

**SEISMIC BEHAVIOR OF SHORT COLUMNS OF AN RC-FRAME ON
PLANE AND SLOPING GROUND**

DISSERTATION-II

Submitted by
MUZAFAR AHMAD GANIE

In partial fulfillment for the award of the degree of

MASTERS OF TECHNOLOGY

IN

**STRUCTURAL ENGINEERING
(CIVIL ENGINEERING)**



Under The Guidance of
Miss Sristi Gupta
Assistant Professor

LOVELY PROFESSIONAL UNIVERSITY

Phagwara – 144401, Punjab (India)

LOVELY PROFESSIONAL UNIVERSITY

DECLARATION

I hereby declare that the project work entitled “SEISMIC BEHAVIOR OF SHORT COLUMNS OF AN RC-FRAME ON PLANE AND SLOPING GROUND” is an authentic record of my work carried out as requirements of Dissertation-II Project for the award of M.Tech. in Structural Engineering from Lovely Professional University, Phagwara, under the guidance of Miss. Sristi Gupta during August to December 2017.

Name of Student: Muzafar Ahmad Ganie
Registration Number:11611361

Miss. Sristi Gupta
Department of Civil Engineering

(Signature of Student)

CERTIFICATE

Certified that this project report entitled “**SEISMIC BEHAVIOR OF SHORT COLUMNS OF AN RC-FRAME ON PLANE AND SLOPING GROUND**” submitted by “**MUZAFAR AHMAD GANIE**” registration no 11611361 of Civil Engineering Department, Lovely Professional University, Phagwara, Punjab who carried out this project work under my supervision for the Award of Degree. This report has not been submitted to any other university or institution for the award of any degree.

**Head of Department
Supervisor
(Structural Engineering)**

**Signature of

Miss Sristi Gupta
Assistant Professor
Department of Civil Engineering**

ACKNOWLEDGEMENT

First of all I would like to thank Lovely Professional University Management who has given me an opportunity to work on this project “SEISMIC BEHAVIOR OF SHORT COLUMNS OF AN RC-FRAME ON PLANE AND SLOPING GROUND”. I would also like to thank my parents for always being concerned, supporting and encouraging me to work on this project.

I express my deep gratitude and special thanks to my mentor of this Project “Miss. Sristi Gupta” for her valuable suggestions and guidance rendered in giving shape and coherence to this endeavor. I would also like to thank my friend Krishna Kant who helped me to get an access to the research papers that were necessary for the project.

ABSTRACT

RC structures on sloping ground are comprised of columns of different height within one storey. Brittle shear fail is generally noticed in short columns of RC frame as it takes the maximum shear during severe dynamic excitation by earthquake forces. It has been observed in previous earthquakes, Reinforced Concrete frame buildings having columns of unequal heights within 1 storey, more damage is indicted to short columns as they take most of the shear due to higher stiffness characteristics. Hence Short column suffers catastrophic damage during earthquake than long column as it fails by crushing. The dynamic analysis of buildings on varying slopes of ground and different seismic zones has been accomplished and the results compared to the building on flat ground. Dynamic evaluation of short columns is based on the parameters including shear ratio that determines the nature of column, energy dissipation capacity, shear resistance and ductility. Lack of these properties in existing structures is dealt with various retrofitting measures including CFRP, GFRP, Steel jackets, concrete and Ferro cement jacketing techniques. To improve hysteretic behavior of column, large bar diameters and higher percentage of longitudinal reinforcement must be avoided. The transverse reinforcement improves the hysteretic behavior & ductility of the column to a certain extent. For the dynamic analysis of RC frame, various softwares have been used for analysis purpose. In this study, number of research papers are reviewed that include the analysis of buildings on flat and sloping ground and suggest various strengthening techniques for the rehabilitation of short column.

TABLE OF CONTENT

CHAPTER DISSERTATION	PAGE No.
DECLARATION.....	II
CERTIFICATE.....	III
ACKNOWLEDGEMENT.....	IV
ABSTRACT.....	V
CONTENT.....	VI
CHAPTER 1 INTRODUCTION.....	1
CHAPTER 2 SCOPE.....	2
CHAPTER 3 OBJECTIVE.....	2
CHAPTER 4 LITERATURE REVIEW.....	3
CHAPTER 5 RESEARCH METHODOLOGY.....	14
CHAPTER 6 MODELLING AND ANALYSIS.....	15
CHAPTER 7 DISCUSSION AND CONCLUSION.....	25
CHAPTER 8 REFERENCES.....	27

The north and north east of India generally consists of hilly landscape which mostly have been characterized in zone IV and V respectively. The building on sloping ground have columns at different levels of ground slope. This phenomenon gives rise to the formation of short columns at the ground level. Short column suffers catastrophic damage during earthquake than long column as it fails by crushing due to higher stiffness. The mass and stiffness of such buildings vary along horizontal and vertical plane due to the vertical irregularity, due to which the center of mass and Center of stiffness of different stories do not correlate each other [6, 11]. This leads to the torsion response of building during dynamic excitation and hence needs special attention during analysis and design. Short column with lesser shear length and depth ratio is more vulnerable to brittle shear failure [2]. Columns that were built before 1970, were designed on the basis of strength, however it was observed that there was an abrupt non-ductile shear failure of short columns before its flexural strength was attained, when it was subjected to cyclic horizontal displacement [2]. Hence these buildings demand retrofitting for strengthening and to be safe in case of dynamic excitation. Various retrofitting methods have been used for strengthening including GFRP [2], CFRP [2], Steel [9], concrete and Ferro-cement jacketing to enhance shear resistance and energy dissipation capacity of columns. In this study, different configuration of building is considered and investigated considering its live load, dead load and earthquake load. Generally the STAAD pro V8i software has been used and various analytical methods including RSA, Pushover analysis, nonlinear Time History Analysis have been used to study the behavior of building in varying configurations and to evaluate its response parameters under dynamic excitation.

SCOPE

- ❖ To assess the impact of short columns on the response of RC frame on sloppy ground during static and dynamic loading.
- ❖ To ascertain the remedial measures to be taken to prevent the failure of short column of buildings on sloping ground during construction or after the construction of the building.

OBJECTIVE

- ❖ Analysis for static and dynamic loading of short and long columns of an RC frame on plane and sloping ground.
- ❖ Solution for improvement in strength and stiffness of short columns of an RC frame on a sloping ground.
- ❖ To evaluate various parameters through which it becomes cost effective.

S.S. Nalawade & B.G. Birajdar (2004) Studied “Dynamic assessment of Buildings resting on sloppy Ground”, in which they have analyzed 24 RC buildings with three dissimilar arrangements including “Step Back and Step Back cum Set Back buildings on sloppy ground and a Set Backed building on a level surface”. They have used the Response Spectrum analysis to evaluate torsional effect. The dynamic Response properties have been studied with reference to the suitability of building arrangement on sloping surface. The slope of ground for the analysis of building has been taken as 27 degrees. Analysis and Evaluation of the three configurations of the building indicate that Set Back buildings on plain ground attract less seismic forces.

K. Galal et al. (2005) Studied “Retrofit of RC Short Columns”, in which they have experimentally worked to assess the performance improvement of RC short columns with elevated and less steel content while they are retrofitted by using FRC’S. They have used Carbon and Glass FRP for retrofitting the RC short columns. They come up with calculated seven RC tiny columns & tested them under agile cyclic loading and sustained axial loading. They found during the experiment that short columns suffered the brittle shear failure including those designed according to current codal provisions. Their test proves that it is possible to enhance shear capacity of tiny columns so that flexural ductile failure occurs by promote plastic hinges on either ends of the of column. Small columns with lesser shear span ratio are susceptible to shear failure. They found that anchoring fiber wraps to the columns increased the shear and energy dissipation capacity of short columns. They divided the seven column specimens into two groups. Group one includes samples SCone, SCTwo, SConeR, SCTwoR & SConeU [2]. Group two includes samples SCthree AND SCthreeR [2]. In group 1, the SCone column was unstrengthened (not retrofitted) and was tested to be as a control model [2]. The column SCTwo was strengthened using 3 coats of CFRP [2]. For strengthening specimen SC1R, 4 layers of unidirectional G-FRP at the plastic hinge locality at the above and below ends of column were used [2]. The quantity of G-FRP at the plastic hinge locations of SConeR was selected so that it provides shear capacity nearer to that contributed by the C-FRP of specimen SCTwo [2]. SConeU was given strength by 3 coats of C-FRP alike to model SCTwo but without anchors [2]. In Group 2 (SC3 &SC3R) had less ratio of transverse steel according to 1986 ACI

design practice. SCthree was given strength utilising 3 coats of C-FRP [2]. SC3R was retrofitted using 6 & 3 layers of GFRP [2]. The use of carbon fiber coats for strengthening reinforced concrete tiny columns enhances the shear resistance and energy dissipation capacity rather than anchored glass fiber sheets. Moreover the use of carbon fiber sheets decreases the strains in the steel ties and the FRP along the height of column. The strains in both the transverse steel ties and fiber material decreased by increasing the number of FRP layers.

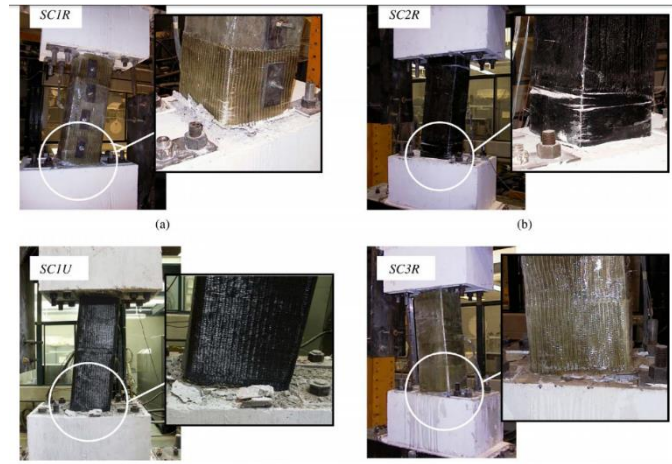


Fig. 1. Test models at maximum lateral drift:

(a) SConeR;[2] (b) SCTwoR;[2] (c) SConeU;[2] and (d) SCthreeR[2].

M. Moretti (2006) studied the ‘response of tiny columns suspected to cyclic shear drifts’. This is an experimental research in which 8 RC columns have been analyzed, in which the 7 columns have been analyzed as short columns with shear ratio less or equal to 2.5 [6 columns shear ratio=1.0 & for 7th shear ratio=2.0] and the 8th column analyzed as a slender column with shear ratio equal to 3.0 for comparison. The axial load ratio for all specimens was in general equal to 0.3 except for specimen 2 for which the value was 0.6, which increased the axial compressive force by 25% at which diagonal cracking of this specimen appeared compared to the other 7 specimens. The main objective of the experiments was to calculate the strains in the longitudinal and transverse steel. Specimens. Columns with shear ratio=1 failed in a brittle way while as columns with shear ratio=2 & 3 failed in a ductile way even with shear fissures at end segment. In specimens 5 & 6 part of the classical longitudinal reinforcement was exchanged by bi-diagonal steel and experimental analysis proved that it enhanced the hysteretic behavior and energy characteristics in case of short columns. The analysis of strain along the longitudinal reinforcement for varying values of shear load in a short column indicated that there was non-

linear distribution of strain along longitudinal reinforcement of short column specimens at the shear load value at which diagonal cracking appears. For specimen 8 [long column], there was flexural cracking at the flexural shear load value and the distribution of strain along longitudinal reinforcement was linear. The experimental results v/s analytical FEM prognosis of longitudinal reinforcement at different cross sections of a 1m column under external shear loads showed a difference in the strain for the specimen 2 and this difference goes on decreasing as the analysis is proceeded to successive sections of the column. It was observed from the load-displacement diagram, increase in %age of longitudinal reinforcement for specimen with shear ratio=1, there is 10% increase in the shear load at which diagonal cracking of specimen appears but simultaneously there is larger response and stiffness degradation, however if bi-diagonal reinforcement is also provided to the same specimen with increase in %age of longitudinal reinforcement, the behavior is flexure rather than shear and there is less shear response and stiffness degradation in the specimen[short column]. Hence to enhance hysteretic behavior, in terms of response decrease and energy dissipation of short column, large %age of longitudinal reinforcement and bar diameter must be avoided, however higher transverse steel enhances ductility. More over partial substitution of bi-diagonal reinforcement improves hysteretic behavior and is thus favorable for the short column stability

A. Kheyroddin & A. Kargaran (2009) Studied “Seismic Behavior of tiny Columns in RC Structures”, in which they have studied the behavior of short columns of duplex structures of 4, 8 and 10 story structures, all having difference in story floors relative to each other that leads to discontinuity in floor diaphragm inflicting vital vary in period, stiffness distribution of dynamic force and dynamic loading of structure. Diaphragms transfer the lateral inertial earthquake forces between resistant parts of building but the disorder or separation in diaphragm floor, due to story height difference cause stress absorption in their junctions with upright parts (columns). Depending on stiffness disparity of columns, most of these forces reach to tiny columns of floor & incase of unsuitable designing, severe damage is caused to the short column. In this analysis, seismic behavior of short column in 3 duplex structures has been surveyed that have height level distinction of 1.6m. The plan of all three duplex structures (4, 8 and 10 storey) is same and variable in height. In this analysis, at first, seismic behavior of short column development is established, then, nonlinear response of RC tiny columns in four, eight and ten story structures with story level distinction is examined. tiny columns and cited structures are examined using the Elcentro schedule of earthquake with completely varying

higher base acceleration with IDARC package that is nonlinear dynamic assessment program. During this assessment, the conclusion of optimum response, base shear, global damage index, drift time history and impact of tiny column in structural failure is examined [4]. The damage in structural elements occur in a progressive manner and the trend includes damage stage in small scale (material failure), progressive gathering includes medium scale damage (damage to structural elements: Beams and columns) and this includes gradual increasing of cracks and their expansion resulting in wide scale damage (structure failure). To assess the definite response of structure due to earthquake, nonlinear analysis program is adopted using IDARC software. In results of Elcentro earthquake show that the average damage rate in short column in distinct story increases with the increase in Peak Ground Acceleration in all the three duplex structures. Extent of Seismic damage of short column in consecutive floors of 3 duplex structures enhances in structural height especially in upper story degree of damage has been increased. It is observed that in eight and ten story structures, failure in short columns of 4th and 6th story is least. The displacement history of last short column in four, eight and ten story structures is higher than tiny column in overall structures by enhancing Peak Ground Acceleration. Drift time history of ist and medium tiny column in four story structures and end short column in ten story structure is more relative to other structures. Investigation of Shear force history established that the median of shear force record in 1st short column in 4 story structures, mid short column in eight story structures and last short column in ten story structures has the majority amount relative to other columns. Damage index concluded that the fraction of end short column and lower part of ist short column in eight and ten story structures get largely damage. By increasing height and story of story of structures it is concluded that damage at upwards and downwards of mid short column has decreased.

Dinesh J.Sabu and Dr. P.S. Pajgade (2012) studied “Seismic Evaluation of Existing Reinforced Concrete Building”, in which he opted for the Response Spectrum analysis procedure for the analysis of an RC frame without infill and frame with infill and soil impact so as to look at the response of those models. After executing the RSA of those models, reinforcement needed in each case is established and have suggested the retrofitting with reference to it. They have studied totally different retrofitting strategies and established that impact of infill plays a crucial role within the analysis of existing reinforced concrete building. For analysis of three models, they have used STAAD Pro. They concluded that the durability of the existent structure could be enhanced as needed by using concrete jacketing type of

retrofit. It enhanced the stiffness of the building and the building exhibited better results in terms of maximum displacement. They concluded that if actual reinforcement is higher the reinforcement needed within the brick infill and soil interaction impact, then there is no requirement to retrofit the particular section, as the amount of actual reinforcement is sufficient to carry the seismic forces (zone III).

Y Singh et al. (2012) studied “Dynamic response of structures lying on slopes- analysis study and remark of Sikkim earthquake”, they have performed analytical study to observe the dynamic behaviour of structures on sloppy ground. They compared dynamic response of buildings on sloppy ground with reference to buildings on level ground to assess the fundamental time of vibration, form of inter-story displacement, column shear and plastic hinge development pattern [6]. The dynamic response of 2 arrangements of hill buildings is assessed utilizing the linear and nonlinear time history analysis. The dynamic characteristics of the hill buildings vary to that of buildings on level ground as buildings on sloppy ground are uneven and unsymmetrical in the horizontal and vertical directions [6]. The irregular difference in stiffness and mass of buildings on sloping ground result in the creation of torsion due to lateral loading as the midpoint of stiffness and midpoint of mass don't coincide each other. The unequal column height within a story of buildings on sloping ground result in large difference in the stiffness of columns within said story. The short and inelastic columns on the upper hill side allure more lateral forces and are subjected to damage. For the purpose of analytical investigation, they have compared 4 types of buildings including a nine story reinforced concrete frame building with 2 varying hill arrangements and two buildings on flat ground of 9 and 3 stories respectively. The nine storey reinforced concrete frame building with 2 varying hill arrangements has 6 stories below road level and 3 above it. To assimilate the behaviour with even buildings, 2 even buildings on level ground with 9 and 3 stories have been considered. The 1st building (type S-1) on a slope of 45 degree is stepping back at every floor level up to 6 stories and has 3 stories over the road level. 2nd structure named as “Type S-two”, is stepping back at 6th floor level only and possess 3 stories above road level. The nine and three storied even structures on level ground are named as “Type P-three” and “Type P-four”, respectively as shown in fig: 3.2. Investigation reveals that due to the irregularity in arrangement, the mass play in fundamental mode in case of building on sloping ground is much lower than the regular building on flat ground. They also observed that there is no considerable lateral drift beneath the 6th floor level (road level) in “Type S-one” building, due to high

rigidity of columns. The deflected form of the “Type S-two” building is same to a vertical cantilever propped at 6th floor level [6]. In case of “Type S-I” building all the storey shear beneath 6th floor level is thwarted by short columns [6]. In case of “Type S-two” arrangement, the columns in the lower storey and stories immediately up and down the road level (6th & 7th storey) are susceptible to higher forces. In “Type S-one” building torsion is noticed in all stories while in “Type S-two”, torsion is noticed in upper 3 stories only [6]. The investigation has concluded that hill buildings experience significant torsion impact due to excitation perpendicular to the slope. Under excitation in the direction of slope, the differing elevations of columns cause stiffness unevenness, and the tiny columns resist almost the entire storey shear. The linear and non-linear dynamic assessment shows that the storey at the road elevation is most susceptible to damage.

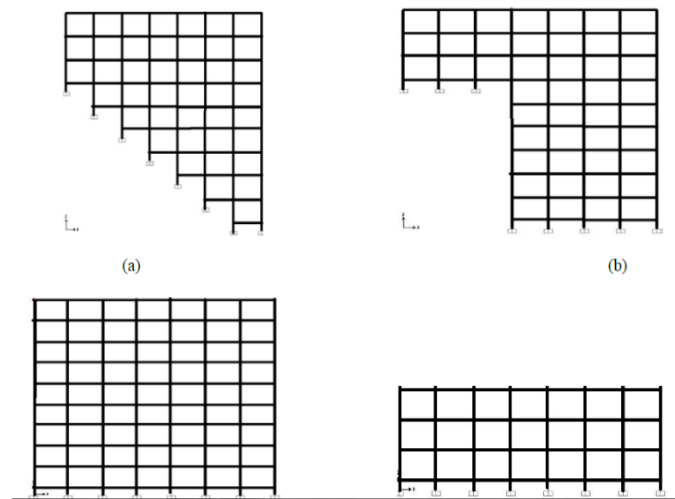


Fig. 2 Elevation view of buildings:

(a) Type S-one;(b) Type S-two; (c) Type P-three; (d) Type P-four

Keyvan Ramin and Foroud Mehrabpour (2014) Studied seismic behavior of an RC building resting on a sloping ground using STAAD pro v8i. The study makes assessment and distinguishes 2 four-story RC moment opposing frame (MRF) buildings with average deformation, one amongst that is found on a level ground and the another one is on a ground slanting by twenty degrees. Conjointly Sap2000 computer code had been utilised to establish that the drift of floors is more for a level building than a sloppy structure. However, the rise in shear was established to be more in shorter columns when compared to similar ones and a higher moment ought to be tolerated by structures on sloppy ground. The nonlinear static

pushover analysis proved structure to be stiffer. The main function of this column is to transmit the inertia force generated from earthquake to columns. Major proportion of these forces is taken over by the tiny column as the stiffness differs from column to column. Thus, the tiny column shows a colossal potential for serious harm by earthquake within the case of associate inappropriate design. As per the results, short column are supposed to possess more resistant sections and are should be reinforced with more bars. Hence additional reinforcement ought to be utilised as stirrups compared to that of longitudinal bars. Moreover for existing structures, shear capability of short columns ought to be retrofitted by fibre reinforced polymer, Steel coat or other materials [7].

Hugo Rodrigues et al. (2015) studied “dynamic response of strengthened reinforced concrete columns under biaxial loading”, in which they have presented the results referenced to of shear drift, stiffness decrease, ductility and energy diffusion. They have done the experimental characterization so as to improve the ductility and strength and this purpose was met either by efficient jacket coating or enhancing the amount of longitudinal and transverse steel. They have presented the results of 9 tested specimens, in which 7 columns were strengthened with plates of steel and CFRP. These 9 columns were made with similar geometric characteristics and reinforcement detailing and were tested for 2 varying loading histories. In order to ascertain the characteristics of response of the strengthened column model, many loading conditions were taken. Cyclic lateral drifts were imposed at the top of the column with steadily increasing displacement levels. Two Hz. drift path types were adopted, including “Diagonal-45 degree and Diamond”, respectively. The 9 columns with similar geometry and reinforcement detailing were subjected to similar biaxial horizontal displacement paths and with equal constant axial load. In this campaign 6 RC columns were tested under different loading histories in order to evaluate the influence of the biaxial loading in the cyclic response of the columns. After that 4 of the tested columns were repaired and were subjected to different retrofit strategies so as to replace their original characteristics, so as to improve their ductile behaviour to respond well under cyclic loads. The retrofit techniques that have been used in the present work include: increasing the number of stirrups, steel packet jacketing and CFRP sheets and plate jacketing. After strengthening these columns with these retrofit strategies, were again tested biaxially. The results have been presented in terms of shear drift, stiffness degradation, ductility and energy dissipation and these results were compared with the original un-strengthened column results. They concluded that the columns subjected to the diamond horizontal load path and the

ones with C-FRP retrofit exhibit more energy dissipation capability when compared with diagonal load path. The experimental results on the column retrofitting show that the initial stiffness is typically lower and softening starts for higher drift demands. Also it was observed that there was increase in strength of retrofit columns up to maximum of 20%. It was also observed that for the same drift demand, the damage in original columns was more pronounced compared to that of retrofitted columns.

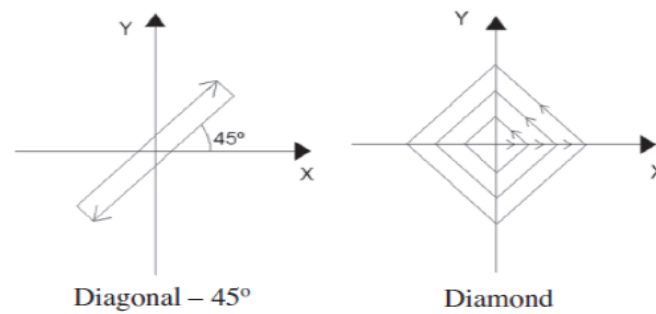


Fig. 3 Horizontal displacement paths types

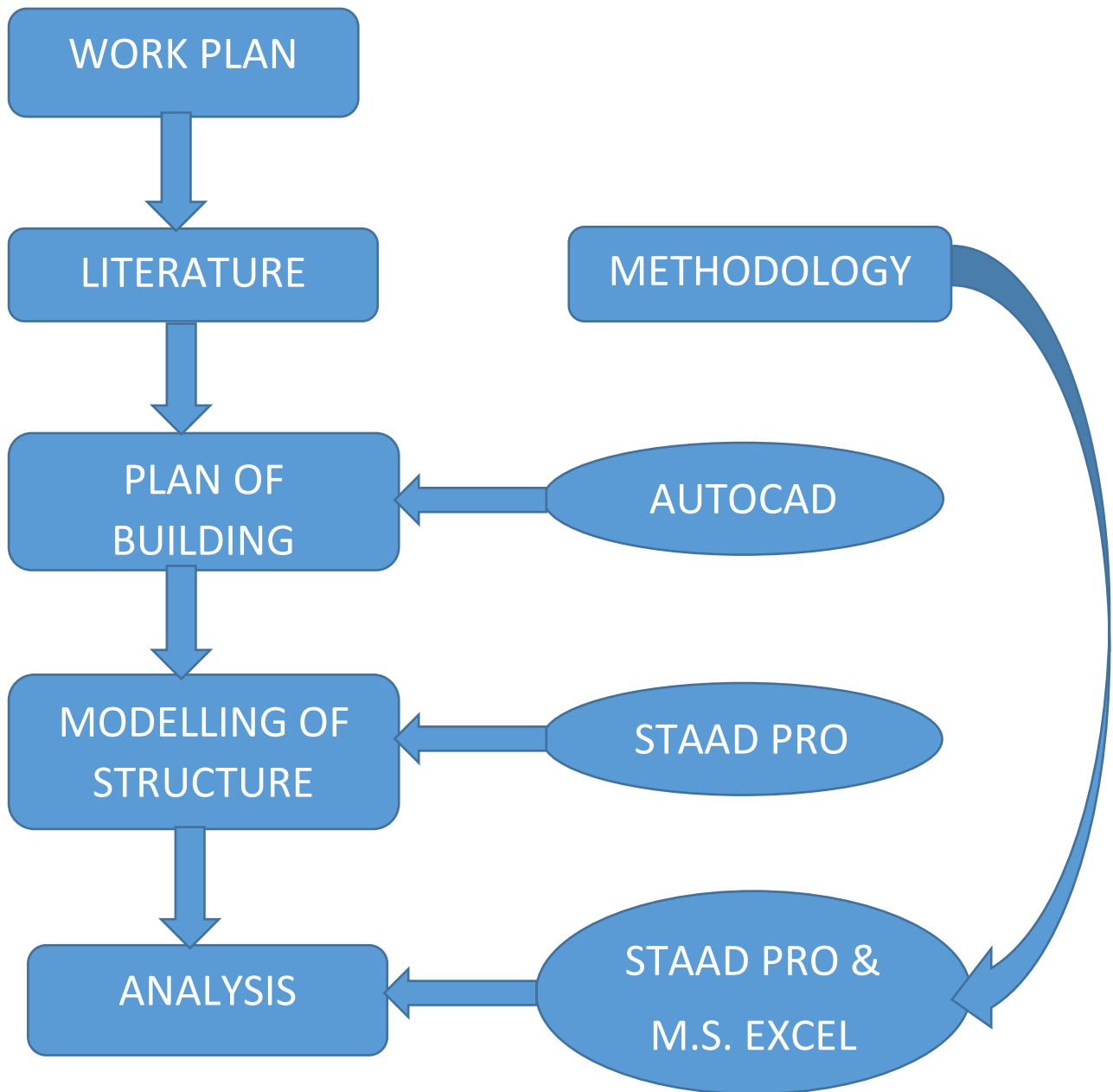
Mahmoud F. Bilal et al. (2015) studied “Behaviour of reinforced concrete columns strengthened by steel jacket”, in which they have analyzed the response of RC columns strengthened by steel jacket technique. RC columns often need strengthening to increase their capacity to sustain the applied loading. Their objective was to determine the effect of shape of the main strengthening retrofit system, size and number of batten plates on the behavior of RC column. They investigated the behavior of 7 RC columns divided into 2 un-strengthened control specimens and 5 strengthened specimens, strengthened with different steel jacket configurations. All steel jackets were 200x200 mm in cross section with 1200 mm height, which concludes that L/d ratio is less than 12 and the columns taken for investigation are the short columns. The vertical steel elements (angles, c-sections and plates) meant for jacketing technique were chosen to the same total horizontal cross sectional area. They placed the specimens in the testing machine between the jack head and the steel frame. The strain gages, load cell and linear voltage displacement transducer (LVDT) were all connected to the data acquisition system attached to the computer. The load was continuously monitored by a load cell of 5000 KN capacity and was transmitted to the RC column through steel plates to provide uniform bearing surfaces. They used a controlled data acquisition system to continuously

record readings of the electrical load cell, the two dial gauges of 0.01 mm accuracy (LVDT instrument) that measures the horizontal deformations of the column in two perpendicular directions, the reinforcement strain gages and also the steel jacket strain gages. In order to ensure that the failure should take place in specimen's body and not the head, the top and bottom ends of the specimens were more confined with steel boxes made from 10 mm thick steel plates. All the test records were automatically saved on the computer data acquisition file for further data manipulation and plotting. They developed a finite element specimen to study the behaviour of these columns. The specimen was verified and ascertained using the experimental results. The research showed that the different strengthening methods have large impact on the column capacity. They concluded that the size of the batten plates had substantial effect on the failure load for models strengthened with angles, whereas the number of batten plates was more impactful for models strengthened with C-channels. They investigated, analyzed and verified their behaviour by using finite element (F.E) package ANSYS 12.0. The test result showed a good match between both experimental tests and F.E. models. They concluded through the experimental results that the modes of failure and failure loads varied depending on the configuration of the steel jacket as well as its arrangement. It was not possible to observe the initial cracks or the cracking load for specimen as the strengthening element covered most of the specimen, hence only the failure load was recorded. The failure load is considered the maximum recorded load during testing and at which the specimen could not carry any extra load. They concluded that the column capacity was increased to a minimum of 20% by strengthening the RC columns using steel jacketing technique. The controlled RC column specimen failed in a brittle manner while the specimens strengthened with steel jacketing failed in more flexible and ductile mode. It was observed that the failure load was increased in case of specimens strengthened with angles or channel sections with batten plates. The specimens strengthened with 4 angle sections encountered less deformation than other specimens, but it was also observed that increasing the number of batten plates in 4 angle series of specimen didn't help increase the failure load while as the failure load was increased in case of specimen strengthened with 2 channel sections. The use of Channel sections with batten plates or the use of plates only needs special attention to be given in the perspective of the buckling due to their thin thickness. The simulation of strengthened RC column using F.E analysis in ANSYS 12.0 program is quite well, since mode of failure, failure loads and displacements predicted were very close to those measured during experimental testing, but it was observed that F.E package ANSYS 12.0 overestimated failure loads for strengthened models when compared with the experimental results.

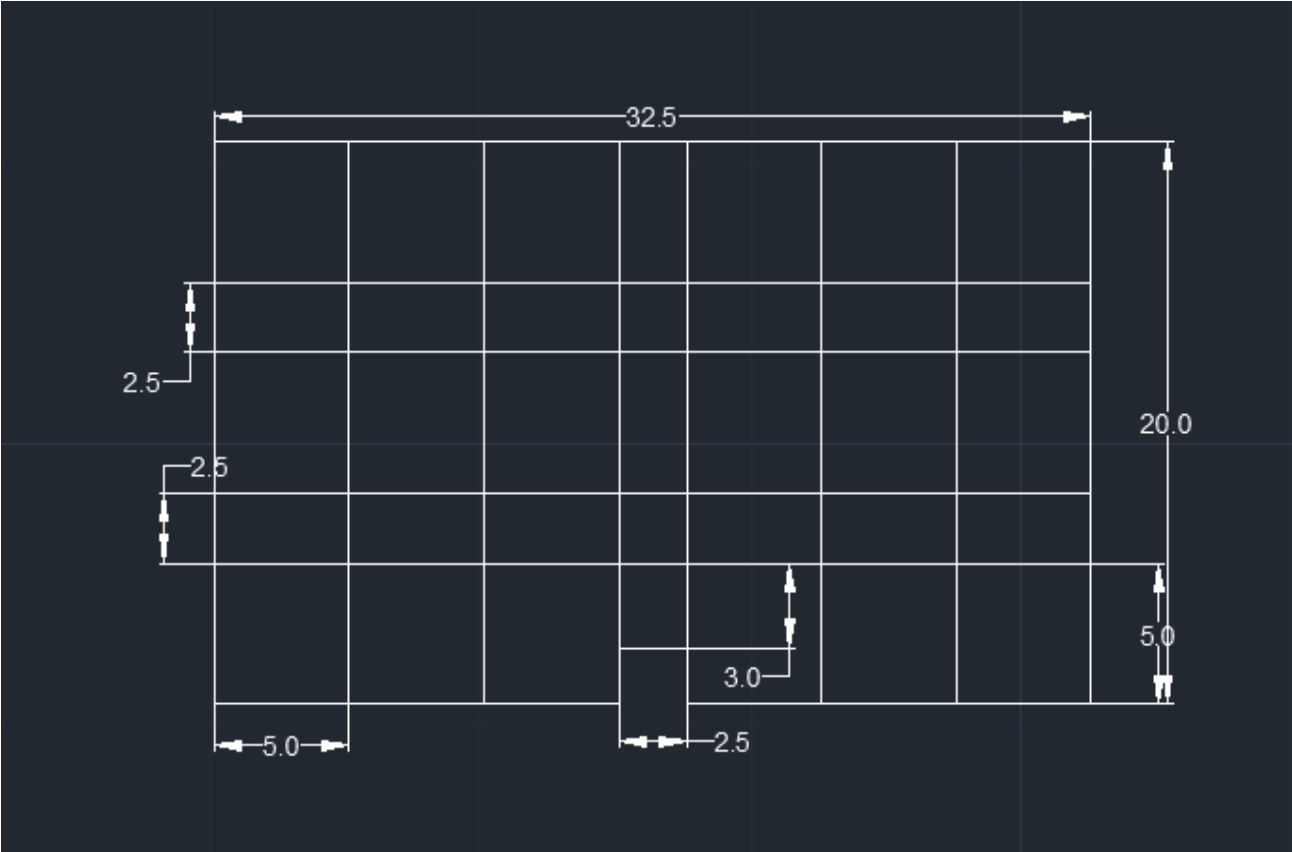
Vinay Mohan Agarwal and Arun C (2015) studied “comparative study on fundamental period of RC framed building”, in which their study proposes that it is hard to measure the unevenness in a setback structure with any one criterion. It is recommended by most of the national and international design codes to opt for dynamic analysis for the design of a setback building with elevated base shear, referenced to the fundamental time as according to the empirical formula given in the code. However the empirical formula for calculating the fundamental period of a setback building given in the code depend of building height, which is ambiguous [10]. The ambiguity has itself been observed by them, as the analysis has shown that the fundamental time of a setback structure varies when the arrangement of building changes, even though if the height of building remains the same. The fundamental time of setback buildings is always found to be lesser than that of identical regular buildings. The fundamental time of a bare building, depends not only on the elevation of the structure but also on the bay width, irregularity and many other structural and geometrical parameters [10]. Hence correlating the fundamental time of a setback structure to its elevation only is not adequate. In setback buildings, there is a staggered abrupt reduction in floor area along the elevation of the structure and there are a consequent drops in mass, strength, stiffness, mid of mass and mid of stiffness along the elevation of the structure. This results the variation in the dynamic characteristics of the setback building with the regular building. The empirical equations given in the design codes for calculating the fundamental period of a setback building are function of height of building only. Hence based on free vibration assessment of 90 setback building frames with constant height and varying unevenness, their study critically reviews the empirical code formula for fundamental time, to make it applicable for setback buildings. The calculation of the fundamental time period of building is generally determined by the empirical formula recommended by the codes including IS 1893:2000, ASCE 7:2010, Euro code 8 or New Zealand code of practice. These codes also define different types of irregular structures. Different methods for determining the fundamental time and the definition of irregularity as per available design codes are discussed. The following formulas were checked and the results were calculated and the comparison of the fundamental time of setback structure with that obtained from the equation based on IS 1893:2002 was carried out and was presented. The comparison indicates that the empirical formula in the IS code provides lower bound of the fundamental time than obtained by Modal Analysis and Rayleigh method. Therefore IS 1893:2002, always provides estimates of fundamental time period of setback buildings with

six to thirty storeys. They also concluded that the values for fundamental periods of setback buildings determined by the Rayleigh's method are underestimated, which is also conservative for selected buildings. The ASCE 7:2010 approach for calculating the building fundamental periods is found to be most conservative, as it only considers the number of storeys of the building and doesn't consider the height of the building. Rayleigh's formula has been found more rational approach for calculating the fundamental time period as the formula depends on structural properties and deformational characteristics of resisting parts. Their study indicates that there is a very weak correlation between fundamental periods of a setback building with any of the parameters used to define the setback irregularity by the design codes, like the way these codes define the setback irregularity by only geometry is found to be inadequate. The fundamental period is different for the structures with similar maximum elevation and similar maximum width, depending on the amount of unevenness present in the setback structures. This difference in fundamental periods due to difference in unevenness is found to be more in taller structures and less for shorter structures respectively. This observation has been put forth from both Rayleigh and modal analysis, and the fundamental periods calculated by these two methods have been observed to be quite similar.

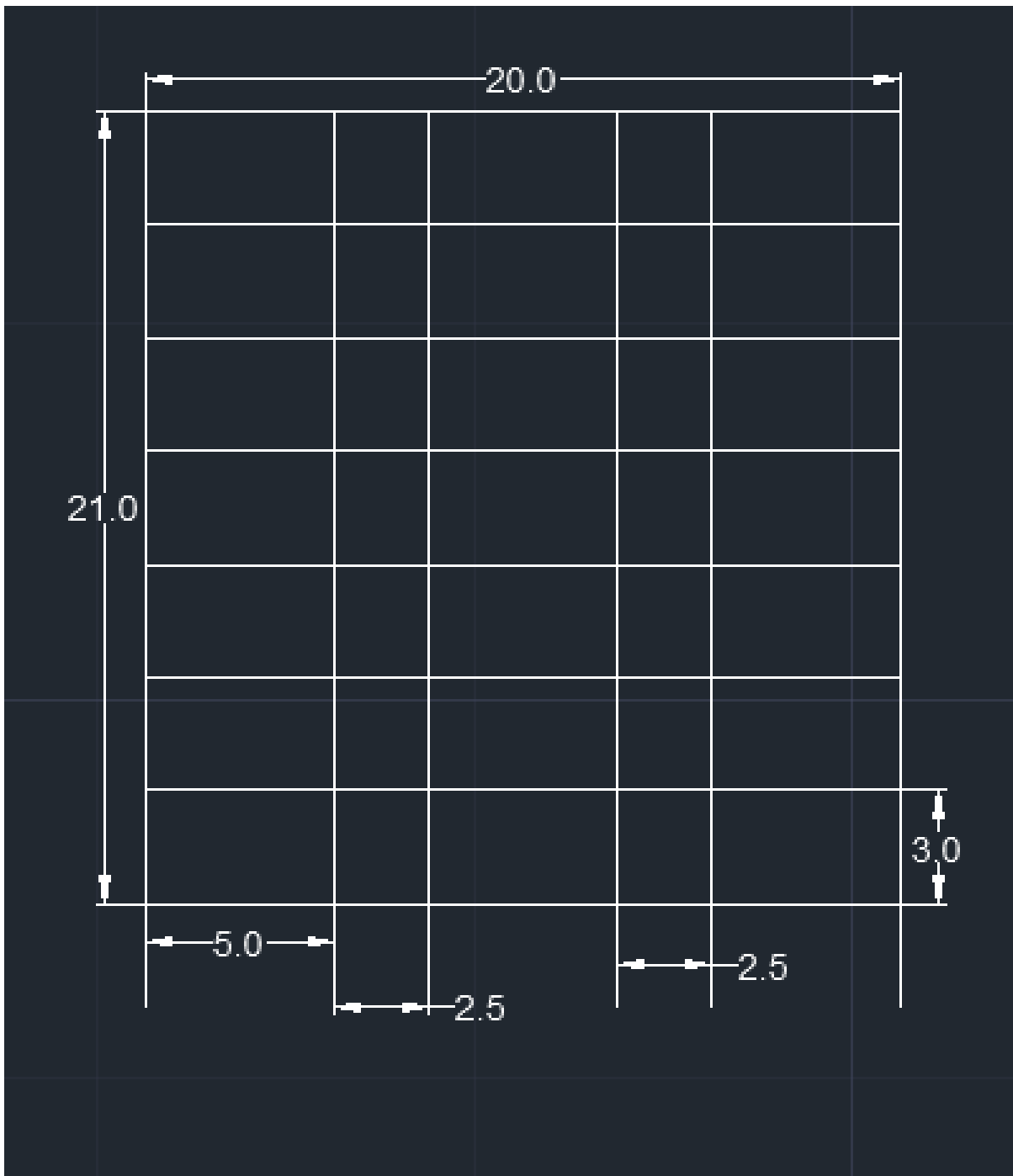
Rajkumar Vishwakarma and Anubhav Rai (2017) studied "Analysis of RCC frame tall structure using STAAD Pro V8i, on different seismic zones considering ground slopes", in which they have done comparative analysis of seismic behavior of a G+10 RCC frame building with different slopes of ground, taken as 0 degree, 7 degree and 14 degree with three different soil types. The method that they have opted for analysis is response spectrum analysis as per IS 1893-2000. They have made a comparison of the analytical results in terms of maximum displacement, maximum bending moment and maximum shear force for the building on three slopes as mentioned and with three different soil types (soft, medium and hard). They concluded that with the increase in angle of a slope, the maximum displacement, maximum bending moment and maximum shear force on the columns of a building also increases. Also the change in soil type has shown its effect on the analytical results.



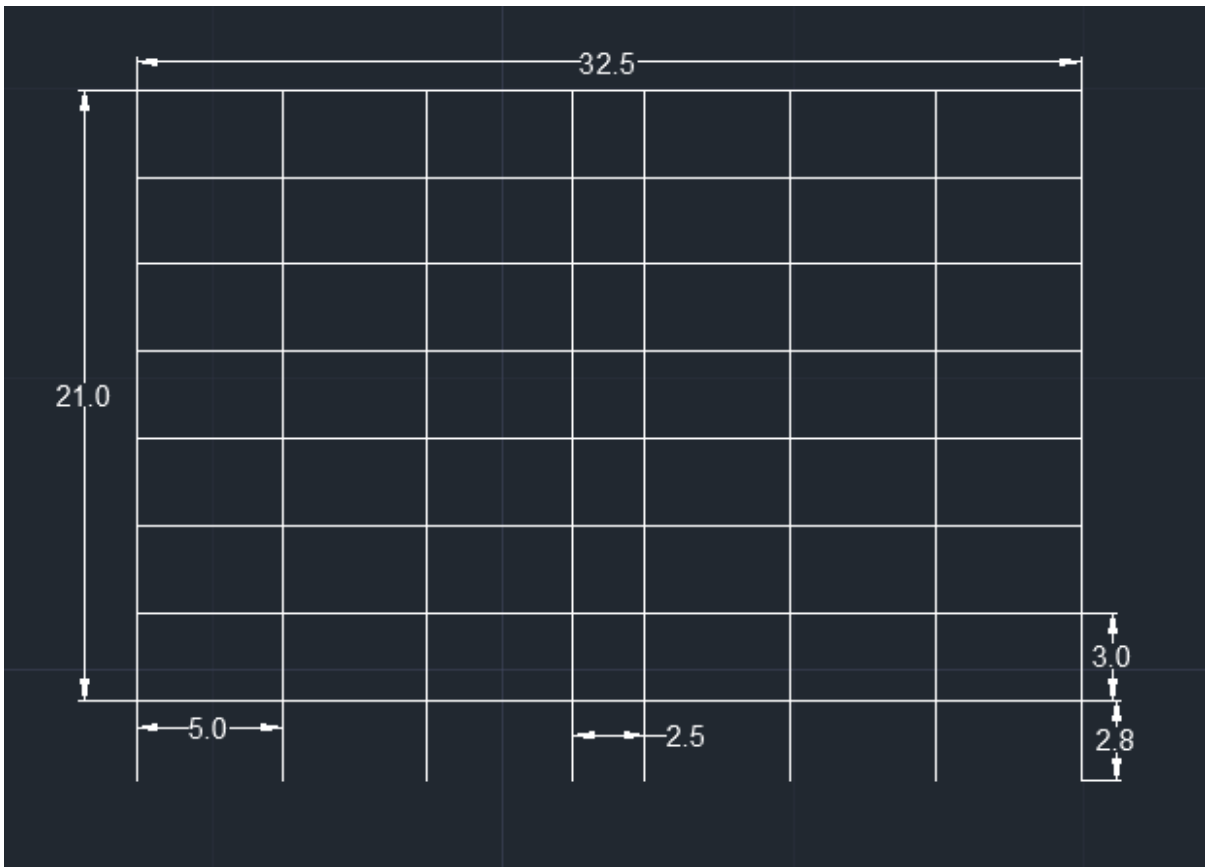
5.1 Plan of Building



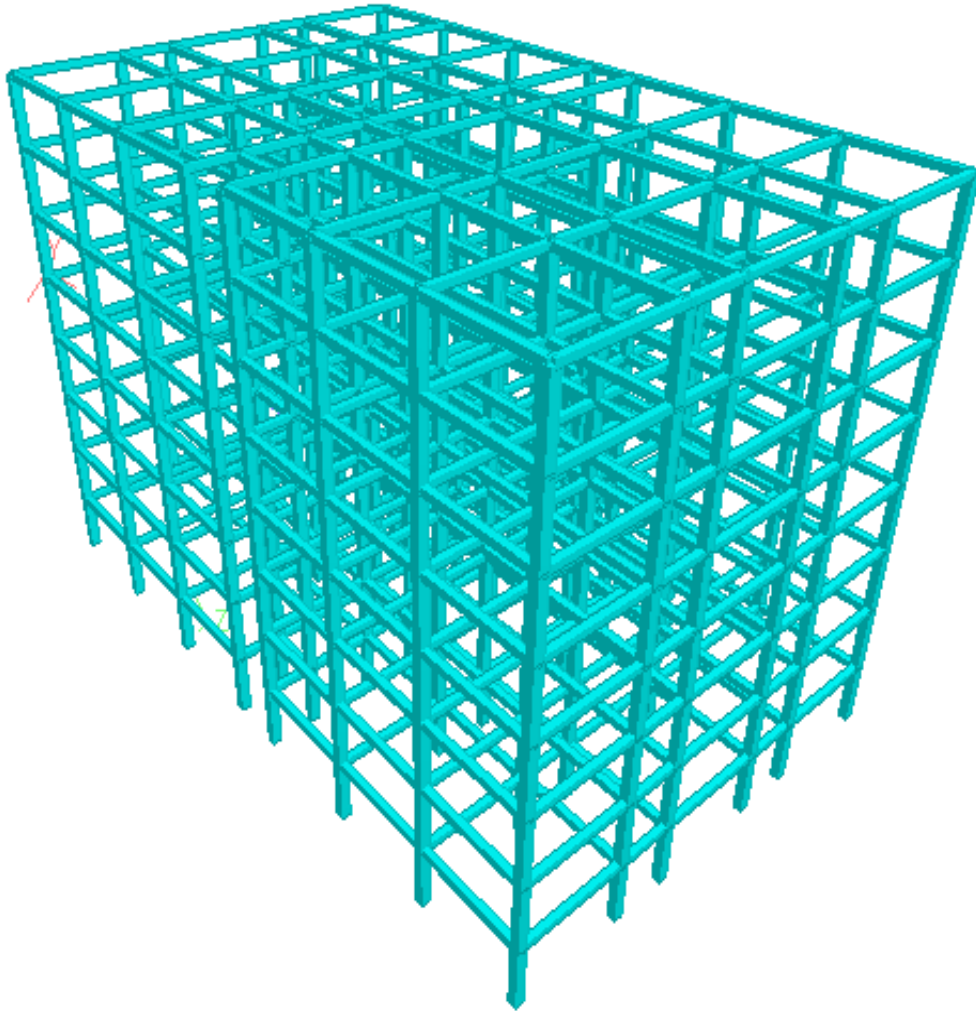
5.2 Front Elevation of Building



5.3 Side Elevation of the Building



5.4 Model of the Building



5.5 Design on Excel

LOAD CALCULATION

1 SLAB-			
Lenth,l=	5	m	5000 mm
width,b=	2.5	m	2500 mm
depth,d=	0.23	m	230 mm
Unit wt of conc. γ =	25	kN/m ³	
Load (for/m lenth and width=	lxbxD γ		5 kN/m ²
Slab load will distribute in Triangle and Trapezoidal form			

2 BEAM-			
For slab 5x5			
Lenth,l=	5	m	
width,b=	5	m	
Area of triangle(Δ)=	0.5*b*h		6.25 m ²
Load on beam, W=	Δ *length of adjacent beam		31.25 kN/m
Reactions or Load on Supports=	W/2		78.125 kN

For slab 2.5x2.5			
Lenth,l=	2.5	m	
width,b=	2.5	m	
Area of triangle(Δ)=	0.5*b*h		1.5625 m ²
Load on beam, W=	Δ *length of adjacent beam		3.90625 kN/m
Reactions or Load on Supports=	W/2		4.8828125 kN

For slab 5x2.5			
Lenth,l=	5	m	
width,b=	2.5	m	
Area of triangle(Δ)=	0.5*b*h		1.5625 m ²
Area of trapezoid(Δ)=	0.5*(h/2)*b/2		4.6875 m ²
Load on beam-			
W=	Δ *length of adjacent beam		3.90625 kN/m
(a) with triangle			9.765625 kN
Reactions or Load on Supports=	W/2		23.4375 kN/m
(b) with triapooidal			58.59375 kN
Reactions or Load on Supports=	W/2		

Beam	Load(in kN/m)	Rxn=W/2(in kN)
B1	31.25	78.125
B2	62.5	156.25
B3	54.6875	136.71875
B4	3.90625	4.8828125
B5 & B6	7.8125	9.765625

3 COLUMN-	C1(in kN)	C2(in kN)	C3(in kN)	C4(in kN)	C5(in kN)	C6(in kN)	C7(in kN)
6th Storey	156.25	219.7265625	312.5	439.453125	292.96875	292.96875	214.84375
5th Storey	312.5	439.453125	625	878.90625	585.9375	585.9375	429.6875
4th Storey	468.75	659.1796875	937.5	1318.359375	878.90625	878.90625	644.53125
3rd Storey	625	878.90625	1250	1757.8125	1171.875	1171.875	859.375
2nd Storey	781.25	1098.632813	1562.5	2197.265625	1464.84375	1464.84375	1074.21875
1st Storey	937.5	1318.359375	1875	2636.71875	1757.8125	1757.8125	1289.0625
Ground Storey	1093.75	1538.085938	2187.5	3076.171875	2050.78125	2050.78125	1503.90625



TWO WAY SLAB DESIGN

Size of Slab	Lx=	5	m	5000	mm	
	Ly=	2.5	m	2500	mm	
Live Load				3	kN/m ²	
Finish Load				1	kN/m ²	
Conc Grade	f _{ck}			20	N/mm ²	
Unit Weight of Conc.				25	kN/m ³	
Steel Grade	f _y			500	N/mm ²	
beam thickness		0.15	m	150	mm	
Effective Short Span				2650	mm	
Span to effective depth ratio			1.5*20	30		
depth	d=	88.33333	mm	200	mm	
Assume						
Clear cover	d'			30	mm	
Dia of bar				10	mm	
Overall depth	D=Depth+Clear Cover			230	mm	
Effective depth	d=D-Clear Cover-0.5Dia of bar			195	mm	
Effective Span		l _{ex} =		5195		
		l _{ey} =		2695		
		r=		0.518768		
Loads on Slabs	Self Wt			5.75	kN/m ²	
	Finish			1	kN/m ²	
	Live			3	kN/m ²	
			Total		9.75	kN/m ²
Factored load	P _u =			14.625	kN/m ²	
	α _x =	(r ⁴ /(1+r ⁴))/8		0.008442		
	α _y =	(r ² /(1+r ⁴))/8		0.031368		
Design Moments	M _{ux} =α _x *W _u *l _{ex} ²			3.331982	kNm	Shorter Span
	M _{uy} =α _y *W _u *l _{ex} ²			12.38101	kNm	Longer Span
Design of Reinforcement						
Ast = ((1-(1-(4.6Mu/(f _{ck} bd ²))) ^{0.5})/2f _y)						
Required Ast	A _{st,x} =			28.87733	mm ² /m	
	A _{st,y} =			135.947	mm ² /m	
Area of one bar	a _{st} =(π*d ²)/4			78.5	mm ²	

Spacing X dir 2718.396 mm
 Y dir 577.4308 mm

Maximum spacing for primary reinforcement=300mm

Total No. of bars in X-direction Nos.,x=Lx*Ast/ast 1.83932
 Total No. of bars in Y-direction Nos.,y=Ly*Ast/ast 4.329523

Hence Provided no. of bars

X- DIR 2
 Y- DIR 5

Hence Provided Ast

157 mm² 31.4 mm²/m
 392.5 mm² 157 mm²/m

Provided Spacing

X- DIR 2500 mm C/C 2500 mm C/C
 Y- DIR 500 mm C/C 500 mm C/C

Checks-

1 Deflection- (l/d)max 30
 (l/d)prov 26.64103 Safe

2 Shear- $\tau_v = \frac{V_u}{bd}$
 $V_u = w_u(0.5l_y - d)$ 15.42938 kN/m
 $S_o, \tau_v =$ 0.079125 N/mm²
 $P_t = \frac{100A_{st,prov}}{bd}$ 0.016103 %
 $\tau_c =$ 0.172882 N/mm²

For τ_c (From Table 19 IS 456)			
Pt	M15	M20	M25
0.15	0.28	0.28	0.29
0.25	0.35	0.36	0.36
0.5	0.46	0.48	0.49

BEAM DESIGN

Length of Beam, l		8 m
Load, W		31.25 kN/m
Conc. Grade, fck		35 N/mm ²
Steel Grade, fy		415 N/mm ²
Unit weight of Conc		25 kN/m ³
Dia of bar, φ		16 mm
Factored Load	Wu=1.5*W	46.875 kN
Design Moment	Mu=Wu*l ² /8	375 kNm

$$X_{u,max}/d = 700 / (1100 + 0.87f_y) = 0.479107$$

d/b ratio varies between 1.5 to 3

Take $d/b = 2$

Limiting Moment $M_{u,lim} = R_u * b * d^2$

$$R_u = 0.36f_{ck}(X_{u,max}/d)(1 - 0.416(X_{u,max}/d))$$

$$R_u = 4.833577$$

Now equating $M_{u,lim}$ and M_u we got-

$$d = (2M_u / R_u)^{1/3} = 537.3586 \text{ mm}$$

$$b = d/2 = 268.6793 \text{ mm}$$

For Ast-

$$A_{st} = ((1 - (1 - (4.6M_u / (f_{ck}bd^2)))^{0.5}) / (f_{ck} / 2f_y))$$

$$A_{st} = 2411.358 \text{ mm}^2$$

Provide 16 mm dia bar

$$a_{st} = 3.14 * \phi^2 / 4 = 200.96 \text{ mm}^2$$

$$\text{No of bars} = A_{st} / a_{st} = 12$$

TABLE- Value of $R_{u,lim}$ For balance section			
Grade concrete	Reinforcement		
	Fe 250, lim = 0.1489 fck	Fe 415, lim = 0.1381 fck	Fe 500, lim = 0.1330 fck
M15	2.333	2.071	1.995
M20	2.978	2.761	2.66
M25	3.722	3.452	3.325
M30	4.467	4.142	3.99
M35	5.211	4.833	4.655
M40	5.956	5.523	5.32

Design of Long Column

Axial Load	Pu =				1093.75 kN	
Effective Length-	lex=	5	m		5000 mm	
	ley=	5	m		5000 mm	
b=		0.5	m		500 mm	
D=		0.5	m		500 mm	
Unsupported Length-	lx=	5.5	m		5500 mm	
	ly=	5.5	m		5500 mm	
Concrete Grade, fck=					20 N/mm ²	
Steel Grade, fy=					415 N/mm ²	
	M1x=				12 kNm	
	M2x=				54 kNm	
	M1y=				8 kNm	
	M2y=				52 kNm	
Slenderness Ratio-	lex/D=				10	
	ley/b=				10	
Hence Long Column/Short Column						
Minimum eccentricity-		$ex=(lx/500)+(D/30)$			27.66667 mm	
		$ey=(ly/500)+(b/30)$			27.66667 mm	
Moment due to minimum eccentricity		$Mx=Pu \times ex$			30.26042 kNm	
		$My=Pu \times ey$			30.26042 kNm	
Initial Moment-		$Muix=(0.6M_{2x}-0.4M1x)$			27.6 kNm	Major Axis
		$Muiy=(0.6M_{2y}-0.4M1y)$			28 kNm	Minor Axis
Additional eccentricity-		$eax=D/2000(lex/D)^2$				
		$eay=b/2000(ley/b)^2$				
Moment due to additional eccentricity		$Max=Pu \times eax \times Ka$			110 kNm	
		$May=Pu \times eay \times Ka$			55 kNm	
Compare Initial Moment and Moment due to minimum eccentricity		$Mux,min=$			30.26042 kNm	
		$Muy,min=$			30.26042 kNm	
Final Design Moment		$MDX=Mux,min+Max$			140.2604 kNm	
		$MDY=Muy,min+May$			85.26042 kNm	

Design of Short Column

Axial Load, P=		868.75	kN	
Factored Load, Pu=	1.5P	1303.125	kN	
fck=		20	N/mm ²	
fy=		415	N/mm ²	
Assume Minimum Eccentricity				
		emin<=	0.05D	
So, Pu=		0.4fck*Ac+0.67*fy*As		
Assume, As=	(1% of Ag)=	0.01Ag		
	Ac=	(Ag-As)=	0.99Ag	
So, Ag=		Pu/(0.396fck+6.7*10 ⁻³ fy)		
		Ag=	121781.7	mm ²
Assuming Square Cross-section				
b=d=	Ag ^{0.5}	348.9723	mm	
Area of Longitudinal Steel				
	As=	1217.817	mm ²	
Provide Dia of Bar, φ=				
		16	mm	
	ast=	pi*φ ² /4	200.96	
Nos. of bar				
	Nos.=	Ast/ast	7	
For Lateral ties provide dia of bar				
		6	mm	
Spacing	1	Least Lateral Dimension=	349	mm
	2	16*Dia of main bar=	256	mm
	3	48*Dia of Lateral ties=	288	mm

Hence provide 6mm dia bars @ 250 mm C/C spacing

1. As witnessed along the course of literature review, dynamic evaluation of short columns is based on the parameters including shear ratio that determines the nature of column, energy dissipation capacity, shear resistance and ductility.
2. Lack of these properties in existing structures is dealt with various retrofitting measures including FRP and new Ferro-cement jacketing techniques.
3. To improve hysteretic behavior of column, large bar diameters and higher % of longitudinal reinforcement must be avoided.
4. The transverse reinforcement improves the hysteretic behavior & ductility of the column to a certain extent.
5. Anchored FRP jacket enhances the column shear resistance and energy dissipation capacity.
6. Anchored FRP jacketing increases the confinement of column cross section and as such decreases the strain in the transverse steel.
7. Increasing the % of transverse reinforcement decreases the strain in the FRP jacket along the elevation of the column.
8. Increasing the no. of FRP layers, lessens the strain in both the transverse steel ties & FRP retrofit.
9. Anchored CFRP has been found effective compared to GFRP, as these columns depicted 12% increment in strength.
10. Concluded that there is 20% increase in maximum strength in columns that are retrofitted.
11. Steel jacketing technique has been found effective as it increases a minimum of 20% column strength.
12. Columns strengthened with angles and channel sections with batten plates attained higher energy dissipation capacity and shear strength and failed in a more ductile manner.
13. The use of C-sections with batten plates or plates only for retrofitting has buckling vulnerability due to their thin thickness, hence needs more caution while using it for retrofitting.
14. Buildings on sloping ground experience significant torsion effects when subjected to excitation perpendicular to the direction of sloppy ground.
15. Under excitation along the direction of slope, there is stiffness irregularity because of successive varying heights of column.
16. Tiny columns resist almost the entire storey shear, as they attract more shear forces to it.

17. Concrete jacketing technique has been found an economical method, easy to execute and effective for column strength.
18. Infill panels increase the stiffness of structure and has significant effect on dynamic behavior of structure & in this case less reinforcement is required bare frame.
19. Deflection is more in frame without infill compared to that of infill frame, as its stiffness is more.
20. It has been observed that step back building is more vulnerable during dynamic excitation compared to that of step-back set-back building as in this case torsion moments are less.
21. Columns on the extreme left at ground level has been observed worst effected during dynamic excitation, hence more attention needs to pay to these columns in design and detailing.
22. Empirical formulae for calculating the fundamental period of a setback building are function of building elevation only, which is ambiguous, as its fundamental period tends to change with the change in building configuration, even if its height remains same.
23. Fundamental period of setback buildings has been found less compared to that of regular buildings.
24. Analytical comparison indicates that empirical formula in the IS code provides lower bound value of the fundamental time than obtained by Modal analysis and Rayleigh assessment.
25. Rayleigh method has been found more rational approach for calculating fundamental period as the formula depends on properties of structure and deformational behavior of resisting elements.
26. The variation in fundamental time due to variation in unevenness is found to be more in taller buildings and less for shorter buildings.

1. B.G. Birajdar & S.S. Nalawade. "Seismic analysis of buildings resting on sloping ground", 13th World Conference on Earthquake Engineering: 1-6, August 1-6, 2004.
2. K. Galal, A.Arafa, A. Ghobarah. "Retrofit of RC square short columns", Elsevier, *Engineering structures*: 801-813, 27, 2005.
3. M. Moretti, T.P. Tassios. "Behavior of short columns subjected to cyclic shear displacements: Experimental results", *Engineering structures*: 2018-2029, December 29, 2006.
4. Kheyrodin, A. Kargaran, A. "Seismic Behaviour of Short columns in RC structures". *3rd International conference on concrete & development*: 287-299, December, 2009.
5. Dinesh J.Sabu and Dr. P.S. Pajgade. "Seismic Evaluation of Existing Reinforced Concrete Building", *International Journal of Scientific & Engineering Research*: 1-8, June, 2012.
6. Y Singh et al. "Seismic behavior of buildings located on slopes- An analytical study and some observations from Sikkim earthquake", 15th World Conference on Earthquake Engineering: 101-108, September, 2012.
7. Keyvan Ramin and Foroud Mehrabpour. "Seismic behavior of an RC building resting on a sloping ground", *open journal of civil engineering*: 24-34, November, 2014.
8. Hugo Rodrigues et al. "Seismic behaviour of strengthened RC columns under biaxial loading: An experimental characterization", Elsevier: 393-405, 2015.
9. Mahmoud F. Bilal et al. "Behavior of RC columns strengthened by steel jacket", *Housing & Building Research Center [HBRC]*: 201-212, November, 2015.
10. Vinay Mohan Agarwal and C. Arun. "Comparative study of fundamental period of RC framed building", *International Journal of Engineering, Technology, Management and Applied sciences*: 207-215, April, Volume 3 Issue 4, ISSN 2349-4476, 2015.
11. Rajkumar Vishwakarma, Anubhav Rai. "Analysis of a RCC frame Tall Structure using Staad Pro on Different Seismic Zones Considering Ground Slopes", *International Research Journal of Engineering and Technology (IRJET)*: 1271-1275, Volume: 04, Issue: 03-March -2017.
12. Vissamaneni, S. "Determination of hill slope buildings damage due to earthquake", *International journal of advance research in science and engineering*: 7-16, December, 2014.