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ANALYSIS OF BUILDING CONSTRUCTED ON SLOPING GROUND CONSIDERING DIFFERENT PARAMETERS USING STADD.PRO

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Abstract: Involves the analysis of simple 2-D frames of varying floor heights and varying no of bays using a very popular software tool STAAD Pro. Using the analysis results various graphs were drawn between themaximum axial force, maximum shear force, maximum bending moment, maximum tensile force and maximum compressive stress being developed for the frames on plane ground and sloping ground. The graphs used to drawn comparison between the two cases and the detailed study of “SHORT COLOUMN EFFECT” failure was carried up. In addition to that the detailed study of seismology was undertakenand the feasibility of the software tool to be used was also checked. Till date many such projects havebeen undertaken on this very topic but the analysis were generally done for the static loads i.e. dead load, live load etc, but to this the earthquake analysis or seismic analysis is to be incorporated. To create a technical knowhow, two similar categories of structures were analyzed, first on plane ground and another on a sloping ground. Then the results were compared. At last the a structure would be analyzed and designed on sloping ground for all possible load combinations pertaining to IS 456, IS 1893 and IS 13920manually.

Key words- earthquake analysis, structures, shear force, bending moment, compressive, stress.

1.1INTRODUCTION

Seismology is the study of vibrations of earth mainly caused by earthquakes. The study of these vibrations by various techniques, understanding the nature and various physical processes that generate them from the major part of the seismology. Elastic rebound theory is one such theory, which was able to describe the phenomenon of earthquake occurring along the fault lines. Seismology as such is still a very unknown field of study where a lot of things are yet to be discovered. The above Picture is showing the fault lines and we can see that epicenters are all concentrated all along the fault lines. The reason for seismic activities occurring at places other than the fault lines are still a big question mark. Also the forecasting of earthquake has not been done yet and would be a landmark if done so. There is general saying that it’s not the earthquake which kills people but its the bad engineering which kills people. With industrialization came the demand of high rise building and came dangers with that. A seismic design of high rise buildings has assumed considerable importance in recent times. In traditional methods adopted based on fundamental mode of the structure and distribution of earthquake forces as static forces at various stories may be adequate for structures of small height subjected to earthquake of very low intensity but as the number of stories increases the seismic design demands more rigorous. During past earthquakes, 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. Two examples of buildings with short columns in buildings on a sloping ground and buildings with a mezzanine floor can be seen in thefigure given below.

1.2Behaviour of short columns

Poor behaviour of short columns is due to the fact that in an earthquake, a tall column and a shortcolumn of same cross section move horizontally by same amount which can be seen from the given figure below. There is another special situation in buildings when short-column effect occurs. Consider a wall *(masonry* or RC) of partial height built to fit a window over the remaining height. The adjacent columns behave as short columns due to presence of these walls. In many cases, other columns inthe same storey are of regular height, as there are no walls adjoining them. When the floor slab moves horizontally during an earthquake, the upper ends of these columns undergo the same displacement. However, the stiff walls restrict horizontal movement of the lower portion of a short column, and it deforms by the full amount over *the short height* adjacent to the window opening. On the other hand, regular columns deform over the *full height.* Since the effective height over which a short column can freely bend is small, it offers more resistance to horizontal motion and thereby attracts a larger force as compared to the regular column. As a result, short column sustains more damage. X-cracking in a column adjacent to the walls of partial height. Innew buildings, *short column effect* should be avoided to the extent possible during *architectural design* stage itself. When it is not possible to avoid short columns, this effect must be addressed in structural design. The Indian Standard IS:13920-1993 for ductile detailing of RC structures requires special confining reinforcement to be provided over the full *height* of columns that are likely to sustain short column effect. The special confining reinforcement *(i.e.,* closely spaced closed ties) must extend beyond the short column into the columns vertically aboveand below by a certain distance. In existing buildings with short columns, different retrofit solutions can be employed to avoid damage in future earthquakes. Where walls present, the simplest solution is to close the openings by building a wall of full height - this will eliminate the short column effect.

If a short column is not adequately designed for such a large force, it can suffer significant damage during an earthquake. This behaviour is called ***Short Column Effect****.* The damage in these short columns is often in the form of X-shaped cracking - this type of damage of columns is due to ***shear failure .***

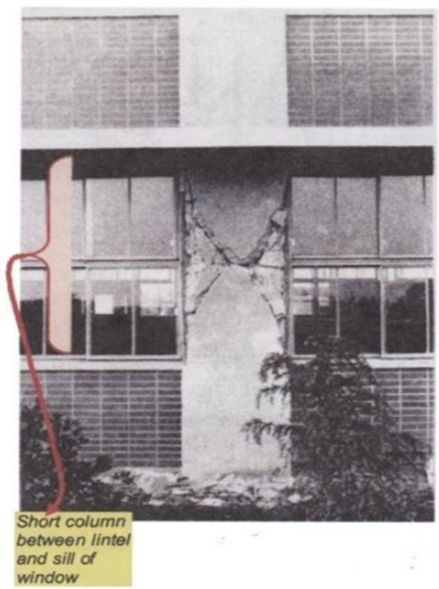


Figure-5 Short column between lintel and sill of window

Many situations with short column effect arise in buildings. When a building is rested on slopedground, during earthquake shaking all columns move horizontally by the same amount along with the floor slab at a particular level (this is called ***rigid floor diaphragm action****).* If short and tall columns exist within the same storey level, then the short columns attract several times largerearthquake force and suffer more damage as compared to taller ones. The short column effect alsooccurs in columns that support mezzanine floors or loft slabs that are added in between two regular floors.

1.3 OBJECTIVES

Following are the main objective of the present study:

a). To investigate the seismic performance of a multi-story steel frame building.

- When unbraced and then with different bracing arrangement such as cross bracing 'X' and diagonal bracing using Equivalent Static analysis, Response Spectrum analysis and linear Time History analysis.
- Under different earthquake loading and loading combinations.

b). To investigate the seismic response of a multi-story steel frame building.

c). Under same bracing configuration but with varying number ofstory i.e. with varying height of the building.

1.4 METHODOLOGY

a) A thorough literature review to understand the seismic evaluation of building structures and application of Equivalent

Static analysis,Response Spectrum analysis, and linear Time History analysis.

b) Seismic behaviour of steel frames with various concentric bracings and ecentric bracing geometrical and structural details.
c) Modeling the steel frame with various concentric bracing by computersoftware Staadpro.
d) Carry out Equivalent Static analysis, Response Spectrum analysisand linear Time History analysis on the models and arrive at conclusion.

1.5 SCOPE OF THE PRESENT STUDY

In the present study, modeling of the steel frame under the three analysis mentioned above using Staad Pro software is done and the results so obtained are compared. Conclusions are drawn based on the tables andgraphs obtained .

2.1 LITERATURE REVIEW

Blasi et al. (2018) numerically evaluated the 8 storey RC frames with infill panels for the interaction of infill panels with the frame along with failure modes. They designed frame according to the Euro code and perform incremental dynamic analysis for the assessment of analysis results. They modeled RC frame by lumped plasticity approach in which column are considered as linear elastic beam elements with shear and flexure springs connected in series and in beams flexure hinge is provided. The infill panels are modeled as equivalent struts with 3 strut models for local interaction and single strut model for global interaction. They found that the infill properties significantly modifies the structure, as the low shear resistance of panel governs the flexure failure and increased shear strength increases the shear failure modes. They also observed that the local interaction of infill and frames is correctly found with multi strut models and due to huge variability in infill panels single strut models is used for global interaction.

Chandel and Sreevalli (2018) conducted numerical study to understand the behavior of full infill RC masonry frame in comparison with Open Ground Storey frame. The modeling and analysis is performed using finite element technique with the help of ABAQUS 6.14 software. Different parameters are used for study includes aspect ratio (height to length ratio), number of bays and number of storey’s. It is found that the infill with lower aspect ratio have higher resistance to lateral loads due to the formation of diagonal strut at lower aspect ratios. The stiffness of an infill frame is higher than Open Ground Storey followed by bare frames. The stiffness change shows the failure of the interaction between the masonry and the infill. The ground storey columns are failed in both fully infill frame and OGS.

Bhatt and Narayan (2018) studied the comparison between conventional slabs and flat slabsfor 16 storey building considering shear wall with flat slabs. The assessment of seismic behavior done by using elastic time history method analysis in seismic zone IV with the help of Etabs software and the analysis is done according to the Indian standard code. They compared different parameters such as storey drift, storey displacement, time period and baseshear for different types of model. They observed that the flat slab have more storey drift and displacement, it is reduced after the addition of shear wall. The natural time period is reducedafter the add-on of shear wall.Base shear increases with increase in mass and stiffness of Building and the story displacement of all models increases with the height of the Structure.

Shendkar and kumar (2018) investigated two types of infill panels as unreinforced masonryand semi interlocked masonry infill for the out of plane behavior of in fills. The nonlinear staticpushover analysis is performed to investigate the nonlinear behavior of infill panels by using seismostruct finite element analysis software. The panels are model as double strut diagonal model to account nonlinear out of the plane behavior of in fills. The response reduction factorsfor RC frames are also calculated. For this study seven different numerical models are used foranalysis including bare frame, open ground storey, and full infill, side bay in filled with URMand later with semi interlocked masonry in fills. They observed that the ductility reduction factor as well as ductility reduces when frame have in fills. On the other hand strength factor increases when add- on of infill in the frames. The response reduction factor and base shear value of semi interlocked in fills are higher than other type. And the R factor is depends on the materials and its geometric configuration in the frames.

Abdelaziz et al. (2019) conducted study to understand the seismic performance of Reinforced concrete structures with fully and partially in filled with masonry. They modeled high-rise, medium- rise, and low-rise buildings with different configuration of infill along with varying story height. The configuration includes the bare frame, the in filled frame, the open ground story frame, and the partially opened ground story frame. The dynamic time–history analysis is performed and double strut nonlinear cyclic model for infill walls implemented using the structural software package Seismo Struct software. The analysis results are parametrically compared along with static pushover and dynamic analysis results. It is observed that infill walls configurations affect the frames. Its regular distribution improves the performance of theRC frames in terms of story drifts, lateral capacity and displacement control. The soft story phenomena, increases the drift ratios at that levelwhere infill walls are removed and the columns in this story are more vulnerable to collapse.

2.2 CRITIQUE/OUTCOME OF LITERATURE REVIEW

The various researchers have done analytical and experimental investigation for the study of flat slabs for various seismic parameters with different structural elements to identify the response of the structure under seismic loading. The outcome of the literature review is givenas follows by studying the work done by different researchers as a reference:

1. The researchers have done experimental and analytical study, using different computer software viz. STAAD-PRO, SAP2000, E-TABS, SAFE, and MIDAS/SDS etc.
2. Several comparative study of flat slat and conventional slabs in the literature shows thatflat slabs have poor lateral resistance against lateral forces.
3. The flexibility of the flat slabs structure is more than normal conventional frames because ofthe absence of beams.
4. The fundamental period and displacements of the flat-slab system is more in comparison tothe framed system and the flat-slab system have lower stiffness capacity and strength capacity.

3.1 Analysis of simple 2-dimensional reinforced concrete frames

SECTION- 2

Under phase 2, with full confidence on the STAAD Pro. Design tool, we proceed with the analysis of simple 2 dimensional frames. The analysis was done for both the static loadconditions and dynamic load conditions. The 2nd phase involves the analysis of frames on a plane ground and then on a sloping ground. This 2nd phase can be again broadly divided into following:-

- we first start with 2 storey frame. First we went with double bay and up to 4 bays both on a plane ground and on as sloping ground. we then compare theresults.
- we then go for 4 storey frame. For the same we start with double bay and up to 4 bays both on a plane ground and on a sloping ground.we then compare the results
- we then go for 6 storey frame. For the same we start with double bay and up to 4 bays both on a plane ground and on a sloping ground.we then compare the results.

3.2 PROBLEM STATEMENT:-

3.2.1 MEMBER PROPERTIES

- All Beams: Rectangular, 400 mmwidth X 500 mmdepth
- All Columns: Rectangular, 400 mm width X 500 mm depth.

3.2.2 MEMBER ORIENTATION

- All members : Default

3.2.3 MATERIAL CONSTANTS

- Modulus of Elasticity : 22 KN/sq.mm
- Density : 25 kn/cu.m
- Poisson's Ratio : 0.17

➢ **SUPPORTS**

- Base of all columns : Fixed
- LOADS

➢ **Load case 1 : Earth Quake Load**

- Zone- III(Z= 0.16)
- SPECIAL REVISITING MOMENT FRAME(RF= 5)
- Importance factor = 1
- SOIL TYPE MEDIUM
- RC frame
- DAMPING RATIO=5
- Self weight of the structure.
- 1893 LOAD IN GLOBAL X DIRECTION

➢ **LOAD CASE 2 : DEAD LOAD**
- Self weight of the structure.
- BEAMS : 30 KN/M IN GLOBAL Y DOWNWARD

➢ **LOAD CASE 3 : LIVE LOAD**
- Beams : 200 kN/m in global Y downward

➢ **Load Case 4 : DEAD + LIVE**
- L2 X 1.5 + L3 X 1.5

➢ **LOAD CASE 5 : DEAD +LIVE+EARTH QUAKE**
- L1 X 1.2 + L2 X 1.2+ L3X 1.2

➢ **Load Case 6 : DEAD +LIVE-EARTH QUAKE**
- -L1X1.2+L2X1.2+L3X1.2

➢ **LOAD CASE :7 DEAD+EARTHQUAKE**
- L1 X 1.5 + L2 X 1.5

➢ **LOAD CASE 8 : DEAD -EARTH QUAKE**
- -L1 X 1.5 + L2 X 1.5

➢ **LOAD CASE 9 : DEAD +EARTH QUAKE**
- -L1 X 1.5 + L2 X 0.9

➢ **LOAD CASE 10 : DEAD -EARTH QUAKE**
- -L1 X 1.5 + L2 X 0.9\

ANALYSIS TYPE : P-DELTA

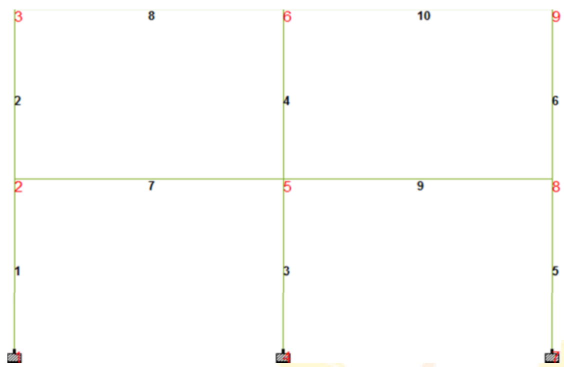
4.1 CONCRETE DESIGN:

- Consider all the load cases.
- Parameters: ultimate tensile strength of steel-415 N/sq.mm
- Concrete strength: 30N/sq.mm
- Clear cover: 30mm.
- Centre to centre distance of each beam- 4 m
- Height of eachstorey

a) First the structure is on level ground all the supporting columns being of 4 mheight.
b) For the second case the we design the frame for same loading combinations buton a sloping ground of I in 5.
c) Each beam length = 5m, So for this the dimensions of the supporting column are 4m, 4.5 m,5m, 5.5m and 6m.

PART 2.1(DOUBLE BAY)

PLANE GROUND



SLOPING GROUND

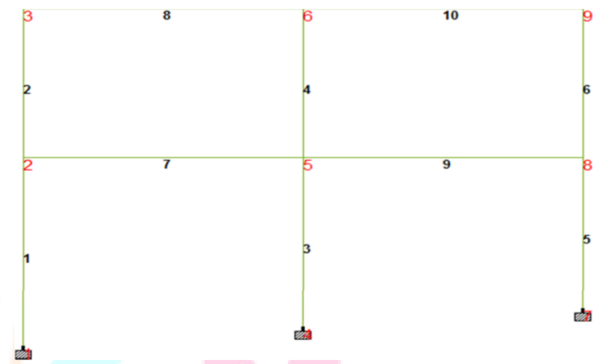


Table 4.1 ANALYSIS RESULTS FOR 2 BAY SYSTEMS ON PLANE AND ON A SLOPINGGROUND FOR TWO STORY FRAME.

BEAM NO	MAXIMUM AXIAL FORCE kN		MAXIMUM SHEAR FORCE kN		MAXIMUM BENDING MOMENT kN-m		MAXIMUM TENSILE FORCE N/ mm^2		MAXIMUM COMPRESSSIVE FORCE N/ mm^2	
	P*	S*	P	S	P	S	P	S	P	S
1	372	283	-26	-18	-51	-45	-2	-2	5	4
2	183	139	-41	-38	93	-84	-5	-5	6	6
3	767	635	-18	-18	-41	-46	-1	-3	5	5
4	390	323	-17	-17	-36	-36	-1	-2	4	3
5	767	635	26	-28	51	59	-2	-3	5	4
6	182	138	42	38	-93	-83	-5	-4	6	6
7	-26	-27	-174	-156	147	131	-9	-8	9	8
8	41	38	-180	-161	157	140	-9	-8	10	9
9	-26	-27	174	156	148	133	-9	-8	9	8
10	41	38	180	161	157	140	-9	-8	10	9

*[P= Plane ground, S= Sloping Ground]**

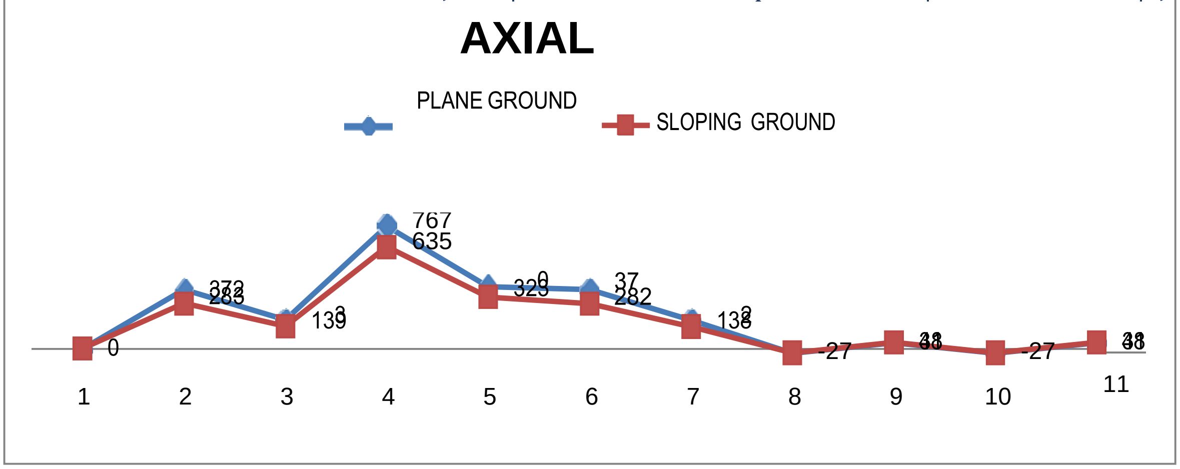


Fig. 4.1 MAXIMUM AXIAL FORCE

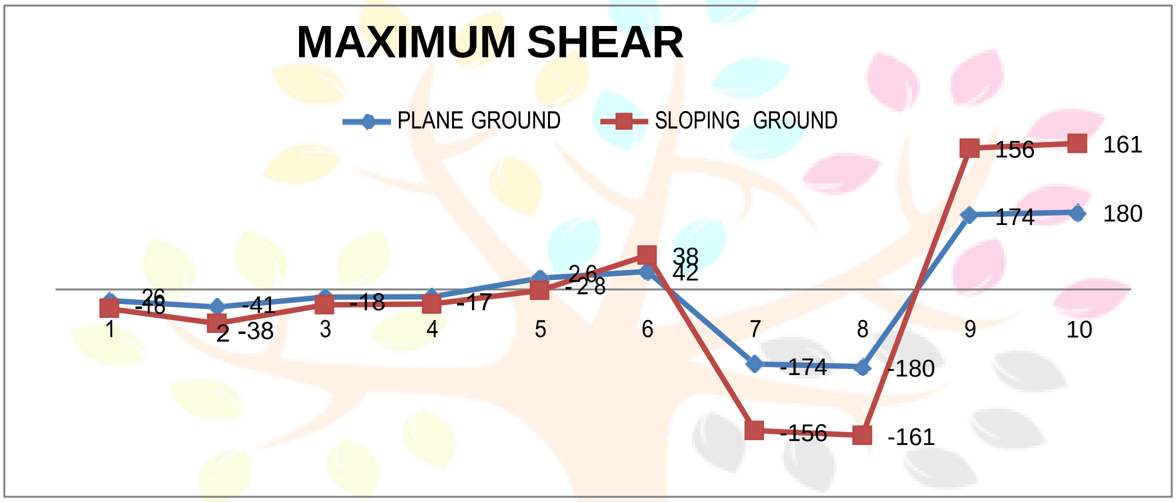


Fig. 4.2 Maximum Shear

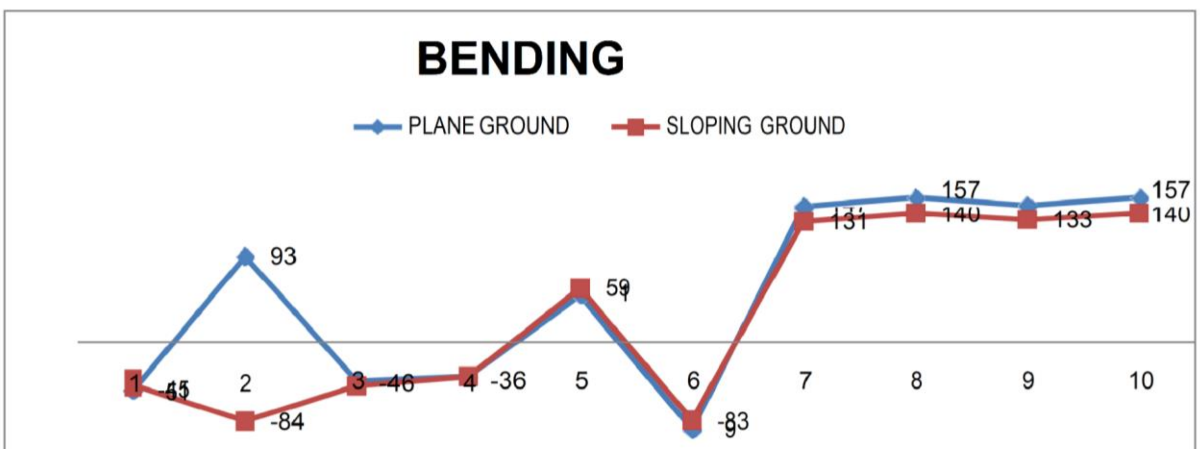


Fig: 4.3 Maximum Bending

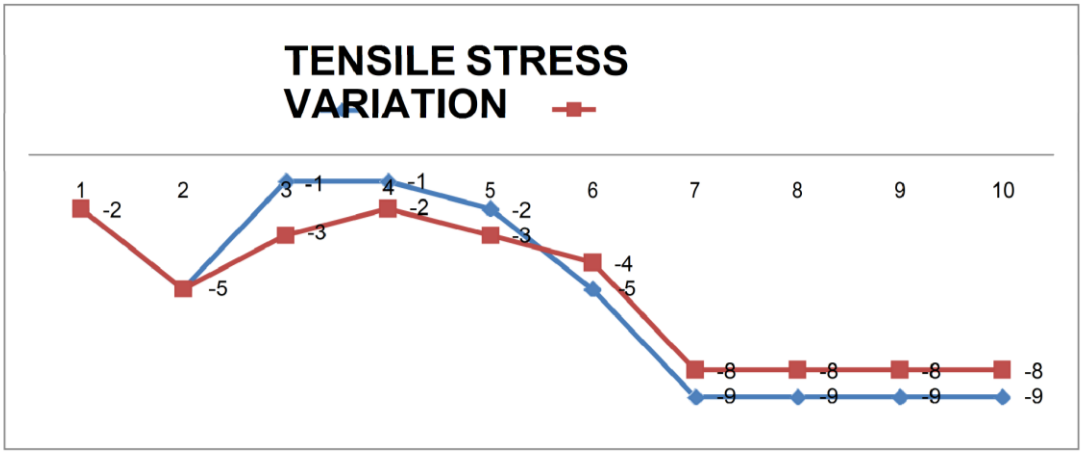


Fig: 4.4 Tensile Stress Variation

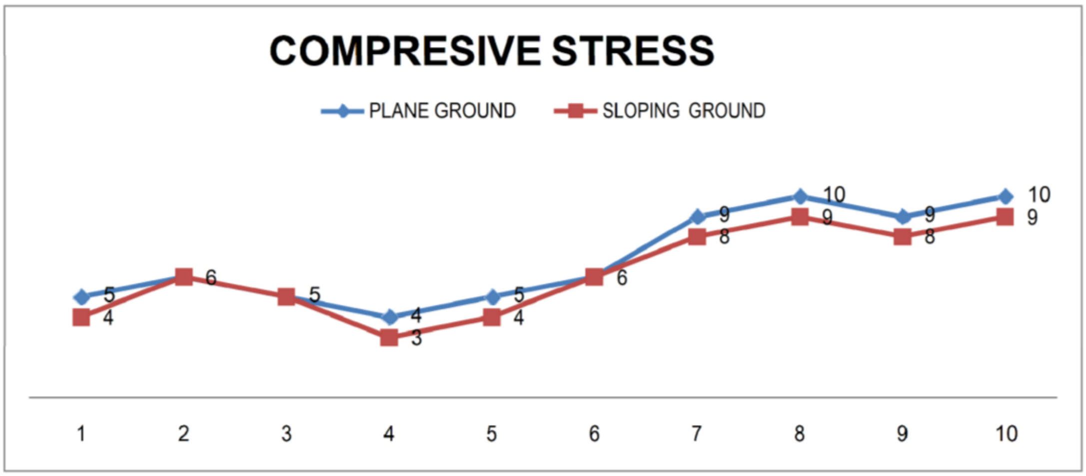
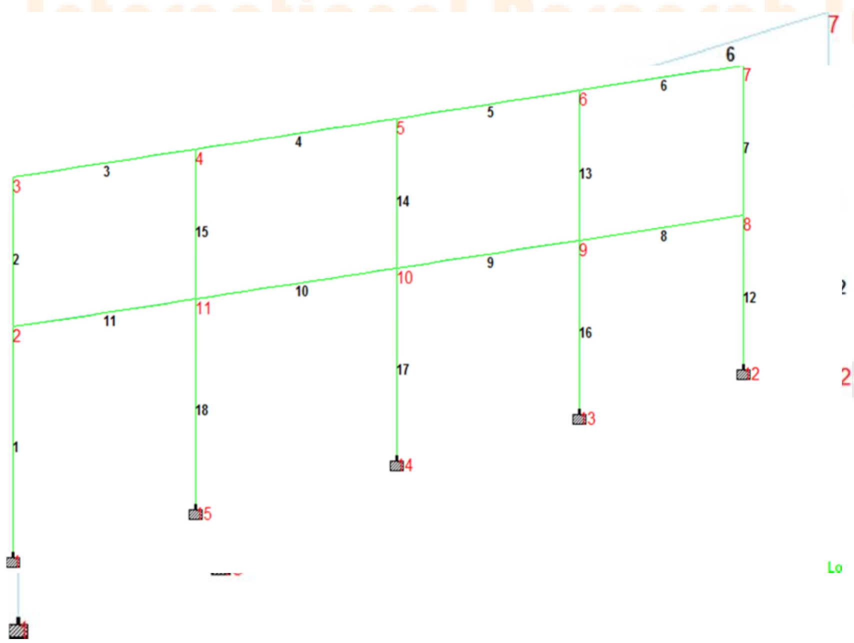


Fig::4.5 Compressive Stress

PART 2.1(4 BAY SYSTEMS)



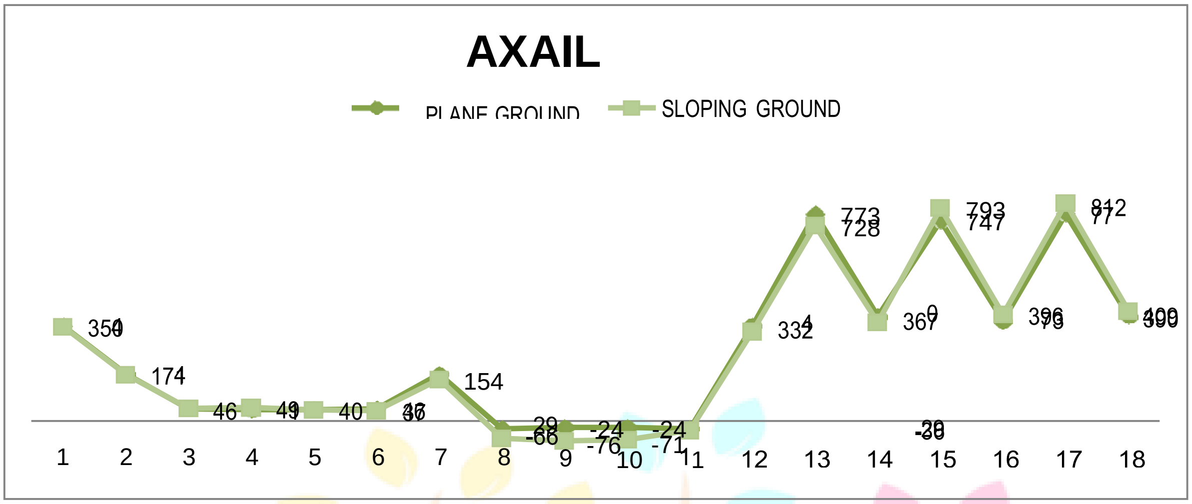
PLANE GROUND

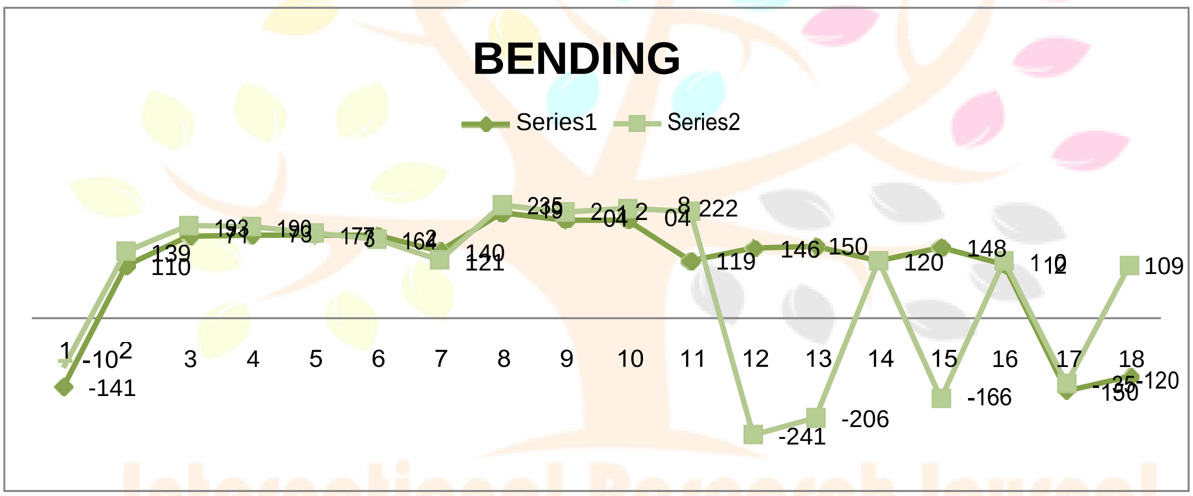
SLOPING GROUND

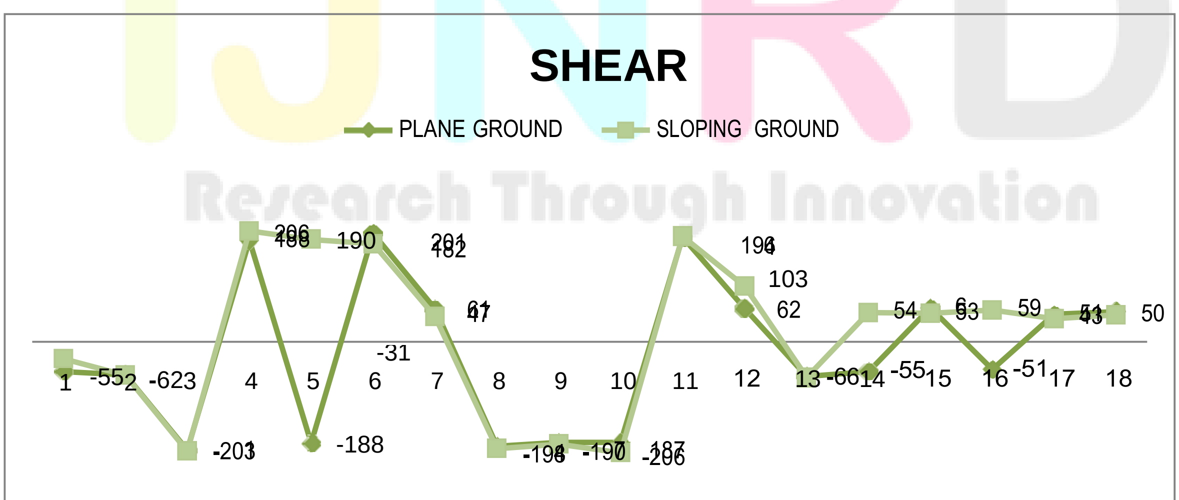
Table No. 2 ANALYSIS RESULTS FOR 4 BAY SYSTEMS ON PLANE AND ON A SLOPING GROUND FOR TWO STORY FRAME.

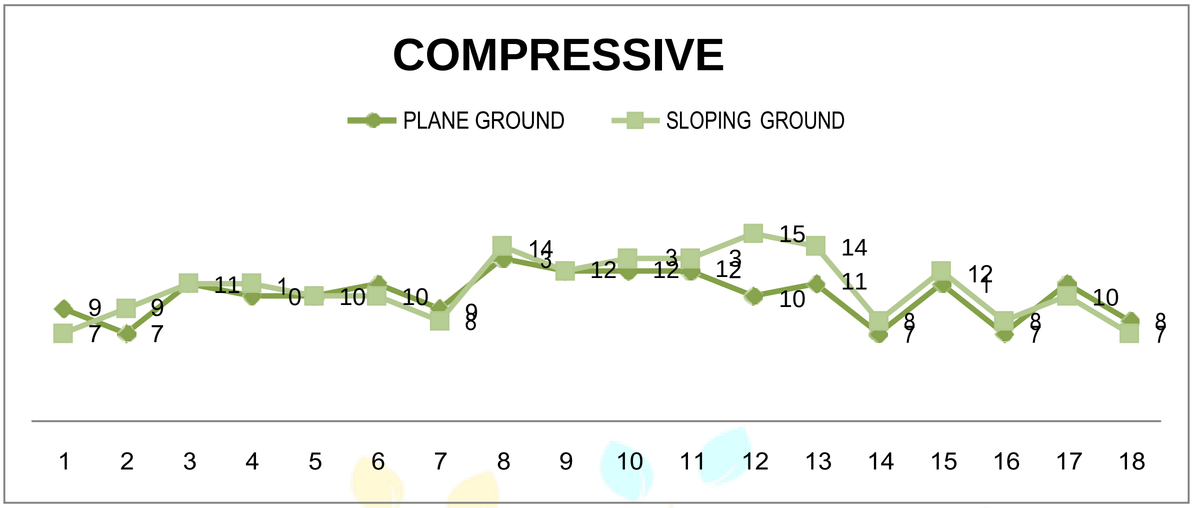
BEAM NO	MAXIMUM AXIAL FORCE kN		MAXIMUM SHEAR FORCE kN		MAXIMUM BENDING MOMENT kN-m		MAXIMUM TENSILE FORCE N/mm^2		MAXIMUM COMPRESSSIVE FORCE N/mm^2	
	P*	S*	P	S	P	S	P	S	P	S
1	-7	4	354	350	9	7	-55	-31	-141	-102
2	-7	7	174	171	7	9	-61	-62	110	139
3	11	11	46	46	11	11	-201	-203	171	193
4	10	11	41	49	10	11	188	206	173	190
5	10	10	41	40	10	10	-188	190	173	177
6	11	9	46	37	11	10	201	182	172	164
7	7	6	174	154	9	8	61	47	140	121
8	13	13	-29	-66	13	14	-194	-198	219	235
9	12	13	-24	-76	12	12	-187	-190	204	221
10	12	13	-24	-71	12	13	-187	-206	204	228
11	13	13	-29	-36	12	13	194	+196	119	222
12	7	10	354	332	10	15	62	103	146	-241
13	7	10	773	728	11	14	-64	-66	150	-206
14	6	6	390	367	7	8	-55	54	120	120
15	7	8	747	793	11	12	63	53	148	-166
16	6	6	373	396	7	8	-51	59	112	120
17	7	7	772	812	11	10	51	43	-150	-135
18	6	5	390	409	8	7	56	50	-120	109

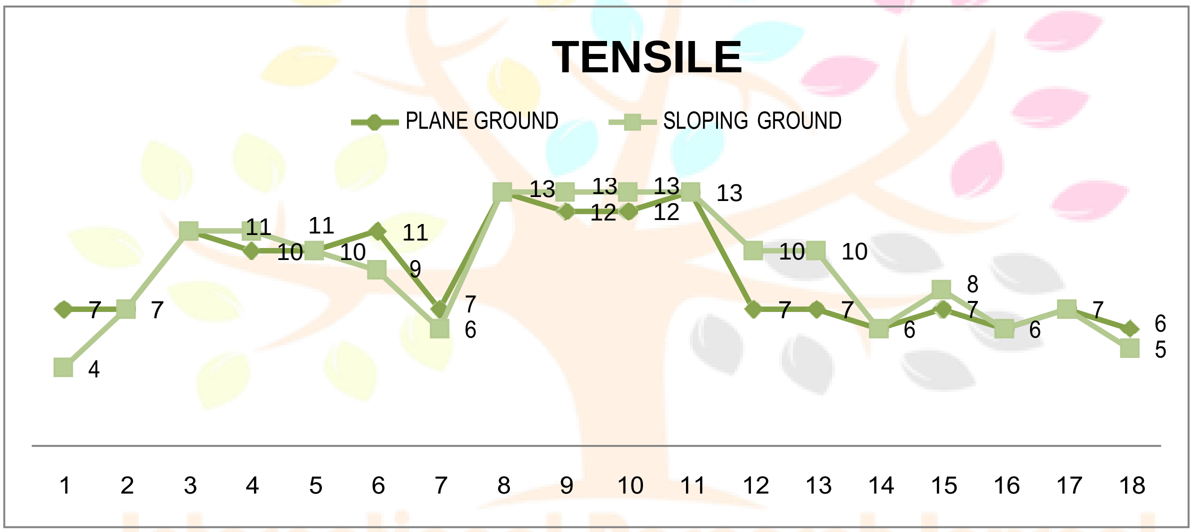
[P= Plane ground, S= Sloping Ground]*


AXAIL
PLANE GROUND
SLOPING GROUND

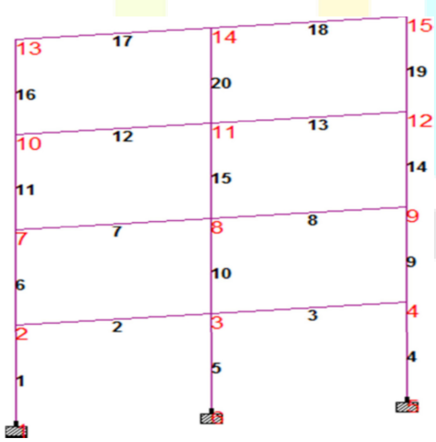

BENDING
Series1
Series2

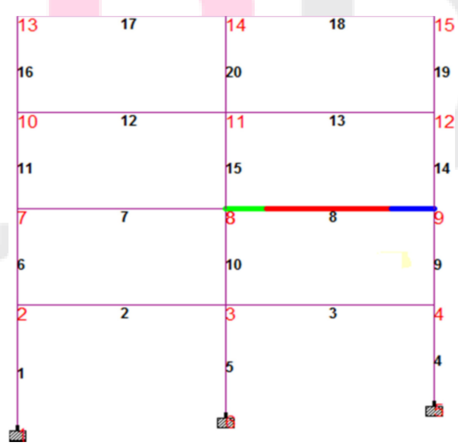

SHEAR
PLANE GROUND
SLOPING GROUND





PART 2.2(DOUBLE BAY) **PLANE GROUND**



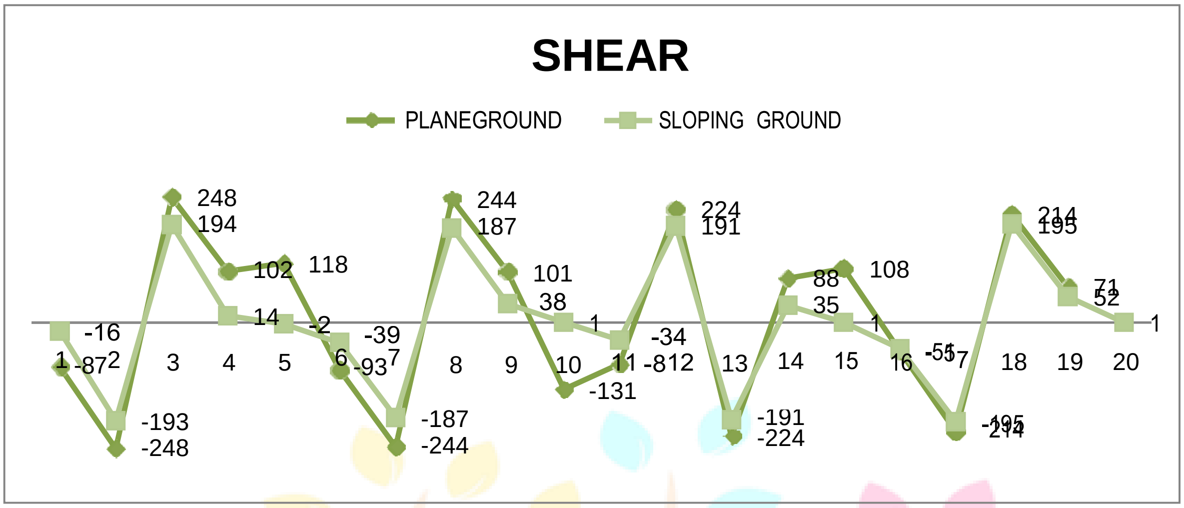


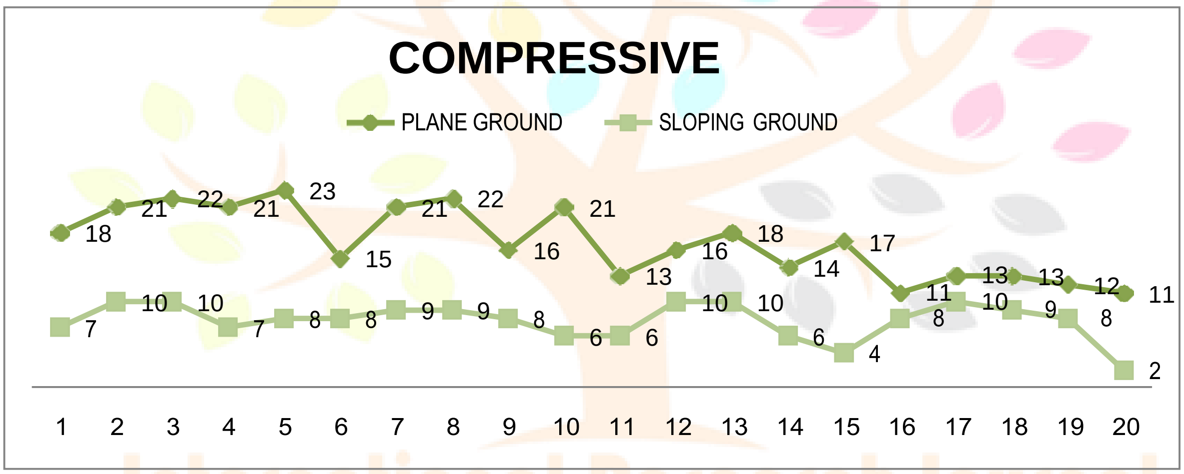
SLOPING GROUND

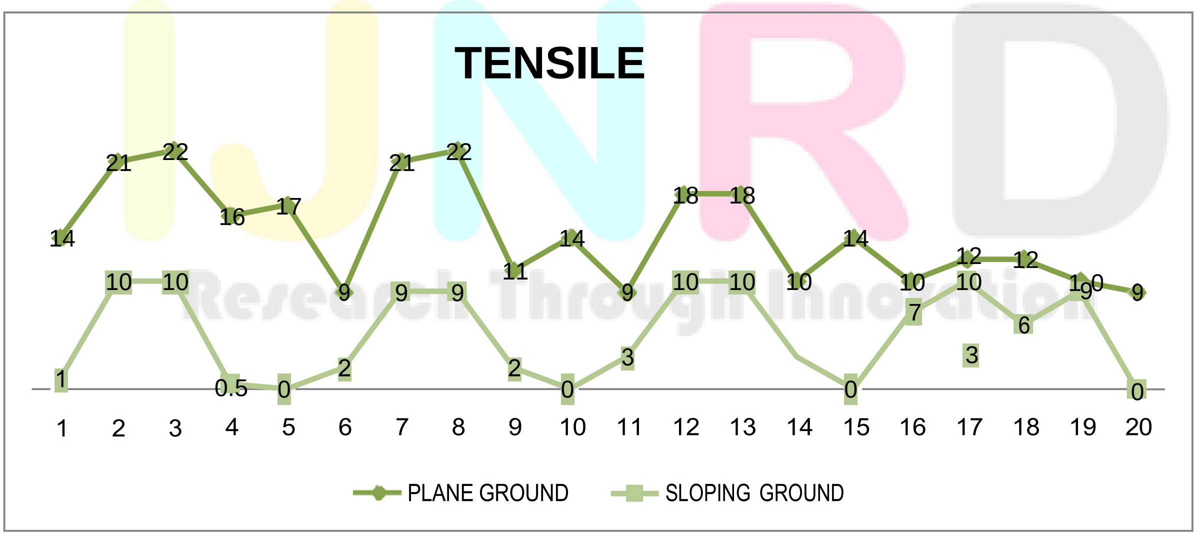
Table No. 3 ANALYSIS RESULTS FOR 2 BAY SYSTEMS ON PLANE AND ON A SLOPING GROUND FOR FOUR STORY FRAME

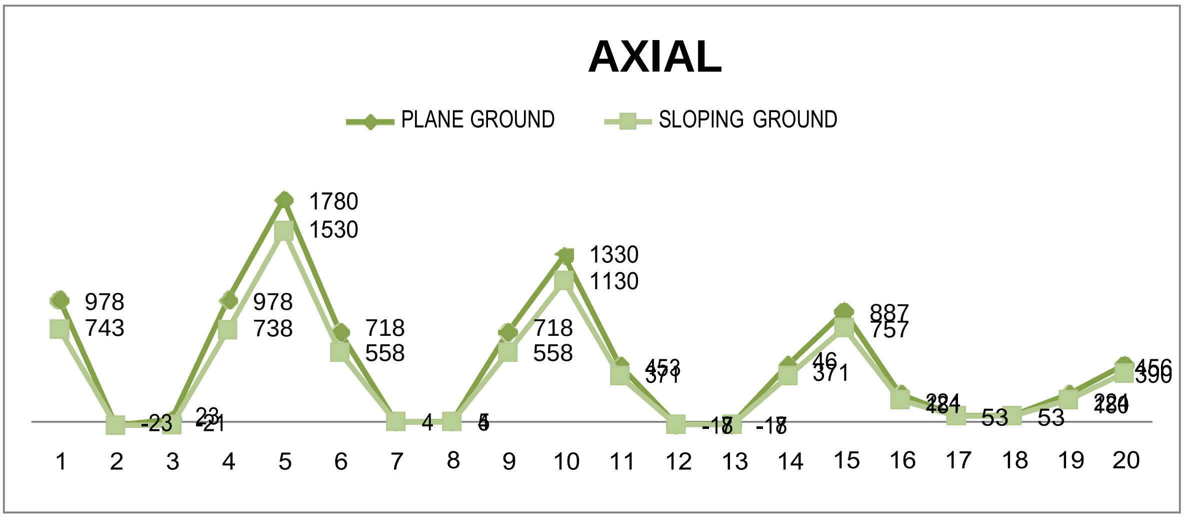
BEAM NO	MAXIMUM AXIAL FORCE		MAXIMUM SHEAR FORCE		MAXIMUM BENDING MOMENT		MAXIMUM TENSILE FORCE		MAXIMUM COMPRESSSIVE FORCE	
	kN		kN		kN-m		N/ mm^2		N/ mm^2	
	P*	S*	P	S	P	S	P	S	P	S
1	978	743	-87	-16	-240	49	14	1	18	7
2	-22	-23	-248	-193	325	159	21	10	21	10
3	23	-21	248	194	325	162	22	10	22	10
4	978	738	102	14	-258	49	16	.5	21	7
5	1780	1530	118	-2	-280	5	17	0	23	8
6	718	558	-93	-39	-187	-78	9	2	15	8
7	4	4	-244	-187	337	147	21	9	21	9
8	5	4	244	187	336	147	22	9	22	9
9	718	558	101	38	-203	-77	11	2	16	8
10	1330	1130	-131	1	266	-1	14	0	21	6
11	453	371	-81	-34	175	-70	9	3	13	6
12	-18	-17	224	191	288	156	18	10	16	10
13	-18	-17	-224	-191	298	156	18	10	18	10
14	461	371	88	35	192	-70	10	3	14	6
15	887	757	108	1	228	-.2	14	0	17	4
16	224	181	-55	-51	173	116	10	7	11	8
17	55	53	-214	-195	197	155	12	10	13	10
18	55	53	214	195	197	155	12	6	13	9
19	224	180	71	52	173	116	10	9	12	8
20	456	390	65	1	-150	-.1	9	0	11	2

*[P= Plane ground, S= Sloping Ground]**


SHEAR
PLANEGROUND
SLOPING GROUND

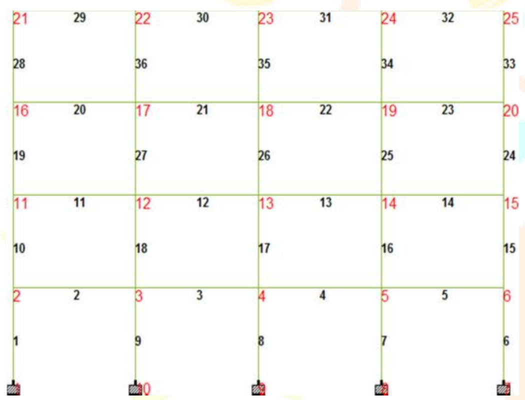

COMPRESSIVE
PLANE GROUND
SLOPING GROUND


TENSILE
PLANE GROUND
SLOPING GROUND



PART 2.2(4 BAY SYSTEMS)

PLANE GROUND



SLOPING GROUND

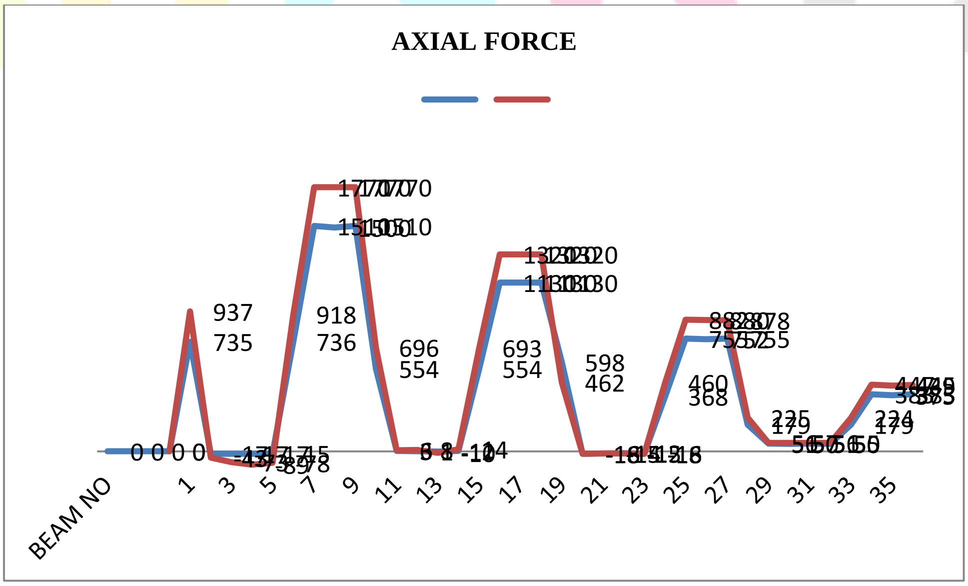
Table no 3. ANALYSIS RESULTS FOR 4 BAY SYSTEMS ON PLANE AND ON A SLOPINGGROUND FOR FOUR STORY FRAME

BEAM NO	MAXIMUM AXIAL FORCE kN		MAXIMUM SHEAR FORCE kN		MAXIMUM BENDING MOMENT kNm		MAXIMUM TENSILE FORCE N/ mm^2		MAXIMUM COMPRESSSIVE FORCE N/ mm^2	
	P*	S*	P	S	P	S	P	S	P	S
1	735	937	-20	-36	54	-137	0.1	4	7	12
2	-17	-43	-193	-222	160	264	10	16	10	16
3	-17	-73	-187	-212	157	259	10	16	10	16
4	-17	-89	-187	-216	157	271	10	17	10	17
5	-15	-78	193	238	160	318	10	19	7	18
6	736	918	21	125	54	-297	0.1	16	7	22
7	1510	1770	-0.4	99	-1.2	-252	0	12	8	21
8	1500	1770	-0.1	-72	-0.5	208	0	9	8	17
9	1510	1770	0.4	-55	1.265	174	0	7	8	16
10	554	696	-37	-77	-75	-156	2	6	8	13
11	3	6	-188	-216	150	267	8	16	9	16
12	1	8	-187	-212	157	256	9	15	9	15
13	-1	-10	-187	-210	157	254	9	15	9	15
14	2	14	188	216	150	267	9	16	9	16
15	554	693	37	66	-75	146	2	6	7	12

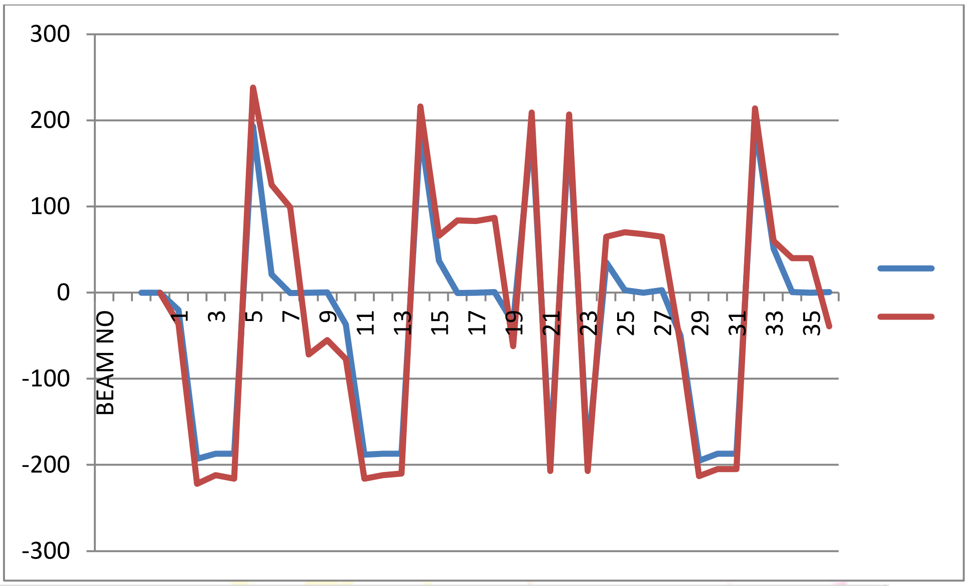
16	1130	1320	-0.5	84	2	173	0	8	6	15
17	1130	1320	0	83	0	170	0	8	6	14
18	1130	1320	0.5	87	-2	-177	0	8	6	15
19	598	462	-35	-62	-70	134	2	7	6	10
20	-16	-18	189	209	153	231	9	14	9	14
21	-14	-15	-188	-207	158	223	10	13	9	13
22	-12	-15	188	207	158	221	10	13	9	13
23	-16	-18	-189	-207	153	230	9	14	9	14
24	368	460	35	65	-70	141	2	6	6	10
25	755	882	3	70	6	148	0	7	4	12
26	752	880	0	68	0	143	0	7	4	11
27	755	878	3	65	7	-146	0	7	4	11
28	179	225	-50	-60	114	145	6	8	8	10
29	51	56	-195	-213	156	169	9	10	10	10
30	50	57	-187	-205	156	174	9	10	10	11
31	51	56	-187	-205	156	174	9	10	10	11
32	50	55	195	214	156	172	9	10	10	11
33	179	224	51	60	114	144	6	8	8	10
34	383	447	0.5	40	-0.2	91	0	5	2	7
35	375	439	0	40	0	92	0	5	2	7
36	383	445	0.5	-39	2	-90	0	5	2	7

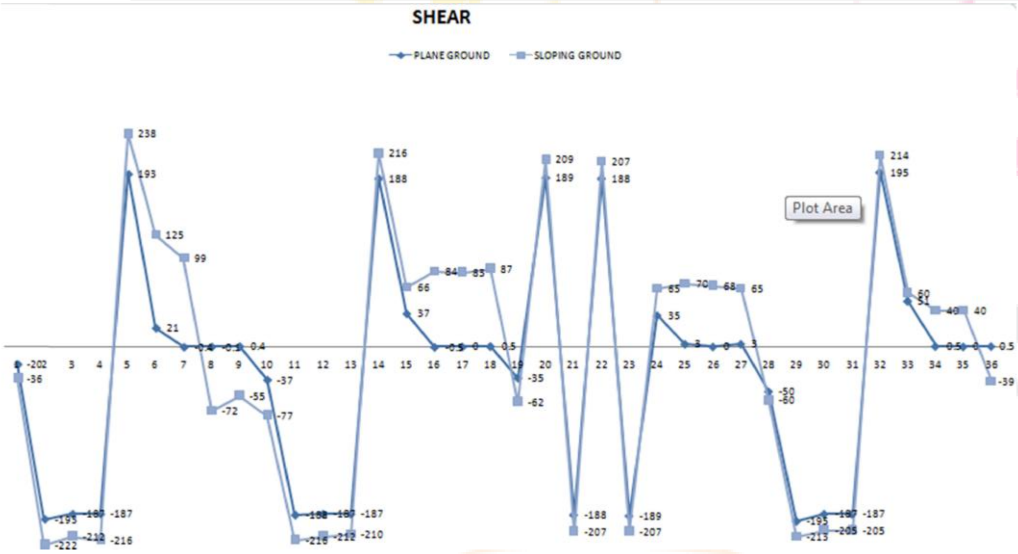
PART 2.1(DOUBLE BAY)

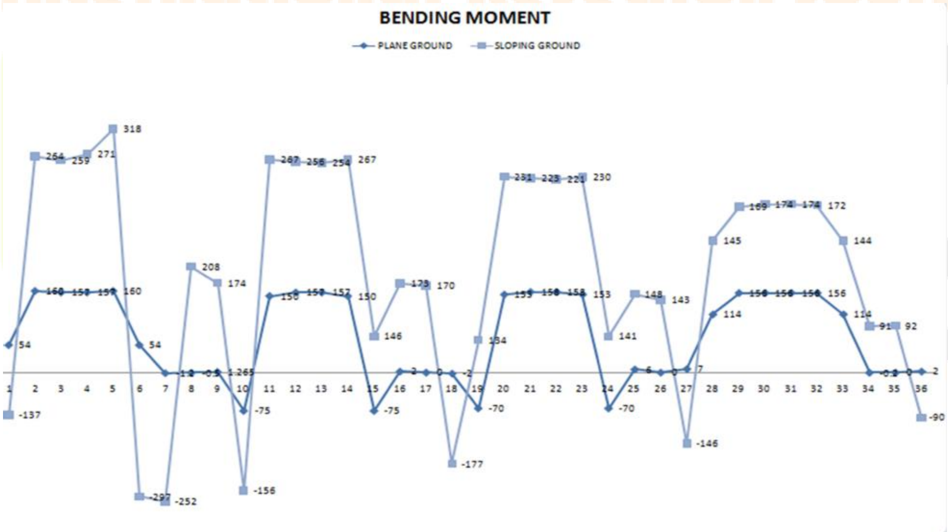
	PLANE GROUND					SLOPING GROUND				
	755	882	3	70	6	148	0	7	4	12
26	752	880	0	68	0	143	0	7	4	11
27	755	878	3	65	7	-146	0	7	4	11
28	179	225	-50	-60	114	145	6	8	8	10
29	51	56	-195	-213	156	169	9	10	10	10
30	50	57	-187	-205	156	174	9	10	10	11
31	51	56	-187	-205	156	174	9	10	10	11
32	50	55	195	214	156	172	9	10	10	11
33	179	224	51	60	114	144	6	8	8	10
34	383	447	0.5	40	-0.2	91	0	5	2	7
35	375	439	0	40	0	92	0	5	2	7
36	383	445	0.5	-39	2	-90	0	5	2	7

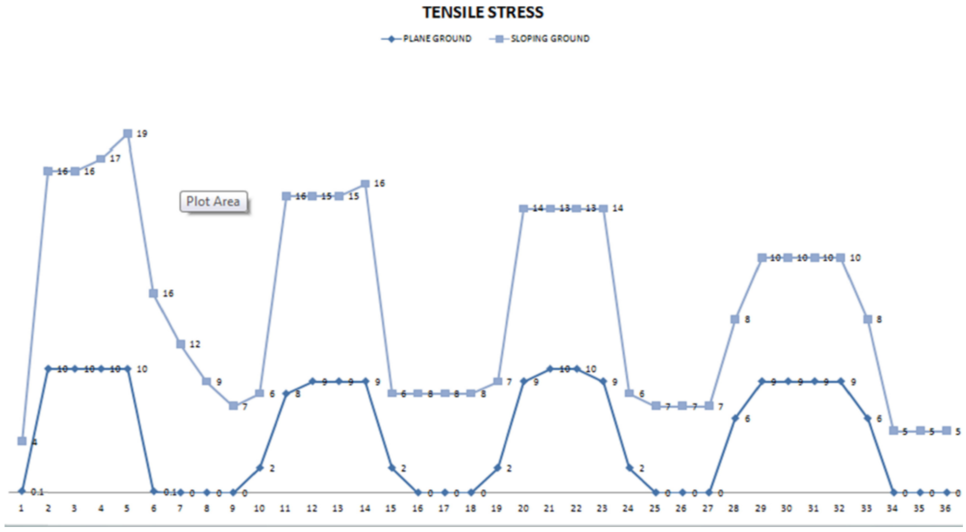


[P= Plane ground, S= Sloping Ground]*








TENSILE STRESS
PLANE GROUND
SLOPING GROUND
Plot Area


7 7 8 8 9
6 15 9
6 29 16 30 10
5 16 10
5 27 17 28 11
4 17 11
4 25 18 26 12
3 18 12


7 7 8 8 9
6 15 9
6 21 16 22 10
5 16 10
5 23 17 24 11
4 17 11
4 25 18 26 12
3 18 12
3
3
4
4
5

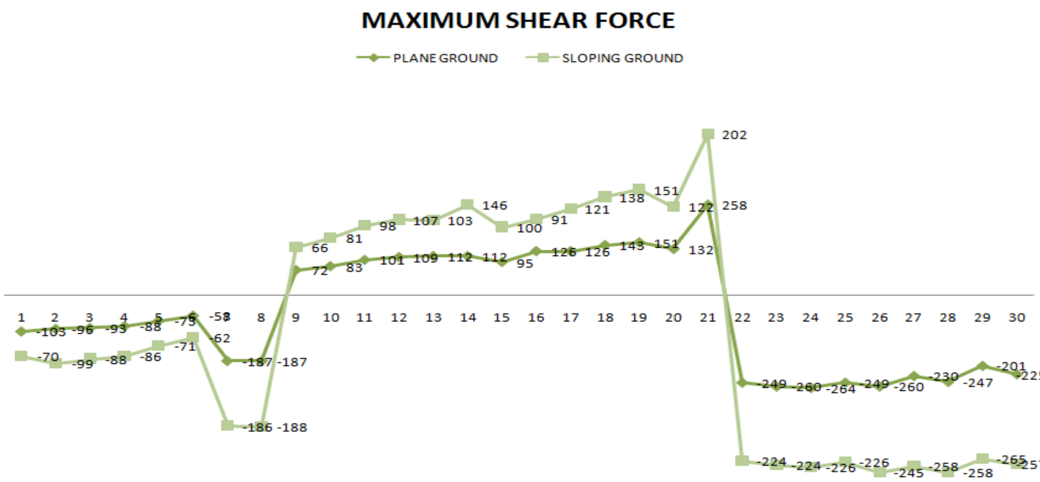
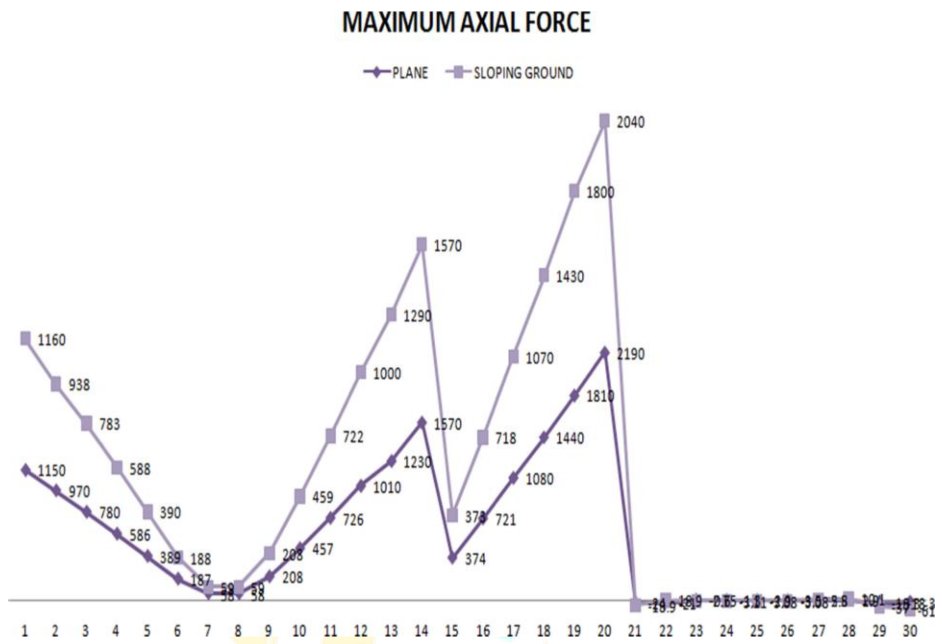
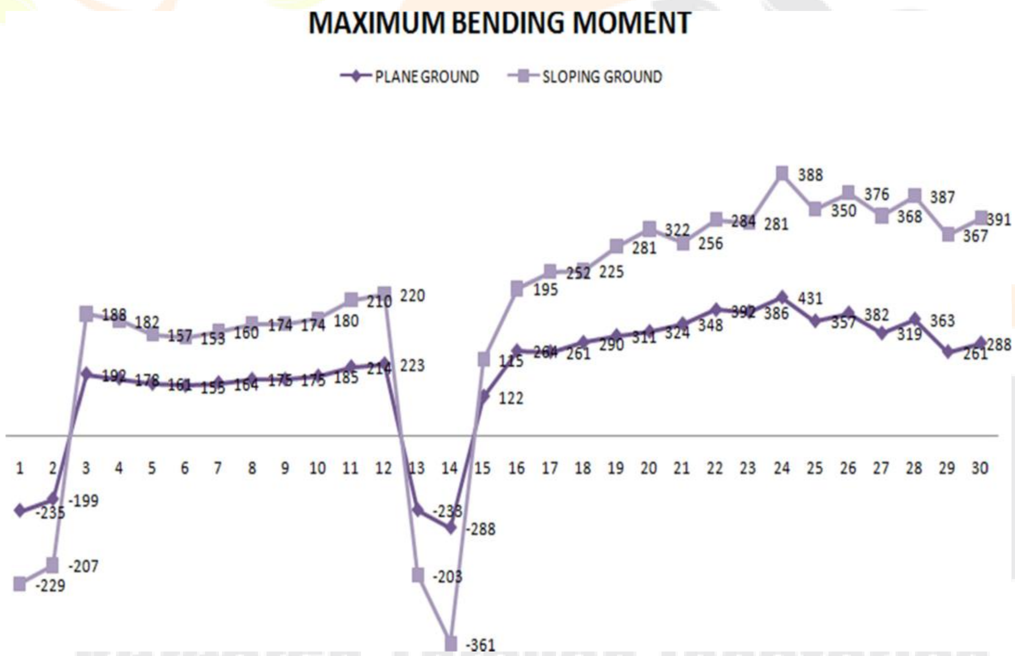

MAXIMUM SHEAR FORCE
PLANE GROUND
SLOPING GROUND

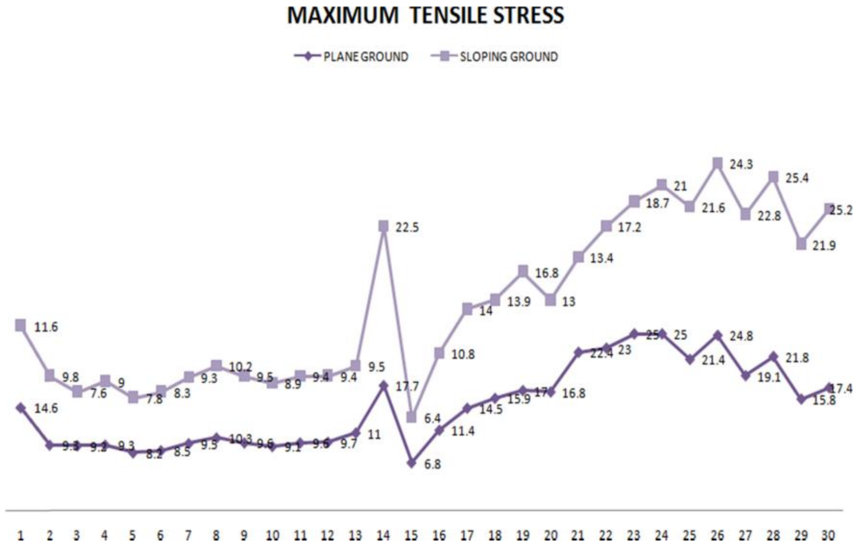
Table No. 4 ANALYSIS RESULTS FOR 2 BAY SYSTEMS ON PLANE AND ON A SLOPINGGROUND FOR SIX STORY FRAME

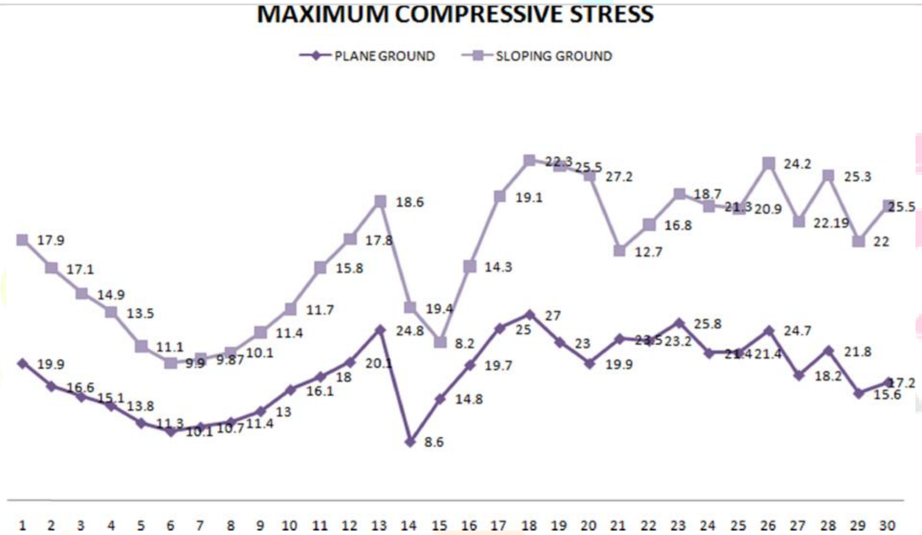
BEAM NO	MAXIMUM AXIAL FORCE		MAXIMUM SHEAR FORCE		MAXIMUM BENDING MOMENT		MAXIMUM TENSILE STRESS		MAXIMUM COMPRESSSIVE STRESS	
	kN		kN		kN-m		N/ mm^2		N/ mm^2	
	P*	S*	P	S	P	S	P	S	P	S
1	1150	1160	-103	-70	-235	-229	14.6	11.6	19.9	17.9
2	970	938	-96	-99	-199	-207	9.3	9.8	16.6	17.1
3	780	783	-93	-88	192	188	9.2	7.6	15.1	14.9
4	586	588	-88	-86	178	182	9.3	9	13.8	13.5
5	389	390	-73	-71	161	157	8.2	7.8	11.3	11.1
6	187	188	-58	-62	155	153	8.5	8.3	10.1	9.9
7	58	59	-187	-186	164	160	9.5	9.3	10.7	9.87
8	58	59	-187	-188	175	174	10.3	10.2	11.4	10.1
9	208	208	72	66	175	174	9.6	9.5	13	11.4
10	457	459	83	81	185	180	9.1	8.9	16.1	11.7
11	726	722	101	98	214	210	9.6	9.4	18	15.8
12	1010	1000	109	107	223	220	9.7	9.4	20.1	17.8
13	1230	1290	112	103	-233	-203	11	9.5	24.8	18.6
14	1570	1570	112	146	-288	-361	17.7	22.5	8.6	19.4
15	374	373	95	100	122	115	6.8	6.4	14.8	8.2
16	721	718	126	91	264	195	11.4	10.8	19.7	14.3
17	1080	1070	126	121	261	252	14.5	14	25	19.1
18	1440	1430	143	138	290	225	15.9	13.9	27	22.3
19	1810	1800	151	151	311	281	17	16.8	23	25.5
20	2190	2040	132	122	324	322	16.8	13	19.9	27.2
21	-24	-18.9	258	202	348	256	22.4	13.4	23.5	12.7
22	-21	18.9	-249	-224	392	284	23	17.2	23.2	16.8
23	-0.6	-2.75	-260	-224	386	281	25	18.7	25.8	18.7
24	-1.3	-3.11	-264	-226	431	388	25	21	21.4	21.3
25	-2.9	-3.08	-249	-226	357	350	21.4	21.6	21.4	20.9
26	-3.5	-3.08	-260	-245	382	376	24.8	24.3	24.7	24.2
27	2.8	5.5	-230	-258	319	368	19.1	22.8	18.2	22.19
28	2.9	10.1	-247	-258	363	387	21.8	25.4	21.8	25.3
29	-18.3	-37	-201	-265	261	367	15.8	21.9	15.6	22
30	-18.3	-61	-225	-257	288	391	17.4	25.2	17.2	25.5

[P= Plane ground, S= Sloping Ground]*


MAXIMUM AXIAL FORCE
PLANE
SLOPING GROUND

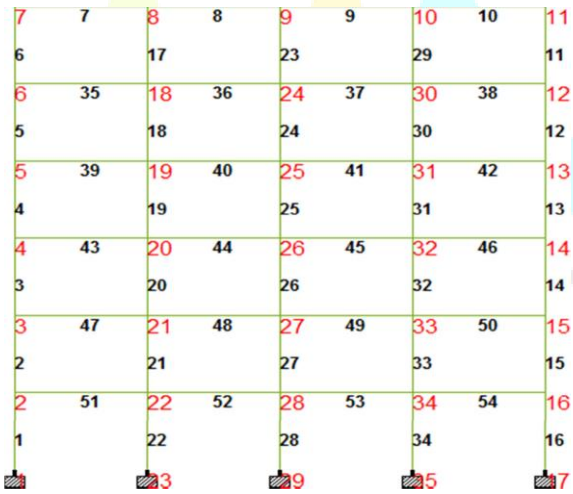

MAXIMUM BENDING MOMENT
PLANE GROUND
SLOPING GROUND





PART 2.3(FOUR BAY)

PLANE GROUND *SLOPING GROUND*



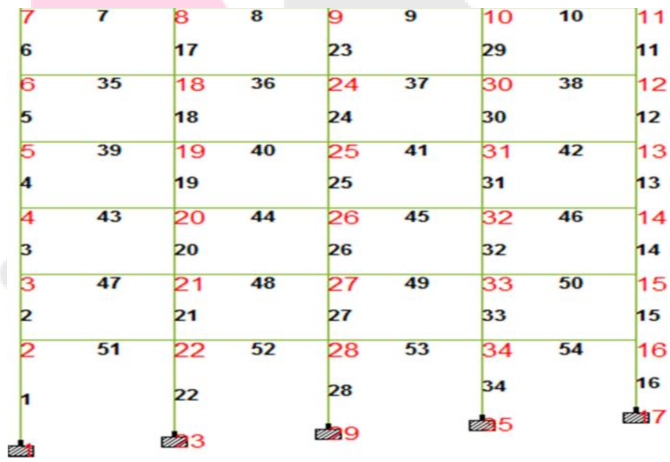


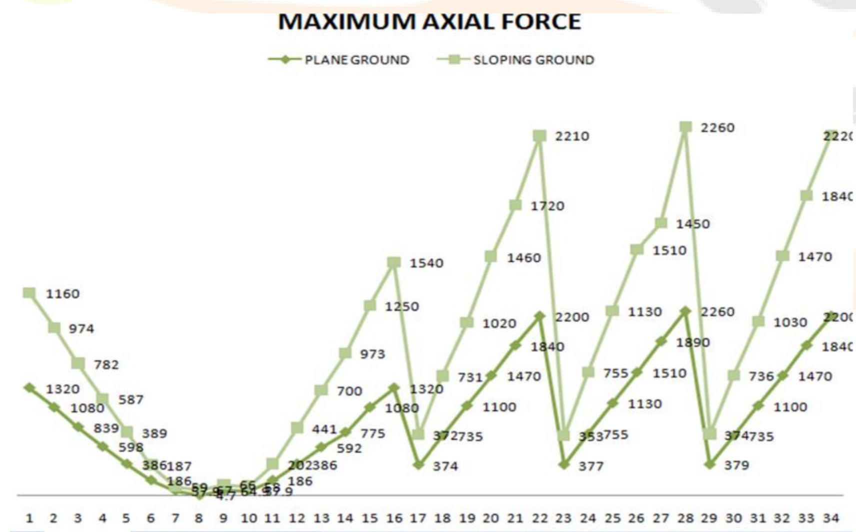
Table No 5. ANALYSIS RESULTS FOR 4 BAY SYSTEMS ON PLANE AND ON A SLOPINGGROUND FOR SIX STORY FRAME

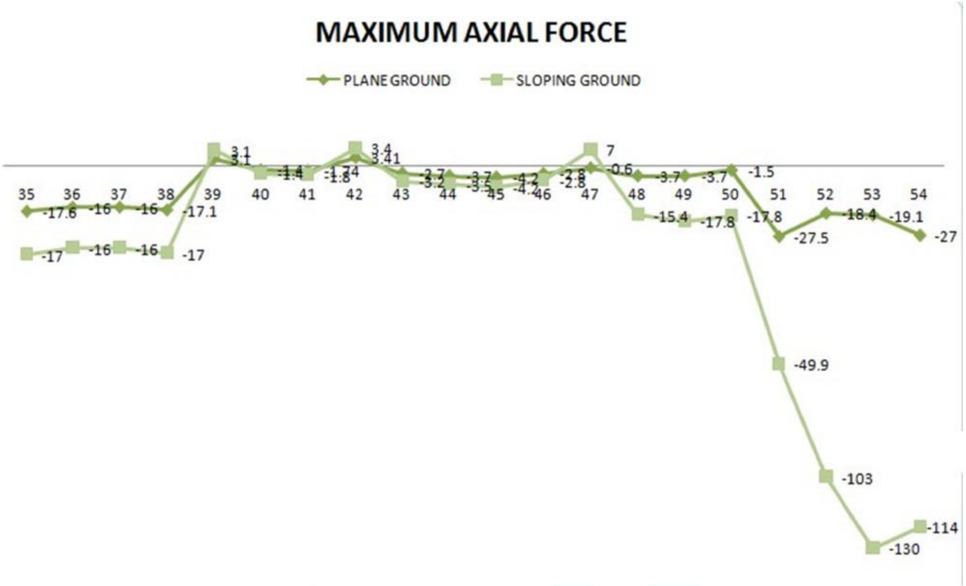
BEAM NO	MAXIMUM AXIAL FORCE		MAXIMUM SHEAR FORCE		MAXIMUM BENDING MOMENT		MAXIMUM TENSILE STRESS		MAXIMUM COMPRESSSIVE STRESS	
	kN		kN		kN-m		N/ mm^2		N/ mm^2	
	P*	S*	P	S	P	S	P	S	P	S
1	1320	1160	-107.6	-53	-282	-203	16.2	9.8	21.5	15.8
2	1080	974	-95.6	-97	-196	-201	8.6	9.3	17.9	16.4
3	839	782	-93	-82	190	179	8.5	7.2	15.6	14.2
4	598	587	-87	-18	-164	-171	8.7	8.3	14	12.7
5	386	389	-87	-67	161	148	8	7.3	11.5	10.4
6	186	187	-74	-59	156	142	8.5	7.8	10.22	9.3
7	57.9	59	-64.1	-187	156	151	9.1	8.8	9.58	9.3
8	4.7	67	-188.5	-189	174	183	10.2	10.6	10.76	11.3
9	64.9	66	-188.6	188	174	180	10.2	10.5	10.76	11.1
10	57.9	58	188.6	188	156	162	9.1	9.5	9.58	9.9
11	186	202	64.1	67	156	149	8.5	8.8	10.22	10.6
12	386	441	76.2	75.8	176	168	8.6	8.2	14.4	11.9
13	592	700	92.65	91.7	184	196	9.2	8.6	14.4	14.8
14	775	973	100.76	102	206	-208	8.7	8.6	16.1	16.9
15	1080	1250	104.7	90.3	-214	199	8.6	8.6	17.7	17.6
16	1320	1540	112.6	187	-289	-455	16.2	27.9	21.5	34.6
17	374	372	-51	-45	-119	-106	6.4	5.9	7.8	6.9
18	735	731	-93	84	-200	-182	10.6	10.1	13.3	12.5
19	1100	1020	-120	-111	-222	-232	13	12.7	17.6	17.2
20	1470	1460	-135	-127	-275	-260	13.9	14.1	20.4	20.5
21	1840	1720	-142	-141	289	289	14.1	15.6	22.9	24.4
22	2200	2210	-131	-85	-314	269	15	14.2	25.6	24.5
23	377	353	-52	48	123	113	6.7	6.2	8.5	7.6
24	755	755	91.7	84	197	-181	10.4	10.1	14.1	13.7
25	1130	1130	118.6	111	-246	231	12.7	12.5	18.1	16.7
26	1510	1510	133	126	270	257	13.4	13.7	20.7	19.3
27	1890	1450	-138	-136	-281	-279	13.4	14.5	22.5	23.7
28	2260	2260	-129	-111	-311	322	14.6	17.1	23.9	27.7
29	379	374	53	49	122	114	6.4	5.8	8.5	8.2
30	735	736	95.2	88	204	191	10.6	12.4	14.5	14.2

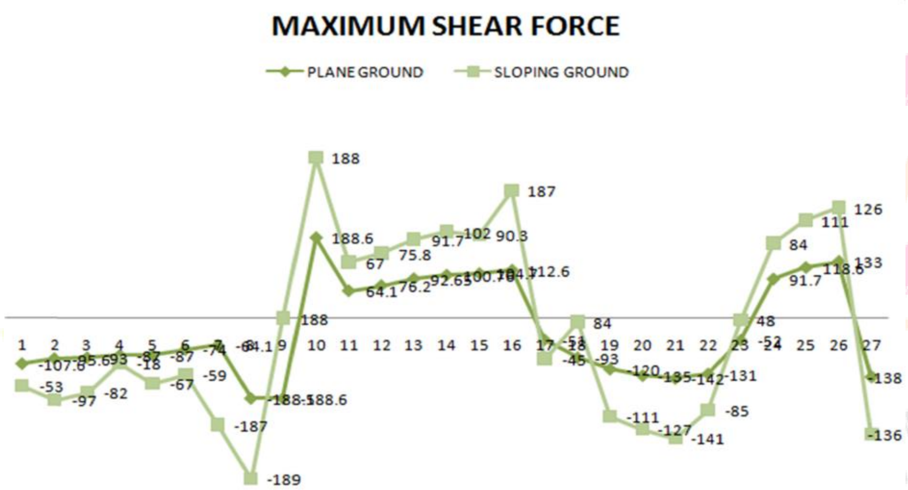
BEAM NO	MAXIMUM AXIAL FORCE		MAXIMUM SHEAR FORCE		MAXIMUM BENDING MOMENT		MAXIMUM TENSILE STRESS		MAXIMUM COMPRESSSIVE STRESS	
	kN		kN		kN-m		N/ mm^2		N/ mm^2	
	P	S	P	S	P	S	P	S	P	S
31	1100	1030	122	115	253	239	13	13.75	8.5	18.4
32	1470	1470	137	130	278	265	13.9	14.4	14.5	21.4
33	1840	1840	142	138	-290	278	14.1	21	18.5	23.5
34	2200	2220	131	-154	-314	398	14.6	21.6	21	31.9

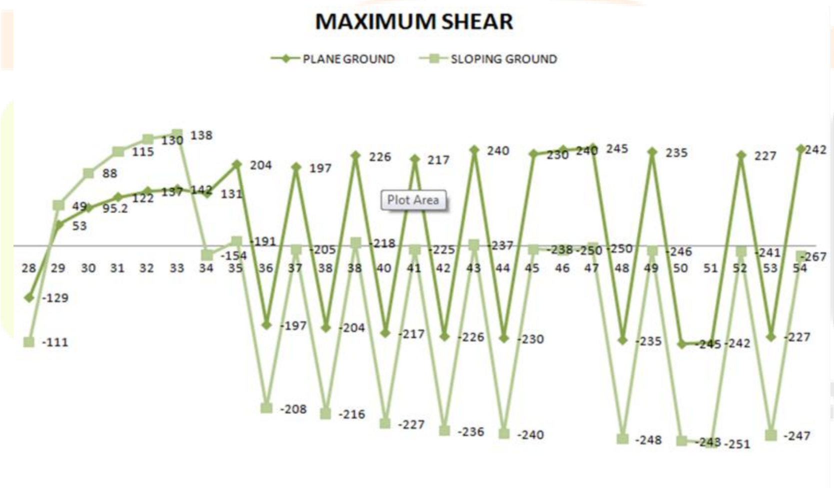
35	-17.6	-17	204	-191	261	239	15.7	14.4	22.8	14.3
36	-16	-16	-197	-208	237	243	14.4	14.7	25.3	4.5
37	-16	-16	197	-205	240	238	14.4	14.3	15.6	14.2
38	-17.1	-17	-204	-216	261	266	15.7	16	19.1	15.9
39	3.1	3.1	226	-218	291	293	19.4	17.6	18.1	17.6
40	-1.4	-1.4	-217	-227	291	302	18.1	17.5	17.9	18.1
41	-1.74	-1.8	217	-225	299	299	17.9	18	19.9	17.9
42	3.41	3.4	-226	-236	331	299	19.14	19.8	19.8	19.8
43	-2.7	-3.2	240	-237	355	330	21.3	19.8	21.3	19.8
44	-3.7	-3.5	-230	-240	343	344	20.6	20.6	20.5	20.6
45	-4.2	-4.2	230	-238	340	340	20.4	20.4	20.4	20.4
46	-2.8	-2.8	240	-250	355	379	22.8	22.7	22.7	22.7
47	-0.6	7	245	-250	369	361	21.7	21.6	22.1	21.6
48	-3.7	-15.4	-235	-248	336	368	21.6	22.18	21.6	22
49	-3.7	-17.8	235	-246	358	365	21.5	22	21.5	21.8
50	-1.5	-17.8	-245	-243	400	368	24	21	24	23.8
51	-27.5	-49.9	-242	-251	343	352	21.4	24	21.1	21.2
52	-18.4	-103	227	-241	336	355	20.2	20.9	20.1	19.4
53	-19.1	-130	-227	-247	335	370	20.1	21.6	20.1	22.7
54	-27	-114	242	-267	375	455	22.4	26.8	22.5	27.8

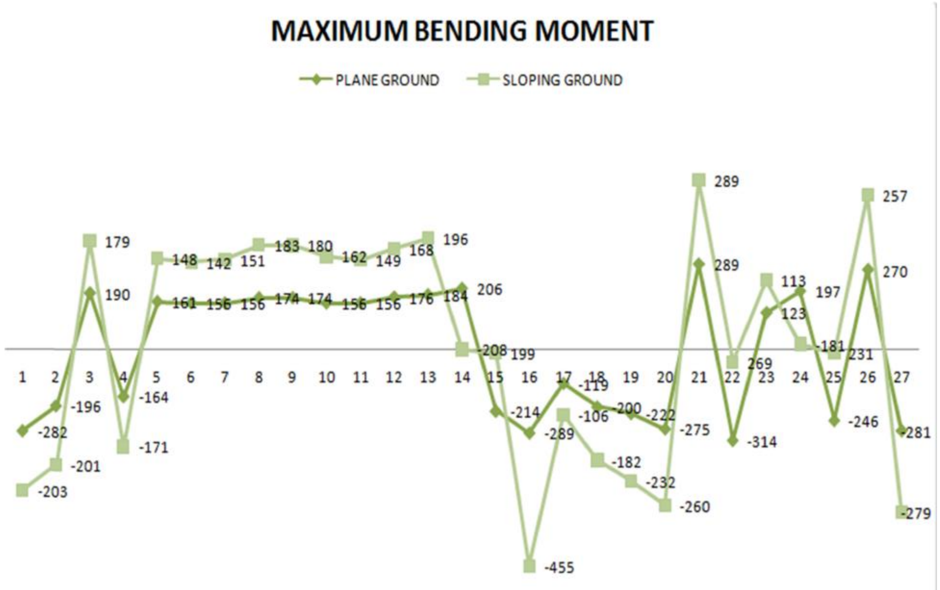
[P= Plane ground, S= Sloping Ground]*

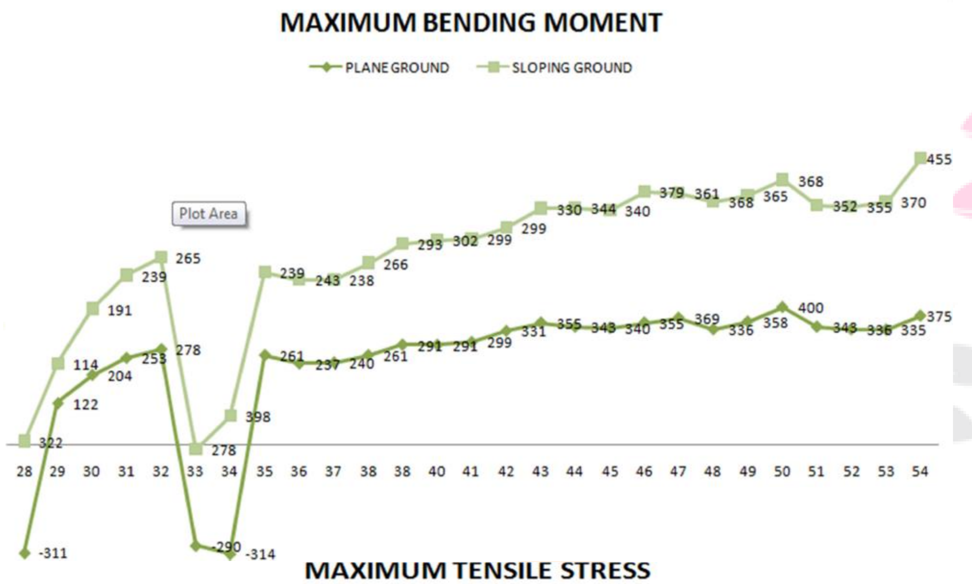


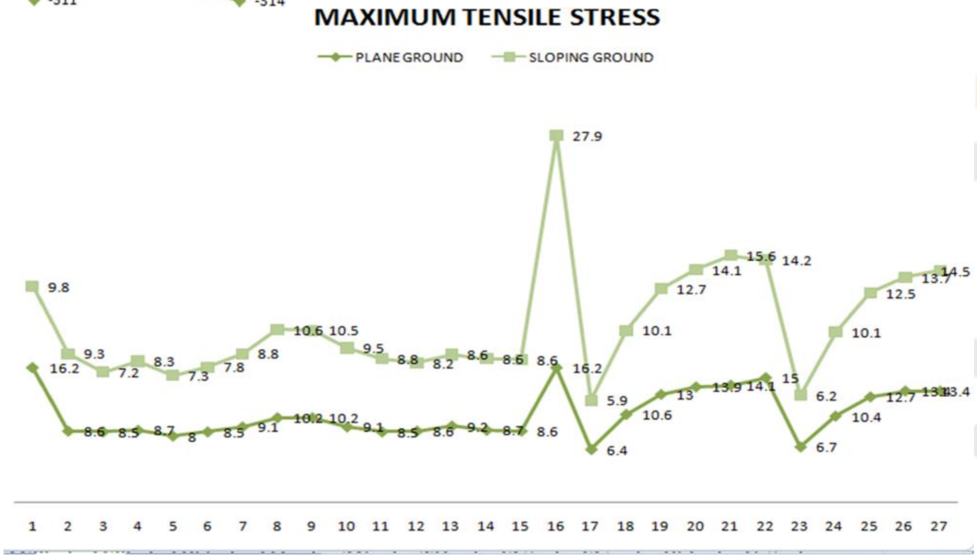

MAXIMUM AXIAL FORCE
PLANE GROUND
SLOPING GROUND

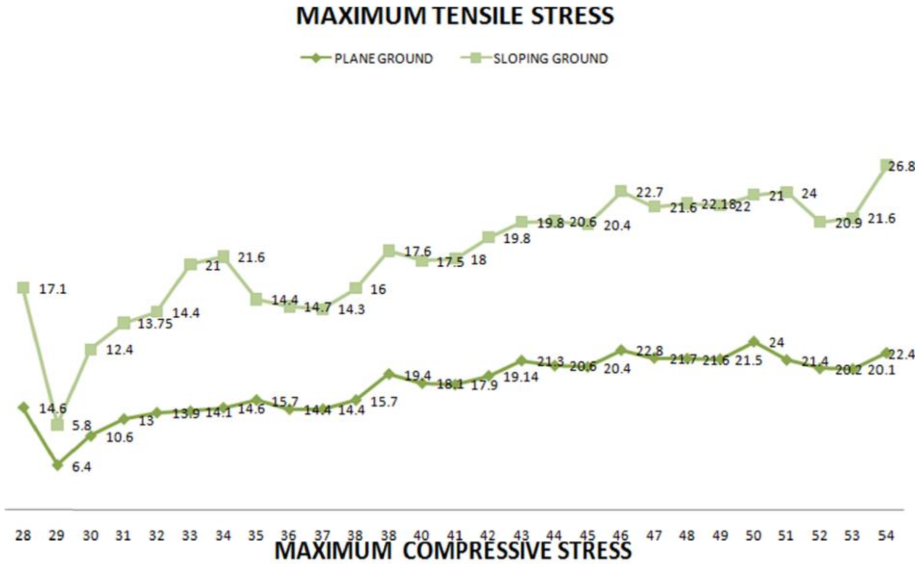

MAXIMUM SHEAR FORCE
PLANE GROUND
SLOPING GROUND

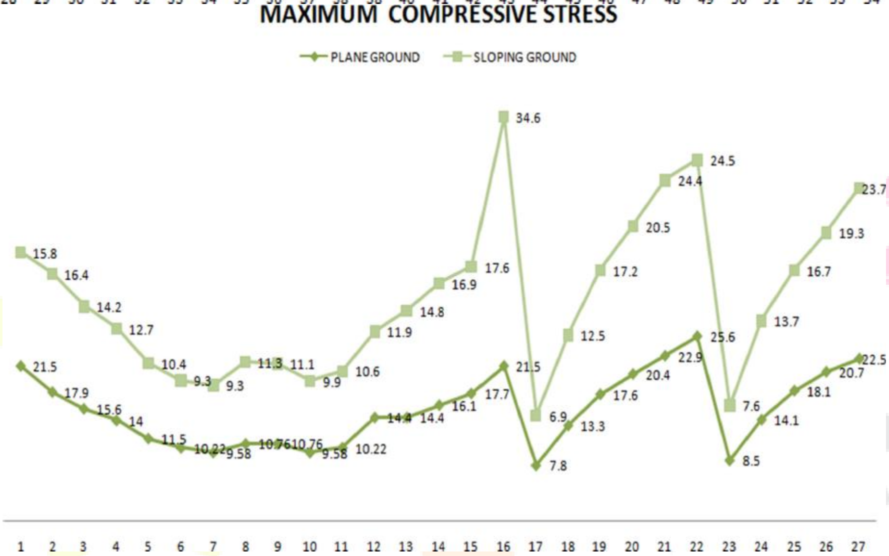

MAXIMUM SHEAR
PLANE GROUND
SLOPING GROUND

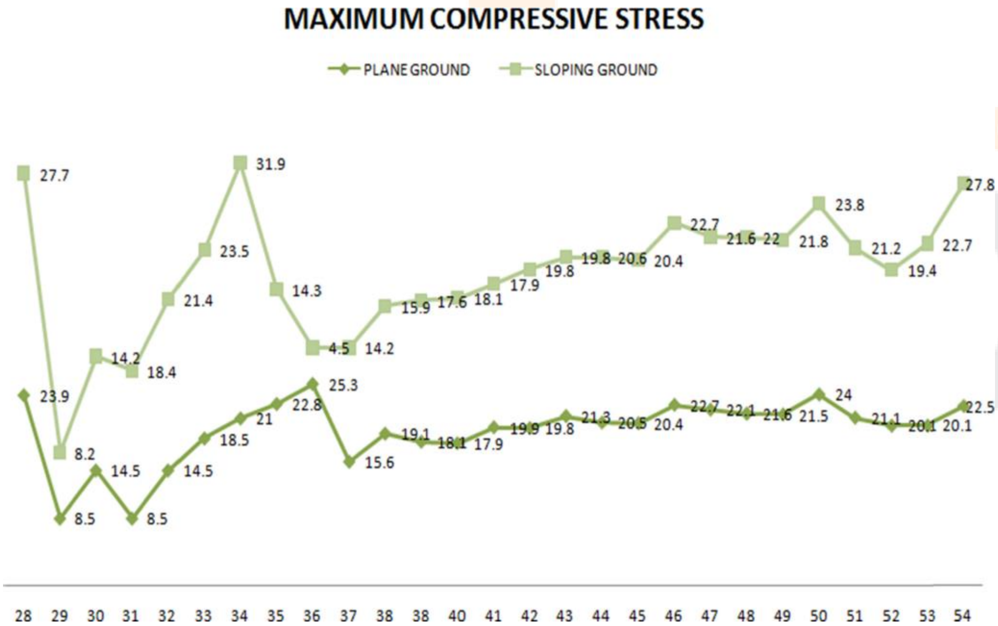

MAXIMUM BENDING MOMENT
PLANE GROUND
SLOPING GROUND
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27


MAXIMUM BENDING MOMENT
PLANE GROUND
SLOPING GROUND
Plot Area
28 29 30 31 32 33 34 35 36 37 38 38 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54


MAXIMUM TENSILE STRESS
PLANE GROUND
SLOPING GROUND
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27







5.1 DUCTILITY DESIGN AND DETAILING.

SECTION-3

Under the **PART- 3** a detailed design of a frame has been carried out with the design aid of IS456 and IS 13920:1993

This 3rd phase can be again broadly divided into following:-

Design of an flexuralmember.

3.2) design of an exteriorcolumn.

3.3) design of an interior column.

To illustrate the design of a sub-frame a flexural member with maximum bending momenthas been carried out .

General specification

- the member is designed according to IS 456:2000
- building > 3 storey height, minimum grade of concrete M20 – we used M30
- steel reinforcement of FE 415 used.

Design of the flexural

member*[2]* **General**

1) Factored axial stress less than 0.1 f_{ck}
2) The member should preferably have a width to depth ratio of more 0.3Width/depth=400/500=0.8 > 0.3 ,hence ok
3) Width should not be less then 200mm . But we provided width of 400 mm which is ok.
4) depth should not be greater than 0.25(clear span) i.e.(5000- 400)= 4600mm.

Longitudinal reinforcement (node 16)

We will find the reinforcement due to a sagging moment of 455 kN-m.
Assuming 25 mm dia bars with 25 mm clear coverEffective depth(d)= 500-25-(25/2)=465mm
From table D of SP 16 :1980 $M_{u,lim}/bd^2 = 4.14$ *(for M-30 and FE 415)*
$M_{u,lim} = 4.14 \times 400 \times 465^2 = 358.1$ kN-m.
Actual moment 455 kN-m is greater than 358.1 kN-m ,so we go for the doubly reinforcedsection.
Reinforcement from table 50 of SP 16: 1890$M_u/bd^2 = 5.26$
$d^1/d = 0.08$
P_t (bottom) = 1.7028P_c (top) = 0.784

Reinforcement due to the sagging moment

328.98kN-m is the design hogging moment which is not greaterthan 358.1kN-m ,so we gofor the singly reinforced section.
$M_u/bd^2 = 3.80$
From table 4 we have,
P_t (top)=1.282
now required reinforcement is the maximum of 0.784 and 1.282,so finally we have

Reinforcement due to the hogging moment

P_t (top) = 1.282
P_c (bottom) = 1.7028
Reinforcement at top (At)=1.282x 400x465 = 2384 mm^2. Reinforcement at the bottom=1.7028x400x465 =3167.20 mm^2.
Checks
1) the top and bottom reinforcements should atleast contain 2 bars which is the case here.
2)tension steel ratio p min≤ 0.24 (fck/fy) 1/2=0.258 but we have 1.7028. hence okLongitudinal reinforcement(node 34)

We will find the reinforcement due to a sagging moment of 235 kN-m**.**Assuming 25 mm dia bars with 25 mm clear cover
Effective depth(d)= 500-25-(25/2)=465mm
From table D of SP 16 :1980 $M_{u,lim}/bd^2 = 4.14$*(for M-30 and FE415)* $M_{u,lim} = 4.14 \times 400 \times 465^2 = 358.1$ kN-m.
Actual moment 235 kN-m is less than 358.1 kN-m ,so we go for the singly reinforced section.Reinforcement from table 4 SP 16: 1890
$M_u/bd^2 = 2.717$, $d^1/d=0.08$ P_t (bottom) =0.80

Reinforcement due to the sagging moment

349 kN-m is the design hogging moment which is not greater than 358.1kN-m ,so we go forthe singly reinforcedsection.
$M_u/bd^2 = 4.03$

From table 4 of SP 16, we have P_t (top)=1.391
so finally we have P_t (top) = 1.391 P_c (bottom) = 0.80
Reinforcement at top =1.391 x 400 x 465 = 2587 mm^2.Reinforcement at the bottom=0.808x400x465 =1503 mm^2. **Checks**
1) the top and bottom reinforcements should atleast contain 2 bars which is the case here.

2)tension steel ratio p min≤ 0.24 (fck/fy) 1/2=0.258 but we have 0.808 hence ok

Shear reinforcement requirement

Shear force under consideration will be the maximum of the :-

1) *Calculated shear force (V = 375)*
2) ***Shear force sue to the formation of the plastic hinges. At both the ends of thebeam.***

At node no 16

P_t =3216/(400x465)=2.31% (at top)

$M_{u,lim}/bd^2 = 6.9$ ($P_t = 2.31$, d'/d = 0.08)
$M_{u,lim} = 6.9 \times 400 \times 465^2 = 596$ kN-m (maximum hogging moment)
$P_t = 2450/(400 \times 465) = 1.32\%$ (at bottom) $P_t = 8.1$ (1.32% , d'/d= 0.08)
$M_{u,lim} = 8.1 \times 400 \times 465^2 = 700.56$ kN-m(maximum sagging moment)

At node 34

$P_t = 1960/(400 \times 465) = 1.05\%$(at bottom)
$M_{u,lim}/bd^2 = 7.3$($P_t = 1.05\%$, d'/d = 0.08)
$M_{u,lim} = 7.3 \times 400 \times 465^2 = 631$ kN-m (maximum sagging moment)
$P_t = 2613/(400 \times 465) = 1.40\%$(at top)
$M_{u,lim}/bd^2 = 4.15$ ($P_t = 1.05\%$, d'/d=0.08)
$M_{u,lim} = 4.15 \times 400 \times 465^2 = 358$kN-m(maximum hogging moment)
$V_{34}^{D+L} = V_{16}^{D+L} = 1.2 \times (30 + 20) = 60$

for sway to right:

$$V_{u,a} = V_a^{D+L} - 1.4\left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}}\right]$$

$$\text{and } V_{u,b} = V_b^{D+L} + 1.4\left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}}\right]$$

for sway to left:

$$V_{u,a} = V_a^{D+L} + 1.4\left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}}\right]$$

$$\text{and } V_{u,b} = V_b^{D+L} - 1.4\left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}}\right]$$

For sway to right

$V_{u,34} = 60 - 1.4[631+596]/4.6 = -313$kN $V_{u,16} = 60 + 1.4[631+596]/4.6 = 433$

For sway to left

$V_{u,34} = 60 + 1.4[358+700.56]/4.6 = -382$ kN. $V_{u,16} = 60 - 1.4[358+700.56]/4.6 = -262$ kN

The minimum percentage of steel used is = 1.05%
$\tau_c = 0.66$ N/ mm^2.
$\tau_v = 433/(400 \times 465) = 2.32$ N/ mm^2.
τ 1c,max for M 30 = 3.5 N/ mm^2.
$V_{us} = V_u - \tau_c bd = 433 - 122 = 310$ N/ mm^2 We adopt 8 mm two legged stirrups
$A_{sv} = 100.52$ mm^2 S_{max} is minimum of a) d/4 = 465/4 = 116
b) 8 d_{min} = 8 x 25 = 200
c)S=.87 x 415 x 100.5 x 465/(310 x 1000)= 54.42 = 60 mm.
So we provide stirrups @ 60 mm c/c.

Design of exterior column*[2]*

In this example the columns of the ground floor are designed for illustrations. The exterior columns no 1 is designed for the forces based on maximum interaction ration (1 in this case)

- We have size of the column 400mm x 500 mm
- Concrete mix M 30
- Vertical reinforcement Fe 415
- Axial load 1160 kN
- Moment from load 229 kN

The general requirement of the column for the ductility will follow IS 13920:1993 and vertical reinforcement of the column is designed according to IS 456:2000. The transverse and the special confinement reinforcement will be d by following the IS 13920:1993 and IS 456:2000.

General (Column subjected to bending and axial load)

- IS 13920:1993 will be applicable if the axial stress > 0.1 f 1ck.
  1160 x 1000/ (400 x 500) = 5.8 > 0.1 f 1ck = 3.
- Minimum dimension of the member ≥ 250 and we have taken 400 which is ok.
- Shortest cross section dimension / perpendicular dimension ≥ 0.4 and we have the same ratio as 0.8

Vertical (longitudinal) reinforcement

Assume 20 mm ¤ with 40 mm cover (d1= 40 +10= 50 mm, d1 / D = 50/ 500 = 0.1)
From chart 44 SP 16: 1980 (d1/ D = 0.1, 415 N/ mm^2)

$P_u/ f_{ck}bD$ = 1160 x 1000 / (30 x 400 x 500)= 0.19
$M_u/ f_{ck}bD^2$ = 0.075
Reinforcement on four sides from chart 44 SP 16: 1980P/f_{ck}=0.045, reinforcement in % =0.045 x 30 = 1.35 %
A 1s = pbd/ 100= 1.35 x 400 x 500 / 100 = 2700mm^2(8 @ 22mm ¤ bars of area 3040 mm^2)Some important notes:-

- Lap splices only at the central half of the member
- Hoops over the entire splice length at a spacing <150
- Not more than 50 % of the bars are spliced at one section.

Transverse reinforcement
Some important points:-

- Hoop requirement as per fig 7 in IS 13920:1993
- If the length of the hoop > 300 mm a cross tie shall be provided as shown in fig
- Hoop spacing should not exceed half the least lateral dimension of the column i.e.400/2 = 200mm

The design shear force for the column shall be the maximum of the the following:-

a) Calculated shear force as per the analysis which is 53.29 kN.
b)*Factored shear force as given by* $V_u = 1.4\ [M^{bL}_{u,lim} + M^{bR}_{u,lim} / h_s]_t$
where M ^{bL} $_{u,lim}$ and M^{bR} $_{u,lim}$ are the moments of opposite sign of beams framing in to the column from opposite faces and h_{st} is the storey height.
moment of resistance of the beam at node 2 ,assuming all other beams of the same floor is designed for the same critical design conditions as that of beam 54, we have

At node 2

P_t = 1960/(400x465) = 1.05%(atbottom)
$M_{u,lim} /bd^2$ =7.3(P_t = 1.05%, d'/d = 0.08)
$M_{u,lim} = 7.3 \times 400 \times 465^2$ = 631 kN-m (maximum saggingmoment) P_t =2613/(400x465)=1.40%(at top)
$M_{u,lim} /bd^2$ = 4.15 (P_t =1.05%, d'/d=0.08)
$M_{u,lim} = 4.15 \times 400 \times 465^2$ = 358kN-m(maximum hoggingmoment) V_u = 1.4 [631/6]= 147.23kN.
¤ = 1 +3 P 1u/A 1g f 1ck = 1 + (3 x 1160 x 1000) / [(400x500-3040) x 30] = 1.5890> 1.5

So w use ¤ = 1.5
Now corrected τ_c = 1.5 x 0.724 =1.086 N/ mm^2.Vc = τ_c bd = 1.086 x 400 x 500 = 217.2 kN.
Thus nominal shear reinforcement is to be provided in accordance to IS 456. We use 8 mm ¤, two legged stirrups of area 100.5 mm2.
For minimum stirrups we have $S_v \leq A_{sv}\ 0.87 f_y / 0.4$ i.e. ≤ 225The spacing shall be lesser of the a) 0.75 d= 0.75 x 475 = 356
b) 256 as calculated.
So we provide 8 mm phi two legged stirrups @ 225 mm. c/c

Special confining reinforcement
Special confining reinforcement is to be provided over a length of l_0 towards the mid span ofthe column
$l_0 \geq$ *depth of the member = 500 mm+
[1/6 of the clear span which is 1 m in this case][450 mm]
The spacings of the hoop shall not exceed
S_{max} < [¼(minimum member dimensions) = 100 in this case][Should not be less than 75]
[Should not be greater than 100].
Minimum area of cross-section of the bar forming hoop is$A_{sh} = 0.18\ sh\ f_{ck} / f_y\ (A_g / A_k - 1)$
We use s = 100 mm from above and h= 400So we have A_{sh} = 130 mm^2.
Using 10 mm dia bar (78.53 mm2) at a spacing of 100 x 78.53 / 130= 60 mm i.e . @ 60 mmc/c.

Design of interior column*[2]*
In this example the columns of the ground floor are designed for illustrations. The exteriorcolumns no 28 is designed for the forces based on maximum interaction ration (1 in thiscase)

- We have size of the column 400 mm x 500 mm
- Concretemix M 30
- Verticalreinforcement Fe 415
- Axial load 2220 kN
- Moment from load 398 kN

The general requirement of the column for the ductility will follow IS 13920:1993 and vertical reinforcement of the column is designed according to IS 456:2000. The transverse and the special confinement reinforcement willbe d by following the IS 13920:1993 and IS456:2000.

General(Column subjected to bending and axial load)

- IS 13920:1993 will be applicable if the axial stress > 0.1 f 1ck.
  2260 x 1000/ (400 x 500) = 11.3 > 0.1 f 1ck = 3.
- Minimum dimension of the member ≥ 250 and we have taken 400 which is ok.
- Shortest cross section dimension / perpendicular dimension ≥ 0.4 and we have thesame ratio as0.8

Vertical (longitudinal) reinforcement

Assume 20 mm ¤ with 40 mm cover (d1= 40 +10= 50 mm, d1 / d = 50/ 500 = 0.1)
From chart 44 SP 16: 1980 (d1/ d = 0.1, 415 N/ mm^2)
$P_u/ f_{ck}bD$ = 2220 x 1000/(30 x 400 x 500)= 0.36
$M_u/ f_{ck}bD^2$ = 0.1327.
Reinforcement on four sides from chart 44 SP 16: 1980P/f_{ck}=0.08%, reinforcement in % = 0.08 x 30 = 2.4%
A_s = pbd/ 100= 2.4 x 400 x 500 / 100 = 4800 mm^2(13 @ 22mm ¤ bars of area 4940mm^2)Some important notes:-

- Lap splices only at the central half of the member
- Hoops over the entire splice length at a spacing <150
- Not more than 50 % of the bars are spliced at one section.

Transverse reinforcement

Some important points:-

- Hoop requirement as per fig 7 on IS 13920:1993
- If the length of the hoop > 300 mm a cross tie shall be provided as shown in fig
- Hoop spacing should not exceed half the least lateral dimension of the column i.e.400/2 = 200mm

The design shear force for the column shall be the maximum of the the following:-

a) calculated shear force as per the analysis which is 154 kN.

b) factored shear force as given by $V_u = 1.4 [M^{bL}_{u,lim} + M^{bR}_{u,lim} / h_s]_t$

where $M^{bL}_{u,lim}$ and $M^{bR}_{u,lim}$ are the moments of opposite sign of beams framing in to the column from opposite faces and h_{st} is the storey height.

moment of resistance of the beam at node 2 ,assuming all other beams of the same floor is designed for the same critical design conditions as that of beam 54, we have

<u>At node 34</u>

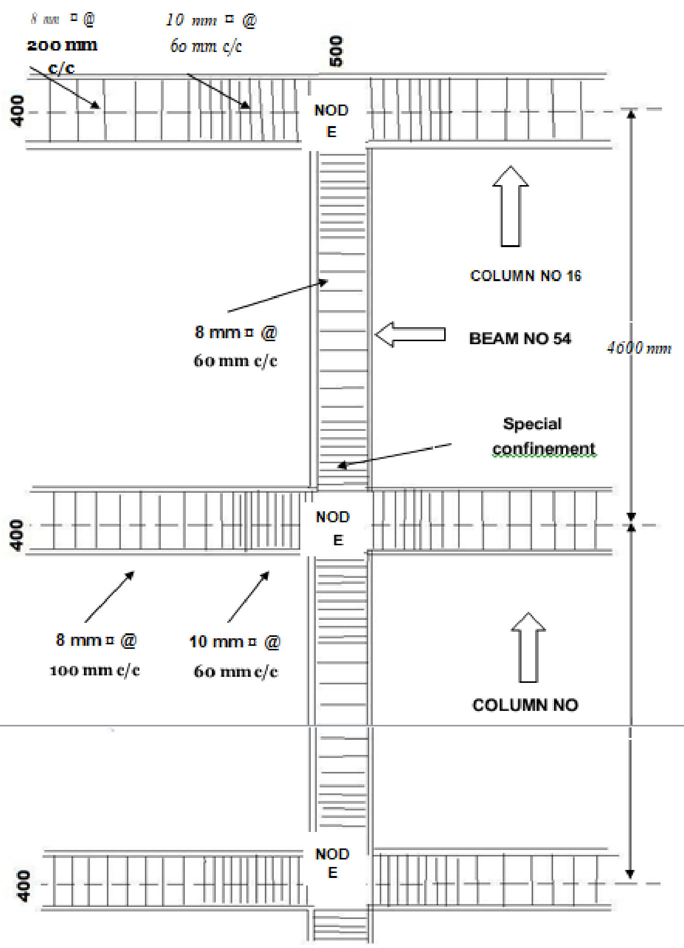
P_t = 1960/(400x465) = 1.05%(at bottom)
$M_{u,lim} /bd^2$ =7.3(P_t = 1.05%, d'/d = 0.08)
$M_{u,lim}$ = $7.3 \times 400 \times 465^2$ = 631 kN-m (maximum sagging moment) P_t =2613/(400x465)=1.40%(at top)
$M_{u,lim} /bd^2$ = 4.15 (P_t =1.05%, d'/d=0.08)
$M_{u,lim}$ = $4.15 \times 400 \times 465^2$ = 358kN-m(maximum hogging moment) V_u = 1.4 [631+ 358/5.5]= 251.7 kN
¤ = 1 +3 P 1u/A 1g f 1ck = 1 + (3 x 2220) / [(400x500- 4940) x 30] = 2.13 > 1.5

So w use ¤ = 1.5
Now corrected τ_c = 1.5 x 0.87 = 1.305 N/mm2.
Vc= τ_c bd = 1.305 x 400 x 500 = 261 kN.> 251 kN
Thus nominal shear reinforcement is to be provided in accordance to IS 456. We use 8 mm dia two legged stirrups of area 100.5 mm2.
For minimum stirrups we have
$S_v \le A_{sv}$ 0.87f_y / 0.4 i.e. ≤ 225The spacing shall be lesser of the a) 0.75 d= 0.75 x 475 = 356
b) 256 as calculated.
So we provide 8 mm phi two legged stirrups @ 225 mm. c/c

Special confining reinforcement

Special confining reinforcement is to be provided over a length of l_0 towards the mid span ofthe column
$l_0 \ge$ *depth of the member = 500 mm+
[1/6 of the clear span which is 1 m in this case][450 mm]
The spacing of the hoop shall not exceed
S_{max} < [¼(minimum member dimensions) = 100 in this case][Should not be less than 75]
[Should not be greater than 100]
Minimum area of cross-section of the bar forming hoop isA 1sh = 0.18 sh f_{ck} / f_y (A_g /A_k – 1)
A_{sh} = 0.18 sh f_{ck} / f_y (A_g /A_k – 1) We uses = 100 mm from above andh= 400So we have A_{sh} = 130 mm2.
Using 10 mm dia bar (78.53 mm2) at a spacing of 100 x 78.53 / 130= 60 mm i.e . @ 60 mmc/c

DUCTILITY DETAILING:-



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6.1 CONCLUSION & SCOPE FOR FURTHER STUDEY

The tasks of providing full seismic safety for the residents inhabiting the most earthquake-prone regions are far from being solved. However in present time we have new regulations in place for construction that greatly contribute to earthquake disaster mitigation and arebeing in applied in accordance with world practice. *[8]* [4]**

6.1 CONCLUSION

In the regulations adopted for implementation in India the following factors have been found to be critically important in the design and construction of seismic resistant buildings:

- sites selection for construction that are the most favourable in terms of the frequency of occurrence and the likely severity of ground shaking and ground failure;
- high quality of construction to be provided conforming to related IS codes such as IS1893 , IS 13920 to ensure good performance during future earthquakes.
- To implement the design of building elements and joints between them inaccordance with analysis .i.e. ductility design should be done.
- structural-spatial solutions should be applied that provide symmetry and regularity in the distribution of mass and stiffness in plan and in elevation.
- Whereas such the situations demands irregularity maximum effort should be given to done away with the harmful effects like that of "SHORT COLUMN EFFECT"

Researchers indicate that compliance with the above-mentioned requirements will contribute significantly to disaster mitigation, regardless of the intensity of the seismic loads and specific features of the earthquakes. These modifications in construction and design can be introduced which as a result has increase seismic reliability of the buildings and seismicsafety for humanlife.

6.2 Scope for further study

- The present study was conducted to find out comparison between seismicparameters such as:-
- base shear, roof displacement, time period, storey drift, storey displacement forsteel bare frame with knee braced patterns are studied.
- In this study moment resisting steel bare frame with knee bracing patterns are analysed using pushover analysis, equivalent static analysis, response spectrum analysis.

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