SEISMIC ANALYSIS OF RESIDENTIAL BUILDING WITH SHORT COLUMN EFFECT

A Thesis

Submitted in partial fulfillment of the requirements for the award of the degree of

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IN

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With specialization in

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Under the supervision of

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CERTIFICATE

This is to certify that the work which is being presented in the dissertation title **"Seismic Analysis of Residential Building with Short Column Effect"** in partial fulfillment of the requirements for the award of the degree of Master of technology in civil engineering with specialization in "**Structural Engineering**" and submitted in Civil Engineering Department, Jaypee University of Information Technology, Waknaghat is an authentic record of work carried out by **Sristi Gupta (142657)** during a period from July 2015 to June 2016 under the supervision of **Mr. Chandra Pal Gautam**, Assistant Professor, Civil Engineering Department, Jaypee University of Information Technology, Waknaghat. The contents of this dissertation, in full or in parts, have not been submitted to any other institute or university for the award of any degree.

The above statement made is correct to the best of my knowledge.

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ABSTRACT

Short column effect is cause to failure of columns which may result in severe damages or even collapse during earthquakes. The scope of the study is mainly to reveal the effect of short column on the holistic behavior of the buildings. The adverse effect of the short column on the response of buildings is shown in terms of the total load factor and displacement capacity of building. The response of buildings in terms of ground storey displacements is presented in figures and discussed. Various other factors influencing the short column behavior are discussed. A G+4 storey RCC building responses are checked for both the building constructed on plane and at an inclined ground using STAAD pro. Both the static and dynamic analysis are performed on both the buildings. The buildings members are thus compared with various important components of structural analysis such as for shear force, bending moments, displacements, deflections and torsion. Shear wall as a solution to the prevention of short column effect is designed and used at different positions and checked for the changes in terms of the torsion, displacements, shear and frequency of vibration and time period of vibration through mode shapes.

Keywords: Short Column, RCC building, Earthquake, Response spectrum analysis, Shear wall

CONTENTS

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LIST OF ABBREVIATIONS

CFRP	Carbon fiber-reinforced polymer
GFRP	Glass fiber – reinforced polymer
RCC	Reinforced cement concrete
STAAD Pro	Structural analysis and design computer program
N	Number of blows required to achieve a penetration
CSM	Capacity spectrum method
CTSRC	Circular tube steel reinforced column
STSRC	Square tube steel reinforced column
f_{ck}	Characteristic strength of concrete
f_y	Yield strength of steel
Hz	Hertz
cm	Centimeter
kN	Kilonewton
ASCE	American society of civil engineers
FRP	Fiber reinforced polymer
GFRP	Graphite fiber reinforced polymer
CFRP	Carbon fiber reinforced polymer
ETABS	Extended three dimensional analysis of building systems
SAP2000	Structural analysis program
LVDT	Linear voltage displacement transducer
Type S- I	Building at slope with shear wall
Type S-II	Building at slope with bracing
NIT	National Institute of Technology

CHAPTER – 1

INTRODUCTION

1.1 GENERAL

Post-earthquake damages investigation in past and recent earthquakes has illustrated that the building structures are vulnerable to severe damage and/or collapse during moderate to strong ground motion. In this investigation, the results of maximum response in terms of base shear, displacement, time history are evaluated. The aim of this paper is to represent a general study on the short column behavior originated on sloping lots during earthquake referring to hilly areas of zone IV. Reinforced concrete (RC) frame buildings that have columns of different heights within one storey, suffered more damage in the shorter columns as compared to taller columns in the same storey and also the short column could be due to presence of intermediate beams or due to other reasons such as staircase landing slab, half infill wall. The great stiffness of the short columns enables them to absorb large amount of energy.

The seismic analysis of G+4 storey RCC building on varying slope angles is studied and compared with the same on the flat ground. The structural analysis software STAAD Pro V8i is used to study the effect of short column on building performance during earthquake in zone IV. According to the study of past short column behavior results, short column are required to have more resistant sections and are suggested to be reinforced with more bars. It has been observed that the footing columns of shorter height attract more forces, because of a considerable increase in their stiffness, which in turn increases the horizontal force (i.e. shear) and bending moment significantly. In addition more steel should be used as stirrups than as longitudinal bars. Also for existing structures, shear capacity of short columns should be retrofitted by FRP, steel jacket or other materials. North and northeastern parts of India have large scale of hilly regions, which are categorized under seismic zone IV and V. Major seismic events during the past years in hilly areas such as Kangra, 1905 earthquake M8, Kinnaur ,1975 earthquake M6.2 , Uttarkashi uphill's, 1991earthquake M6.6 ,Nepal/Sikkim (India) border area in 2011 earthquake M6.9 ,where there is level difference of sloping lot the short column failure is also seen in damaged buildings which is one of the vertical irregularity. In the present study differently configured R.C framed building are described and studied from structural seismic safety point of view under the action of dead, live and earthquake loads. Plane land in hill is scare and therefore sloping land is being increasingly used for buildings. The unequal height of the columns causes twisting and damage to the short columns of the building. It is because shear force is concentrated in the relatively stiff short columns which fail before the long columns. Short columns demand special attention in building structures. As far as possible, such configuration shall be avoided during plan phase itself; as failure of such columns could be quite brittle in nature hence disastrous. A G+4 storey RCC building responses are checked for both the building constructed on plane and at an inclined ground. Comparison is made by using software such as STAAD. Pro, ETABS and manual calculations on MS-excel. The static and dynamic response for the building on plane and sloping ground are compared and checked for the changes in terms of shear force, bending moments and deflection in same elements at an earthquake shaking of same magnitude. In a static model for both the buildings a comparison is made between the bending moments and shear forces of the elements at same nodes in both the structures. Thereby, concluding the changes in the shear force and bending moments of same elements in structure constructed on plane and sloping ground.

1.1.1 STAAD Pro V8i

STAAD Pro. V8i is a structural analysis and design computer program originally developed by research engineers at Yorba Linda, CA in year 1997. In late 2005, Research Engineers International was bought by Bentley Systems. STAAD Pro. Is one of the most widely used structural analysis and design software. It supports several steel, concrete and timber design codes.

STAAD Pro. V8i is a comprehensive and integrated finite element analysis and design offering program. It is capable of analyzing any structure exposed to static loading, a dynamic response, wind, earthquake and moving loads.

1.1.2 Shear wall

Shear wall may be defined as vertical elements of horizontal force resisting system, composed of braced panels to bear the effect of lateral load acting on a structure. Shear wall is designed using STAAD Pro. V8i to carry the seismic forces in (G+4) residential building safely to the foundation and reduce the effect of lateral forces in short columns at base of the building at sloping ground. It is a reinforced concrete (RC) vertical plate-like RC wall, in addition to slabs, beams and columns. These walls generally start at foundation level and are continuous throughout the building height. The thickness of the wall can vary from as low as 150 mm or as high as 400 mm in high rise buildings. Shear walls are usually provided along both length and width of buildings. These carry earthquake loads downwards to the foundation. Also, shear walls in buildings must be symmetrically located in plan to reduce ill-effects of twist in buildings. These are the various positions of shear walls for which results on displacements, torsion, time of vibration, frequency are studied.

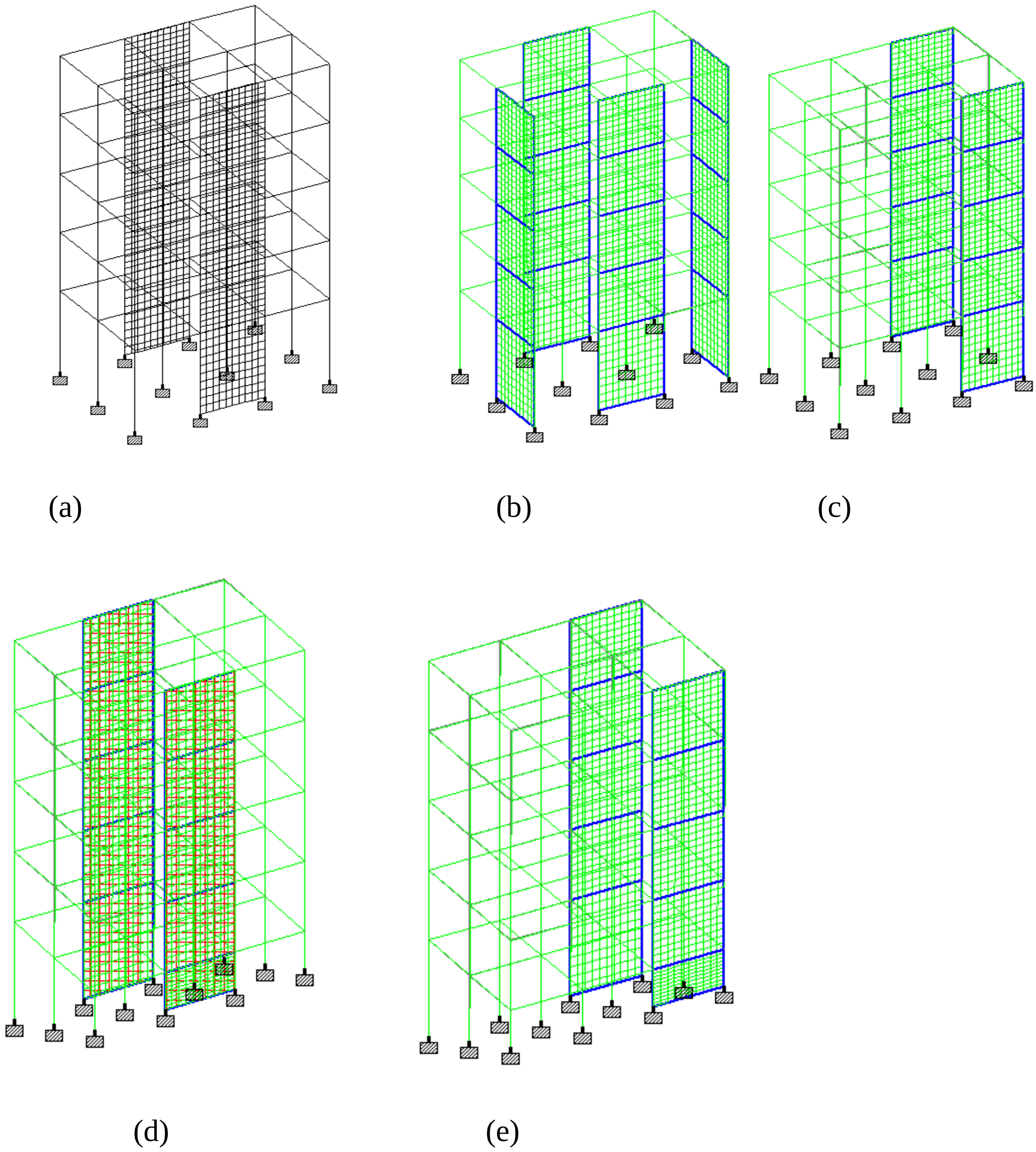


Figure 1.1: Various positions of shear walls in building at slope and plain

1.2 MODELING, ANALYSIS AND DESIGN ASSUMPTIONS

1.2.1 Material and geometrical properties

Following properties of material have been considered in the modeling

I. Density of RCC: 25 KN/m^3

II. Young's modulus of concrete: 5000 $\sqrt{fck}$

III. The foundation depth is considered at 1.5 m below
IV. Beam cross section : 300 x 400 mm
V. Column cross section : 300 x 450 mm , 300 x 300 mm and 450 x 300 mm
VI. Ht. of column (short) : 0.6 m , 2.3 m
VII. Ht. of column (long) : 4 m
VIII. Ground level and the floor height is 4 m.
IX. Thickness of shear wall is 230 mm

1.2.2 Loading conditions

Dead Loads: as per IS: 875 (part-1) 1987

Self-wt. of slab

Slab = 0.15 x 25 = 3.75 kN/m^2 (slab thick. 150 mm

Assumed)

Floor Finish load = 1.47 kN/m^2

Total slab load = 4.75 kN/m^2

Live Loads: as per IS: 875 (Part-2) 1987

Response Spectrum Analysis: as per IS 1893 (Part-1) 2002

Design seismic base shear, $V_b = A_h W$ (Clause 7.5.3)

Design Spectrum $A_h = ZIS_a/2Rg$

Z (zone) = .24 (Clause 6.4.2) Table 2

I (Importance factor) = 1 (for all general buildings)

R (Response reduction factor) = 3 (ordinary moment resisting frame) Table 7

S_a/g = Average response acceleration coefficient

Soil strata = Soft soil as N<10 refers to the soft soil in Clause 6.3.5.2, where N is 13.8, so, have considered a medium soil, the corrected value, for the depth of foundation below ground is 1.5 m <5 ,so, the N values is referred as 15 in Table 1. As per the report (NIT, Kurukshetra), we have N= 13.8, so, for safe designing let us consider the soil be as soft soil.

When calculating the seismic weight of the building, in clause 7.3.1 it is specified that the earthquake forces shall be calculated for the full dead load plus the percentage of imposed load on the floors and the live load on roof is considered to be as zero when design seismic forces are calculated. Table 8 gives percentage of imposed load to be considered in seismic weight calculation, since we have L.L of 2 KN/m^2 on floor, we took 25 percent of imposed load on floors.

Wind Load as per IS (Part-3) 1987

$V_z = V_b \times k_1 \times k_2 \times k_3$

Where, V_b = 39 m/s (for Shimla) in Appendix A of IS 875 (Part-3)

k_1 = factor for maximum design life

Since, the building is a residential building, clause 5.3.1 and Table 1 of IS 875 (Part-3), k_1= 1 (for all general buildings, having return period of 50 years)

K_2= factor of terrain, height and structure clause 5.3.2

Category 3 is adopted as per the note which says this category includes well wooded areas and shrubs, towns and industrial areas full or partially developed.

Clause 5.3.2.2 states variation of wind speed with height for different sizes of structures in different terrains is k_2 dependent. Assuming Class A structures and/or their components such as cladding, glazing, roofing etc, having maximum dimension (greatest horizontal or vertical dimension) less than 20 m. Also, the wind speed till 10 m height of the building is constant and varies after that.

Clause 5.3.3.1 states that the value is taken to be 1 for factor k_3 when slope is less than 3°. When slope is greater than 3° the value is taken to be 1 to 1.36 for slopes greater than 3°.

$P_z = 0.6\, V_z^2$

Where, P_z is the wind pressure

For the slopes greater than 3°, Appendix C of IS 875 (Part-3)

Table 1.1: Value of actual length of the upwind slope in the wind direction as per Appendix C of IS 875 (Part – 3)

Slope	L_e
Slope > 17°	Z/ 0.3

L= actual length of the upwind slope in the wind direction

Z (effective height of the feature) = 150 m (assumed)

ɵ (upwind slope in the wind direction) = $\tan^{-1}(1/2.05) = 26°$

Topography factor, k_3

$K_3 = 1+ Cs$

C= 0.36 (from Appendix C)

To get slope, s, referring figure 15 for hill and ridge

L_e=500 m

X/L_e = -.3

$H/L_e = .04$

Therefore, s= .54

$K_3 = 1+ (0.36*0.54) = 1.1$

1.3 PREVENTION

In buildings with heavy mass, earthquake induced forces are more so one way is to reduce the mass of the building. In modern overall high rise buildings use of light weight prefab panels in place of brick masonry walls is done. Also hollow concrete blocks could be used to achieve the benefit of its light weight. Also irrespective of all these asymmetric, irregular shapes and vertical irregularities in building configuration it can be made safe in earthquake if proper modeling and analysis of concerned structure were carried out. To make short columns more resistant sections and are suggested to be reinforced with more bars, in addition more steel should be used as stirrups. All these points are to be kept in mind at time of construction of a building. For existing structures shear capacity of short columns should be retrofitted by FRP, steel jacket, concrete jacketing or other materials.

1.4 OBJECTIVES

- ❖ Study the behavior of buildings constructed on plane ground and sloping ground for static load and dynamic load by use of appropriate software (STAAD Pro, ETABS)
- ❖ Strengthening solutions for a short column

1.5 SCOPE OF STUDY

The scope of the study is mainly to reveal the effect of short column on the whole behavior of the buildings and ultimately finding the measure of prevention for the buildings which are being in its construction phase or the already built buildings where local repair is required thus, studying various prevention measures to control short column failure can be implemented at the stage of construction of when building is newly constructed. From analysis of structure on software we can measure the behavior of overall building and the elements particularly the short columns and long columns at base of structure.

CHAPTER – 2

LITERATURE REVIEW

2.1 LITERATURE

Harumi Yashiro et al. (1990) studied "Shear failure mechanisms of reinforced concrete short columns", experimental and analytical study on the shear failure mechanisms of reinforced concrete short columns of shear span ratio of 1.5 was carried out. They used 31 specimens for the experimental study and finite element analysis was applied for the analytical study by considering bond-splitting cracks of concrete surrounding tensile steels, the failure processes, until the maximum shear load, were followed. As a result, of the study they carried was the failure processes and stress condition of reinforced concrete short columns of shear span ratio of 1.5 are as follows: first, bending cracks and bending shear cracks occur in the end positions; next, shear cracks in the end positions occur due to bending yielding. Therefore, the solution for a seismic design of reinforced concrete structures such as beams, columns are necessary to be ductile enough to make sure the structures should not fail in brittle state under earthquake shear loading. For all column specimens, the cross section is 25x25 cm, the length is 75 cm, the ratio of shear span to depth is 1.5 and tensile steel ratio (p_t) is 0.96 %. To cut bond between tensile steel bars and concrete, tensile steels are coated with wax at first, coated with grease next and finally covered with soft paper. The region of end portion is 25 cm in length from the member end and the middle portion is the central 25 cm-length. After, the results were found, they concluded that specimens under higher axial load and with lower tie ratio are more brittle than those under lower axial load and with higher tie ratio. From the crack patterns, for specimens in which the bond of tensile steels at the middle portion is cut, shear failure is caused in the middle portion for the case of low tie ratio and bond splitting failure is caused in the middle portion for the case of high tie ratio. Shear load and deformation relationship is not greatly affected by the number of tensile steels but crack patterns are. The results conclude from analytical study, specimens of 15t of axial load, bending shear failure is caused. The variables which are considered to affect the behavior of reinforced concrete short columns subjected to axial load (N) and shear load (Q) are as follows:

(1) axial load:

(N): 15t (24 kg/cm': l/l0F_C),

30t (48 kg/cm’: 2/ 10F_C),

45t (72 kg/cm’: 3/10F_C),

(2) tie ratio:

$\boldsymbol{P_w}$ = 0.56% - @90 mm (2-9φ),

$\boldsymbol{P_w}$= 0.85% - @60 mm (2-9φ),

$\boldsymbol{P_w}$= 1.28% - @60 mm (3-9φ),

(3) bond condition at middle and end portions of the member: ordinary and cut,

(4) number of tensile steel:

3-D16 $\boldsymbol{P_{t.}}$ = 0.96%, ψ= 15.0 cm,

2-D19 $\boldsymbol{P_{t.}}$ = 0.92%, ψ = 12.0cm,

4-D13 and l-D10 $\boldsymbol{P_{t.}}$ = 0.93%, ψ = 20.0cm,

Where, $\boldsymbol{F_c}$ is the compression strength of concrete, $\boldsymbol{P_w}$ is tie ratio, $\boldsymbol{P_{t.}}$ is tensile steel ratio and ψ is total length of tensile steels circumference.

For the specimens of 30t of axial load combined bending shear and bond splitting failure is caused and for specimens of 45t of axial load bond splitting failure is caused. The stresses in ties in the middle portion are rather larger. These values become larger as the axial loads become larger. This corresponds to the occurrence of bond splitting cracks. Under lower axial load, the stresses in tensile steels are larger than those under higher axial load. This corresponds to the deflection. Therefore, they concluded the specimens under higher axial load exhibit a tendency to the bond splitting failure and the specimens under lower axial load exhibit a tendency to bending shear failure. Also, the specimens with lower tie ratio have the tendency of failure in the end portions. Columns with smaller tie ratio are greatly affected by these shear cracks and lose ductility after a few cycles of shear load in this stage. Columns with rich tie reinforcement are not affected so much by these shear cracks and still keep ductile behavior in regular loading cycles. On comparing the stress distributions in tensile steels, for specimen’s bond, which is cut in the end portions, stress gradient in the middle portion is larger and is smaller in the end portions than those for ordinary specimens. For specimen’s bond, which is cut in the middle portion, stress gradient in the middle portion is smaller and that in the end portions is larger. The effects for distribution of tie stresses by bond conditions are not clear for the present analysis performed by them. It is concluded that axial loading, tie ratio and bond conditions have significant effect on the critical deformation of reinforced concrete short columns.

K. Galal et al. (2005) studied “Retrofit of RC square short columns”, they analysed the performance of seven reinforced concrete short columns under lateral cyclic loading and constant axial load. Carbon or glass fiber reinforced polymers were used to strengthen the short columns. It is demonstrated experimentally that it is possible to strengthen the shear resistance of short columns such that a flexural ductile failure occurs by developing plastic hinges at both ends of the column. Anchoring of the fiber wraps to the columns was found to be effective in increasing the shear resistance and energy dissipation capacities of the columns. Low shear span/depth ratio makes a brittle column failure. Three layers of CFRP are applied. The unstrenghtened columns failed in shear were rehabilitated and later exhibited ductile behavior and enhanced shear resistance. The seven specimens had the same column overall dimensions. The specimens were divided into two groups: Group 1 includes SC 1 which is unstrengthened, SC2, SC1R, SC2R and SC1U and are strengthened with high content of transverse reinforcement. In Group 2 includes SC3 and SC3R has low content of transverse reinforcement. The column SC2 was strengthened using 3 layers of CFRP. SC1R included 4 layers of unidirectional glass FRP. SC1U was strengthened by 3 layers of CFRP similar to specimen SC2 but without anchors. In Group 2 (SC 3 and SC3R) had low transverse reinforcement ratio according to 1968 ACI design practice. SC3 was strengthened using 3 layers of CFRP. SC3R was retrofitted using 6 and 3 layers that provided by the 3 CFRP layers of SC3. Using anchored carbon fiber sheets rather than anchored glass fiber sheets for strengthening RC short columns increases both the shear force and the energy dissipating capacity. It also decreases the strains in the steel ties and the FRP along the column height.

M. Moretti and T.P. Tassios (2006) studied “Behavior of short columns subjected to cyclic shear displacements: experimental results”, they studied eight reinforced concrete columns subjected to constant axial load and reversed statically imposed displacement. The parameters tested were ; (a) the shear ratio αs (b) the amount of longitudinal reinforcement (c) the amount of transverse reinforcement (d) the axial load ratio (e) two different main reinforcement layouts (conventional and a combination of conventional and bi- diagonal reinforcement from all the parameters above, they measured the strains of reinforcement (longitudinal and transverse) and of concrete along inclined force paths. They concluded that the columns with low shear ratio had a brittle failure and as a remedial measure bi- diagonal reinforcement is provided in short columns for improved hysteresis behavior and energy characteristics. Specimens with shear ratio αs =1 failed in brittle manner along the main diagonals. The longitudinal reinforcement did not yield at the max. shear force, V_{max}, as is usually the case of columns with $\alpha s < 2$, with the exception of specimen 2 (high axial load ratio $\mu = .60$) in which the longitudinal reinforcement yielded in compression. Specimens 7,8 with $\alpha s = 2$ and $\alpha s = 3$ failed relatively in more ductile manner despite the shear crack near the end sections. Specimen 8, with $\alpha s = 3$ is characterized as normal 'long' column, because as compared to other columns, the onset of cracking along the diagonals ($V = V_{d,cr}$) of column with $\alpha s = 1$ induced non-linearity in distribution of strains along the longitudinal reinforcement, a fact which is not observed in specimen 8, they concluded that (a) the shear strength is larger compared to longer but otherwise identical columns due to low shear value also the mechanism of the diagonal concrete strut, is more activated compared to the truss mechanism of force resistance, a fact which leads to increased diagonal cracking of concrete and enhanced brittleness. Large bars and high percentage of longitudinal reinforcement ought to be avoided. To some extent higher transverse reinforcement improves ductility.

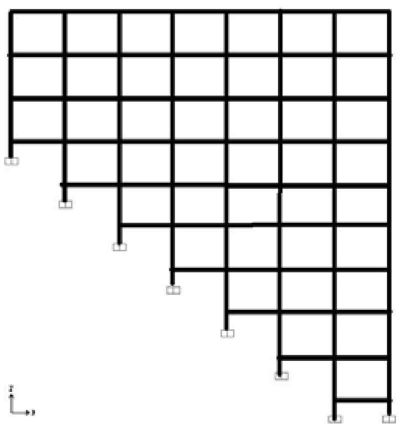
A. Kheyroddin and A. Kargaran (2009) studied "seismic behavior of short columns in RC structures", they have studied the short column phenomenon on sloping ground and duplex structures, storey floors with level difference relative to each other are made in two or different height levels. In this research, at first, seismic behavior of short column phenomenon is determined, then, nonlinear behavior of RC short columns in 4, 8 and 10 storey structures with storey level difference is investigated. Short columns and mentioned structures are analysed under the earthquake record of Elcentro with different peak ground acceleration with IDARC software which is nonlinear dynamic analysis program. In this investigation, the results of maximum response, base shear, global damage index and displacement time history and effect of short column in structural failure is evaluated. In this research, seismic behavior of short column in 3 duplex structures has been surveyed that have height level difference 1.6 meter. Plan of all 3 structures is same and have variable height and include 4, 8 and 10 storey. In results of Elcentro earthquake the concluded that the seismic degree damage of short column in floor building in all structures increase of structures height especially in upper storeys damage index of short column has been increased. Out of all the other storeys in 8 storey structures has the lowest failure in short column. The displacement history of last short column in 4, 8 and 10 storey structures is more than first short column in all structures by increasing PGA. Displacement time history of first and medium short column in 4 storey structures and last short column in 10 storey Structures is high relative to other structures. Investigation of Shear force history concluded that the average of shear force history in first short column in 4 storeys structure and medium short column in 8 storeys structure and last short column in 10 storeys structures has the most amount than other column. Damage index concluded that the part of last short column and down part of first short column in 8 and 10 storeys structure has more damage.

Xuhong Zhou and Jiepeng Liu (2010) studied "Seismic behavior and strength of tubed steel reinforced concrete SRC short columns", they tested eight specimens subjected to combined constant axial compression and lateral cyclic load. Out of which three were circular tube SRC and three were square tube SRC and two common SRC columns were taken for comparison. On comparison, they found that the steel prevented the shear failure of the concrete more effectively in the circular columns from that in the square ones. They also mentioned that shear connector studs should be used in CTSRC and STSRC short columns to prevent bond failure between concrete and flanges of the steel section. Tubed SRC short columns exhibit higher lateral load strength, displacement ductility, more stable hysteresis loops and greater energy dissipation ability than common SRC short columns in respect of the effective confinement of the thin tube to the core concrete.

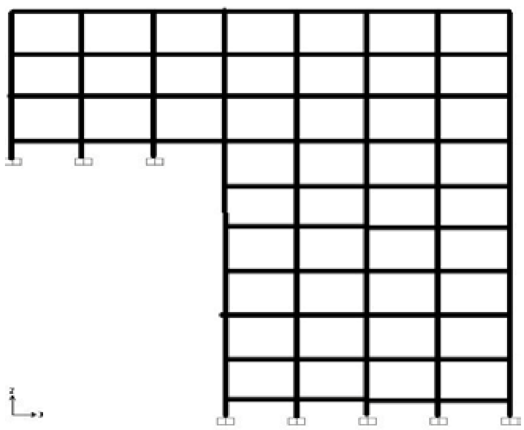
Y. Singh et al. (2012) studied "Seismic behavior of buildings located on slopes- An analytical study and observations from Sikkim earthquake of September 18, 2011", they concluded the response of setback buildings at varying slopes. They have performed an analytical study to investigate the peculiar seismic behavior of hill buildings. Dynamic response of hill buildings is compared with that of regular buildings on flat ground in terms of fundamental period of vibration, pattern of inter-storey drift, column shear, and plastic hinge formation pattern. The seismic behavior of two typical configurations of hill buildings is investigated using linear and non-linear time history analysis. The irregular variation of stiffness and mass in vertical as well as horizontal directions held these buildings to significant torsional response. Unequal height of columns within a storey, results in drastic variation in stiffness of columns of the same storey. They have compared 4 types of buildings, a 9 storey RC frame building with two different hill configurations as shown in figure 2.6.

The building has 6 storeys below the road level and three storeys above the road level. To compare the behavior with regular buildings, two regular buildings resting on flat ground with 3 and 9 storeys and having the same plan considered. The first building named 'Type S-I' is stepping back at every floor level on a slope of about 45̊, up to six storeys and has three storeys above the road level. Second building named ‘Type S-II’ is stepping back at sixth floor level only and has three storeys above road level. The 9 and 3 storeyed regular buildings on flat ground are labeled as ‘Type P-III’ and ‘Type P-IV’. They have concluded that due to irregularity of configurations, mass participation in fundamental mode in case of buildings on slopes is much lower than the regular buildings. Also, it is observed that no significant lateral displacement occurs below the sixth floor level (road level) in Type S-I building, due to high rigidity of short columns.

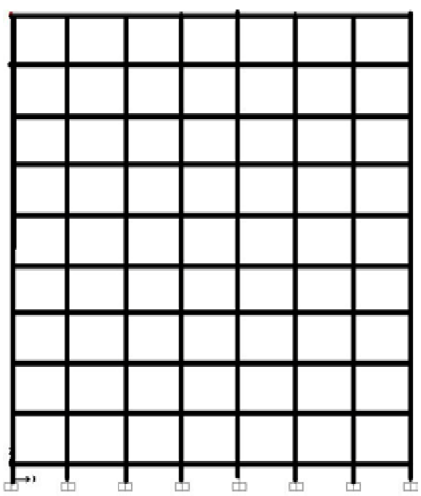
The deflected shape of the Type S-II building is similar to a vertical cantilever propped at sixth floor level. In Type S-II configuration, the columns in the bottom storey and storeys immediately above and below the road level (sixth and seventh) storey are subjected to maximum forces. In Type-S-I building, torsion is observed in all the storeys whereas in Type S-II, torsion is observed in top three storeys only.



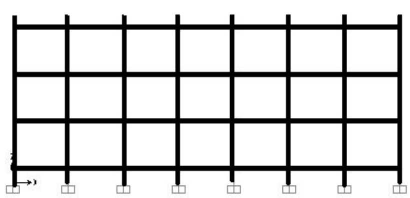
(a) Type S-I



(b) Type S-II



(c) Type P-III



(d) Type P-IV

Figure 2.6: (a) Type S-I ; (b) Type S-II ; (c) Type P-III ; (d) Type P-IV

They have said that, the behavior of hill buildings differs significantly from the regular buildings on flat ground. The hill buildings are subjected to significant torsional effects under cross- slope excitation. Under along-slope excitation, the varying heights of columns cause stiffness irregularity, and the short columns resist almost the entire storey shear. The linear and non- linear dynamic analysis shows that the storey at road level, in case of downhill buildings, is most susceptible to damage.

Dinesh J. Sabu and Dr. P.S. Pajgade (2012) studied "Seismic Evaluation of Existing Reinforced concrete building and applied Response Spectrum analysis procedure", for the evaluation of existing design of a reinforced concrete bare frame, frame with infill and frame with infill and has proved infill a better performance frame as the displacement of such frame is less comparatively and has illustrated it from a bracing system in a frame. After performing the analysis reinforcement required in each format is determined and retrofitting is suggested accordingly. He gave concrete jacketing as a method of retrofitting. Analysis is done using software STAAD Pro, it is concluded that the frame with infill gave much better result in terms of maximum displacement of the building and stiffness. Also, if actual reinforcement is more than reinforcement required in the brick infill and soil interaction effect than there is no need to retrofit the actual section, it is sufficient to carry the seismic forces.

A.B.M.A Kaish et al. (2012) studied "Improved ferrocement jacketing for restrengthening of square RC short column", they have proposed improved techniques over conventional jacketing . Three new square ferrocement jacketing techniques such as square jacketing with single layer wire mesh and rounded column corners (RSL); square jacketing using single layer wire mesh with shear keys at the center of each face of column (SKSL) and square jacketing with single layer wire mesh and two extra layers mesh at each corner (SLTL) are considered for this purpose. Test results and crack patterns tested specimens show that all three improved square ferrocement jacketing. Confinement with the ferrocement encasement improves the ultimate load carrying capacity and the axial and lateral deflections of square RC column. Type SLTL jacketing shows highest load carrying capacity as well as good ductility properties over all other improved types of jacketing as well as non- jacketed specimens under concrete mode of loading whereas type RSL jacketing shows best performance under eccentric mode of loading.

Sandeep Vissamaneni (2014) studied “Determination of hill slope buildings damage due to earthquake”, he concluded that during earthquake when buildings are subjected to earthquake loads and lateral loads, they result in torsional response. A parametric study is carried out on buildings considered, by using ground motion records for IDA (Incremental Dynamic Analysis). The short column is stiffer compared to the tall column and it attracts large earthquake force. Stiffness of a column means resistance to deformation, the larger is the stiffness, larger is the force required to deform it. A study is carried out changing the position of shear wall and varying column height of ground storey columns. By trial and error sizes placing shear wall on flexible side (bottom of hill across slope direction) achieved balanced stiffness of flexible side with rigid side to avoid torsion, or to make ($\Delta max/\Delta avg$ minimum) has studied at each storey of the models. By placing the shear wall at top of the hill or bottom of the hill along the slope direction or providing by bracings reduce column forces that resting on sloping ground. Remedies for such buildings are given i.e. by providing shear wall and bracings in step back buildings on slopes as shown He has compared the dynamic characteristics of hill buildings on slope and plane lots. The torsional response is due to irregular variation of stiffness and mass in vertical as well as horizontal directions, results in center of mass and center of stiffness of a storey not coinciding and not being on a vertical line for different floors.

Majorly, due to unequal heights of columns which result in variation of stiffness. The buildings having step back and set back step back studied for their behavior during earthquake. The buildings were classified as Type S-I and Type S-II. Type S- I building, ground columns height varied from 1m to 3m also the shear wall is introduced to see the change in readings as a remedy measure. In Type S-II , again the shear wall position was changed to various positions and results were noticed. In Type S-I building torsion is observed in all the storeys, whereas in Type S-II building torsion is observed in top three storeys (above road level) only. He has said that by providing bracings, ground supported columns in Type S-I building, and was relieved from excessive shear. However, the building were relieved from excessive shear but could not be torsionally balanced. The various positions plotted in the model are as follows to increase the strength and stiffness of the particular building built on a sloping lot. Whereas increasing the height of ground supported columns to 2.5 m and placing the shear wall on the downhill side of Type S-I and Type S-II building resulted in torsionally balanced configuration and shear force in ground supported columns reduced to reasonable level.as shown below:

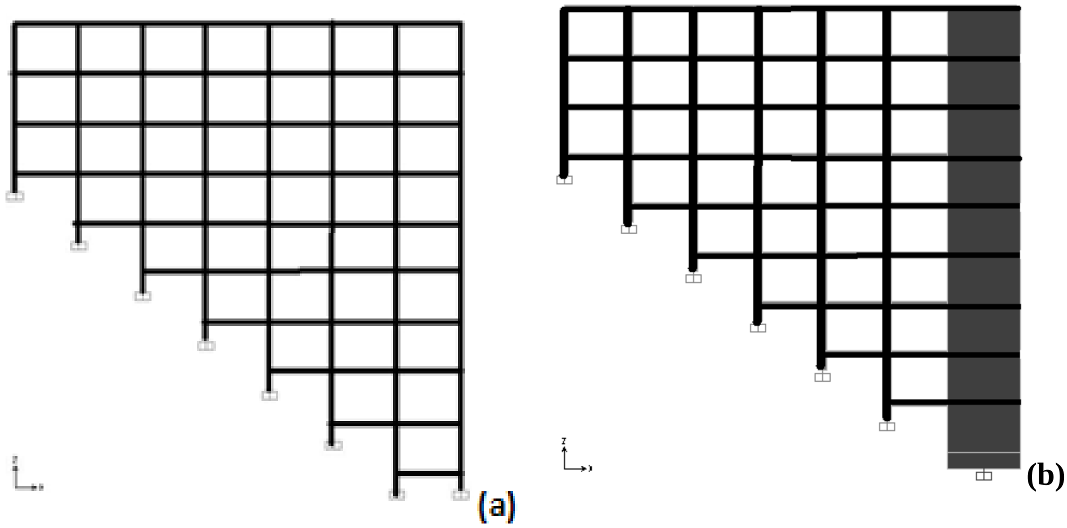


Figure 2.1: Type S-I building (a) 1 *m* long ground supported columns;
(b) Shear wall at bottom of hill along-slope direction

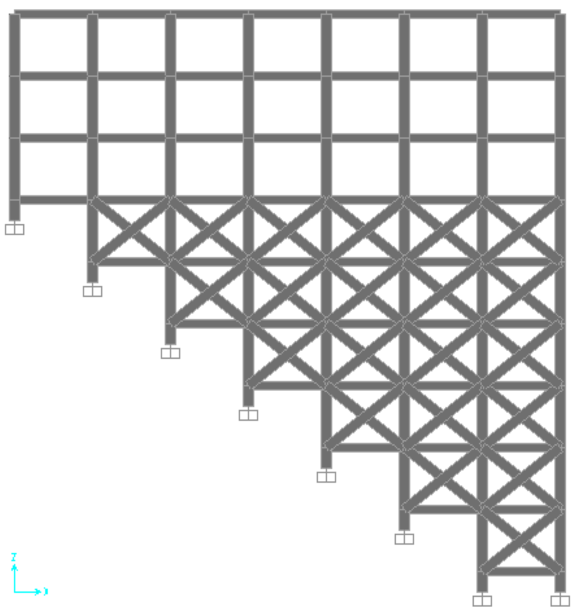
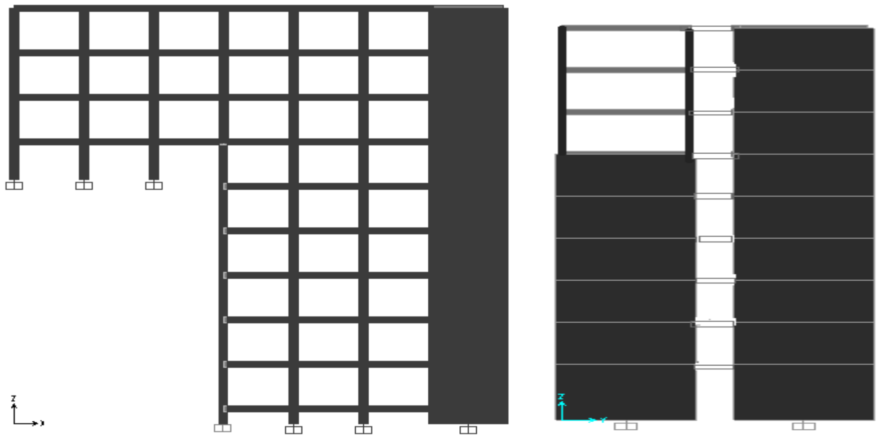


Figure 2.2: Type S-I building with bracing as a remedial measure on hill slope

Using the shear wall at full height of building also gives a good result in terms of stability of building, showing lesser displacement and a good energy dissipation characteristic, overall good behavior at slopes.



(a) (b)

Figure 2.3: Type S-II configuration building (a) Shear wall at bottom of hill along-slope direction; (b) Shear wall at down-hill end in cross-slope direction

Keyvan Ramin and Foroud Mehrabpour (2014) studied seismic performance of buildings resting on sloping ground using STAAD Pro V8i. Also Sap2000 software had been used to show displacement of floors is greater for a flat lot building than a sloping lot building. Seismic behavior of buildings constructed on slopes. The chief role of this column is to transfer the inertia force originated from earthquake to columns. The main part of these forces is exerted on the short column since the stiffness varies from column to column. Thus, the short column shows an enormous potential for serious damage by earthquake in the case of an inappropriate design. Poor behavior of short columns is due to the fact that in an earthquake, a tall column and short column of same cross section move horizontally by same amount which can be seen from the figure below. The study of behavior of building on a sloping ground (inclination say 7˚ and 15˚) under earthquake forces. The comparison of sloping ground and plane ground building is done . G+5 storey building is taken and same load is applied. The response of the building frames is studied for useful interpretation of the results.

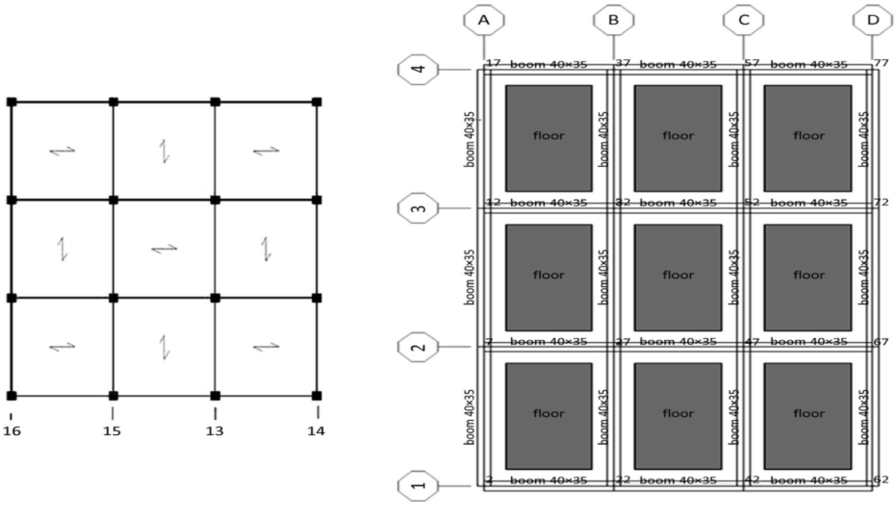
V.Varalakshmi et al. (2014) studied "Analysis and Design of G+5 Residential building", which was constructed at Kukatpally, Hyderabad, India is designed (Slabs, Beams, Columns and Footings) using Auto CAD software. The loads are calculated namely the dead loads, which depend on unit weight of materials used (concrete, brick) and the live loads, which according to the code IS: 456-2000 and HYSD BARS FE415 as per IS: 1786 -1985. Safe bearing capacity of the soil is adopted as 350 KN/m^2 at a depth of 6 ft. and same soil should extent 1.5 times the width of footing below the base of footing. Footings are designed based on the safe bearing capacity of soil. For designing of columns and beams, it is necessary to know the moments they are subjected to. For this purpose frame analysis is done by limit state method. Designing of the slab depends upon whether it is a one-way or a two- way slab, the end conditions and the loading. From the slabs, the loads are transferred to the beam. Thereafter, the loads (mainly shear) from the beams are taken by the columns. Finally, the section must be checked for all the floor components with regard to strength and serviceability.

Hugo Rodrigues et al. (2015) studied "Seismic behavior of strengthened RC columns under biaxial loading" and concluded that the performance of 9 strengthened columns including one unstrengthened, it is kept original for the comparison of result. The column specimens were subjected to several loading condition. Cyclic displacements were imposed at the top of the column with steadily increasing displacement levels. The columns were retrofitted using CFRP plates and steel plates bonded with epoxy resin, retrofitted results are comprised with the original ones which were not retrofitted, in terms of shear drift, degradation, ductility and energy dissipation and the adopted load paths were diagonal and diamond. The experimental campaign was carried out on 9 RC columns with same geometries and reinforcement subjected to similar biaxial horizontal displacement paths with equal constant axial load. Their focus was on the influence of different strengthening strategies on the behavior of columns under certain load conditions. They found that the initial stiffness was not significantly affected. The strengthened columns present higher strength capacity of about 12% (in particular in columns under diamond biaxial horizontal load path). The strength degradation in strengthened columns starts for higher levels of drift demand. The strengthened columns tend to have lower levels of cumulative dissipated energy when compared with the original solution for the same drift levels. This fact is related with the concentration of damage in the base of columns. The columns submitted to the diamond horizontal load path, and also with the CFRP strengthening show higher energy dissipation capacity when compared with the diagonal load path.

Hugo Rodrigues et al. (2015) studied "Seismic rehabilitation of RC columns under biaxial loading", he has done an experimental characterization is done in order to improve the ductility and / or strength characteristics and it was obtained through concrete ductility with efficient jacketing or increasing the amount of longitudinal or transverse steel . The results are presented in terms of shear-drift, stiffness degradation, ductility and energy dissipation. The retrofitted results are comprised with the original ones, in terms of shear drift, degradation, ductility and energy dissipation. This campaign composed of 6 RC columns that 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, four of the tested columns were repaired and submitted to different retrofit strategies in order to replace the original characteristics, and mainly to provide the columns a good ductility capacity to respond well under cyclic loads. The retrofit techniques used in the present work were: increasing the number of stirrups, steel packet jacketing and carbon fiber reinforced polymer (CFRP) sheets and plate jacketing. After the retrofit these columns were biaxial tested. The results are presented in terms of shear- drift, shear drift envelopes, ductility, energy dissipation and stiffness degradation and are compared with the results of the original one. The experimental results on the column retrofitting show that the initial stiffness is typically lower and softening starts for higher drift demands. Also retrofitted columns tend to have an increase of the maximum strength around 20% maximum. The damage in original column is more pronounced when compared to the retrofitted for the same drift demand.

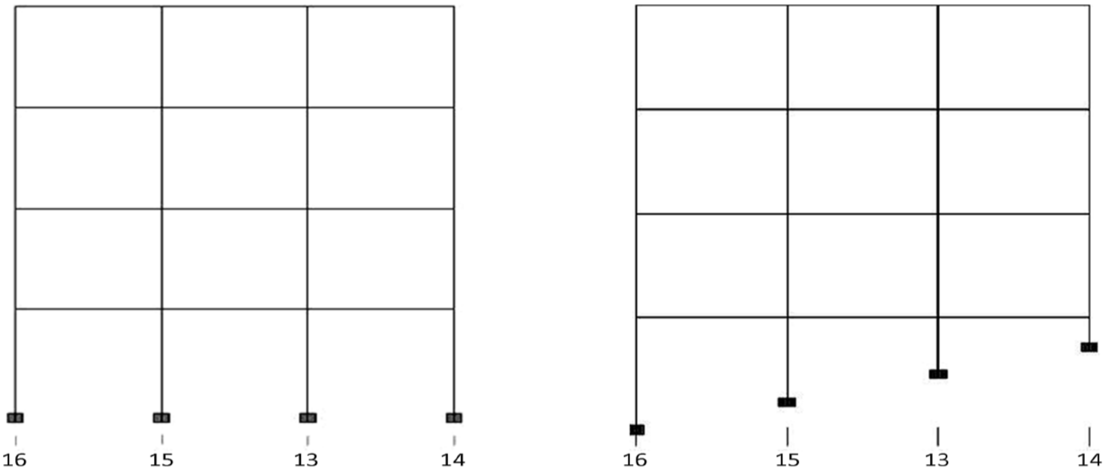
Various steps involved in the comparison of horizontal forces

Step 1: selection of building geometry and seismic zone Chamba, Kullu, Kangra, Una, Hamirpur, Mandi, and Bilaspur Districts lie in Zone V. The remaining districts of Lahual and Spiti, Kinnaur, Shimla, Solan and Sirmaur lie in Zone IV. So, let us take seismic zone IV as per IS code 1893 (Part-1):2002 for which zone factor Z is .24.Step 2: Load combinations are formed Types of Primary Loads and Load Combinations: The structural systems are subjected to Primary Load Cases as per IS 875:1987 and IS 1893: 2002. Six Primary load case and thirteen load combinations are used for analysis. Step 3: Modeling of building frames using STAAD Pro Software .Step 4: Study of structural behavior in terms of bending moments and horizontal footings, axial force and bending moments in columns. They compared two four-storey reinforced concrete moment resisting frame (MRF) buildings with medium deformability, one of which is located on a flat lot and the other one is on a lot sloped by 20 degrees . Flexural frame for building on slope and plane ground is shown in Figure 3, 4 shown in next page.



(a) **(b)**

Figure 2.4: Plans (along X- coordinate) of the studied structures.



(a) (b)

Figure 2.5: Frames of the studied structures (a) The flexural frame for the structure on flat lot and (b) The flexural frame for the structure on sloping lot

Mahmoud F. Belal et al. (2015) studied "Behavior of reinforced concrete columns strengthened by steel jacket", he has performed an experimental and analytical method to show the appropriate results, RC columns often need strengthening to increase their capacity to sustain the applied load. This research investigates the behavior of 7 RC columns strengthened using steel jacket having dimension of 200x200 mm in cross-section with 1200mm height, which also concludes that the L/d ratio is less than 12 so, are termed as short columns. The specimens were divided into two groups: the first group includes two control specimens without strengthening and second group includes five specimens strengthened with different steel jacket configurations. Vertical steel elements (angles, channel and plates) were chosen to have the same total horizontal cross sectional area. The specimens were placed in the testing machine between the jack head and the steel frame. The strain gauges, load cell and linear voltage displacement transducer (LVDT) were all connected to the data acquisition system attached to the computer. The load was monitored by a load cell of 5000 kN capacity and transmitted to the reinforced concrete column through steel plates to provide uniform bearing surfaces. Behavior and failure load of the strengthened columns were experimentally investigated on seven specimens divided into two un-strengthened specimen and five strengthened ones. A finite element model was developed to study the behavior of these columns. The model was verified and tuned using the experimental results. The research demonstrated that the different strengthening schemes have a major impact on the column capacity. The size of the batten plates had significant effect on the failure load for specimens strengthened with angles, whereas the number of batten plates was more effective for specimens strengthened with C- channels. Then by using finite element (F.E) package ANSYS 12.0 their behavior was investigated analyzed and verified. Experimental results stated that modes of failure and failure loads varied depending on the configuration of steel jacket as well as its arrangement. Because the strengthening elements covered most of specimen, it was not possible to observe either the initial cracks or the cracking load for specimens. So, only failure load was recorded. Failure load is considered the maximum recorded load during testing and at which specimen could not carry any extra load. The results showed 20% of the minimum increase in the column capacity, also the failure turned from brittle to ductile with steel jacket. Specimens strengthened with angles or channel sections with batten plates recorded a higher failure load than that with strengthened plates. And the simulation of strengthened RC columns 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, for strengthened models, F.E package ANSYS 12.0 overestimated failure loads compared to experimental results.

Vinay Mohan Agrawal and Arun C (2015) studied “comparative study on fundamental period of RC framed building”, they concluded it is difficult to quantify the irregularity in a setback building with any single parameter such as overall building height. Fundamental period of all the selected building models were estimated and Empirical equations given in the design codes and the results were critically analysed. The fundamental time period is calculated as per given in available design codes for earthquake resistant building including

IS 1893:2002, ASCE 7:2010, Euro code 8 or New Zealand code of practice, recommends an empirical formula for the determination of Fundamental time period of building. The following formulas were checked and the results were calculated and the comparison of fundamental period of setback buildings with that obtained from equation based on IS 1893:2002 was carried out and is presented, it stated that empirical formula in IS Code provides the lower- bound of the fundamental periods obtained from Modal Analysis and Raleigh method. Therefore, IS 1893:2002 always gives conservative estimates of fundamental period of setback buildings with 6 to 30 storeys. It was also concluded that the Raleigh method underestimates the fundamental periods of setback buildings slightly which is also conservative for the selected buildings. ASCE 7:2010 does not consider the height of the building but it considers only the number of storeys of the building and this approach is most conservative among other code equations. This study indicates that there is very poor correlation between fundamental periods of three dimensional buildings with any of the parameters used to define the setback irregularity by the previous studies or design codes.

Vrushali S. Kalsait and Dr. Valsson Varghese (2015) studied “Design of earthquake resistant multistoried building on a sloping ground”, The purpose of their paper was to perform linear static analysis of medium height RC buildings and investigate the changes in structural behavior due to consideration of sloping ground. They have studied the buildings at varying slopes and countered their behavior in terms of mode shapes, fundamental time of vibration, lateral displacements, moments in column and axial shear force. The response of G+15storey building at varying slopes of 0°, 7.5°, 15°, 22° was studied and as per his conclusions made from STAAD Pro V8i, the displacement of building resting on sloping ground have more lateral displacement compared to the buildings on plain ground, the critical axial force in columns increases as slope increases. He found that critical bending moments increased on 22° slope than 7.5° slope and 15° slope ground. The calculated frequency decreases as slope of ground increases whereas time period increases as slope of ground increases. Also, the steel quantity on sloping ground is more than on plain ground for same cross section of column and beam, thus, it is concluded that cross section required more steel on sloping ground to make earthquake resistant structures.

2.2 SUMMARY OF LITERATURE REVIEW

Various software such as ETABS, STAAD Pro, ANSYS 12.0, SAP2000 are used for the analysis of combination of loads on which column would fail. The shear wall and cross bracings can be effective if used in a step back or step back set back buildings for better performance of a building in hilly areas having short column. For a better performance of a short column various parameters such as shear ratio α, energy dissipating property, ductility, shear resistance has to be checked and various preventive measures should be applied to repair and retrofitting of existing structures. Short columns can be made safe at the time of new construction and can be retrofitted in existing buildings by means of FRP, new ferrocement jacketing techniques. Cost effective measures can also be understood from the papers read so far. Short column with higher shear ratio value has high energy dissipation capacity and more ductile failure when subjected to cyclic displacements. To some extent higher transverse reinforcement improves ductility. More layers of FRP composites applied on a column with proper anchorage bars improves the strain and minor cracking also leading to a flexural ductile failure. CFRP gives more lateral force capacity and high strain bearing capacity as compared to the GFRP. The CFRP strengthened columns present higher strength capacity of about 12% (in particular in columns under diamond biaxial horizontal load path). The strength degradation in strengthened columns starts for higher levels of drift demand. For columns rehabilitation with CFRP, The experimental results on the column retrofitting show that the initial stiffness is typically lower and softening starts for higher drift demands. Also retrofitted columns tend to have an increase of the maximum strength around 20% maximum.

The damage in original column is more pronounced when compared to the retrofitted for the same drift demand. Seismic behavior of short column in 3 duplex structures has been surveyed that have height level difference 1.6 meter. Plan of all 3 structures is same and have variable height and include 4, 8 and 10 storey using earthquake record of Elcentro with different peak ground acceleration with IDARC software which is nonlinear dynamic analysis program. Comparison of two four-storey reinforced concrete moment resisting frame (MRF) buildings with medium deformability, one of which is located on a flat lot and the other one is on a lot sloped by 20 degrees. The RC columns strengthened with steel jacket results showed 20% of the minimum increase in the column capacity, also the failure turned from brittle to ductile with steel jacket. Specimens strengthened with angles or channel sections with batten plates recorded a higher failure load than that with strengthened plates. The behavior of hill buildings differs significantly from the regular buildings on flat ground. The hill buildings are subjected to significant torsional effects under cross- slope excitation. Under along-slope excitation, the varying heights of columns cause stiffness irregularity, and the short columns resist almost the entire storey shear. The linear and non- linear dynamic analysis shows that the storey at road level, in case of downhill buildings, is most susceptible to damage. . The buildings with same maximum height and same maximum width may have different period depending on the amount of irregularity present in the setback buildings. The variation of the fundamental periods due to variation in irregularity is found to be more for taller buildings and comparatively less for shorter buildings. This observation is valid for the periods calculated from both modal analysis and Raleigh method are quite smaller. Comparison results for (G+15) building is done for different slope and same soil condition on varying slopes of 0°, 7.5°, 15°, 22° concluded that the displacement of building resting on sloping ground have more lateral displacement compared to the buildings on plain ground, the critical axial force in columns increases as slope increases. He found that critical bending moments increased on 22° slope than 7.5° slope and 15° slope ground. The calculated frequency decreases as slope of ground increases whereas time period increases as slope of ground increases. Also, the steel quantity on sloping ground is more than on plain ground for same cross section of column and beam, thus, it is concluded that cross section required more steel on sloping ground to make earthquake resistant structures. It is also concluded from the literature that axial loading, tie ratio and bond conditions have significant effect on the critical deformation of reinforced concrete short columns.

2.3 RESEARCH GAP

In prior study research based on the effect of short column in a building and various prevention factors are discussed. The main point of interest draws upon these factors, the various prevention methods adopted can be compared and it can be found that which among them is cost effective. Moreover, study of failure loads can make it easy for engineers to design a short column loading and thus the whole building components. Despite the increasing attention is given to various prevention methods mainly FRP, there could be other retrofitting criterions which may be more cost effective and also the prevention can be taken at time of construction itself. Also, ferrocement jacketing is also found to be an easy, effective and cost effective method in improving the condition of a degraded column. The scope of the study is mainly to reveal the effect of short column on the whole behavior of the buildings.

CHAPTER – 3

RESEARCH METHODOLOGY

3.1 WORK PLAN AND METHODS USED

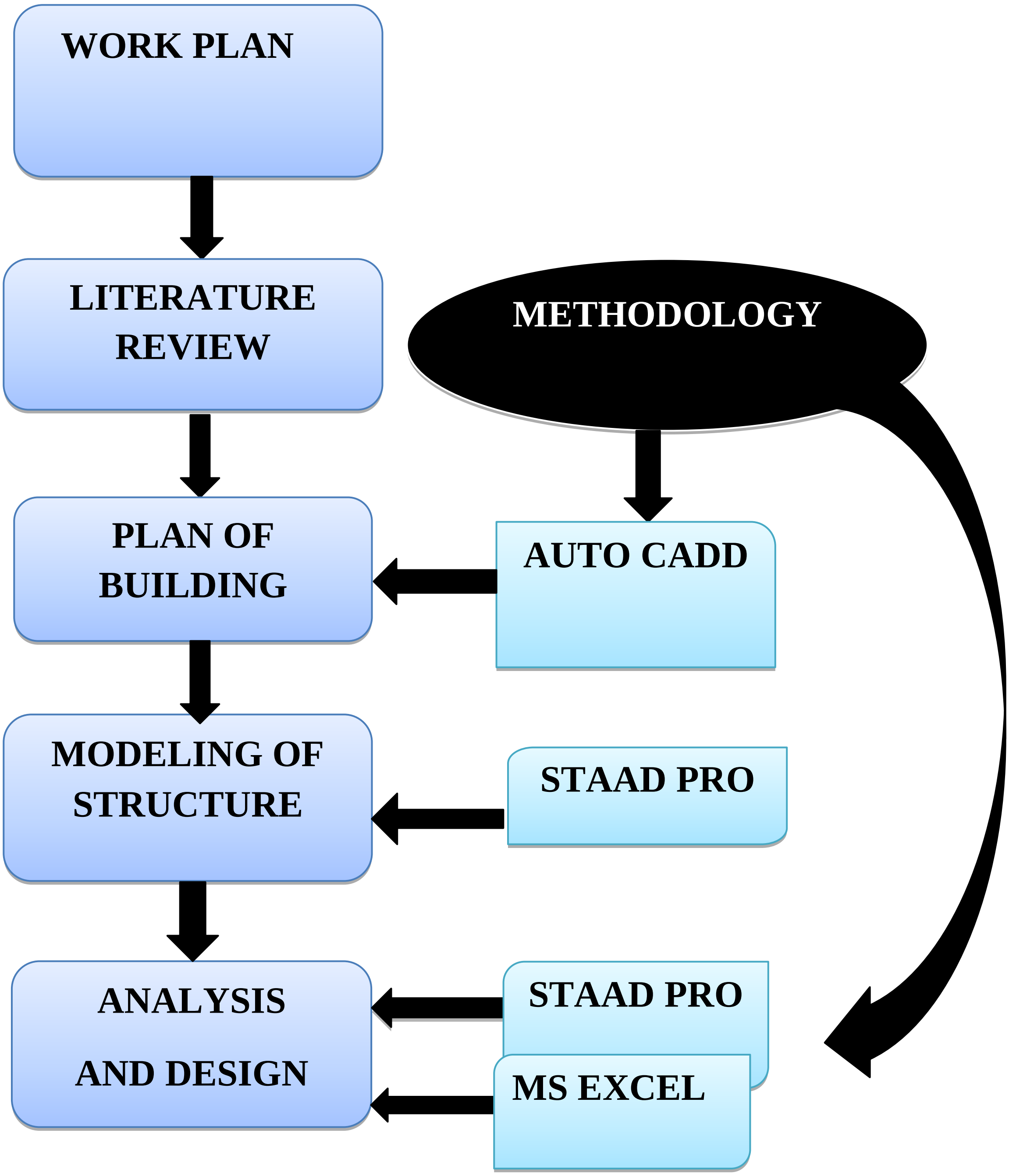


Figure 3.1: Work Plan and methods used

3.2 PLAN OF BUILDINGS

3.2.1 Plan on AUTO CAD

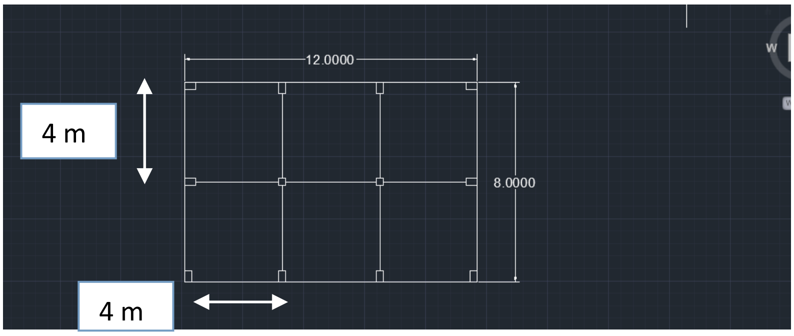
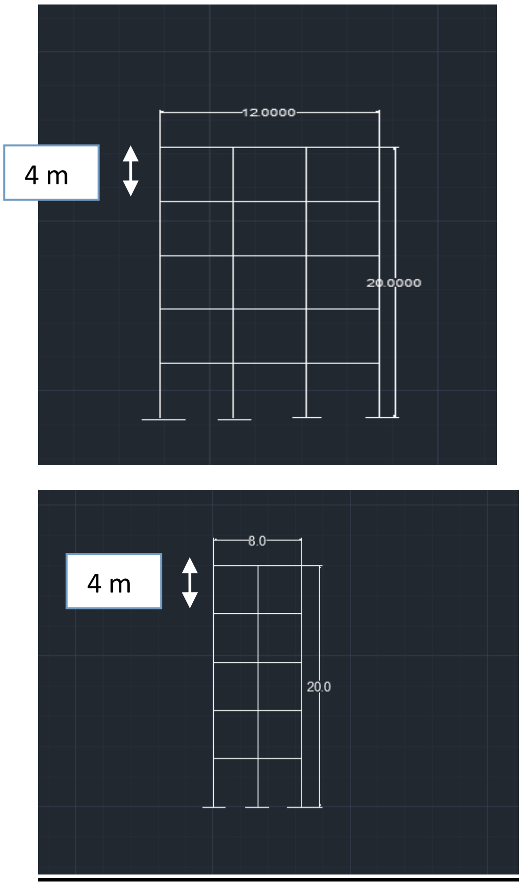


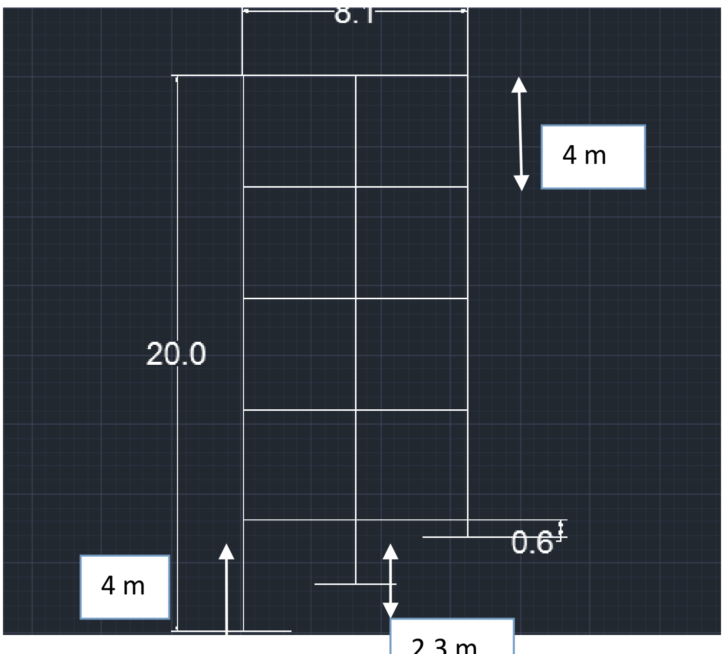
Figure 3.2: Plan for both the structures



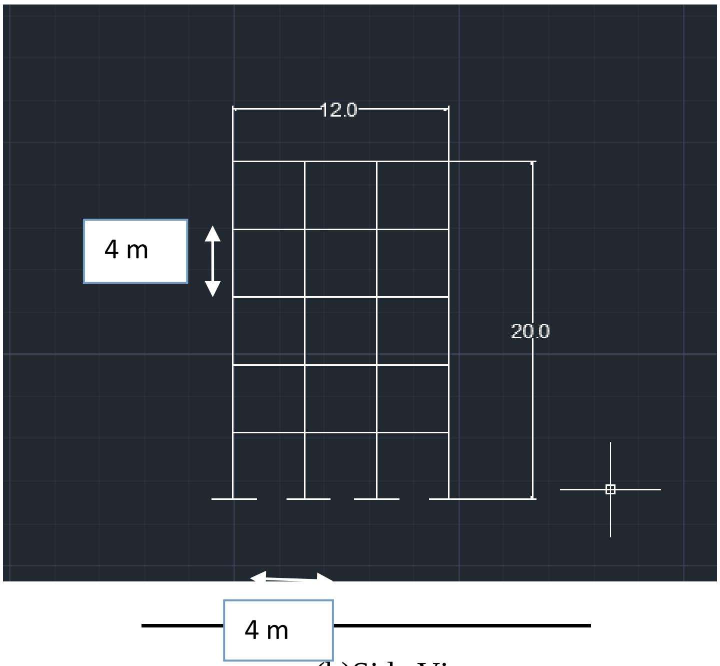
(a) Front view

(b) Side view

Figure 3.3: (a) Front view and (b) Side view of building resting on Plain



(a) Front view



(b)Side View

Figure 3.4: (a) Front view and (b) Side view of building resting on sloping ground

3.2.2 STAAD Pro. V8i Reports

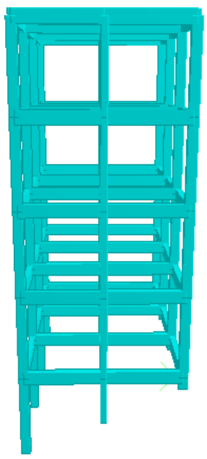


Figure 3.5 Model of RC frame on sloping ground

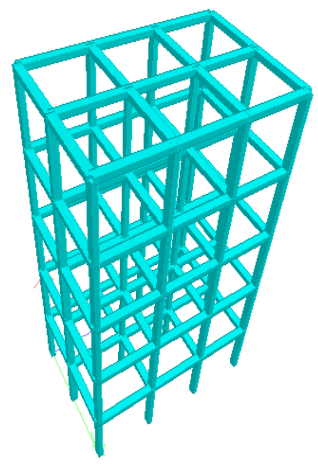
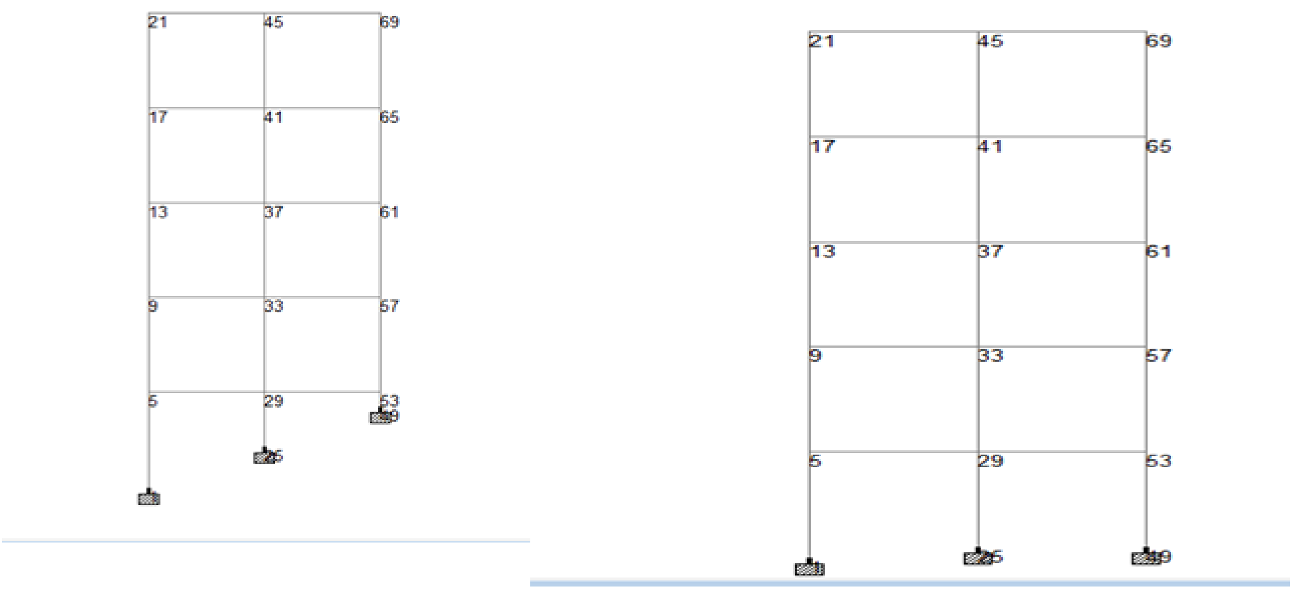


Figure 3.6: Model of RC frame on plain ground



(a) **(b)**

Figure 3.7: Position of nodes in both the buildings (a) building on sloping ground and

(a) building on plain ground

3.2.3 Design sheets on MS-Excel

3.2.3.1 Design of a two-way continous slab

Room measuring = (4X4)m from inside breadth of slab= 1000 mm

Live load = 1.5 KN/m^2 (from IS 875 part II)

Use M30 and Fe415 steel .

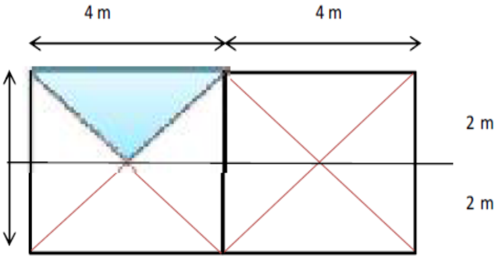
Design constants and limiting depth of N.A

fck = 30 N/mm^2

fy = 415 N/mm^2

for Fe 415, xu,max/d= 0.479107

Ru = 4.143066



Load distribution in a square slab

Area of shaded region = 1/2 (b x h)

Computation of Loading and Bending moment

From deflection point of view , L/d = 20 for simply supported slab
26 for continous supported slab

taking, L/d = 23

Let pt be .2 % for under-reinforced section,

Modification factor, from Fig. 4 of IS - 456:2000 1.68

L/d = 38.64

d = 103.5197 mm

nominal cover = 20 mm

dia = 8 mm

D = 127.5197 mm

However, this panel is a two way slab (and that too a square one), a slightly lower value of d may be assumed

Taking, D = 150 mm for the purpose of calculating the self wt.

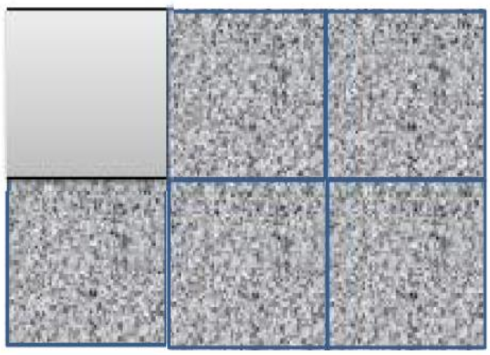
Therefore D = 0.15 m

1. Self wt. = 3750 N/m^2
2. Floor finish = 1470 N/m^2 (for floor finish of 100mm thickness in IS: 875 Part I Table 2)
3. Live load = 1.5 KN/m^2 1500 N/mm^2 m

Total load = 6720 N/m^2

Hence, Wu = 10080 N/m^2

Taking an effective d of = 150 mm, 0.15 m



ly = 4 m

lx = 4 m

Hence, r = ly/lx 1

α_x = 0.035 Table 26 IS 456: 2000

α_y = 0.035 Two adjacent edges discontinous

Mux = 5644.8 N-m

Muy = 5644.8 N-m

For short span width of middle strip = 3 m

width of edge strip = 0.5 m

For long span, width of middle strip = 3 m

width of edge strip = 0.5 m

Computation of effective depth and total depth

d = $(Mu,max/Ru,b)^{.5}$

d = 36.91164 mm

However, from deflection point of view, provide D= 150 mm

provided, d = 126 mm

for shorter span, d= 118 mm

Computation of steel reinforcement

Since d provided is more then d required from B.M, we have an under-reinforced section for which,

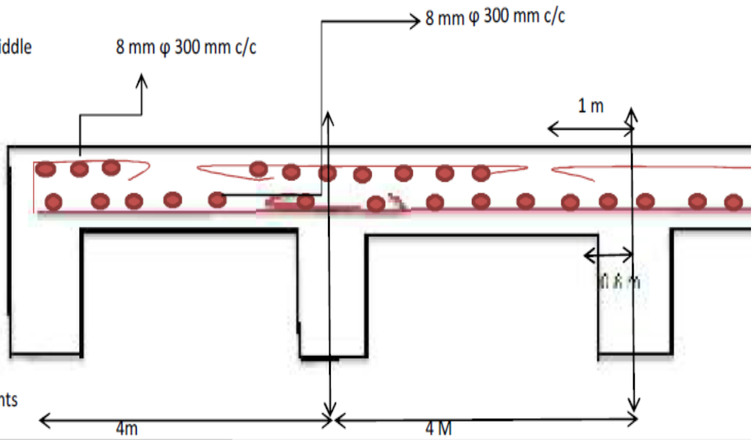
Ast, B = 125.8844 mm^2

spacing,S= 399.0964 mm

Hence, provide 8mm dia bars @ 399.0964 mm c/c for the middle strip of width 3 m
Bend half the bars up at a distance = 0.15 lx
600 mm
from the centre of the support, or at a distance of 680 mm from the edge of the slab
(assuming a bearing of 160 mm on the wall)
Available length of the bars at the top = 460 mm
from the centre of the support, assuming bending of bars at 45 degrees. The length more than 0.1 lx,
400 mm required by the code.
Hence, length of top bars from the edge of slab = 540 mm.
Edge strip is of length = 0.6 m
Reinforcement in the edge strip , 1.2 x D= 180 mm^2
Hence, spacing of 8 mm φ bars , s= 279.1111 mm c/c

Computation of reinforcement for long span

Ast, y = 134.6878 mm^2
using , 8 mm φ bars, s = 373.0107 mm
Hence provide 8 mm φ bars, spacing 373.0107 mm c/c in the middle
strip up at a distance of .15ly = 600 mm
from the centre of the support or 680 mm from the edge of the slab.
Available length of bars of top = 460 mm from edge of strip
Ast, = 180 mm^2
giving rise to spacing, s = 279.1111 mm c/c



Check for Shear and development length in short span

S.F at long edges = (Wu*lx*r)/(2+r) where, r = lx/lr
So, S.F = 13440 N
Therefore, nominal shear stress at long edges, 0.106667
At the long edges the dia of bars shoud be so restricted that the following requirements
is satisfied : 1.3*Mu1/Vu + Lo ≥ Ld

Ast1 at support of short span 279.1111 mm^2
Xu = 9.3308395 mm
M1u = 12306243 N-mm
Vu = 13440 N
Ld = 376 mm
Assuming support width is = 160 mm
and a side cover of, x' 20 mm
Providing no hooks, Lo = ls/2 -x'
Lo = 60 mm
comparing the results, 1250.336 ≥ 376
Hence, code requirements are satisfied.
Also, the code requires that the positive reinforcement should extent into
the support atleast by Ld/3
Hence, minimum support width = Ld/3 + x' = 145.3333 mm
In, our case the support width is 160 mm

Check for shear and development length in long span

S.F at short edges = (1/3 Wu lx), 13440 N
Nominal shear stress = 0.113898 (safe)

Ast = 180 mm^2
Xu = 6.0175 mm
M1u = 7506016 N-mm
Vu = 13440 N
Lo = 60 mm
Ld = 376 mm
comparing, 786.0284 ≥ 376
So, code requirements are satisfied

Torsional reinforcements at corners:

Size of torsional mesh, Lx/5 0.8 m

from the centre of support or 0.88 m from the edge of the slab

Area of torsional reinforcement, = 3/4 of Astx

Ast, T = 94.413286 mm^2

Spacing of dia 8 mm bar is, s = 532.1285 mm

DESIGN OF TWO WAY CONTINOUS SLAB

Room measuring = (4 x 4)m from inside breadth of slab= 1000 mm

Live load = 1.5 KN/m^2 (from IS 875 part II)

Use M20 and Fe415 steel . simply supported on all four edges

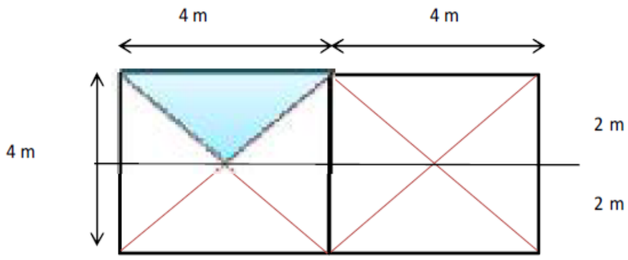
Design constants and limiting depth of N.A

fck = 30 N/mm^2

fy = 415 N/mm^2

for Fe 415, xu,max/d= 0.4791075

Ru = 4.1430657



Load distribution in a square slab

Area of shaded region = 1/2 (b x h)

Computation of Loading and Bending moment

From deflection point of view , L/d = 20 for simply supported slab

26 for continous supported slab

taking, L/d = 23

Let pt be .2 % for under-reinforced section,

Modification factor, from Fig. 4 of IS - 456:2000 1.68

L/d = 38.64

d = 129.39959 mm

nominal cover = 20 mm

dia = 8 mm

D = 153.39959 mm

However, this panel is a two way slab (and that too a square one), a slightly lower value of d may be assumed

Taking, D = 150 mm for the purpose of calculating the self wt.

Therefore D = 0.15 m

1. Self wt. = 3750 N/m^2
2. Floor finish = 1470 N/m^2 (for floor finish of 100mm thickness in IS: 875 Part I)
3. Live load = 1.5 KN/m^2 1500 N/mm^2 m

Total load = 6720 N/m^2

Hence, Wu = 10080 N/m^2

Taking an effective d of = 150 mm, 0.15 m

ly = 4 m

lx = 4 m

Hence, r = ly/lx 1

α_x = 0.037 Table 26 IS 456: 2000

α_y = 0.037 One long edge discontinous

Mux = 5967.36 N-m

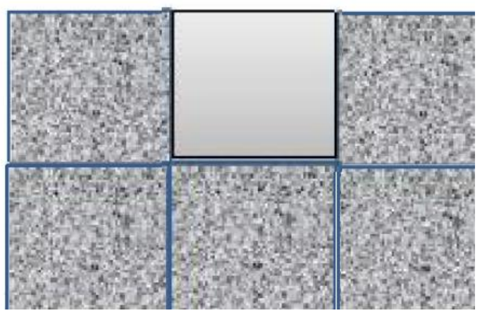
Muy = 5967.36 N-m

For short span width of middle strip = 3 m

width of edge strip = 0.5 m

For long span, width of middle strip = 3 m

width of edge strip = 0.5 m



Computation of effective depth and total depth

d = $(Mu,max/Ru,b)^{.5}$

d = 37.951611 mm

However, from deflection point of view, provide D= 150 mm

provided, d = 126 mm

for shorter span, d= 118 mm

Computation of steel reinforcement

Since d provided is more then d required from B.M, we have an under-reinforced section for which,

Ast, B = 133.18604 mm^2

spacing,S= 377.21672 mm

Hence, provide 8mm dia bars @ 377.2167 mm c/c for the middle strip of width 3 m

Bend half the bars up at a distance = 0.15 lx

600 mm

from the centre of the support, or at a distance of 680 mm from the edge of the slab

(assuming a bearing of 160 mm on the wall)

Available length of the bars at the top = 460 mm

from the centre of the support, assuming bending of bars at 45 degrees. The length more than 0.1 lx,

400 mm required by the code.

Hence, length of top bars from the edge of slab = 540 mm.

Edge strip is of length = 0.6 m

Reinforcement in the edge strip , 1.2 x D= 180 mm^2

Hence, spacing of 8 mm φ bars , s= 279.1111 mm c/c

Computation of reinforcement for long span

Ast, y = 142.51719 mm^2

using , 8 mm φ bars, s = 352.5189 mm

Hence provide 8 mmφ bars, spacing 352.5189 mm c/c in the middle

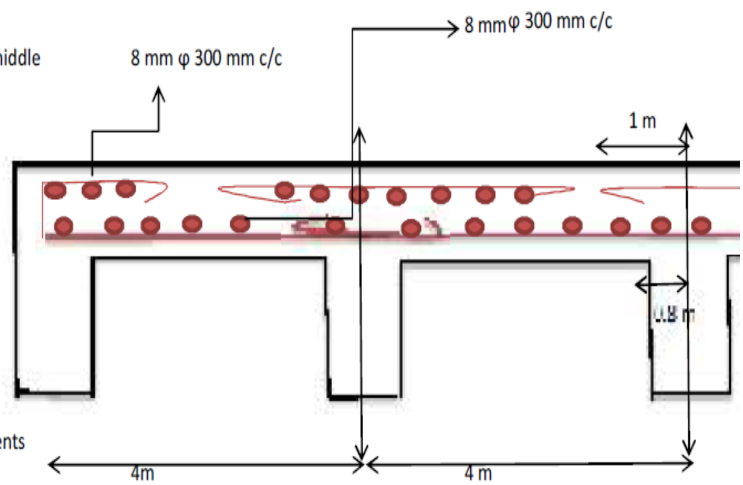
strip up at a distance of .15ly = 600 mm

from the centre of the support or 680 mm from the edge of the slab.

Available length of bars of top = 460 mm from edge of strip

Ast, = 180 mm^2

giving rise to spacing, s = 279.1111 mm c/c



Check for Shear and development length in short span

S.F at long edges = (Wu*lx*r)/(2+r) where, r = lx/lr

So, S.F = 13440 N

Therefore, nominal shear stress at long edges, 0.106667

At the long edges the dia of bars shoud be so restricted that the following requirements is satisfied : 1.3*Mu1/Vu + Lo ≥ Ld

Ast1 at support of short span 279.1111 mm^2

Xu = 9.33084 mm

M1u = 12306243 N-mm

Vu = 13440 N

Ld = 376 mm

Assuming support width is = 160 mm

and a side cover of, x' 20 mm

Providing no hooks, Lo = ls/2 -x'

Lo = 60 mm

comparing the results, 1250.336 ≥ 376

Hence, code requirements are satisfied.

Also, the code requires that the positive reinforcement should extent into the support atleast by Ld/3

Hence, minimum support width = Ld/3 + x' = 145.3333 mm
In, our case the support width is 160 mm

Check for shear and development length in long span

S.F at short edges = (1/3 Wu lx), 13440 N
Nominal shear stress = 0.113898 (safe)

Ast = 180 mm^2
Xu = 6.0175 mm
M1u = 7506016.3 N-mm
Vu = 13440
Lo = 60 mm
Ld = 376 mm
comparing, 786.02837 ≥ 376
So, code requirements are satisfied

Torsional reinforcements at corners:

Size of torsional mesh, Lx/5 0.8 m
from the centre of support or 0.88 m from the edge of the slab
Area of torsional reinforcement, = 3/4 of Astx
Ast, T = 99.88953 mm^2
Spacing of dia 8 mm bar is, s = 502.9556 mm

3.2.3.2 Design of continous beam

Live Load on beam =		80.64 KN		
fck	=	30 N/mm^2		
fy	=	415 N/mm^2		
Span	=	4 m	in mm =	4000
no. of spans	=	2		
cover to tension steel=		50 mm		

Cross- sectional dimensions

As the continous beam supports heavy loads,
the span/ depth ratio is assumed to be 10

effective depth , d = span/10

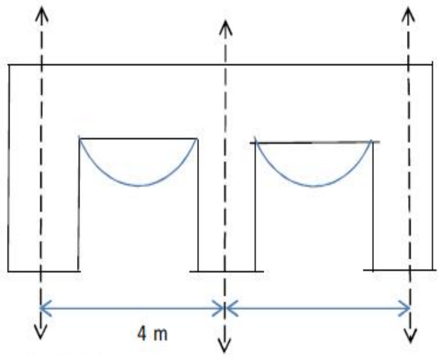
	d	=	400 mm
Taking,	d	=	400 mm
	D	=	450 mm
	b	=	300 mm

Loads

Self weight of beam= 4.5 KN
Live load = 20.16 KN

Beanding moments and shear forces

Reffering to the bending moment and shear force coefficients
(Table 12 & 13) Using IS 456:2000



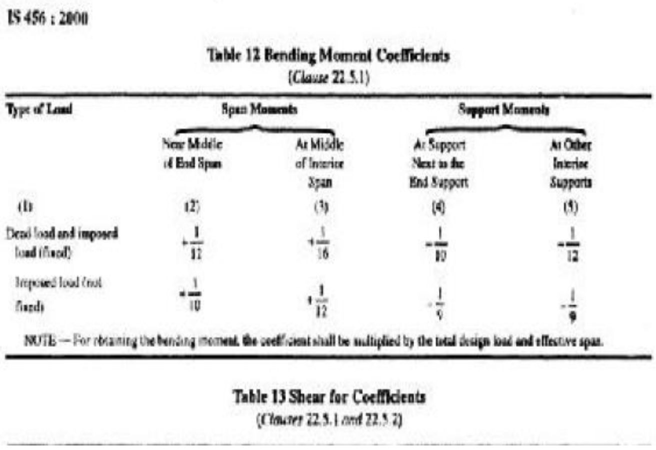

IS 456 : 2000

Table 12 Bending Moment Coefficients
(*Clause* 22.5.1)

Type of Load	Span Moments		Support Moments	
	Near Middle of End Span	At Middle of Interior Span	At Support Next to the End Support	At Other Interior Supports
(1)	(2)	(3)	(4)	(5)
Dead load and imposed load (fixed)	$+\frac{1}{12}$	$+\frac{1}{16}$	$-\frac{1}{10}$	$-\frac{1}{12}$
Imposed load (not fixed)	$+\frac{1}{10}$	$+\frac{1}{12}$	$-\frac{1}{9}$	$-\frac{1}{9}$

NOTE — For obtaining the bending moment, the coefficient shall be multiplied by the total design load and effective span.

Table 13 Shear for Coefficients
(*Clauses* 22.5.1 *and* 22.5.2)

Negative bending moment at interior support

Mu (-ve) = 43.04 KN-m

Positive bending moment at centre of span

Mu (+ve) = 38.256 KN-m

also, **positive bending moment at middle of interior span**

Mu (+ve) = 31.38 KN-m

Maximum shear force at the support section is given by

Vu = 59.184 KN

Limiting moment of resistance

Mu,lim = 198.72 KN-m

Since, Mu < Mu,lim, the section is under- reinforced

Main Reinforcement

Ast = 309.18898 mm^2

Therefore, using 4 bars of dia 10 mm

Ast = 314 mm^2

at supports, for positive bending bending moment the area of steel required

Now, for positive bending moments, the area of steel required,

Ast = 273.659611 mm^2

Hence, provide 4 bars of dia 10 mm on the tension face at mid span

Type of Load	At End Support	At Support Next to the End Support		At All Other Interior Supports
		Outer Side	Inner Side	
(1)	(2)	(3)	(4)	(5)
Dead load and imposed load (fixed)	0.4	0.6	0.55	0.5
Imposed load (not fixed)	0.45	0.6	0.6	0.6

NOTE — For obtaining the shear force, the coefficient shall be multiplied by the total design load.

Table 19 Design Shear Strength of Concrete, τ_c, N/mm^2

(*Clauses* 40.2.1, 40.2.2, 40.3, 40.4, 40.5.3, 41.3.2, 41.3.3 *and* 41.4.3)

$100\frac{A_s}{bd}$	Concrete Grade					
	M 15	M 20	M 25	M 30	M 35	M 40 and above
(1)	(2)	(3)	(4)	(5)	(6)	(7)
≤ 0.15	0.28	0.28	0.29	0.29	0.29	0.30
0.25	0.35	0.36	0.36	0.37	0.37	0.38
0.50	0.46	0.48	0.49	0.50	0.50	0.51
0.75	0.54	0.56	0.57	0.59	0.59	0.60
1.00	0.60	0.62	0.64	0.66	0.67	0.68
1.25	0.64	0.67	0.70	0.71	0.73	0.74
1.50	0.68	0.72	0.74	0.76	0.78	0.79
1.75	0.71	0.75	0.78	0.80	0.82	0.84
2.00	0.71	0.79	0.82	0.84	0.86	0.88
2.25	0.71	0.81	0.85	0.88	0.90	0.92
2.50	0.71	0.82	0.88	0.91	0.93	0.95
2.75	0.71	0.82	0.90	0.94	0.96	0.98
3.00 and above	0.71	0.82	0.92	0.96	0.99	1.01

NOTE — The term A_s is the area of longitudinal tension reinforcement which continues at least one effective depth beyond the section being considered except at support where the full area of tension reinforcement may be used provided the detailing conforms to 26.2.2 and 26.2.3

Shear Reinforcement

τ_v = 0.4932 N/mm^2

pt (%) = 0.2616667

Refer to Table 19 of IS 456:2000

For M30

τ_c = 0.376064

Since, τ_v > τ_c

shear reinforcements need to be designed

When τv > τc, min. shear reinforcement in the form of stirrups shall be provided,

V_{us} = 14.05632

Using, 8 mm dia 2 legged stirrups, the spacing near support ,

Sv = 3 mm

which is greater to, 300 mm

so, provide 300 mm spacing between the stirrups

Check for deflection control

Pt (%) = 0.2616667

Fig., 4 at page 38, in IS 456:2000

Modification factor = 1.24

Neclecting hanging bars, Kc = 1

Kf = 1

(L/d)max = 32.24

(L/d)actual = 10

10 < 32.24

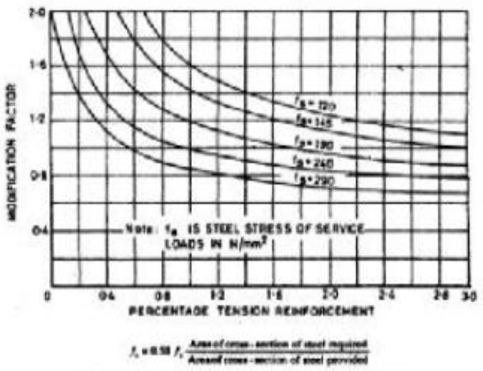


FIG. 4 Modification Factor for Tension Reinforcement

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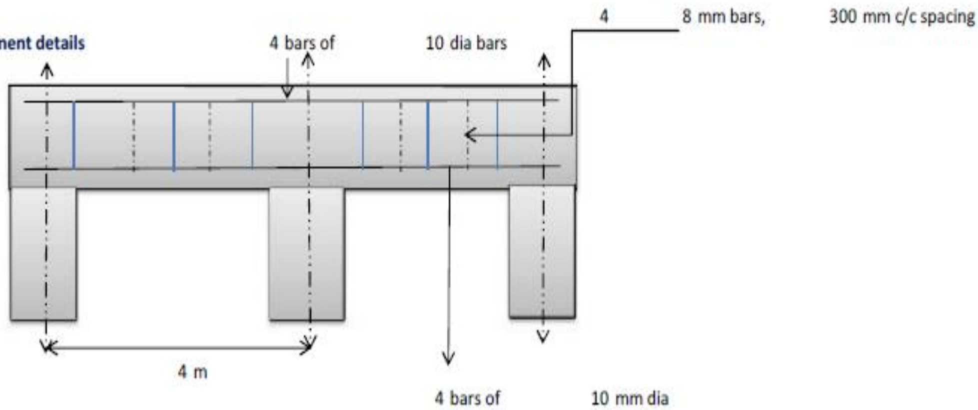
Reinforcement details



3.2.3.3 Design of long columns subjected to combined axial load and biaxial bending moment

P_u = 285.5 KN

$l_{ex} = l_{ey}$ 4 m 4000 mm

b = 300 mm

D = 450 mm

Concrete grade,fck= 30 N/mm²

fy = 415 N/mm²

lex/d = 13.33333 > 12 mm

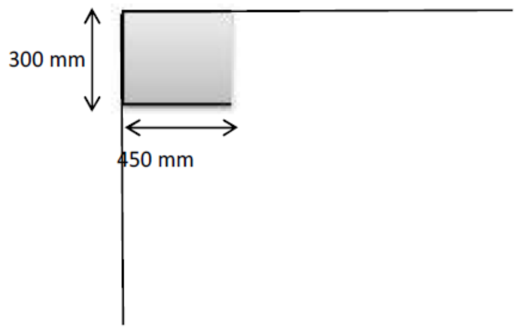
hence, long column

e_x = D/2000 (lex/D)^2

17.77778 mm

e_x = 0.039506

e_{ay} = 0.088889



Calculation of addition moments

M_{ax} = $P_u.e_x$ 3.383704 KN-m

M_{ay} = $P_u.e_y$ 11.42 KN-m

Assuming a trial percentage of 2%

A_g = 135000 mm^2

from chart 63 of SP- 16

P_{uz}/A_g = 19.5

P_{uz} = 2632500 N 2632.5 KN

Calculation of P_b

Assuming 25 mm dia bars with 40 mm clear cover

cover = 40 mm

dia = 25 mm

effective cover = 52.5 mm

$(d'/D)_{x\text{-axis}}$ = 0.175

chart or table for d'/D = 0.15 will be used

$(d'/D)_{y\text{-axis}}$ = 0.116667

chart or table for d'/D = 0.1 will be used

from Table 60 of SP- 16 (page- 171)

$(P_b)_{x\text{-axis}}$ = k1 + k2 *(p/fck)

k1 = 0.184

k2 = 0.203

P_{bx} = 800010 N 80.001 KN

For d'/d 0.15 , fy = 415 N/mm^2 with steel equally distributed on all four sides

$(P_b)_{y\text{-axis}}$ = 800010 N 80.001 KN

kx = 0.919491

ky = 0.919491

Reduction of additional moments

The modified values moments are calculated after multiplying the additional moments by coefficients calculated above

M'ax = Max *kx 3.11128529 KN-m

M'ay = May *ky 10.5005879 KN-m

Design of column (short column)

column ht =	1.2 m	fck =	30 N/mm^2
b=	300 mm	fy =	415 N/mm^2
D=	300 mm		
Ag =	90000 mm^2		
Total load on 1beam	181.53 KN	Pu = Slab wt. + Beam wt.	
Pu=	734.12 kN	slab wt=	161.28 KN
Pu = 0.4*fck*Ac+.67 *fy*As		Beam load=	20.25 KN
Ac = Ag-As		Total load=	181.53 KN

As = 1192.484 mm^2

e min = 12.4 < 20 mm

Min. area of longitudinal reinforcement = 0.9 %

810 mm^2

Hence , As = 1192.484 mm^2

No. of 20 mm bars 3.79771992

Therefore provide, 4 number of bars

As = 1256 mm^2

Using , 8 mm dia ties, spacing is least of the following:

1. Least lateral tie dimension = 300 mm

160 mm

300 mm

Design of uni-axial bending on column

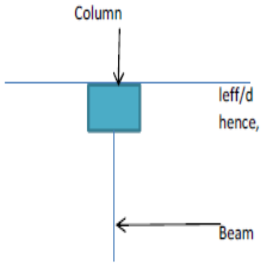
b = 300 mm e = 23 mm

D = 450 mm fck = 30 N/mm^2

carry ultimate load = 141.12 KN wt. from slab = 60.48 KN

effective height = 4 m wt. from beam = 80.64 KN

4000 mm Total load = 141.12 KN



Long column

effective length/least lateral dimension >12

leff/d = 13.33333 > 12

hence, a long column

Assuming effective cover of = 50 mm

e_x = 18 < 20 mm

ey = 23 > 20 mm

Calculating Pu/fck*b*D = 0.03484444

Mu = Pu.e 3.24576 KN-m

Calculating Mu/fck*b*D^2 = 1.7809E-06

Reffering chart 32 (for fy 415, d'/D = 0.111111

p/fck = 0.022

p = 0.66 %

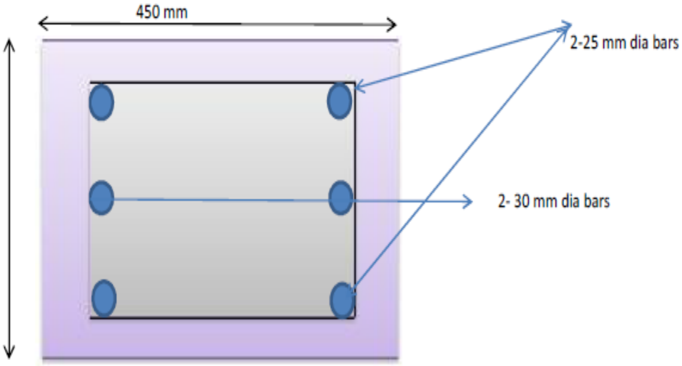
Calculating Asc

Asc = p*bD/100

Asc = 891 mm^2

Using 4-25 mm dia bars and 2-30mm dia bars

Asc = 3375.5 mm^2



Design of lateral ties

Dia of ties = 6 mm

pitch of ties = 400 mm > 300 mm

Hence, provide 6 mm ties @ 250 mm c/c

3.2.3.4 Design of footing (isolated)

Isolated rectangular footing of uniform thickness for R.C column

Design a rectangular isolated footing of uniform thickness for R.C. column bearing a vertical load of 942.008 KN & having a basr size of b= 300 mm & d= 450 mm

The safe bearing capacity of soil may be taken as 116.69 kN/m^2

Use M 20 concrete

Fe 415 steel

Design constants: for fy = 415

fck = 20

$\frac{x_{u,max}}{d}$ 0.479107 R_u = 2.762044

Size of footing: W = 942.008 kN

Let W be equal to 10 % W = 94.2008 KN

A = 8.880014 mm^2

Let ratio of B to L = 0.666667

0.666667 *L*L = 8.880014

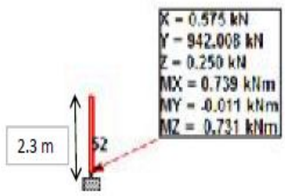
L^2 = 13.32002 m

L = 3.64966 m

B = 2.433107 m

However, provide footing size 2 m x 3 m

Net upward pressure p_0 = 157.0013 kN/m^2



Design of section:

(a) Design on the basis of bending compression

Bending moment M1 about section X-X

$M_1 = p_0 B (L-a)^2/2$ kN-m

M_1 = 2.55E+08 N-mm

M_{1u} = 3.83E+08 N-mm

Bending moment M_2 about section Y-Y is given by

M_2 = $p_0 L (B-b)^2 \times 10^6$ N-mm

1.7E+08 N-mm

M_{2u} = 2.55E+08 N-mm

Thus, $M_{2u} < M_{1u}$

d = 263.2554 mm   Provide effective cover = 75 mm

D = 338.2554

Provide uniform thickness for the entire footing

(b) Depth on the basis of one way shear

V = p0 B (L/2 - a/2 -d)

Vu = 1.5 V

τ_v = Vu/Bd ....(i)

Assuming under reinforced section, with p = 0.3 %

we get, τ_c = 0.384 N/mm² (from IS 456 : 2000) for M 20 concrete

Also, k= 1 for D= 338.255351 mm

Therefore, permissible shear stress = 0.384 N/mm² ....(ii)

equating equation (i) and (ii)

d = 470.6322 mm

(c) Check for two way shear action

For the two way shear action or punching shear action along ABCD

perimenter ABCD = 2*((a+d)+(b+d))

= 3.382529 m = 3382.529 mm

Area ABCD = 0.709469 mm²

Punching shear = 1245.931 kN

τ_v = 0.782656 N/mm²

Allowable shear stress τ_c

τ_c = 1.118034 N/mm²

Ks = (0.5 + ßc)

ks = 1.166667 N/mm²

However, adopt max. ks = 1

$k_s \tau_c$ = 1.118034 N/mm²

This is more then τ_v 0.782656   let dia be d= 12 mm

Hence, safe

Thus, te effective depth d, = 470.6321821 mm O.K

Keeping D = 551.6322

using effective cover = 75 mm

depth,d = 476.6321821 mm in one direction

d = 464.6321821 mm in other direction, with 12 mm ϕ

Design for reinforcement

Since actual d provided is more then that required for bending compression , we have an under - reinforced section,

Hence, area Ast_1 of long bars calculated for B.M M_{1u}

Ast 1 = 2378.907 mm^2

Hence, no. of 16 mm bars= 11.83771643

These are to be distributed uniformly in a width of B = 2 m

effective depth for top layer of reinforcement, d2 = 464.6322 mm

The area Ast_2 of short bars, calculated for M_{2u} ,

Ast_2 = 1558.323 mm^2

The area is to be provided in two distinct band of width B = 2 m

Ast 2 (B) = (2* Ast2)/(ß+1)

1246.658

No. of 16 dia bars = 6.203515347

This number of bars are to be provided in central band width = 2 m

Remaining area in each end band strip = 155.8323 mm^2

No. of 16 dia bars = 0.775439418

However, provide minimum 3 bars in each end band of width = 0.5 m

Check for development length

Ld = 47 ϕ

= 752 mm

providing 75 mm side cover

length available= 812.5 mm

which is greater then Ld, Hence O.K

Check for transfer of load at the base

A_2 = 135000 mm^2

At a rate of spread of 2:1

A1 = 7022500

Therefore, under-root of A_1/A_2 = 7.212386465 > 2

Adopt max value of (A_1/A_2)^.5 as 2

Therefore, permissible bearing stress = 18 N/mm^2

Actual bearing stress = 10.46676 N/mm^2

Hence , O.K

CHAPTER – 4

DATA ANALYSIS

4.1 ANALYSIS

The following observations were made as a result of analysis done on software STAAD Pro for static analysis.

4.1.1 Maximum bending moments and shear forces in each floor in building on sloping ground

Table 4.1: Maximum Bending moments on each floor in building on Sloping ground

FLOOR	BENDING MOMENT (M z) kN-m FLOOR WISE (SLOPING GROUND)
GROUND FLOOR	87.347
FIRST FLOOR	93.973
SECOND FLOOR	87.347
THIRD FLOOR	77.357
FOURTH FLOOR	90.374
TOP FLOOR	38.757

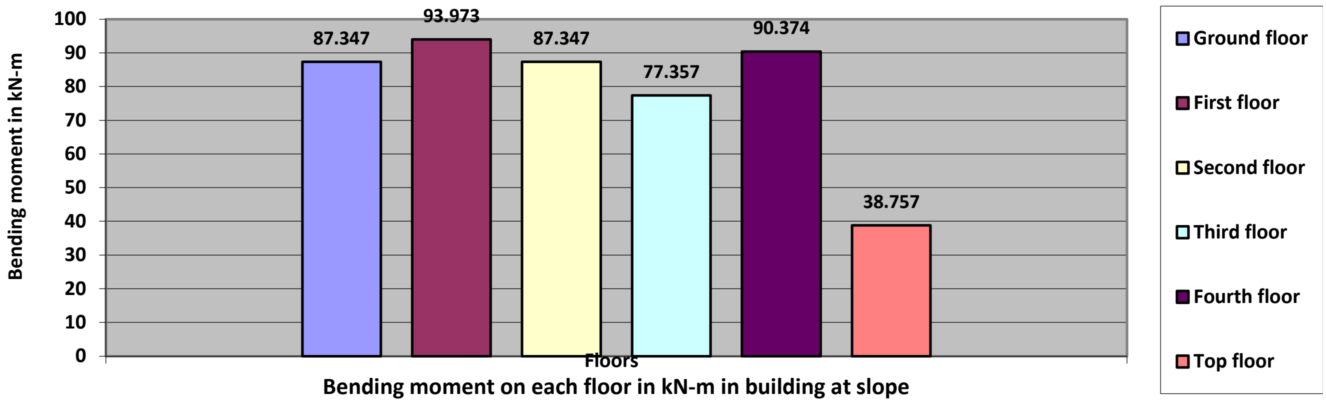


Figure 4.1: Maximum Bending moments on each floor in building on Sloping ground

Table 4.2: Maximum Shear force on each floor in building on Sloping ground

FLOOR	SHEAR FORCE (kN)
GROUND FLOOR	77.337
FIRST FLOOR	79.183
SECOND FLOOR	103.245
THIRD FLOOR	100.140
FOURTH FLOOR	97.867
TOP FLOOR	94.693

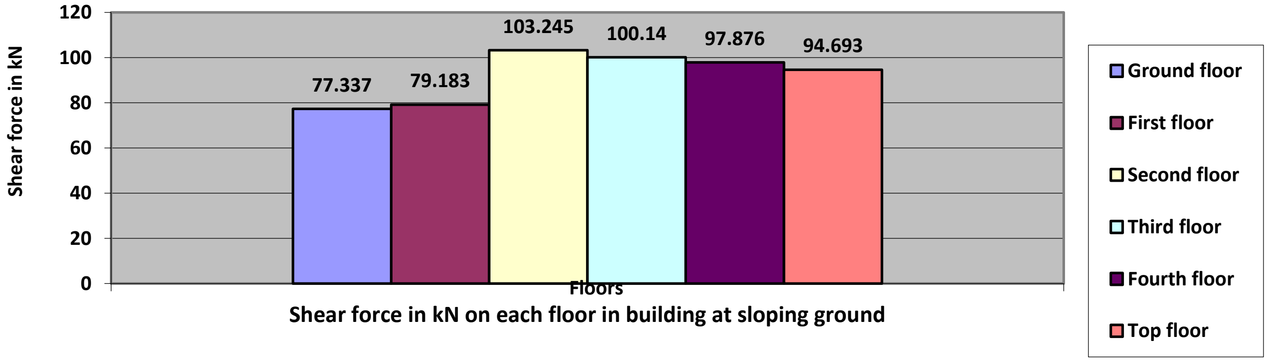


Figure 4.2: Maximum Bending moments on each floor in building on Sloping ground

4.1.2 Maximum bending moments and shear forces in each floor in building on Plain ground

Table 4.3: Maximum Bending moments on each floor in building on Plain ground

FLOOR	BENDING MOMENT (M z) kN-m FLOOR WISE (PLAIN GROUND)
GROUND FLOOR	85.556
FIRST FLOOR	90.278
SECOND FLOOR	80.809
THIRD FLOOR	74.469
FOURTH FLOOR	96.813
TOP FLOOR	38.119

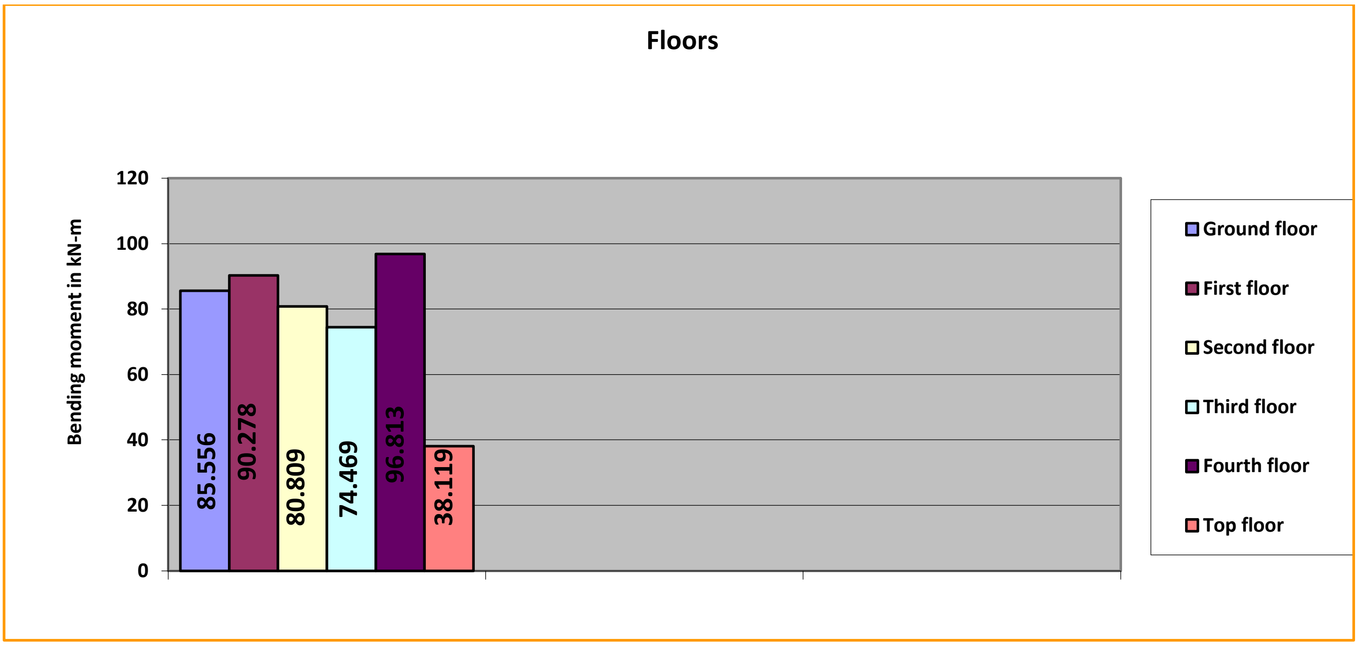


Figure 4.3: Maximum Bending moments on each floor in building on Plain ground

Table 4.4: Maximum Shear force on each floor in building on Plain ground

FLOOR	SHEAR FORCE (kN)
GROUND FLOOR	77.454
FIRST FLOOR	77.134
SECOND FLOOR	96.486
THIRD FLOOR	104.150
FOURTH FLOOR	101.091
TOP FLOOR	80.033

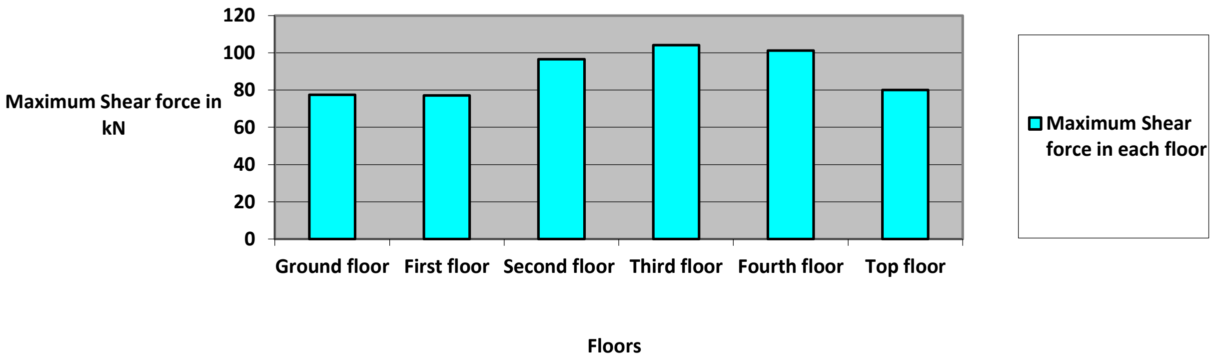


Figure 4.4: Maximum Shear force at various floors in building at plain

4.1.3 Maximum axial forces in columns from top to bottom

Table 4.5: Maximum axial forces in columns from top to bottom

COLUMN AT SLOPING GROUND	AXIAL FORCE (kN)	COLUMN AT PLANE GROUND	AXIAL FORCE (kN)	MANUALLY CALCULATED AXIAL FORCES (kN)
TOP	60.141	TOP	60	58.08
FIRST	174.705	FIRST	173.54	159.38
SECOND	287.839	SECOND	285.39	260.685
BASE (SHORT COLUMN)	460.779	BASE (LONG COLUMN)	461.82	463.29

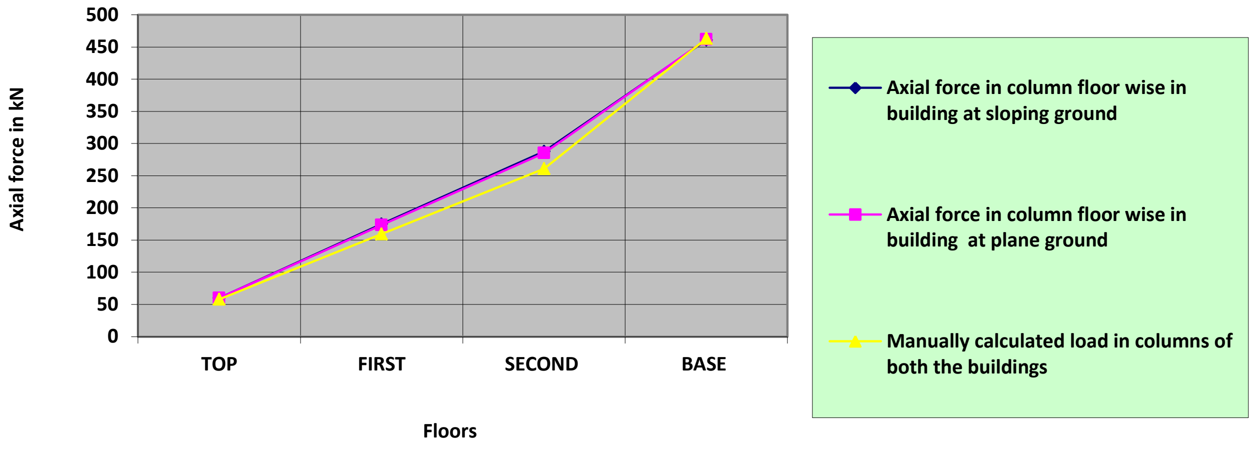
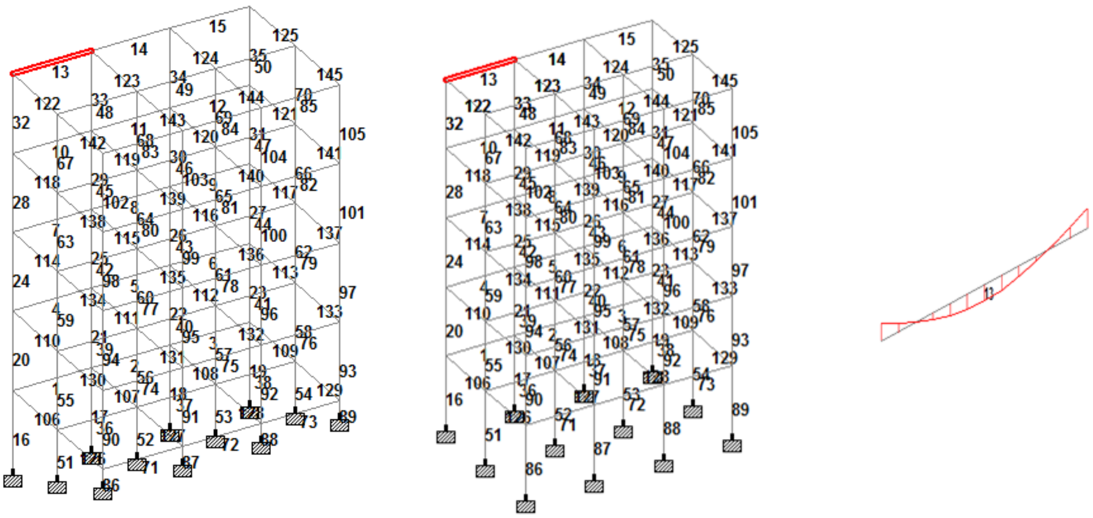


Figure 4.5: Maximum axial forces in columns from top to bottom

The axial forces in column from top to bottom were observed to be almost same to that calculated manually. Also, the axial force in columns of the building at slope is more in comparison to the building columns at plain ground as shown in Figure 4.5. This is the force calculated under static condition of load combination of 1.5 (D.L+L.L).

The moment and shear force in beam element 13 is seen to be almost same for both the buildings at slope and at plain, thus we can see from the figure 4.7, 4.8, 4.9 and 4.10 that the change in moment and shear forces in top element of building is almost alike. Therefore, the changes are to be noticed for the lower building elements for various parameters discussed above.



(a) **(b)** **(c)**

Figure 4.6: Bending moment and shear force diagram Beam 13 (at top) (a) building at sloping ground; (b) building at plain ground (c) building element 13

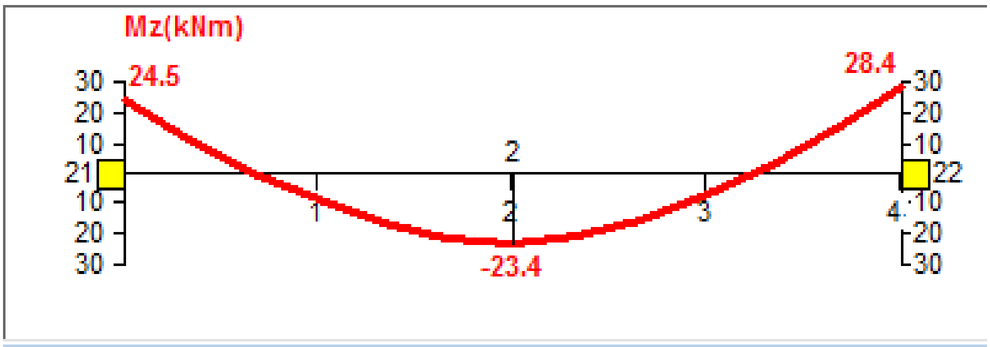


Figure 4.7: Bending Moment in beam 13 (top external) in building at sloping ground

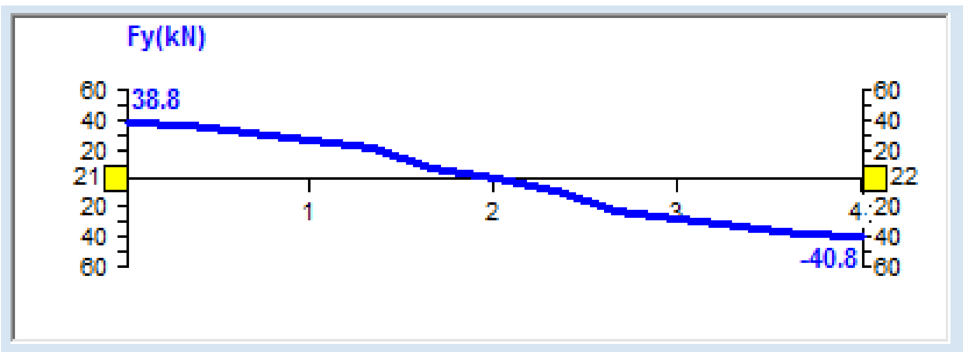


Figure 4.8: Shear force diagram of beam 13 (top external) in building at slope

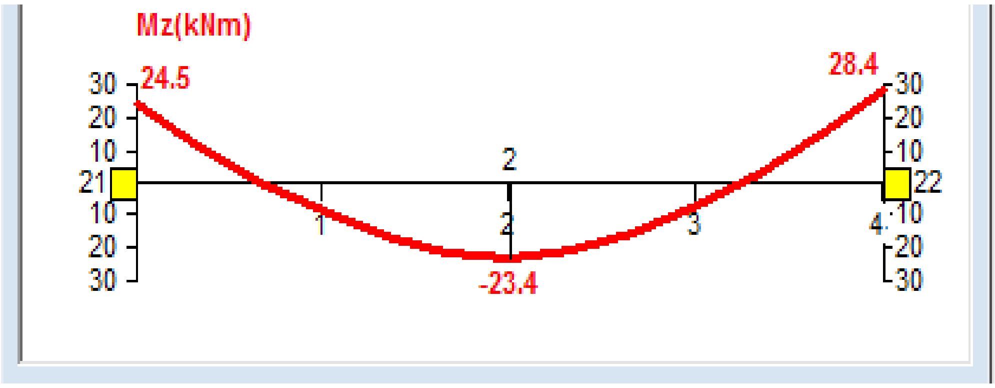


Figure 4.9: Bending moment diagram for beam 13 in building at plain

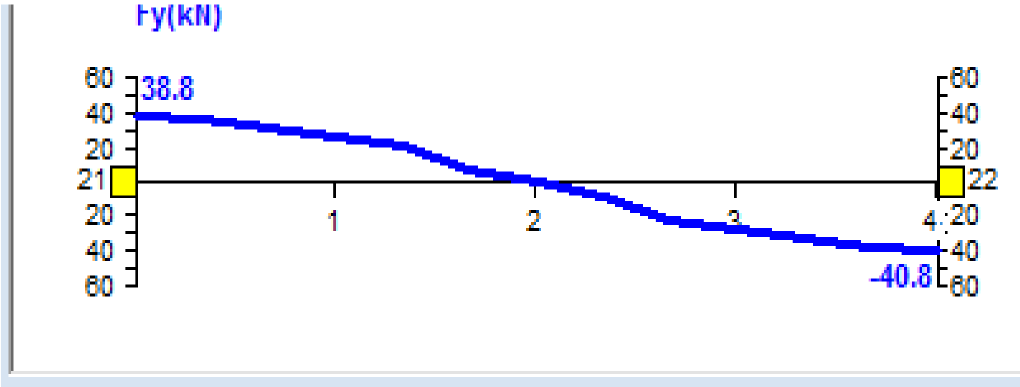


Figure 4.10: Shear force diagram for beam 13 in building at plain

4.1.4 Results and Discussion for response spectrum analysis

The models which were analyzed under static response were found to be safe against the Live loads and dead load criterion after that Response spectrum analysis is done for the same buildings and results in terms of shear forces, bending moment, displacements and torsion are taken out as shown below and compared.

4.1.4.1 Bending moment variation after application of dynamic load

The supports 73, 74, 75, 76 are the supports of short column of length 0.6 m in building at slope, which are exposed to maximum bending moments as compare to any other column in the same floor. Supports 77, 78, 79 and 80 are the supports of short column of length 2.3 m in building at slope, and have exposed to the moments which are almost equal or lesser then the moments at the base of long columns in building at flat ground. The positions of the columns are as shown in figure 4.11.

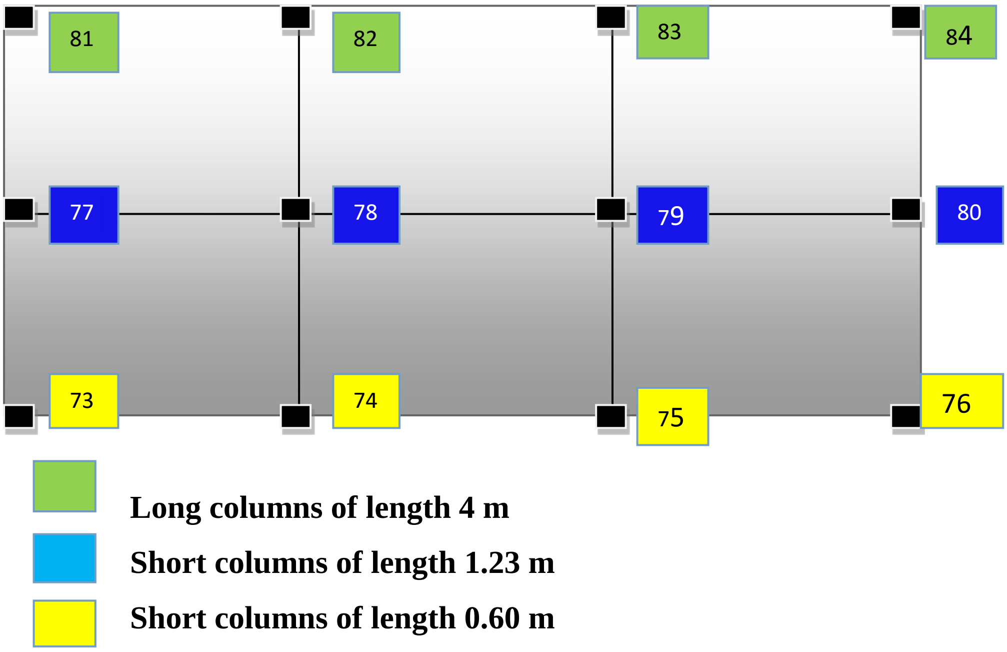


Figure 4.11: Position of columns in building at flat ground and sloping ground with specified lengths of short and long columns

Table 4.6: Bending moments at the base of long and short columns in building at flat and sloping ground

Supports	**Bending Moment Mz in kNm (Plain)**	**Bending Moment Mz in kNm (Slope)**	**Bending Moment Mz in kNm (Plain)**	**Bending Moment Mz in kNm (Slope)**
	Static Loading	Static Loading	Dynamic Loading	Dynamic Loading
73	-5.601	-8.532	161.643	219.022
74	0.043	0.440	182.784	242.583
75	-0.042	0.440	182.784	242.582
76	-5.600	8.532	161.642	219.023
77	-12.593	-14.134	393.074	237.518
78	-0.882	-0.767	115.220	54.441
79	0.882	0.767	115.220	54.441
80	-12.208	0.077	393.074	237.518

81	-5.601	-5.523	161.588	155.466
82	0.043	0.077	182.735	174.559
83	0.042	-0.077	182.735	174.559
84	5.600	5.523	161.589	155.466

4.1.4.2 Axial force variation in Long and short columns subjected to same forces

The increased dynamic load is found to be more on the columns above the supports 73 and 76 which are the shortest heighted columns in the building with biaxial loading can be seen from figure 4.11 and thus will require good detailing work.

Table 4.7: Axial force in columns at static and dynamic loading conditions in building at flat ground and sloping ground.

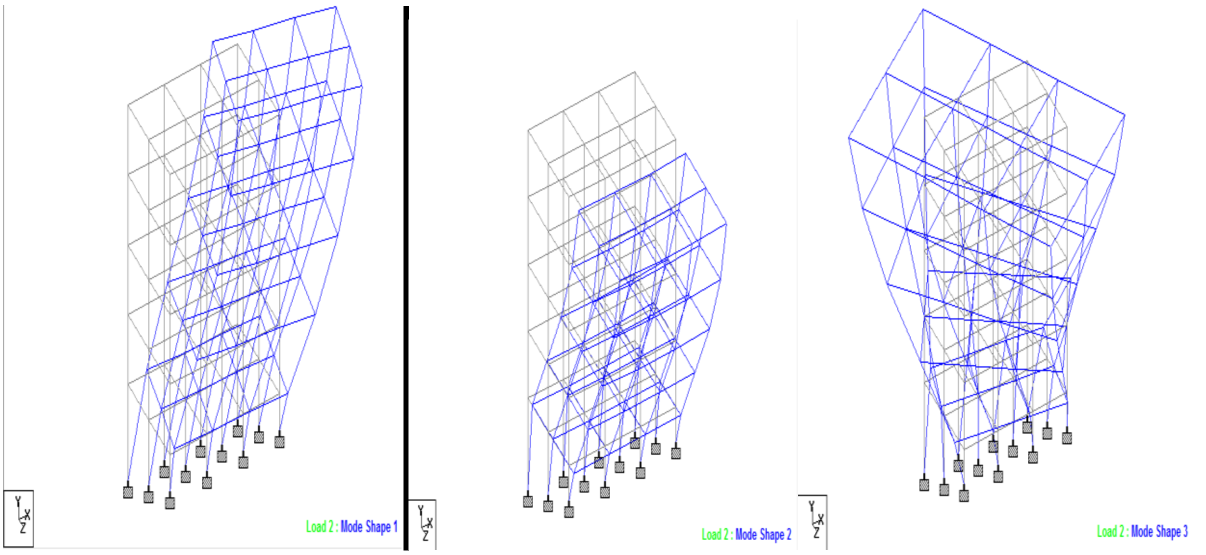
Supports	**Axial force in kN (Plain)**	**Axial force in kN (Slope)**	**Axial force in kN (Plain)**	**Axial force in kN (Slope)**
	Static loading	**Static loading**	**Dynamic loading**	**Dynamic loading**
73	616.273	601.796	616.273 + 236.732 = 853.005	601.796 + 269.270 = 871.066
74	1041.9	1040.318	1041.9 + 36. 049 = 1077.949	1040.318 + 26.016 = 1066.334
75	1041.903	1040.314	1041.314 + 36.090 = 1077.404	1041.903 + 25.921 = 1067.824
76	616.270	601.793	616.270 + 236.733 = 853.003	601.793 + 269.197 = 870.99
77	1007.239	1013.434	1007.239 + 242.134 = 1249.373	1013.434 + 226.977 = 1240.411
78	1463.672	1460.678	1463.672 + 71.121 = 1534.793	1460.678 + 71.915 = 1532.593
79	1463.6	1460.672	1463.6 + 71.126 = 1534.726	1460.672 + 71.126 = 1531.798
80	1007.2	1013.429	1007.2 + 242.141 =1249 .341	1013.429 + 227.048 = 1240.477

81	616.273	626.891	616.273 + 236.799 = 853.072	626.891 + 194.134 = 821.025
82	1041.9	1050.688	1041.9 + 36.112 = 1078.012	1050 + 44.918 = 1094.918
83	1041.903	1050.684	1041.903 + 36.075 = 1077.978	1050.684 + 44.918 = 1095.602
84	616.27	626.88	616.27 + 236.804 = 853.074	616.88 + 194.218 = 811.098

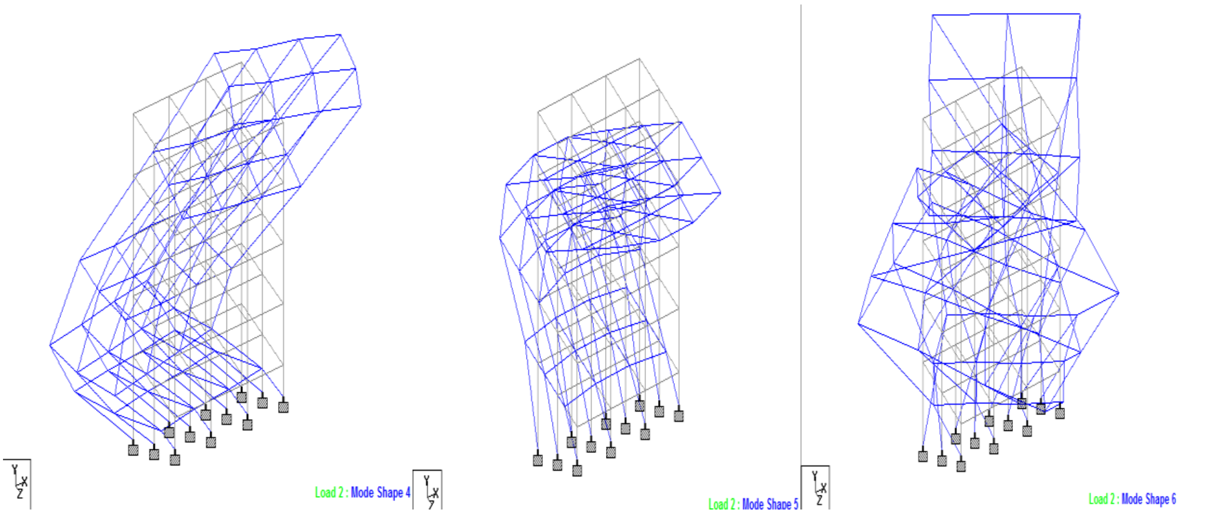
4.1.4.3 Mode shapes

When a system is excited, to describe the response of the system mode shapes are used. A pattern of motion in which all parts of the system move sinusoidally with the same frequency. Calculating the natural frequencies and mode shapes means calculating the linear response of structures to dynamic loading and is called modal analysis. In modal analysis, the response of the structure is decomposed into several vibration modes. A mode is defined by its frequency and shape. The frequency is found to be more in building models at slope for various mode shapes and the time period is inverse to the condition as shown in table 4.8 and 4.9.

- Mode shapes for the building on slope



(a) (b) (c)



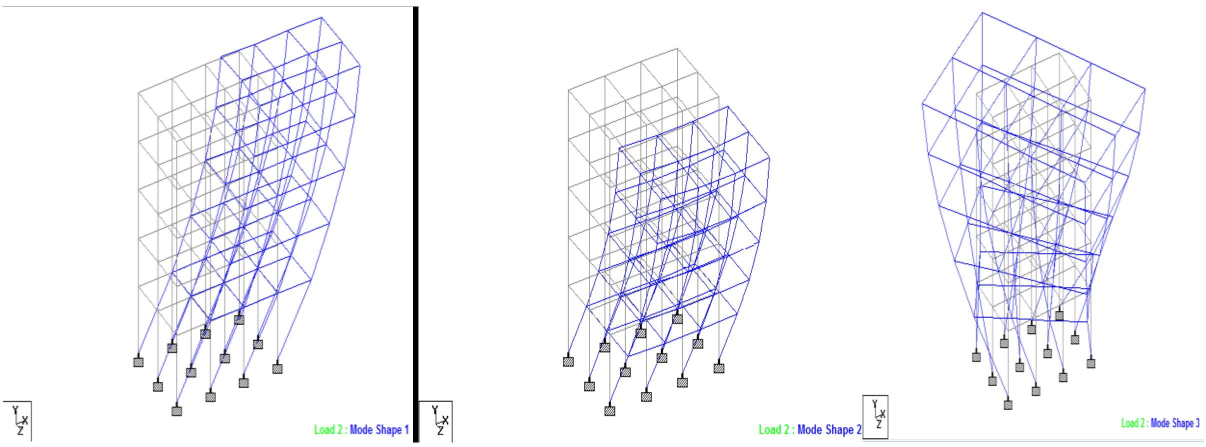
(d) (e) (f)

Figure 4.12: (a) Mode shape 1 ;(b) Mode shape 2; (c) Mode shape 3; (d) Mode shape 4 ; (e) Mode shape 5 ; (f) Mode shape 6 for model at slope

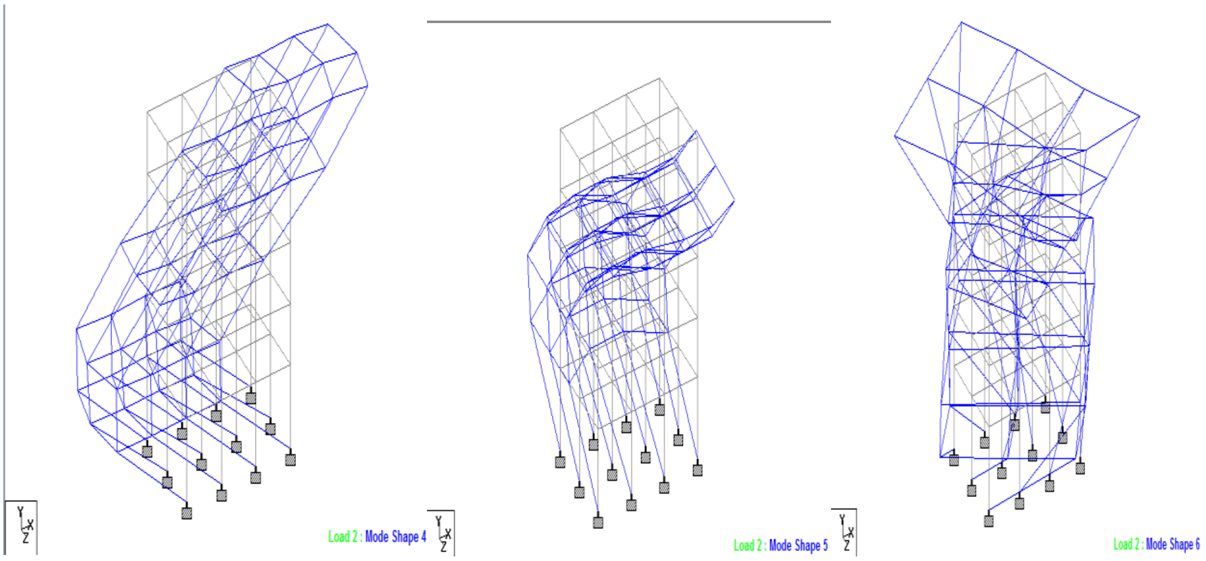
Table 4.8: The frequency in Hertz (Hz) and time period in seconds in detail for building on sloping ground

Mode	Frequency Hz	Period seconds	Participation X %	Participation Y %	Participation Z %
1	1.137	0.879	79.784	0.000	0.000
2	1.227	0.815	0.000	0.000	76.036
3	1.606	0.623	0.653	0.000	0.000
4	3.528	0.283	10.449	0.000	0.000
5	3.890	0.257	0.000	0.000	11.005
6	4.965	0.201	0.010	0.000	0.000

- Mode shapes of building resting on plain ground.



(a) (b) (c)



(d) (e) (f)

Figure 4.13: (a) Mode shape 1; (b) Mode shape 2; (c) Mode shape 3; (d) Mode shape 4; (e) Mode shape 5; (f) Mode shape 6 for model at plain

Table 4.9: The frequency in Hertz (Hz) and time period in seconds in detail for building on plain ground

Mode	Frequency Hz	Period seconds	Participation X %	Participation Y %	Participation Z %
1	1.081	0.925	92.496	0.000	0.000
2	1.157	0.864	0.000	0.000	90.996
3	1.448	0.691	0.000	0.000	0.000
4	3.414	0.293	6.033	0.000	0.000
5	3.598	0.278	0.000	0.000	6.975
6	4.542	0.220	0.000	0.000	0.000

- Results in terms of torsion experienced by a selected beam and a column of both the buildings

4.1.4.4 Torsion in beam and column elements of both the buildings on plain and on sloping ground

Torsion is defined to be twisting or wrenching of some element, and it can lead to a much catastrophic failure of a beam or a column. Therefore, the torsion has to be examined in both the elements namely, beam and columns. The torsion in short column has maximum value as shown in figure 4.18.

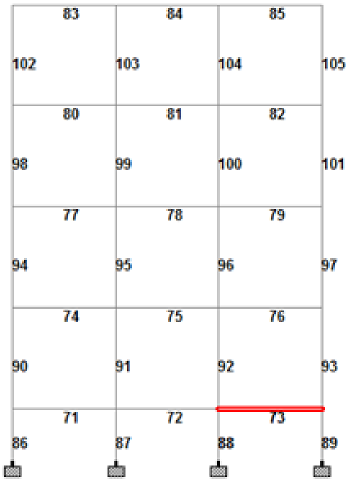


Figure 4.14: Torsion effect in a beam element 73 in both the buildings

Table 4.10: Torsion and moment results for the beam element 73 in building at slope

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
73	2	55	2.037	63.739	18.539	5.172	33.366	124.994
73	2	56	-2.037	-63.739	-18.539	-5.172	-40.791	-129.962

Table 4.11: Torsion and moment results for the beam element 73 in building at Plain

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
73	2	55	14.361	79.263	3.397	0.040	6.014	145.337
73	2	56	-14.361	-79.263	-3.397	-0.040	-7.576	-171.711

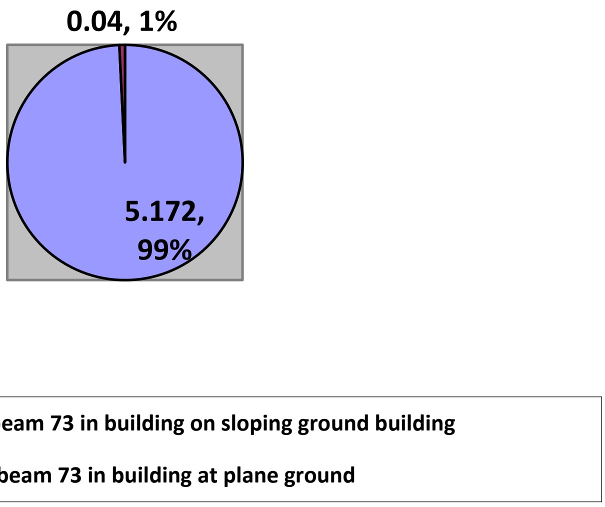


Figure 4.15: Torsion in beam 73 of building at slope and plane ground

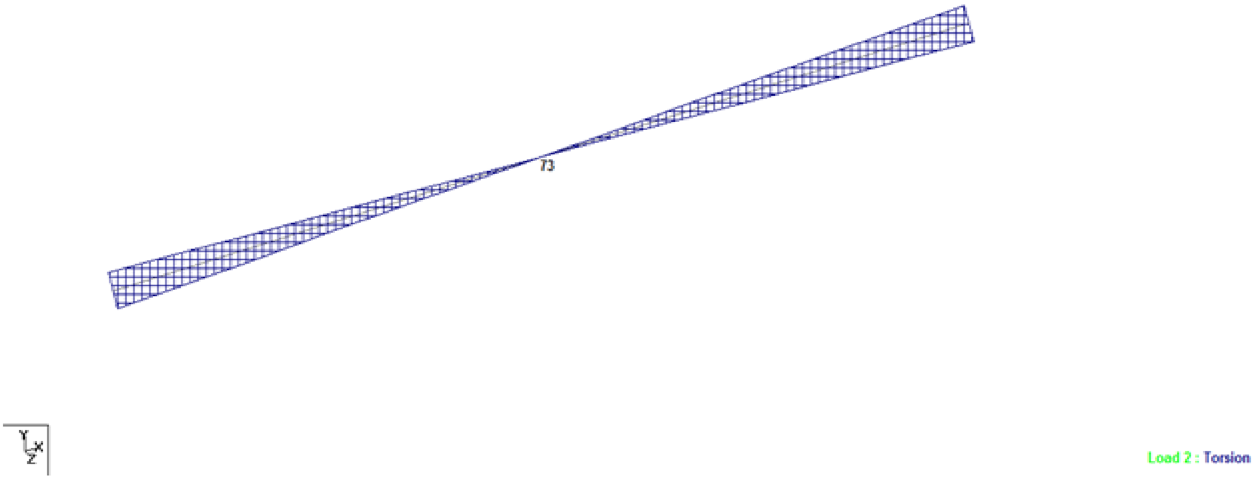
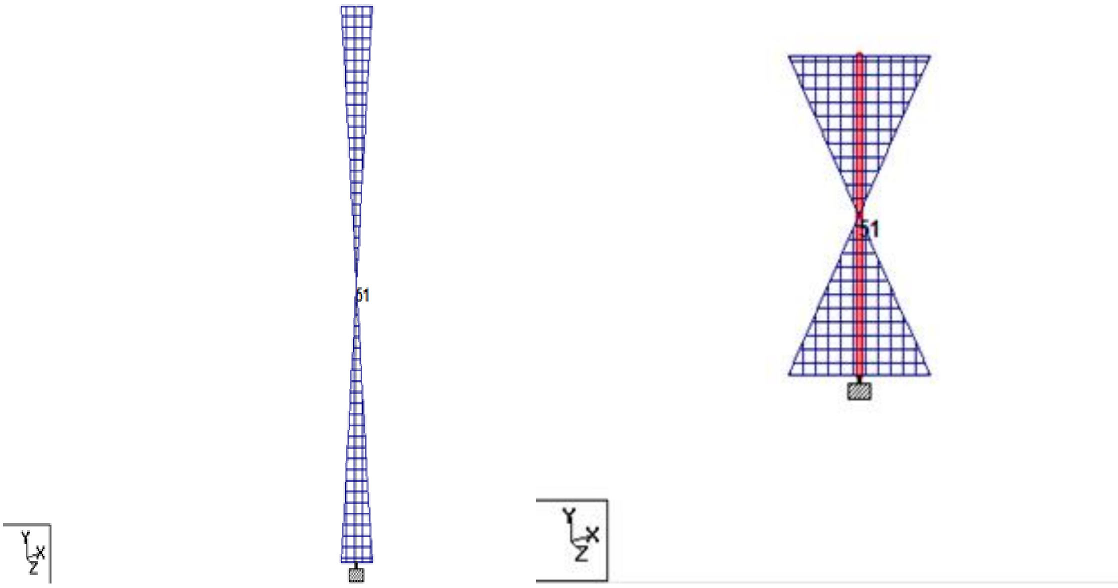


Figure 4.16: Torsion effect in a beam element 73

- Torsion in the column elements 51, of both buildings at plain and slope are shown below:



(a) (b)

Figure 4.17: (a) Torsion effect in a long column element 51in building on plain ground; (b) Torsion effect in a short column element 51 in building at sloping ground

Table 4.12: Column 51 Torsion and moment results for column (long column) element 51 in building at plain

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
51	2	29	241.614	106.747	0.005	0.004	0.015	194.038
51	2	77	-241.614	-106.747	-0.005	-0.004	-0.013	-393.074

Table 4.13: Column 51Torsion and moment results for column (short column) element 51 in building at slope

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
51	2	29	221.874	83.806	16.023	25.212	14.794	18.380
51	2	77	-221.874	-83.806	-16.023	-25.212	-46.352	-300.318

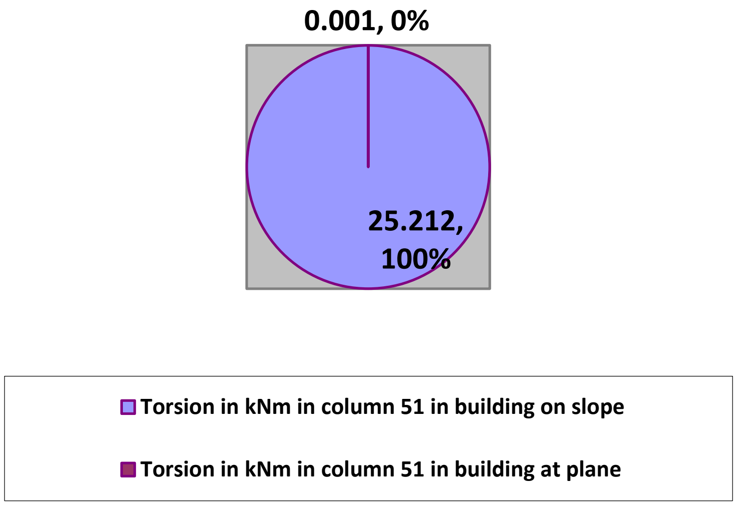
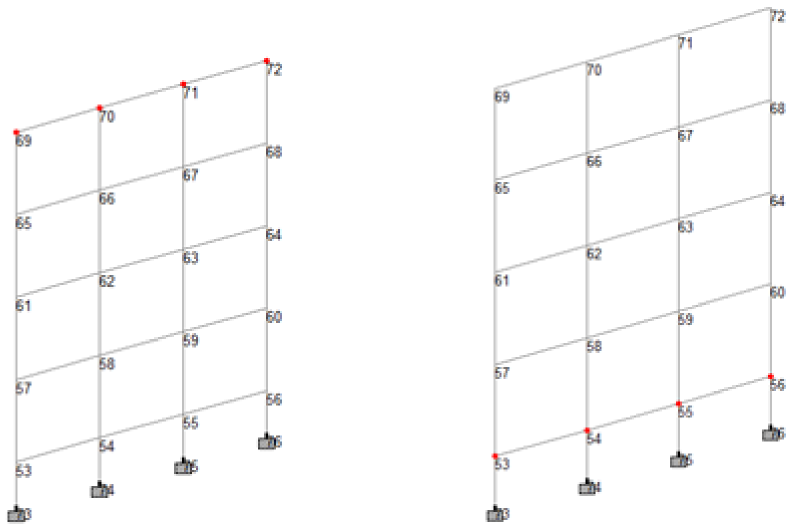


Figure 4.18: Graph showing torsion in column element 51, in building at slope (short column) and building at plain (long column)

4.1.4.5 Displacement in top and bottom nodes of both the buildings



(a) (b)

Figure 4.19: (a) Top Nodes 69, 70, 71, 72 in building at slope; (b) Bottom nodes 53,54,55,56

Table 4.14: Displacement of the nodes 69,70,71,72 in building at slope

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm
69	2	77.256	0.563	12.876	78.324
72	2	77.256	0.563	12.876	78.324
70	2	77.250	0.018	4.397	77.375
71	2	77.250	0.018	4.395	77.375

Table 4.15: Displacement of the nodes 69,70,71,72 in building at plain

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm
72	2	118.444	0.828	0.003	118.447
69	2	118.444	0.828	0.013	118.447
71	2	118.443	0.110	0.003	118.443
70	2	118.443	0.110	0.005	118.443

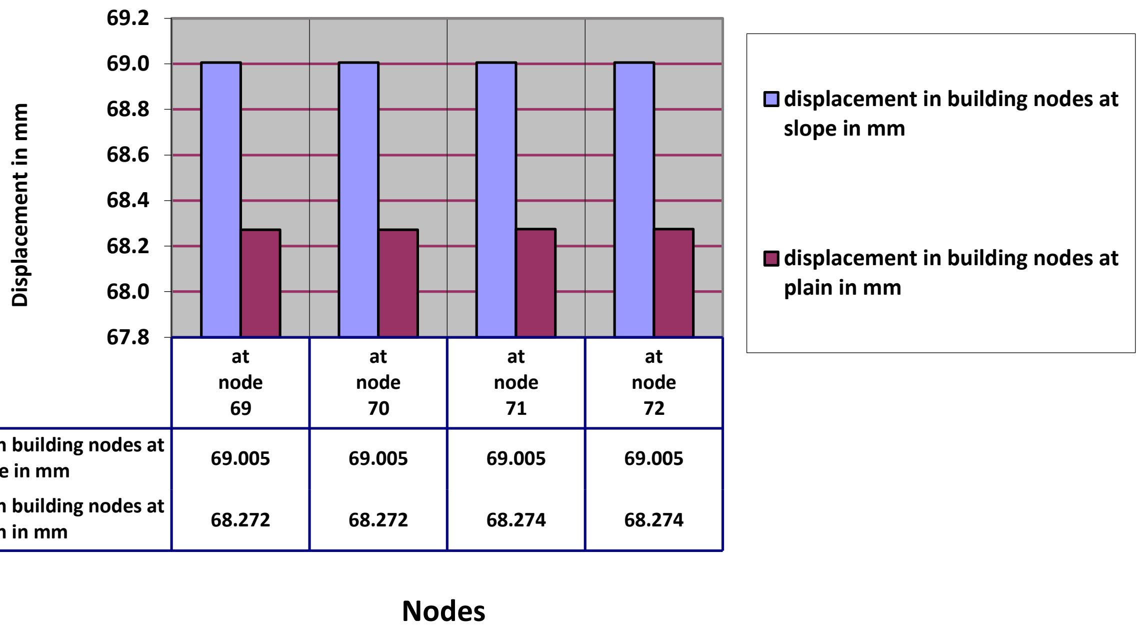


Figure 4.20: Displacement of nodes 69, 70, 71, 72 of building at slope and plain ground

Table 4.16: Displacement of the bottom nodes 53, 54, 55, 56 in building at slope

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm	X-Rotan rad	Y-Rotan rad	Z-Rotan rad
53	2	5.630	0.145	2.660	6.228	0.001	0.001	0.003
54	2	5.632	0.010	0.663	5.671	0.000	0.001	0.002
55	2	5.632	0.010	0.664	5.671	0.000	0.001	0.002
56	2	5.630	0.145	2.663	6.229	0.001	0.001	0.003

Table 4.17: Displacement of the bottom nodes 53,54,55,56 in building at plain

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm
55	2	44.903	0.060	0.004	44.903
54	2	44.903	0.060	0.001	44.903
56	2	44.883	0.402	0.015	44.885
53	2	44.883	0.402	0.005	44.885

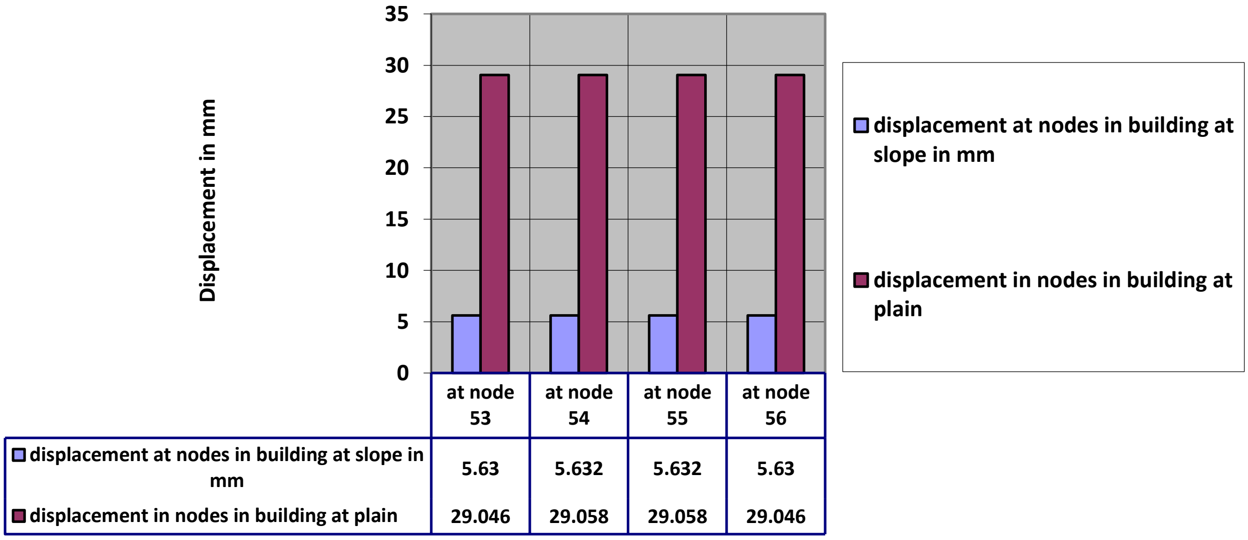
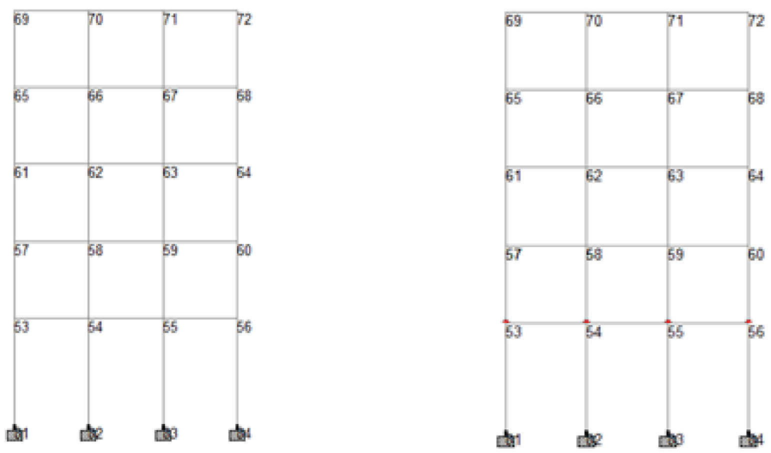


Figure 4.21: Displacement of nodes 53, 54, 55, 56 of building at slope and plain ground



(a) (b)

Figure 4.22: (a) Top Nodes 69, 70, 71, 72 in building at plain; (b) Bottom nodes 53,54,55,56 in building at plain

4.1.4.6 Base shear calculated by STAAD Pro V8i

Base shear is an estimate of the maximum expected lateral force that will occur due to seismic ground motion at the base of the structure. Base shear and mass participation factors in percent for both the buildings at plain and slope are shown below. The total design lateral force or design seismic base shear V_b along any principal direction is determined by the STAAD.

Participation factor Summ-X on 6th mode is calculated to be 98.528 for building at plain and 90.896 for the building at slope. Seismic weight has been achieved using the first three mode shapes. As per clause 7.8.4.2 in IS 1893, the number of modes to be used in the analysis should be such that the sum of total of modal masses of all modes considered as 90% of total seismic mass. The summation has come to be about 90% in STAAD Pro V8i, hence we can conclude that clause 7.8.4.2 has been satisfied.

	MASS PARTICIPATION FACTORS IN PERCENT						BASE SHEAR IN KN		
MODE	X	Y	Z	SUMM-X	SUMM-Y	SUMM-Z	X	Y	Z
1	90.61	0.00	0.00	90.610	0.000	0.000	625.80	0.00	0.00
2	0.00	0.00	88.07	90.610	0.000	88.069	0.00	0.00	0.00
3	0.00	0.00	0.00	90.610	0.000	88.069	0.00	0.00	0.00
4	7.18	0.00	0.00	97.795	0.000	88.069	103.27	0.00	0.00
5	0.00	0.00	8.67	97.795	0.000	96.739	0.00	0.00	0.00
6	0.00	0.00	0.00	97.795	0.000	96.739	0.00	0.00	0.00
					TOTAL SRSS SHEAR		634.27	0.00	0.00
					TOTAL 10PCT SHEAR		634.27	0.00	0.00
					TOTAL ABS SHEAR		729.08	0.00	0.00
					TOTAL CSM SHEAR		729.08	0.00	0.00

```
*********************************************************
*                                                       *
* TIME PERIOD FOR X 1893 LOADING =    0.46470 SEC       *
* SA/G PER 1893=     2.500, LOAD FACTOR= 1.000          *
*      FACTOR V  PER 1893=    0.0975 X    7477.72       *
*                                                       *
*********************************************************
```

For building at plain the base shear is calculated as 729.0777

	MASS PARTICIPATION FACTORS IN PERCENT						BASE SHEAR IN KN		
MODE	X	Y	Z	SUMM-X	SUMM-Y	SUMM-Z	X	Y	Z
1	78.13	0.00	0.00	78.134	0.000	0.000	630.67	0.00	0.00
2	0.00	0.00	73.89	78.134	0.000	73.894	0.00	0.00	0.00
3	0.46	0.00	0.00	78.592	0.000	73.894	5.11	0.00	0.00
4	11.30	0.00	0.00	89.889	0.000	73.894	126.00	0.00	0.00
5	0.00	0.00	11.38	89.889	0.000	85.274	0.00	0.00	0.00
6	0.05	0.00	0.00	89.941	0.000	85.274	0.59	0.00	0.00
					TOTAL SRSS	SHEAR	643.15	0.00	0.00
					TOTAL 10PCT	SHEAR	643.15	0.00	0.00
					TOTAL ABS	SHEAR	762.36	0.00	0.00
					TOTAL CSM	SHEAR	756.69	0.00	0.00

```
**********************************************************
*                                                        *
* TIME PERIOD FOR X 1893 LOADING =     0.46470 SEC      *
* SA/G PER 1893=     2.500, LOAD FACTOR= 1.000          *
*      FACTOR V  PER 1893=     0.0975 X    7760.97      *
*                                                        *
**********************************************************
```

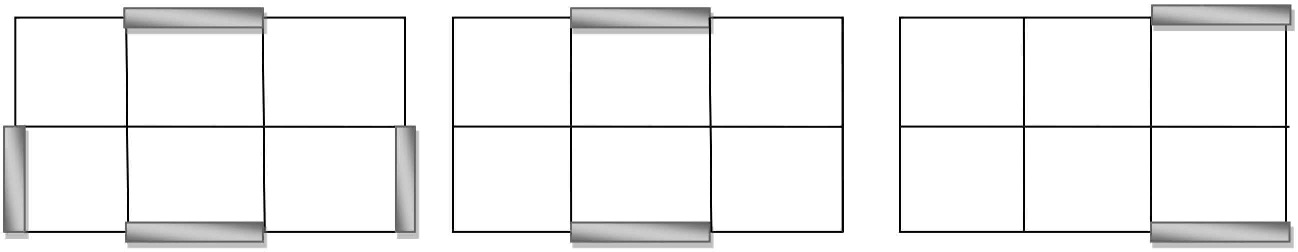
For building at slope the base shear is calculated as 756.6945

4.2 SHEAR WALL

Shear wall is a reinforced concrete (RC) vertical plate-like RC wall, in addition to slabs, beams and columns. These walls generally start at foundation level and are continuous throughout the building height. The thickness of the wall can vary from as low as 150 mm or as high as 400 mm in high rise buildings. Shear walls are usually provided along both length and width of buildings. These carry earthquake loads downwards to the foundation. Also, shear walls in buildings must be symmetrically located in plan to reduce ill-effects of twist in buildings. Details used for shear wall, for various panels is as follows:

```
PANEL  NO.   1 OF SHEARWALL      9  DESIGN PER IS 456-2000


WIDTH :         4.00 M              FC :              30.00 MPA
HEIGHT :        4.00 M              FY :             415.00 MPA
THICKNESS : 230.00 MM               CONC. COVER :    25.000 MM
```

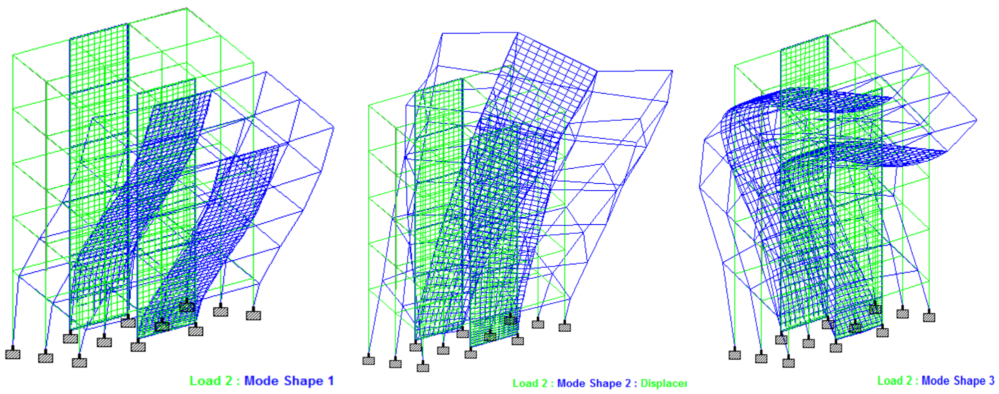


(a) (b) (c)

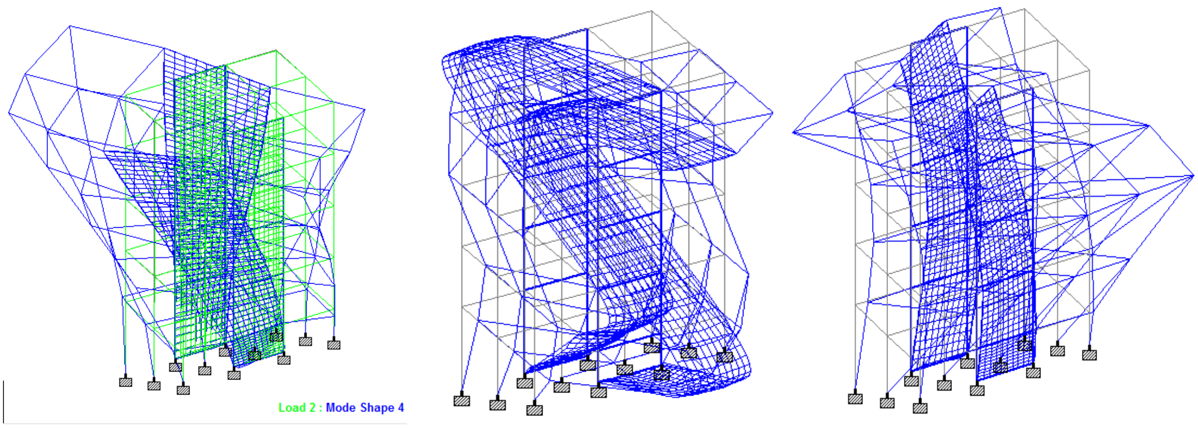
Figure 4.23: Positions of shear wall in both the buildings

4.2.1 Mode shapes after application of shear walls at different position

- Shear wall position at middle as shown in building at slope



(a) (b) (c)



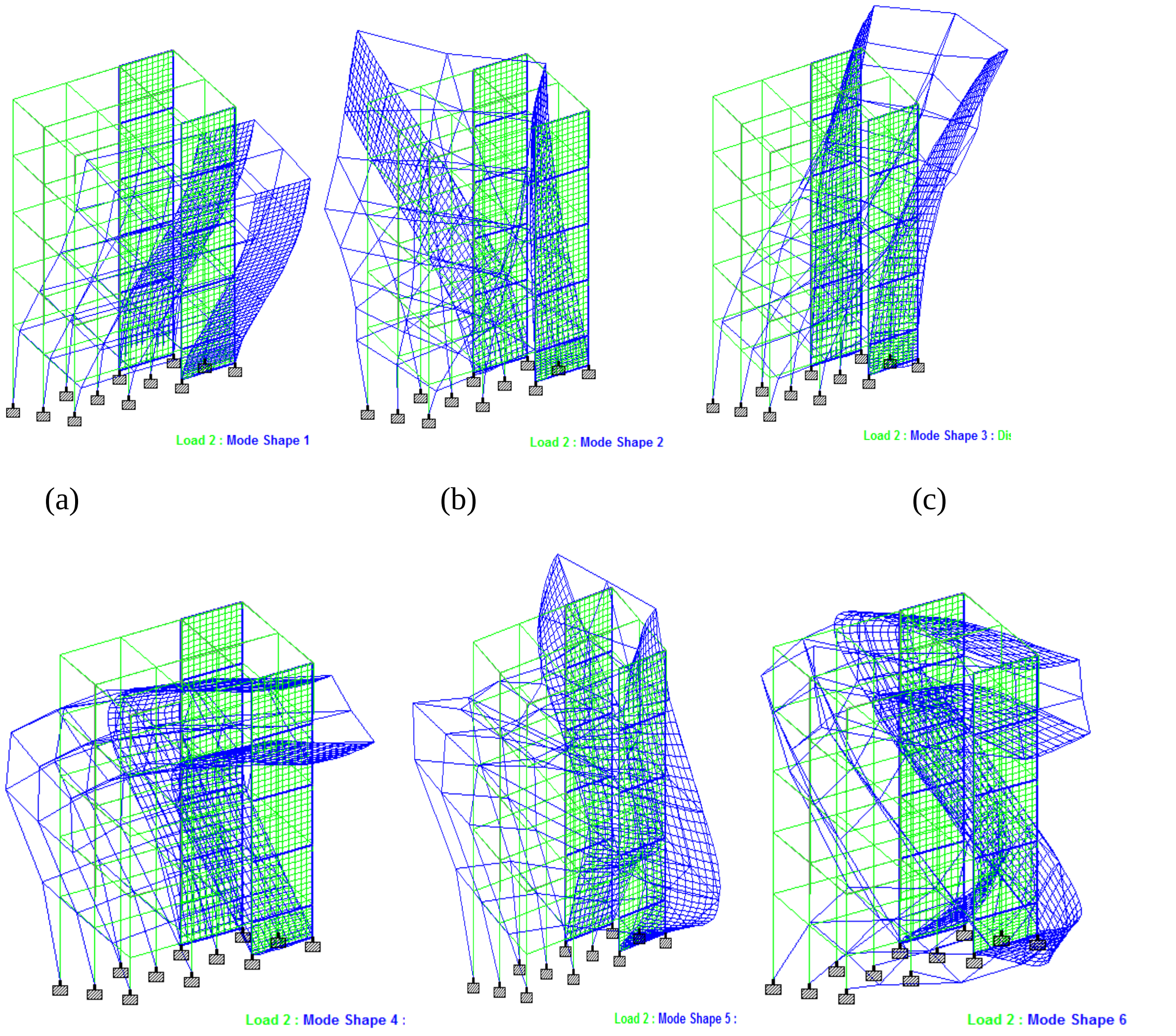
(d) (e) (f)

Figure 4.24: (a) Mode shape 1; (b) Mode shape 2; (c) Mode shape 3; (d) Mode shape 4; (e) Mode shape 5 ; (f) Mode shape 6 for model at slope with shear wall position symmetrically at center

Table 4.18: Frequency of 6 mode shapes, when the shear wall is positioned at middle

Mode	Frequency Hz	Period seconds	Participation X %	Participation Y %	Participation Z %
1	1.237	0.809	0.000	0.000	71.808
2	3.288	0.304	51.418	0.000	0.000
3	3.955	0.253	0.000	0.000	11.784
4	4.133	0.242	17.804	0.000	0.000
5	7.280	0.137	0.000	0.000	6.067
6	7.413	0.135	9.638	0.000	0.000

- When the position of shear walls is revised and built at corner, the mode shapes obtained for the building is as below:



(a) (b) (c)

(d) (e) (f)

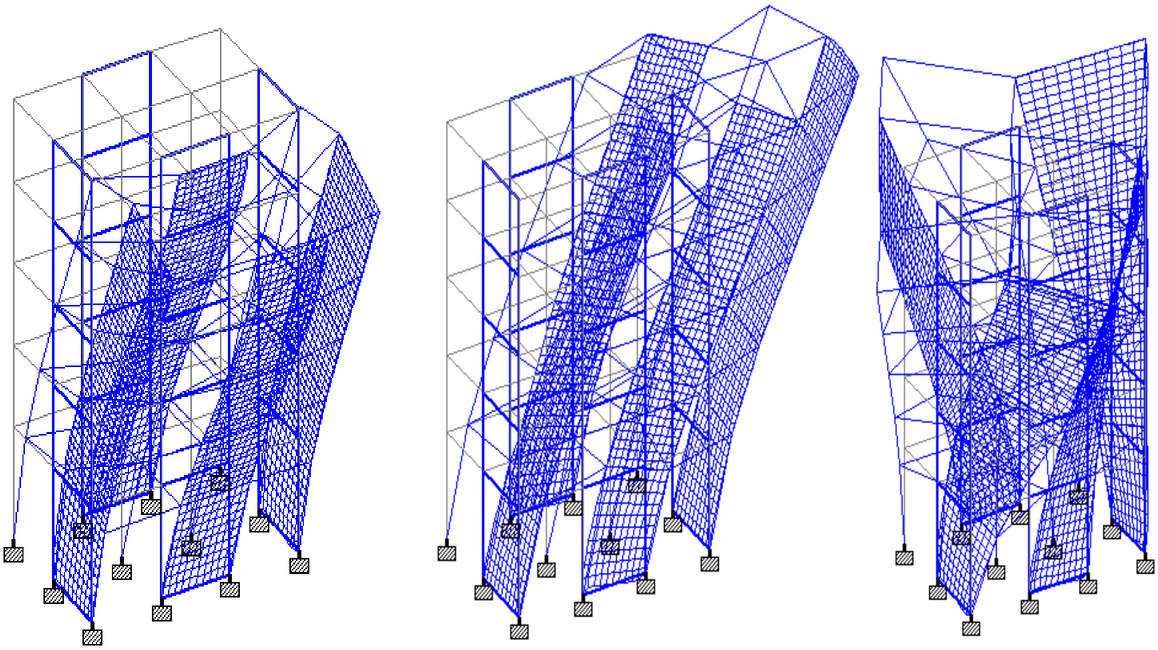
Figure 4.25: (a) Mode shape 1; (b) Mode shape 2; (c) Mode shape 3; (d) Mode shape 4; (e) Mode shape 5; (f) Mode shape 6 for model at slope with shear wall position symmetrically at corners

Table 4.19: Frequency of 6 mode shapes, when the shear wall is positioned at the corner in building at slope

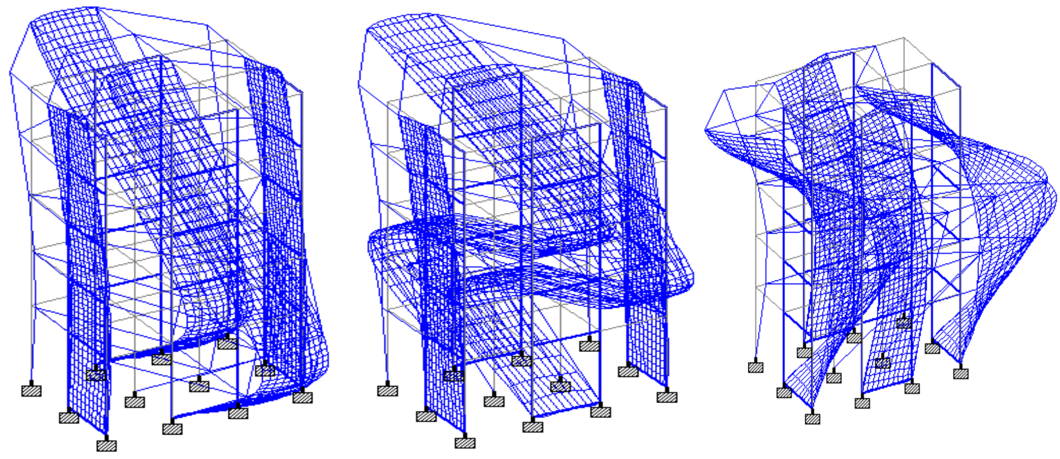
Mode	Frequency Hz	Period seconds	Participation X %	Participation Y %	Participation Z %
1	1.016	0.984	0.004	0.000	73.816
2	2.478	0.404	24.194	0.008	0.018
3	3.129	0.320	44.643	0.021	0.095
4	3.286	0.304	0.453	0.000	11.069
5	5.792	0.173	0.000	0.000	0.227
6	6.117	0.163	0.000	0.000	5.342

- Mode shapes of building model at flat ground as below with different shear wall positions

Shear wall at four sides is designed for the building at plain but it was not possible to be designed for the building at slope, since the shear wall has to be designed from the foundation of the building to the top floor and the panels has to be rectangular throughout. In case of building at slope the bottom panel is possibly not coming rectangular due to slope. Therefore, the response after application of shear walls on all four sides at different bays symmetrically was checked for the building at plain only in terms of mode shapes.



(a) (b) (c)



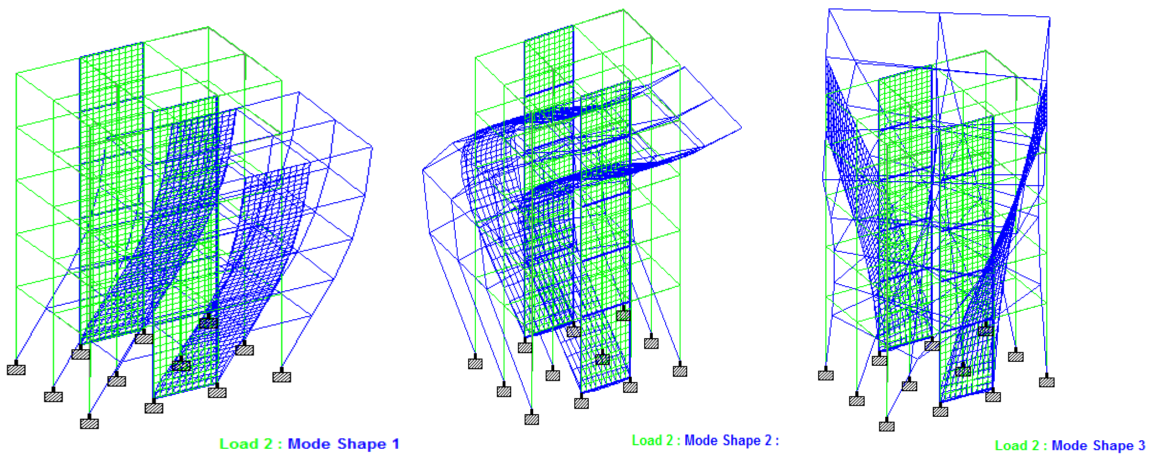
(d) (e) (f)

Figure 4.26: (a) Mode shape 1; (b) Mode shape 2; (c) Mode shape 3; (d) Mode shape 4; (e) Mode shape 5; (f) Mode shape 6 for model at plain with shear wall position symmetrically at 4 sides

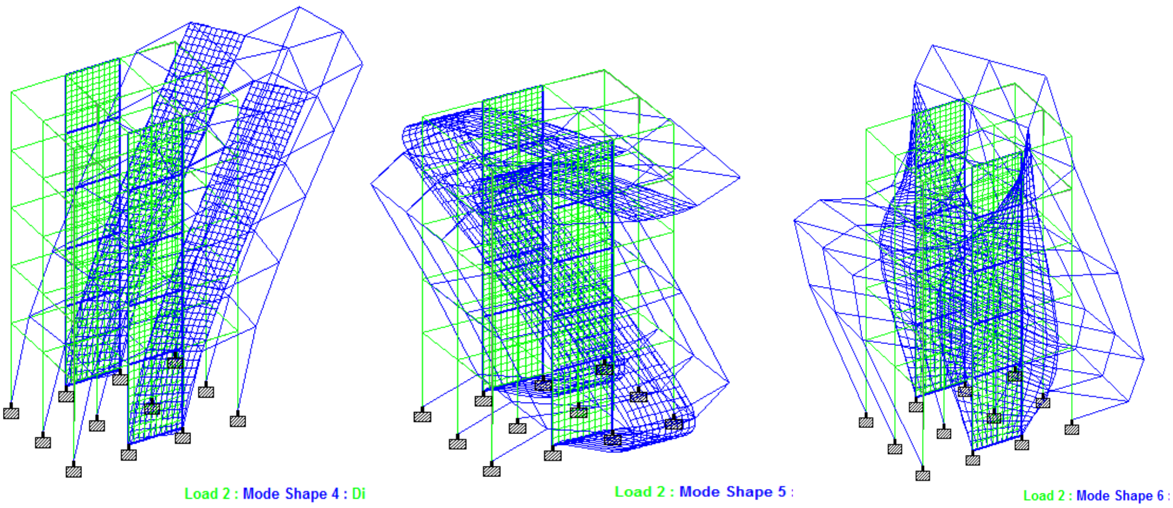
Table 4.20: Frequency and period of vibration of building at plain with shear wall at all four sides placed symmetrically

Mode	Frequency Hz	Period seconds	Participation X %	Participation Y %	Participation Z %
1	2.084	0.480	0.000	0.011	73.773
2	2.284	0.438	72.432	0.000	0.000
3	3.506	0.285	0.137	0.000	0.000
4	4.285	0.233	0.000	0.002	10.946
5	5.704	0.175	0.000	0.002	1.532
6	6.285	0.159	12.549	0.000	0.000

- Mode shapes of building resting at plain with shear walls placed at middle panel as shown below



(a) (b) (c)



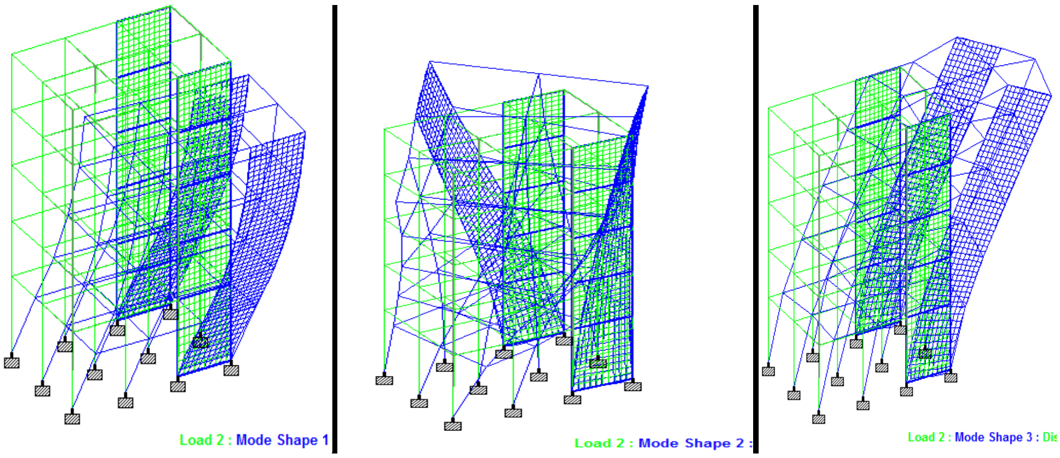
(d) (e) (f)

Figure 4.27: (a) Mode shape 1; (b) Mode shape 2; (c) Mode shape 3; (d) Mode shape 4; (e) Mode shape 5; (f) Mode shape 6 for model at plain with shear wall position symmetrically at center

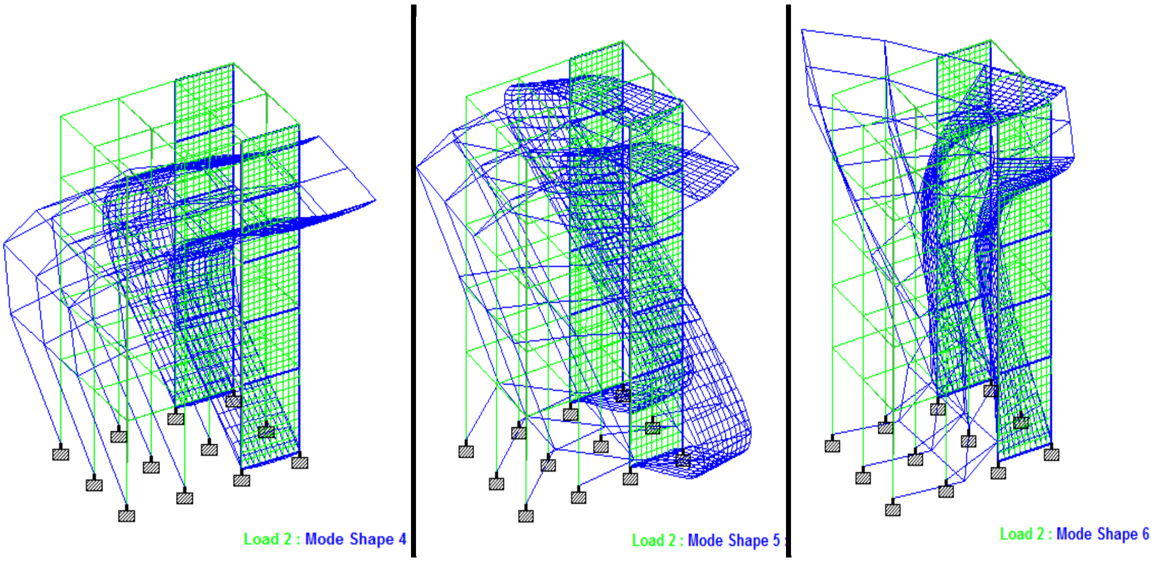
Table 4.21: Frequency and period of vibration of building at plain with shear wall placed symmetrically at center

Mode	Frequency Hz	Period seconds	Participation X %	Participation Y %	Participation Z %
1	0.764	1.309	0.000	0.000	86.461
2	2.475	0.404	0.000	0.000	8.080
3	2.500	0.400	0.000	0.000	0.000
4	2.520	0.397	73.775	0.000	0.000
5	4.608	0.217	0.000	0.000	2.293
6	5.815	0.172	0.000	0.000	0.000

- Mode shapes of building resting at plain with shear walls placed at corner symmetrically as shown below



(a) (b) (c)



(d) (e) (f)

Figure 4.28: (a) Mode shape 1; (b) Mode shape 2; (c) Mode shape 3; (d) Mode shape 4; (e) Mode shape 5; (f) Mode shape 6 for model at plain with shear wall position symmetrically at corners

Table 4.22: Frequency and period of vibration of building at plain with shear wall placed symmetrically at corners

Mode	Frequency Hz	Period seconds	Participation X %	Participation Y %	Participation Z %
1	0.763	1.310	0.000	0.000	86.432
2	2.248	0.445	0.000	0.000	0.021
3	2.447	0.409	73.591	0.027	0.000
4	2.485	0.402	0.000	0.000	8.092
5	4.655	0.215	0.000	0.000	2.201
6	5.005	0.200	0.000	0.000	0.049

Studying the behavior of building through chart representation, the overall changes in frequency and mass participation of the building with no shear wall and with shear wall, placed symmetrically at center and at corners.

The mode shapes has calculated the period of vibration and the frequency of vibration which are related in inverse terms. The graphical representation in figures 4.29 and 4.30 gives an overall view of frequency in Hz and time period in seconds showing the responses with shear wall and without shear wall in buildings at slope and at plain. Also, it can be seen that the frequency increases as the slope increases and period of vibration decreases as the slope increases [12].

- Frequency in Hz

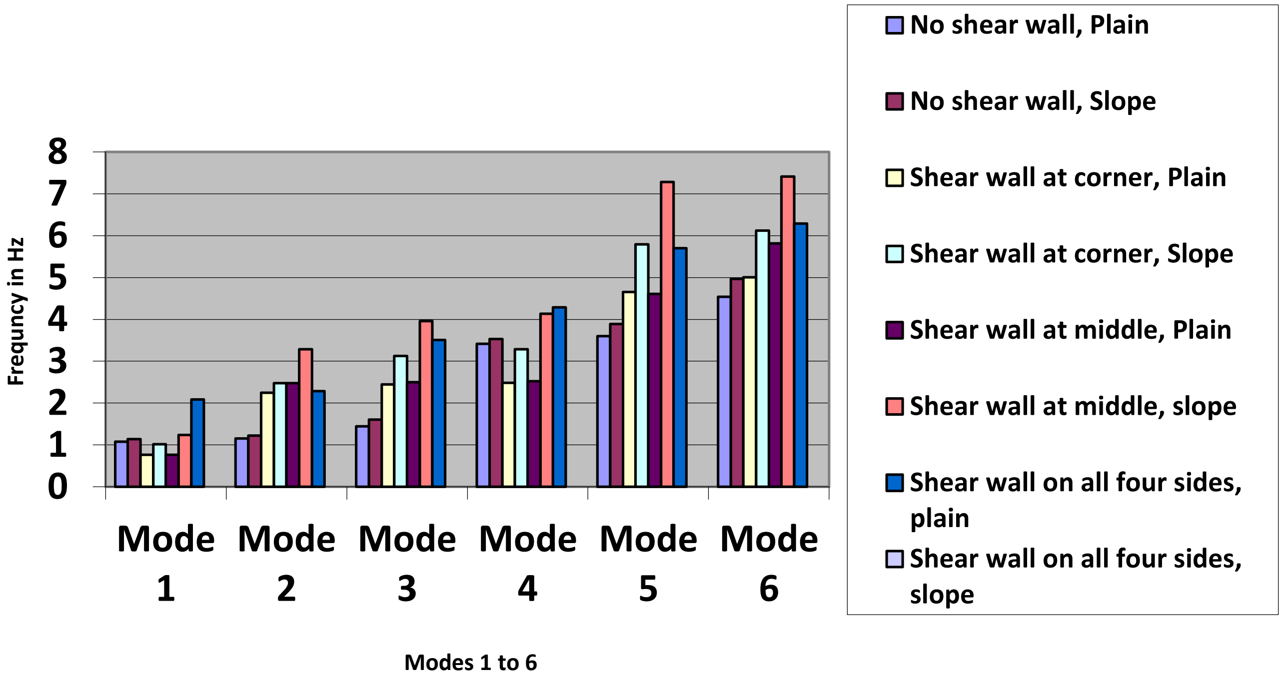


Figure 4.29: Frequency in Hz for various mode shapes of building at plain and slope (with and without shear walls)

- Time period in seconds

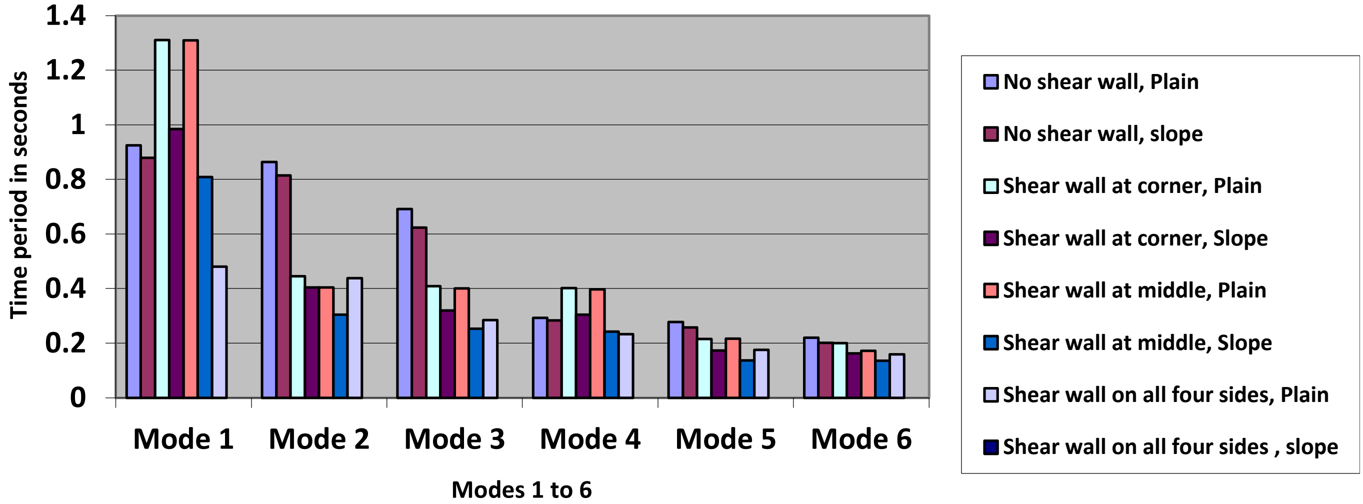


Figure 4.30: Time period in seconds for various mode shapes of building at plain and slope (with and without shear walls)

4.2.2 Torsion effect on beam and column after application of shear walls at different position

4.2.2.1 Torsion effect in beam element 73, after the addition of shear walls symmetrically at various positions in building at plain and slope

- At Plain

Table 4.23: Torsion in beam 73 when shear wall is provided at 2 sides (corner) symmetrically in building resting at plain

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
73	2	56	-6.123	-1.420	-3.812	-1.055	-8.403	-5.523
73	2	55	6.123	1.420	3.812	1.055	6.846	0.155

Table 4.24: Torsion in beam 73 when shear wall is provided at 4 sides symmetrically in building resting at plain

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
73	2	55	22.525	16.668	6.718	0.629	10.895	33.950
73	2	56	-22.525	-16.668	-6.718	-0.629	-15.981	-32.722

Table 4.25: Torsion in beam 73 when shear wall is provided at two sides (middle) symmetrically in building resting at plain

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
73	2	55	8.448	11.474	2.829	0.316	4.884	24.150
73	2	56	-8.448	-11.474	-2.829	-0.316	-6.431	-21.745

- At slope

Table 4.26: Torsion in beam 73 when shear wall is provided at 2 sides (corner) symmetrically in building resting at slope

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
73	2	55	1.956	1.150	3.885	5.062	6.583	1.255
73	2	56	-1.956	-1.150	-3.885	-5.062	-8.961	-3.364

Table 4.27: Torsion in beam 73 when shear wall is provided at 2 sides (middle) symmetrically in building resting at slope

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
73	2	55	10.434	7.158	5.874	3.616	10.071	14.156
73	2	56	-10.434	-7.158	-5.874	-3.616	-13.497	-14.476

Table 4.28: Torsion in beam 73 when no shear wall is provided in building resting at slope

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
73	2	55	2.037	63.739	18.539	5.172	33.366	124.994
73	2	56	-2.037	-63.739	-18.539	-5.172	-40.791	-129.962

Torsion in beam is comparatively bearable as compare to that in columns, therefore, it can be seen in table 4.28 that the torsion is 5.172 kN-m when no shear wall was used whereas this torsion has reduced to be 3.616 kN-m when shear wall is placed at the middle position and is more effective than the corner position of shear wall. Maximum torsion is found in beam element of building at slope and is compared in the figure 4.31.

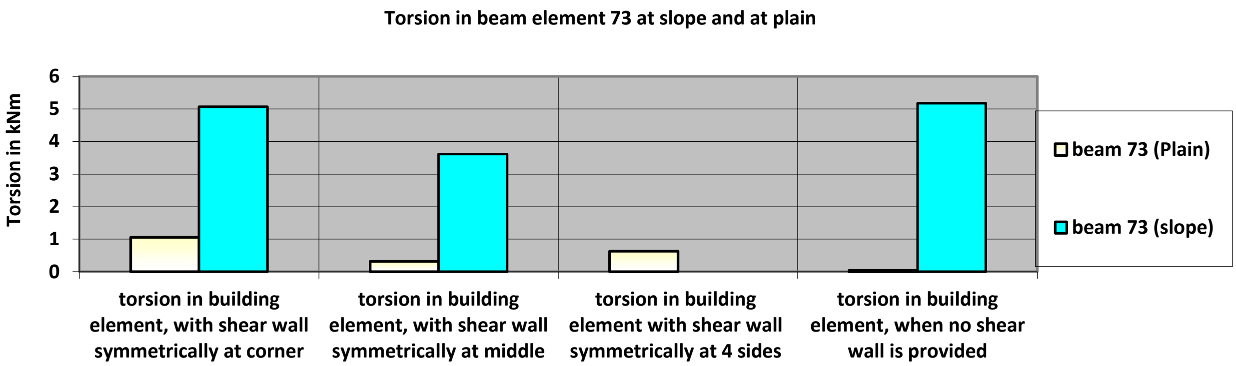


Figure 4.31: Torsion in building element 73, for both the buildings at plain and at slope (with and without shear walls)

4.2.2.2 Torsion effect in column element 51, after the addition of shear walls symmetrically at various positions in building at plain and slope

- At Plain

Table 4.29: Torsion in column element 51 when shear wall is provided at 2 sides (corner) symmetrically in building resting at plain

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
51	2	29	62.074	5.835	0.002	0.000	0.005	9.543
51	2	77	-62.074	-5.835	-0.002	-0.000	-0.004	-22.550

Table 4.30: Torsion in column element 51 when shear wall is provided at 2 sides (middle) symmetrically in building resting at plain

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
51	2	29	71.674	6.095	0.002	0.000	0.005	10.864
51	2	77	-71.674	-6.095	-0.002	-0.000	-0.005	-22.660

Table 4.31: Torsion in column element 51 when shear wall is provided at 4 sides symmetrically in building resting at plain

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
51	2	29	66.264	5.358	0.110	0.153	0.170	9.696
51	2	77	-66.264	-5.358	-0.110	-0.153	-0.438	-19.988

Table 4.32: Torsion in column element 51 when no shear wall is provided in building resting at plain

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
51	2	29	241.614	106.747	0.005	0.004	0.015	194.038
51	2	77	-241.614	-106.747	-0.005	-0.004	-0.013	-393.074

At plain the maximum torsion effect in building element (beam) is found to be reduced the most when shear wall is placed at middle of the frame (exterior). The exact values for torsion are given in table 4.29, 4.30, 4.31 and 4.32 with different criterion for shear wall positions and without shear wall.

- At slope

Table 4.33: Torsion in column element 51 when no shear wall is provided in building resting at slope

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
51	2	29	221.874	83.806	16.023	25.212	14.794	18.380
51	2	77	-221.874	-83.806	-16.023	-25.212	-46.352	-300.318

Table 4.34: Torsion in column 51 when shear wall is provided at 2 sides (corner) symmetrically in building resting at slope

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
51	2	29	43.915	8.450	7.206	2.247	10.689	7.516
51	2	77	-43.915	-8.450	-7.206	-2.247	-16.704	-25.162

Table 4.35: Torsion in column 51 when shear wall is provided at 2 sides (middle) symmetrically in building resting at slope

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
51	2	29	41.974	16.472	7.274	5.836	2.108	13.380
51	2	77	-41.974	-16.472	-7.274	-5.836	-28.876	-52.827

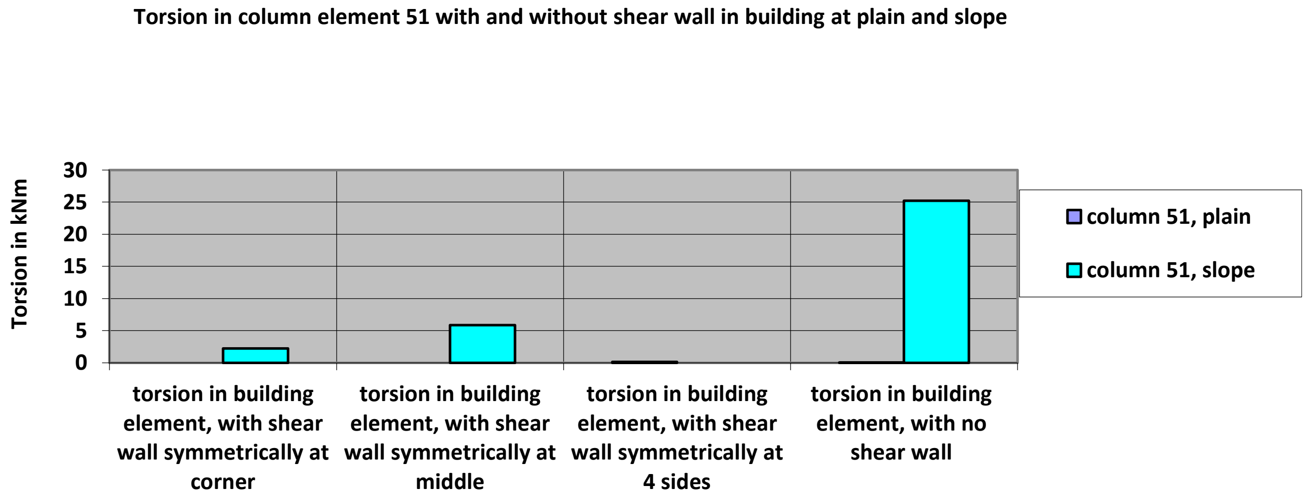


Figure 4.32: Torsion in building element 51, for both the buildings at plain and at slope (with and without shear walls

4.2.3 Displacement at specific nodes of building at plain and slope with and without shear walls

- Displacement of top nodes of building at plain

Table 4.36: Displacement at nodes 69, 70, 71 and 72 when shear wall is provided at 2 sides (corner) symmetrically in building resting at plain

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm
69	2	28.821	0.283	0.015	28.822
70	2	28.802	0.259	0.004	28.804
71	2	28.756	3.190	0.013	28.933
72	2	28.755	3.339	0.023	28.948

Table 4.37: Displacement at nodes 69, 70, 71 and 72 when shear wall is provided at 2 sides (middle) symmetrically in building resting at plain

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm
69	2	26.939	0.546	0.015	26.944
72	2	26.939	0.546	0.018	26.944
70	2	26.928	3.004	0.003	27.095
71	2	26.928	3.004	0.003	27.095

Table 4.38: Displacement at nodes 69, 70, 71 and 72 when shear wall is provided at 4 sides symmetrically in building resting at plain

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm
69	2	24.065	0.065	1.197	24.095
72	2	24.065	0.065	1.197	24.095
70	2	24.055	2.661	0.403	24.205
71	2	24.055	2.661	0.403	24.205

Table 4.39: Displacement at nodes 69, 70, 71 and 72 when no shear wall is in building resting at plain

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm
72	2	118.444	0.828	0.003	118.447
69	2	118.444	0.828	0.013	118.447
71	2	118.443	0.110	0.003	118.443
70	2	118.443	0.110	0.005	118.443

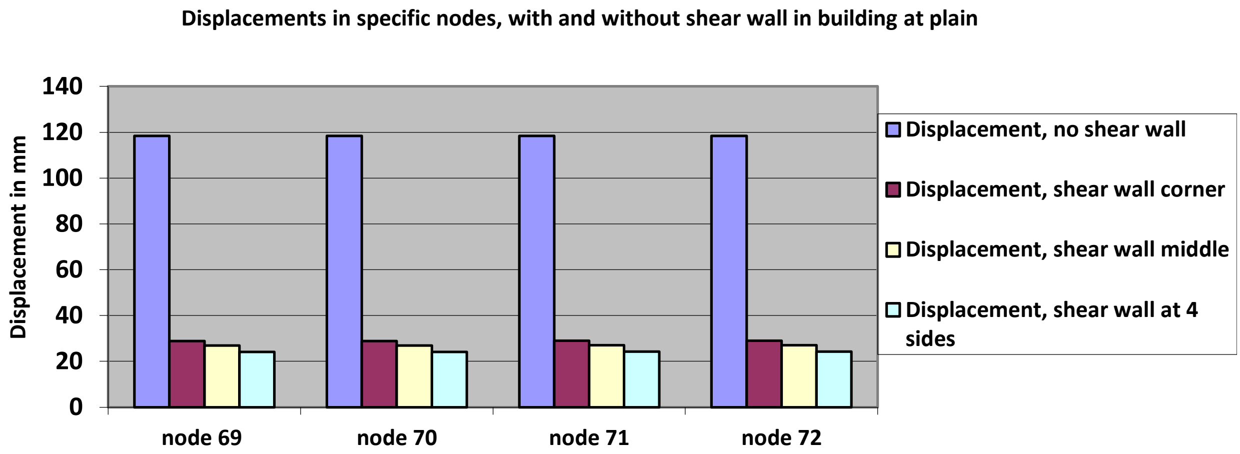


Figure 4.33: Displacement of nodes 69, 70, 71 and 72, with and without shear wall in building at plain

- Displacement of top nodes in building at slope

Table 4.40: Displacement at nodes 69, 70, 71 and 72 when shear wall is provided at 2 sides (corner) symmetrically in building resting at slope

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm
69	2	20.261	0.167	27.977	34.543
70	2	20.234	0.110	12.686	23.882
71	2	20.151	2.661	5.727	21.118
72	2	20.128	2.705	20.766	29.046

Table 4.41: Displacement at nodes 69, 70, 71 and 72 when shear wall is provided at 2 sides (middle) symmetrically in building resting at slope

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm
69	2	10.844	0.136	18.354	21.319
72	2	10.844	0.136	18.354	21.319
70	2	10.824	1.417	6.719	12.818
71	2	10.824	1.417	6.719	12.818

Table 4.42: Displacement at nodes 69, 70, 71 and 72 when no shear wall is in building resting at slope

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm
69	2	77.256	0.563	12.876	78.324
72	2	77.256	0.563	12.876	78.324
70	2	77.250	0.018	4.397	77.375
71	2	77.250	0.018	4.395	77.375

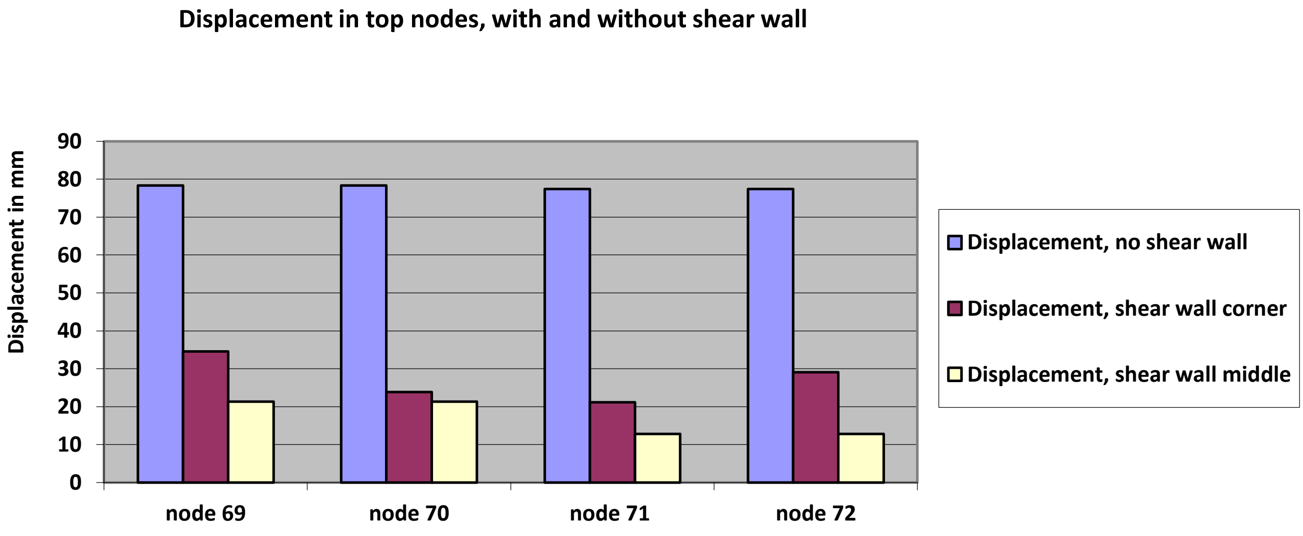


Figure 4.34: Displacement in building at top nodes, for the building at slope (with and without shear walls)

- Comparison of the buildings, the building at plain and the building at slope to note the displacement of top nodes

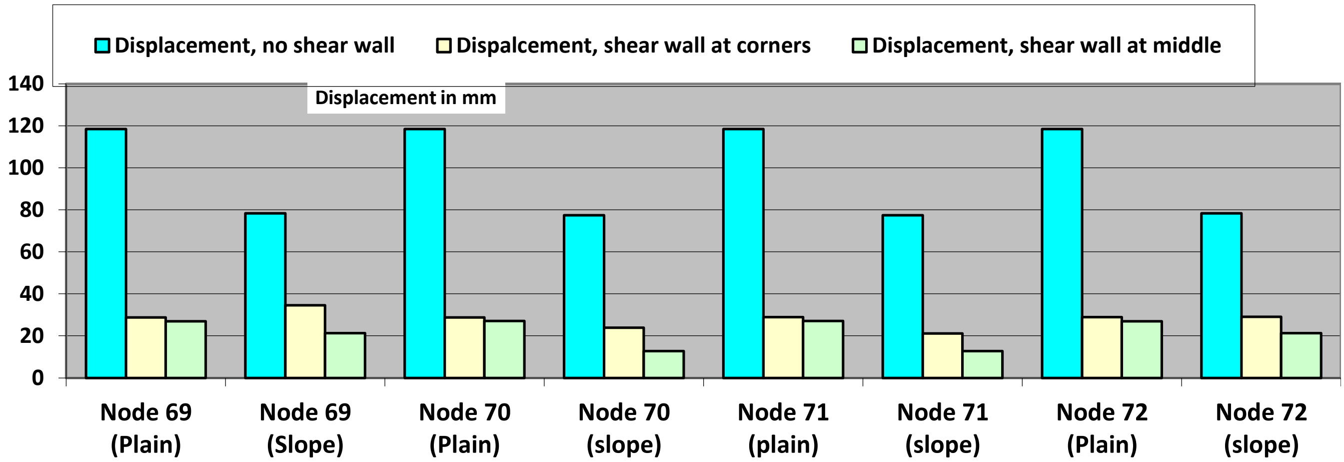


Figure 4.35: Displacement in building at top nodes, for the building at plain and slope (with and without shear walls)

- Displacement of bottom nodes of the building at plain

Table 4.43: Displacement at nodes 53, 54, 55 and 56 when shear wall is provided at 2 sides (corner) symmetrically in building resting at plain

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm
53	2	4.233	0.115	0.007	4.235
54	2	4.219	0.101	0.003	4.221
55	2	4.177	1.917	0.000	4.596
56	2	4.168	1.994	0.004	4.621

Table 4.44: Displacement at nodes 53, 54, 55 and 56 when shear wall is provided at 2 sides (middle) symmetrically in building resting at plain

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm
56	2	4.011	0.219	0.003	4.017
53	2	4.011	0.218	0.005	4.017
55	2	3.993	1.841	0.003	4.397
54	2	3.993	1.841	0.003	4.397

Table 4.45: Displacement at nodes 53, 54, 55 and 56 when shear wall is provided at 4 sides, symmetrically in building resting at plain

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm
56	2	3.642	0.040	0.180	3.646
53	2	3.642	0.040	0.180	3.646
54	2	3.618	1.653	0.090	3.979
55	2	3.618	1.653	0.090	3.979

Table 4.46: Displacement at nodes 53, 54, 55 and 56 when no shear wall is provided in building resting at plain

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm
55	2	44.903	0.060	0.004	44.903
54	2	44.903	0.060	0.001	44.903
56	2	44.883	0.402	0.015	44.885
53	2	44.883	0.402	0.005	44.885

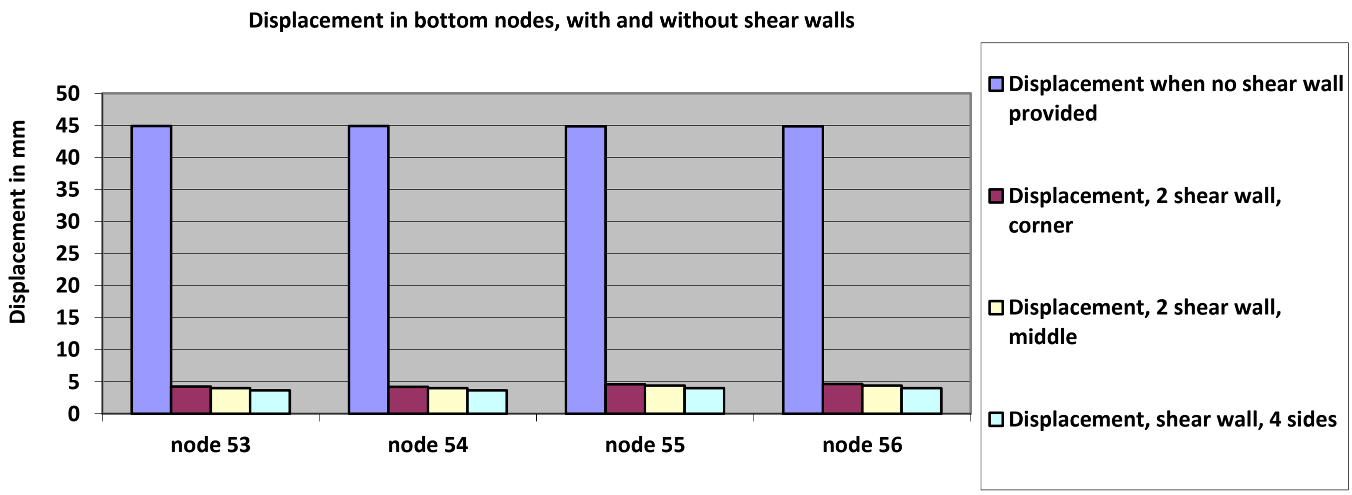


Figure 4.36: Displacement in building at bottom nodes, for the building at plain (with and without shear walls)

Highest displacement is found in building with no shear wall and the building displacement has tended to reduce a lot when shear walls are positioned at the corner and middle (exterior).

- Displacement of bottom nodes 53, 54, 55 and 56 of the building at slope

Table 4.47: Displacement at nodes when shear wall is provided at 2 sides (corner) symmetrically in building resting at slope

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm
53	2	0.898	0.041	2.464	2.623
54	2	0.898	0.020	1.089	1.412
55	2	0.893	0.922	0.786	1.505
56	2	0.890	0.934	2.383	2.710

Table 4.48: Displacement at nodes when shear wall is provided at 2 sides (middle) symmetrically in building resting at slope

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm
54	2	0.543	0.436	0.678	0.972
55	2	0.543	0.436	0.678	0.972
53	2	0.540	0.029	1.673	1.758
56	2	0.540	0.029	1.673	1.758

Table 4.49: Displacement at nodes when no shear wall is provided in building resting at slope

Node	L/C	X-Trans mm	Y-Trans mm	Z-Trans mm	Absolute mm
54	2	5.525	0.003	0.528	5.550
55	2	5.525	0.003	0.526	5.550
53	2	5.522	0.134	2.221	5.953
56	2	5.522	0.134	2.222	5.954

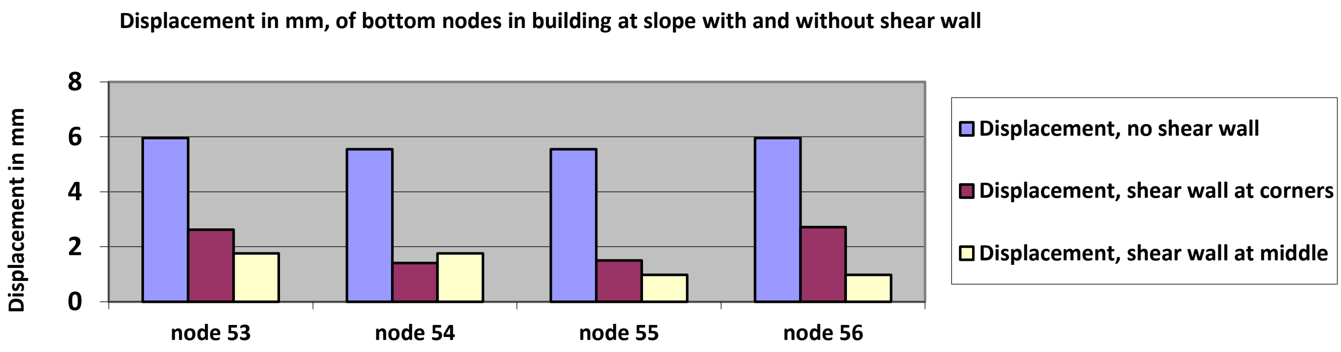


Figure 4.37: Displacement in building at bottom nodes, for the building at slope (with and without shear walls)

- Comparison of the buildings, the building at plain and the building at slope to note the displacement of bottom nodes

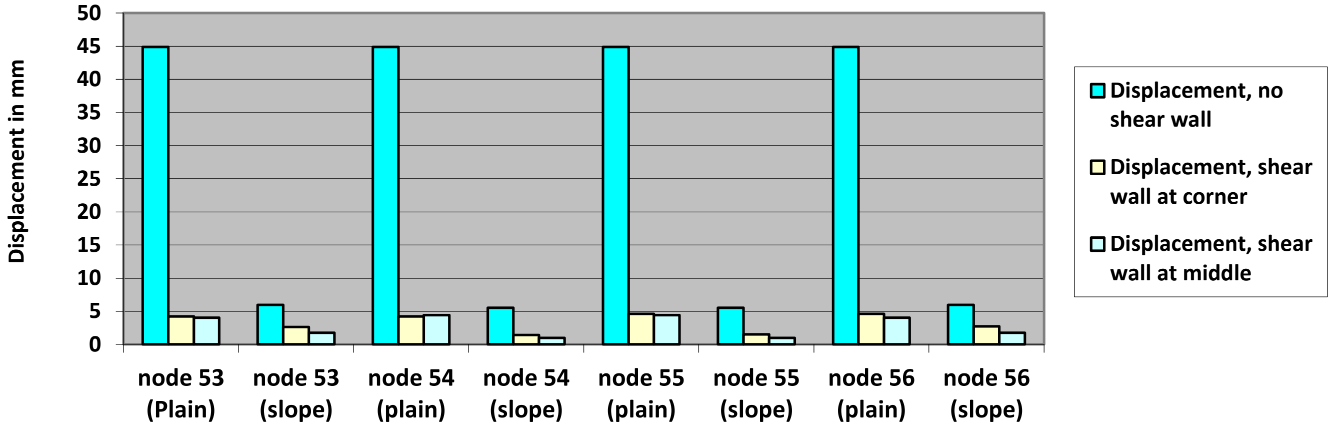


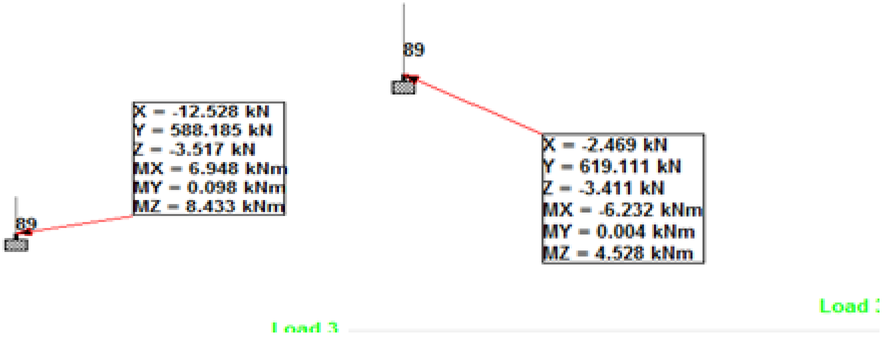
Figure 4.38: Displacement in building at bottom nodes, for the building at plain and slope (with and without shear walls)

4. 3 Wind load study

Table 4.50: Wind pressures

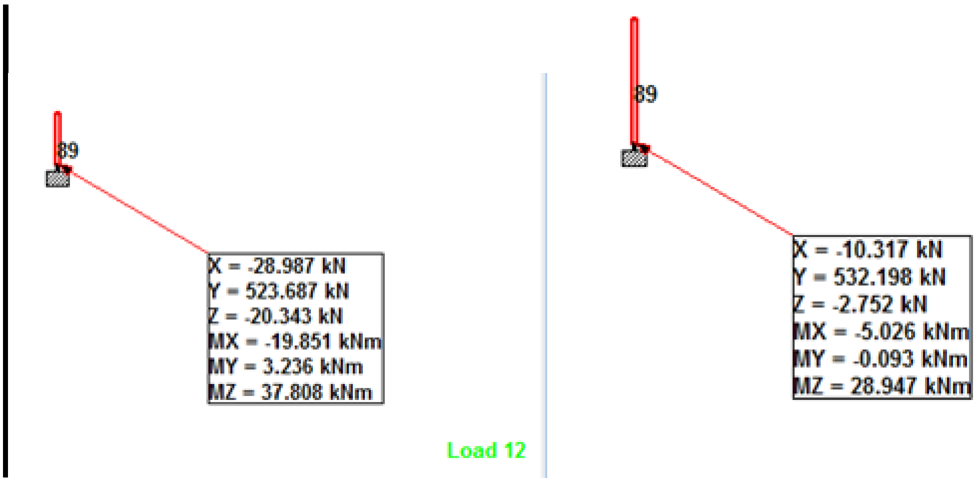
H	K_2	V_z (for plain)	P_z (N/mm^2) (for plain)	P_z KN/m^2 (for plain)	V_z (for slope)	P_z (N/mm^2) (for slope)	P_z (KN/m^2) (for slope)
1-10	.91	35.49	755.7240	.75572	42.3892	1078.10	1.0781
11	.922	35.958	775.78	.775786	42.948235	1106.730	1.1067
12	.934	36.426	796.11	.7961120	43.507214	1135.726	1.1357
13	.946	36.894	816.700	.816700	44.066193	1165.097	1.1650
14	.958	37.362	837.55	.83755	44.625172	1196.265	1.196265
15	.97	37.83	858.665	.858665	45.184415	1224.97	1.22497
16	1.01	39.39	930.9432	.93094	47.0474	1328.074	1.32807
17	1.05	40.95	1006.141	1.0061	48.9106	1435.34	1.43534
18	1.09	42.51	1084.26	1.08426	50.77394	1546.79	1.54679
19	1.13	44.07	1165.298	1.16529	52.63720	1662.404	1.66240
20	1.17	45.63	1249.25	1.249	54.50047	1782.18	1.78218

- After when the wind load is applied on both the buildings the change in the response of two same columns that is column 89 is seen in terms of shear force, bending moments as shown.



(a) (b)

Figure 4.39: (a) Column 89, static model of short column axial loads and the corresponding bending moments; (b) Column 89, static model of long column axial force and corresponding bending moments



(a) (b)

Figure 4.40: (a) Column 89, wind load model of short column axial load and the corresponding moments; (b) Column 89, wind load model of long column axial load and corresponding moments

CHAPTER-5

RESULTS AND DISCUSSIONS

5.1 Comparison of results

Comparing the results for models, it shows that base shear for the building at slope is more than the building at plain for about 4.5 % more at building at slope. Also, as mentioned in the objective of the study, the behavior of multi-storey building frame under dynamic response in terms of displacement, moment, shear, torsion and mode shapes. Response spectrum analysis has increased the effect of torsion, shears force bending moment and deflection in lower elements of buildings and thus are compared. Torsion effect is found to be more in short column in comparison to long column. The displacement of the lower elements in a building at slope is lesser as compared to the displacement of nodes 53, 54, 55, 56. The results obtained for the critical elements are represented in the tables below.

Table 5.1 : Maximum moments floor wise in kN-m

Floor	Sloping ground	Plain ground	Percentage increase %
Ground floor	87.347	85.556	2.05
First floor	93.973	90.278	3.93
Second floor	87.347	80.809	7.48
Third floor	77.357	74.469	3.733
Fourth floor	90.374	92.803	2.7
Top floor	38.757	38.119	1.64

Table 5.2 : Maximum shear force floor wise in kN

Floor	Sloping ground	Plain ground	Percentage increase %
Ground floor	77.337	77.454	.151
First floor	79.183	77.134	2.58
Second floor	103.245	96.486	6.54
Third floor	100.140	104.150	4
Fourth floor	97.867	101.001	3.2
Top floor	94.693	94.033	.69

Table 5.3: Axial force in columns in kN

Column position	Sloping ground	Plain ground	Percentage increase %
Top	60.141	58.08	3.426
First	174.705	159.38	8.77
Second	287.839	260.685	9.43
Base	490.9	468.7	4.522

Table 5.4 : Bending moments in kN-m at column bases after application of dynamic forces

Support no.	Sloping ground	Plain ground	Percentage increase %
73	219.022	161.643	26.2
74	242.583	182.784	24.7
75	242.582	182.784	24.6
76	219.023	161.642	26.2

Table 5.5: Increase in Axial force in kN at base of buildings

Support no.	Sloping ground	Plain ground	Percentage increase %
73	871.066	853.005	2.07
76	870.99	853.003	2.06
82	1094.918	1078.8012	1.47
83	1095.602	1077.978	1.608

Table 5.6: Torsion in kN-m (Beam 73)

Node	Sloping ground	Sloping ground (Shear wall position at middle)	Plain ground	Plain ground (Shear wall position at middle)
55	5.172	3.616	0.040	0.316
56	-5.172	-3.616	-0.040	-0.316

Table 5.7: Torsion in kN-m (Column 51)

Node	Sloping ground	Sloping ground (Shear wall position at middle)	Plain ground	Plain ground (Shear wall position at middle)
29	25.212	2.247	0.004	0.00
77	-25.212	-2.247	-0.004	-0.00

Table 5.8: Displacement of top nodes in mm

Node	Sloping ground	Sloping ground (Shear wall position at middle)	Plain ground	Plain ground (Shear wall position at middle)
69	77.256	21.319	118.44	26.944
70	77.256	12.318	118.443	26.944
71	77.250	12.818	118.443	27.095
72	77.250	21.319	118.444	27.095

Table 5.9: Displacement of bottom nodes in mm

Node	Sloping ground	Sloping ground (Shear wall position at middle)	Plain ground	Plain ground (Shear wall position at middle)
53	5.953	1.758	44.853	4.017
54	5.550	0.972	44.903	4.397
55	5.550	0.972	44.903	4.397
56	5.954	1.758	44.903	4.017

Reduction in shear force at beam element and column element nodes after application of shear wall is as given in the table.

The reduction in shear (Y) in kN after application of shear wall, in beam, 89% in building at slope and 98.2 % reduction in shear in case of building at plain, the values are given in table 5.10.

Table 5.10 : Reduction in maximum shear (Y) in kN (Beam element) after application shear wall

Node	Sloping ground	Sloping ground (Shear wall position at corner)	Plain ground	Plain ground (Shear wall position at middle)
55	63.739	1.150	79.263	1.420
56	-63.739	-1.150	-79.263	-1.420

Table 5.11: Reduction in maximum shear (Y) in kN (column element) after application shear wall

Node	Sloping ground	Sloping ground (Shear wall position at corner)	Plain ground	Plain ground (Shear wall position at middle)
29	83.806	8.450	106.747	5.835
77	-83.806	-8.450	-106.747	-5.835

The reduction in shear (Y) in kN after application of shear wall, in column, 89% in building at slope and 94.533% reduction in shear in case of building at plain.

CHAPTER – 6

CONCLUSION

6.1 Conclusion

1. In this study, performance of residential building frames are studied considering dead load and live load combination, thus studying the static behavior of the building on plane and sloping ground to study short column effect. It has been concluded that a short column is safe under normal loading (D.L and L.L).
2. In beam forces, maximum bending moment and maximum shear force are calculated and it is observed that ground floor is efficient because of direct contact with soil and foundation.
3. In static analysis, there is no considerable difference found between bending moments and shear forces of two building components whereas during response spectrum analysis the change in bending moments of short column in building at slope and long columns in building at flat were noticeably higher to long columns.
4. In column force, maximum axial force is calculated and it is observed that maximum load is in base columns because it resist complete load of residential building and as seen in top floor axial force is reduced up to 3 times of columns in lower floor. Also, that the building is found stable under static forces.
5. Calculated frequency increases as for the building at slope.
6. Calculated time period of vibration is lesser for the building at slope.
7. Response spectrum analysis has increased the effect of torsion, shears force bending moment and deflection in lower elements of buildings and thus are compared. Torsion effect is found to be more in short column in comparison to long column. The displacement of the lower elements in a building at slope is lesser as compared to the displacement of nodes 53, 54, 55, 56.
8. Columns are more affected to torsion in comparison to the beams.
9. Shear walls has tend to reduce the shear effect to 89% in building at slope and almost about 90% in buildings at plain.
10. Shear wall position at the middle is found to be more effective in terms of safer values for restricting displacement of top and bottom nodes.

11. By providing shear wall the buildings were relieved from excessive shear but are not torsionally balanced too much extent.
12. Results indicate more ductility of common structure and although more initial stiffness of sloping lot structures.
13. Also, wind load effect on building has made a change in the moments of beam and column elements.

6.2 Future Scope

1. The study can be further extended to analysis of building at varying slopes.
2. The building of higher degree of irregularities can be analysed.
3. Analysis can be done using software SAP2000, ETAB, ANSYS.
4. Dynamic analysis can be done using Time history method.
5. Comparison of varying analytical methods can be done, such as Time history method and Response spectrum analysis.
6. Analysis can be performed with different seismic zone.

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