.6 / 140.6 = 8.92 \text{ m}$$

$$f_y = 500 \text{ N/mm}^2$$

$$f_{ye} = 1.1 \times 500 = 550 \text{ N/mm}^2 \text{ (yield steel)}$$

$$E_y = 2 \times 10^5 \text{ N/mm}^2$$

$$\epsilon_y = \frac{f_y}{E_y} = \frac{550}{2 \times 10^5} = 0.00275$$

$$h_1 = h_2 \text{ (x-axis) } = 900 \text{ cm}$$

$$\text{Yield draft} = \theta_{y1} = 0.5\epsilon_y \frac{Lbh}{Lbx}$$

Where,

$$Lb_h = 0.9\text{m}$$

$$Lb_x = 5.7\text{m}$$

$$Lb_y = 9\text{m}$$

$$\theta_{yx} = 0.0087$$

$$\theta_{yy} = 0.0111375$$

Equivalent system yield displacement

$$\Delta_y = \frac{((2 \times 0.0087) + (2 \times 0.0111375))}{4} \times H_e = 0.09\text{m}$$

$$\text{Magnitude} = 7.5(M_w)$$

$$\text{Corner period } T_c = 1 + 2.5(M_w - 5.7) = 5.5\text{sec}$$

$$\text{Maximum Displacement} = \frac{C_s \times 10^{(M_w - 3.2)}}{\gamma}$$

$$\gamma = \text{Epicenter} = 10\text{Km (Assumption)}$$

$C_s = 1.4$ (From equation 2.8, Displacement based seismic design of structures, M.J.N. Priestly)

Substitute Structure Equivalent Mass

$$m_e = \frac{\sum m_i \Delta_i}{\Delta_d} = \frac{140.6}{0.22} = 639\text{KN}$$

Spectral Displacement Reduction Factor

$$R_\epsilon = \left(\frac{0.07}{0.02 + \epsilon} \right)^\alpha$$

Where

$$\alpha = 0.5$$

$$\epsilon = \text{Damping motion}$$

$$R\epsilon = \left(\frac{0.07}{0.02+0.156} \right)^{0.5} = 0.63$$

$$\text{Corner Displacement (C}_d\text{)} = 0.63 \times 2793.4 = 1759.8 \sim 1760\text{m}$$

Effective time period of the substitute structure

$$T_e = T_c \times \frac{\Delta d}{c_d}$$

$$= 5.5 \times \frac{0.22}{1.76} = 0.69\text{sec}$$

Substitute structure stiffness

$$K_e = \frac{4\pi^2 \times 6\eta e}{T_e^2} = 52932.5 \text{ KN}$$

Base shear

$$F = V_B = K_e \Delta d \quad (\Delta d = 0.22)$$

$$= 11,645.15 \text{ KN}$$

Column Base moment

$$\Sigma M_{cj} = V_B \times 0.065 H_i = 32548 \text{ KNm}$$

Table- 4.2.2 Shear force and Bending Moment Calculation

Level, i	Height (h _i)(m)	m _i Δ _i	F _i (KN)	V _{si} (KN)	OTM (KNm)	Shear force(KN) (V _{bi})	Moment- beam (KNm) (M _{Bi})
3	11.7	62.15	5147.5	5147	0	255.6	651.8
2	8	51.02	4225.8	9373.3	19044	465.4	1186.8
1	4.3	27.42	2271	11644.3	75852.5	578.1	1474.2
		Σ= 140.6		Σ=26165	Σ=185277		

$$F_i = V_B \times (m_i \Delta_i) / (\sum m_i \Delta_i)$$

$$V_{si} = F_i + F_{i+1}$$

$$\text{OTM (overturning moments)} = \sum_{i=j}^3 F_i (H_j - H_i)$$

$$\text{Column base tensile force } (\sum V_{bj}) = T = \frac{M_1}{2M_1 + 2M_2} \times (\sum \text{OTM} - \sum M_{cj}) / L$$

$$M_1, M_2 = 1$$

$$L \text{ (span of plan area)} = 29.4 \text{ m}$$

$$\text{Tension force } T = 1299 \text{ KN}$$

$$\text{Seismic beam shear } (V_{bi}) = (T \times V_{si}) / \sum V_{si}$$

$$\text{Beam seismic moment } (M_{Bi}) = V_{bi} (L_1 - h_c) / 2$$

$$L_1 = 5.7 \text{ m (distance between the two outer column)}$$

$$h_c = 0.6 \text{ m (clear span between the columns 600 x 600)}$$

The maximum Beam seismic moment = 1474.2 KNm, is taken in the calculation of μ in Reynolds reinforcement concrete designers handbook 11TH edition.

Clause 3.6.2.3 Design of rectangular sections states that $\mu = M / bd^2 \times f_{ck}$,

$$f_{ck} = 40 \text{ KN/m}$$

$$\mu \Rightarrow 0.15 \text{ and } \mu^l = 0.21$$

With the help of Design chart for doubly reinforcement beams (4.9)

The area of steel is calculated $A_{st} f_y / bd f_{ck} = 0.2$

$$f_y = 500 \text{ N/mm}$$

$$f_{ck} = 40 \text{ N/mm}$$

$$b = \text{breadth of beam} = 350 \text{ mm}$$

$$d = \text{depth of beam} = 850 \text{ mm}$$

$$A_{st} = 4760 \text{ mm}^2$$

$$\text{Number of 25 mm steel rebars required} = 4760 / 490.6 = 10 \text{ Nos}$$

$$\pi \cdot d^2 / 4 = 490.1 \text{ mm}^2, d=25 \text{ mm}$$

For 5.7 m as well as 9 m span beam, the longitudinal steel area

$$A_{st} = 4760 \text{ mm}^2, 10 \text{ number of 25 mm } \phi \text{ bars.}$$

$$\text{Total Longitudinal steel area } A_s = N \times A_{st}$$

$$N = \text{Number of sections} = 13$$

$$A_s = 61880 \text{ mm}^2$$

4.6 MODAL RESPONSE SPECTRUM ANALYSIS

BY USING SAP2000 SOFTWARE

The proposed design of sathyabama plan is plotted in SAP2000 software, where the modal analysis, longitudinal steel reinforcement areas for cracked and un-cracked sections are found out. This area of steel is compared with the existing and manual calculations.

The dead load of the slab in all floors is assumed to be 5.75 KN/m (3.75KN/m slab dead weight + 2 KN/m Floor finish)

The live load of the slab is taken as 4 KN/m

The cracked section is the response spectrum load combination of (1.2Dead load + 1.2 live load + 1.2 EL), whereas

The un-cracked section is the collapse Load combination (1.5 (DL + LL))

Both load combinations are taken as per IS code provision 1893:2016 part 1 (clause 6.3.1)

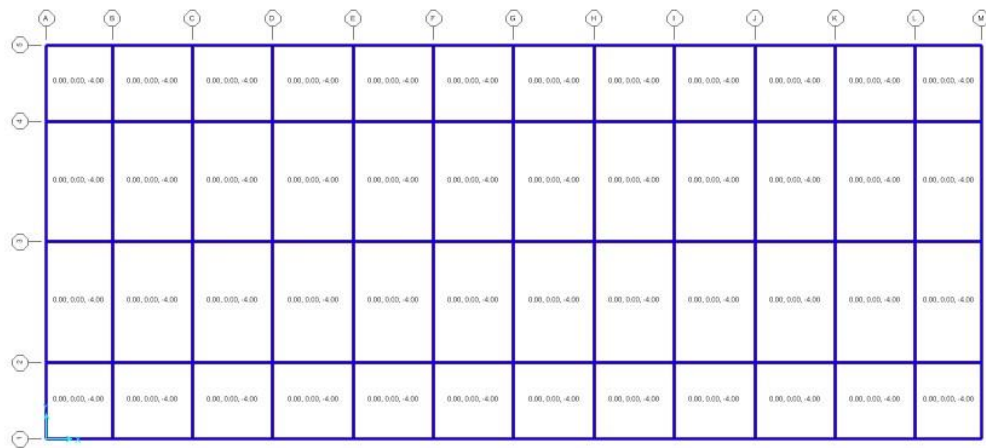


Figure 4.8 Assigning slab live load for Proposed Sathyabama Admin block plan

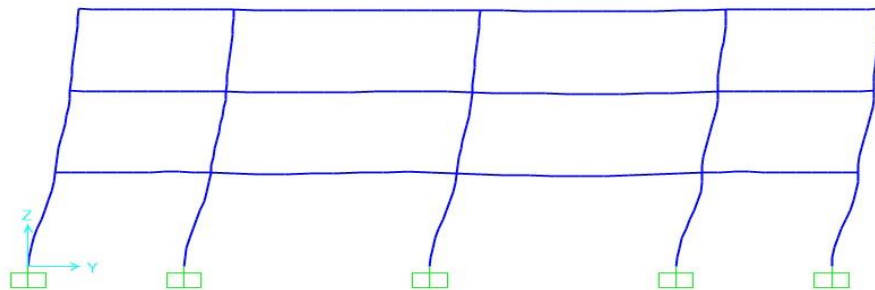


Figure 4.9 Deformed shape after applying EL load combination for Proposed Sathyabama Admin Block plan

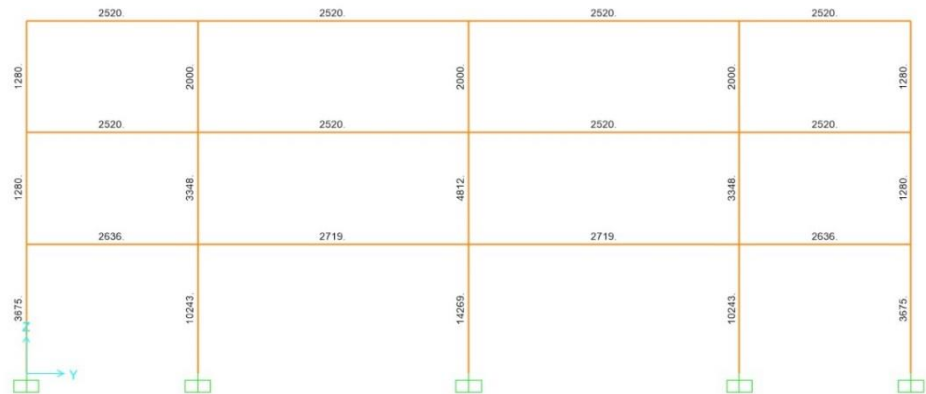


Figure 4.10 Resulted Steel longitudinal area for cracked section for Proposed Sathyabama Admin block plan



Figure 4.11 Steel longitudinal area for un-cracked section for Proposed Sathyabama Admin block plan

The area of steel for 5.7 m and 9 m beam in both cracked and un-cracked sections has been taken and tabulated for comparison with respect to the existing plan.

CRACKED SECTION (Modal Response Spectrum analysis)

For 5.7 m span beam, the longitudinal steel area is resulted to

$$A_{st} = 2636 \text{ mm}^2$$

Total Longitudinal steel area $A_s = N \times A_{st}$

N = Number of frames, (13 Nos)

$$A_s = 34268 \text{ mm}^2$$

For 9 m span beam, the longitudinal steel area is resulted to

$$A_{st} = 2719 \text{ mm}^2$$

$$A_s = 35347 \text{ mm}^2$$

UN-CRACKED SECTION

For 5.7 m span beam, the longitudinal steel area is resulted to

$$A_{st} = 2642 \text{ mm}^2$$

$$A_s = 34346 \text{ mm}^2$$

For 9 m span beam, the longitudinal steel area is resulted to

$$A_{st} = 2722 \text{ mm}^2$$

$$A_s = 35386 \text{ mm}^2$$

EXISTING BEAM LONGITUDINAL REINFORCEMENT STEEL AREA

For 5 numbers of 20 mm ϕ bars in 5.7 m span beam

Total Longitudinal steel area $A_s = N \times n \times A_{st}$

N = Number of frames, (19 Nos)

n = Number of bars in one beam,

$$A_{st} = \text{Area of steel } (\pi d^2/4)$$

d = diameter of steel rods

$$A_s = 29830 \text{ mm}^2$$

For 8 numbers of 25 mm ϕ bars in 9 m span beam

$$A_s = 74616.8 \text{ mm}^2$$

CHAPTER 5

SUMMARY AND CONCLUSION

5.1 SUMMARY

The variation of longitudinal reinforcement steel area of the weak beams have been calculated and tabulated in tables 5.1, 5.2. It shows the increase and savings in percentage of steel area by using Static analysis method, DDBD method and Response Spectrum analysis for cracked and un-cracked section. In IS 1893: 2016 (clause 6.4.3) the equivalent static method and Dynamic analysis by performing Response Spectrum method has been introduced. In this standard, Static Method, Response spectrum Method and Time History Method are adopted. It states that Equivalent static method maybe used for analysis of regular structures with approximate natural period T_n less than 0.4 seconds. In this project for static analysis method the calculated natural time period (T_n) of the building is 0.19 seconds. By combining the longitudinal area of beams from various methods table 5.1 and table 5.2 is tabulated listing the area percentage of steel differences. $P_t \% = \text{difference in steel area} / \text{the existing structure steel area } (A_s)$.

Table 5.1 Difference in longitudinal steel reinforcement of beam 5.7 m Span

SL. No	Method Analysis	Span of beam (m)	Spacing of frame (A_6, B_6, C_6, D_6, E_6)	Spacing Factor (F_s) (6/3.7)	Total Area of steel (A_s) (mm^2)	$F_s \cdot A_s$ (mm^2)	Difference in Steel Area (mm^2)	Percentage (P_t %) of change in steel
1	Existing Structure	5.7	3.7	1.62	29830	48325	-	-
2	Static Analysis	5.7	6	1	69212	69212	-20887	43.2% increase in steel
3	DDBD of 2.5 % Δ	5.7	6	1	61880	61880	-13555	28% increase in steel
4	SAP2000 Modal Response Analysis							
	(a) Cracked section	5.7	6	1	34268	34268	14057	29.1% Savings
	(b) Un-cracked section	5.7	6	1	34346	34346	13979	29% Savings

Table 5.2 Difference in longitudinal steel reinforcement of beam 9 m Span

SL. No	Method Analysis	Span of beam (m)	Spacing of frame (A ₆ , B ₆ , C ₆ , D ₆ , E ₆)	Spacing Factor (F _s) (6/3.7)	Area of steel (A _s) (mm ²)	F _s .A _s (mm ²)	Difference in Steel Area (mm ²)	Percentage (P _t %) of change in steel
1	Existing Structure	9	3.7	1.62	74619	119387	-	-
2	Static Analysis	9	6	1	91598	91598	+27789	23% savings
3	DDBD of 2.5 % Δ	9	6	1	61880	61880	+57507	48% savings
4	SAP2000 Modal Response Analysis (a) Cracked section	9	6	1	35347	35347	84040	70% Savings
	(b) Un-cracked section	9	6	1	35386	35386	13979 54001	70% Savings

5.2 CONCLUSION

- Earthquake can be defined as the shaking of earth caused by waves moving on and below the earth's surface and causing: surface faulting, tremors vibration, liquefaction, landslides, aftershocks and/or tsunamis.
- The seismic provisions of Indian code IS 1893: 2016 (clause 7.11.1) storey drift limit is taken into consideration and the area of Equivalent longitudinal steel reinforcement is calculated and compared with ASCE code Table 12.12.1; Allowable storey drift (ASCE/SEI 7-10).The code provisions concludes the maximum allowable drift limit is respect to the height of the building. IS 1893: 2016 (clause 7.11.1) states that “storey drift in any storey shall not exceed 0.004 times the storey height“.
- It means that for sathyabama admin block of storey height 11.7 m can have maximum storey drift $\delta_{\max} = 0.004 \times 11.7 = 0.047$ m in the top level. Similarly in ASCE code Table 12.12.1, the maximum allowable deformation $\Delta_{\max} = 0.025 h_e$, for structures of 4 stories or less. h_e represents the storey height. $\Delta_{\max} = 0.025 \times 11.7 = 0.293$ m. The storey drift as per ASCE standard is nearly 6.23 times greater than IS standards.
- This change in storey drift capacity is taken into consideration, and manual calculation of Static Analysis and DDBD method is done and compared to the existing steel area of the longitudinal beams and the excess, savings of steel area are tabulated.
- Lastly by using SAP2000 software the Modal Response Spectrum analysis is done and the resulted area of steel for the longitudinal beam is compared with the collapse load combination (un-cracked section) and Earthquake Load combination (cracked section) as per IS provisions.
- Considering Strong-Column-Weak Beam concept in the frame analysis, difference in the percentage of longitudinal beam rebar area is shown.

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