CHAPTER - 1

INTRODUCTION

1.1 GENERAL

Many reinforced concrete (RC) buildings have either collapsed or experienced different levels of damage during past earthquakes. Many investigations have been carried out on buildings that were damaged or ruined by different earthquake. Low-quality concrete, poor confinement of the end regions, weak column-strong beam behavior, short column behavior, inadequate splice lengths and improper hooks of the stirrups, faulty design consideration, were some of the important structural deficiencies. Most of those buildings were constructed before the introduction of modern building codes but these buildings have now more importance value. They usually cannot provide the required ductility, lateral stiffness and strength, which are definitely lower than the limits imposed by the modern building codes. Due to low lateral stiffness and strength, vulnerable structures are subjected to large displacement demands, which cannot be met adequately as they have low ductility.

Now a days, most of the strengthening strategies are based on global strengthening schemes as per which the structure is usually strengthened for limiting lateral displacements in order to compensate the low ductility. In these schemes, global behavior of the system is transformed. Another approach is modification of deficient elements to increase ductility so that the deficient elements will not reach their limit state conditions when subjected to design loads. However, the later strategy is more expensive and harder to implement in cases of many deficient elements which is the reason that the global strengthening methods have been more popular than element strengthening.

This method has increased the lateral load capacity and strength of the structure as well. It should also be noted that the method requires the sides of the buildings to be unobstructed for installation of new shear walls. The literature reviews presents numerous strengthening techniques. However, most of them require long-term construction works inside the building, rendering the building out of service for that period of time. On the other hand, external strengthening techniques offer advantages with respect to cost and ease of construction. This study investigates the performance of Exterior RC Shear walls (ESW) that are placed parallel to the building's sides. In reality, installing a shear wall to a structural system will surely improve the seismic capacity of the structure. The main concern is whether the design methods for the connection of old and new elements can satisfy codes. To make it clear, a program was carried out on RC models in E-tabs software. The program includes a reference model and strengthened models and the results obtained were discussed in detail in this present study.

1.2 OBJECTIVE

This thesis aims to know the effect of external shear wall connection to the existing building for strengthening purpose to resist the lateral load acting on it.

The principal objectives of the study are as follows:

1. Generation of model - 1 of a building (G+8) with rigid diaphragm on E tabs–software.
2. The model is subjected to earthquake load in both x and y direction. Considering the building symmetrical in plan and is located in zone IV on medium soil strata site.
3. Analyze of lateral displacement in both the direction, storey shear forces, modal information, Column forces, beam forces etc.
4. Again another model as model – 2, of same dimension and properties as of model – 1, is created but this time external shear wall are connected to the building model with the help of steel bars as links and then the result of same parameters, as for model -1 is obtained.
5. Again another model as model – 3, is created same as that of model – 2, but this time opening are provided in the external shear wall and the model is analyzed, then for same parameters results are obtained.
6. Last comparisons of results and discussion of same parameters of different model, is being done, and finally Conclusion has been drawn.

1.3 SCOPE

The main aim of the External shear wall to strengthened the existing structure and to investigate the different ways in which the structures can be stabilized against the effects of strong seismic loading. Some other reasons why we use shear walls are, structures can be constructed which reduces the area used and we can accommodate a large population in that particular area. Other objective is to construct a cost effective structure in less period of time. This study helps in the investigation of strength and ductility of walls. The scope is to analyze the constructed shear wall that is to be constructed. Firstly the model is implemented into known computer software E-tabs and then it is analyzed based on the investigation of strength and ductility and the results are obtained quickly.

Further scope in civil engineering field is that we can retrofit the damaged or building susceptible to damage due various forces acting on it, by providing external shear wall which not only reduces the time of construction but also disturbance to the occupant is also removed. So strengthening of structure can be effectively done by the use of external shear wall and found to be economical also and this has been cleared by the results came out by this study and this method has wider application in countries like India where more than 60% of population lives in earthquake prone area and buildings are susceptible to damage.

1.4 METHODOLOGY

1) Study of various research papers and literature regarding the use of external shear wall of different types to strengthen the structure.
2) Study of IS 456:2000 Plain and reinforced concrete -code of practice.
3) Study of IS 1893(part 1):2002 Criteria for earthquake resistant design of structures.
4) Study of IS 13920:1993 Ductile detailing of reinforced concrete structures subjected to seismic forces — code of practice.
5) Study of IS 875:1987 Code of practice for design loads (other than earthquake) for buildings and structures.
6) Study of full manual of E-tabs software related to design of concrete framed structure and design of shear wall.
7) Several Models have been made and analyzed for different combinations of load conditions as per code.
8) Comparison of a reference model (without external shear wall as model 1) and strengthened model(with external shear wall without opening and shear wall with opening provided as model 2 and model 3 respectively) and compare their results of displacements and forces and many more parameter.
9) Various tables and graph of storey displacements, storey shear, time period, forces in beams and column…..etc.
10) Finally Conclusion on the bases of result obtained.

CHAPTER - 2

LITERATURE REVIEW

2.1 Seismic Strengthening Of RC Structures With Exterior Shear Walls By Hasan Kaplan, Salih Yilmaz, Nihat Cetinkaya And Ergin Atimtay.

In this study on seismic strengthening of the RC buildings by exterior shear walls has been carried out. Structures of the two storey framed model were tested under the imposed reversed cyclic lateral sway to simulate seismic loadings. It is observed that the implementation of shear walls to the structural system has improved the capacity of the bare frame as expected.

2.2 Guidelines for Epoxy Grouted Dowels in Seismic Strengthening Projects.

Frequently, it is necessary to strengthened existing concrete structures for improved seismic performance, either after a damaging earthquake or in preparation for a future extent. This work always involves attaching a new concrete or steel members to the existing structures .epoxy grouted dowels are ideal for this task due to the strength and ease of installation of epoxy resins to anchor dowels or threaded rods. The short term loading on dowels from seismic loading precludes creep concern and since dowels are grouted within the concrete mass, there is sufficient insulation to protect the epoxy from heat sources such as fires.

2.3 Recent Advancements in Retrofit of RC Shear Walls by K. galal And H. El-sokkary.

This literature says that Reinforced concrete (RC) shear walls has widely application in medium and high-rise buildings used for various residential as well as commercial purposes to provide the lateral strength, stiffness and energy dissipation capacity required to resist lateral loads arising from earthquake or wind loading. In past decades, there have been considerable modifications adopted in the design of RC shear walls for new construction as well as for old constructions. The modern adopted performance evaluation methods and capacity design principles are examples of these important advancement in earthquake engineering. Therefore, there is a mandatory need to enhance the seismic performance of old building using RC shear walls. So they meet the need of the new performance-based seismic design techniques. Several retrofit techniques using different materials are reported in this literature. These vary from using steel, concrete, fiber-reinforced polymers, and shape memory alloys as retrofitting materials used in different methods of application. This paper presents different types of retrofit techniques that are used to upgrade the seismic resistance of existing building with RC shear walls. The paper discusses the various types of advantages and disadvantages of each retrofit technique and their characteristic enhancements. The objective of this paper is to provide modern techniques on the recent advancements and challenges in the area of retrofit of RC shear walls.

2.4 An Experimental Investigation for External RC Shear Wall Applications by M. Y. Kaltakci, M.Ozturk, And M. H. Arslan.

This literature dealing with the strength and rigidity of various reinforced concrete (RC) buildings in Turkey, which are recently damaged by destructive earthquakes, is not at a sufficient level. Therefore earthquake causes a significant loss of life and property. The strengthening method most commonly preferred for these types of RC buildings is the application of RC infill walls (shear walls) in the frame openings of the building.

The whole building has to be vacated and huge costs needed during this type of retrofitting, most of the population preferred not to rehabilitate their buildings although they were in heavy risk. Therefore, it is immediately necessary to develop easier method to apply and more effective methods for the retrofitting of residential purpose buildings and public buildings which cannot be vacated during the strengthening process (such as hospitals and schools). This study empirically provide the different methods which can meet this need. In these methods, named “external shear wall application”, RC shear walls are applied on the external surface of the structures, along the frame plane despite of inside the building. To this end, 7 test samples in 1/2 and 1/3 geometrical scale were designed to analyzed the performance of the retrofitting technique where the shear wall leans on the frame from outside of the building (external shear wall application) and of the strengthening technique where a specific space is left between the frame and the external shear wall by using a coupling beam to connect elements (application of external shear wall with coupling beam).

2.5 Guideline for Seismic Upgrading Of Building Structures.

Most of the buildings in earthquake prone areas across Canada were built before there was better understanding of earthquake resistance. Most of these buildings would be deemed unsafe by present building codes and the same condition in India also, because code requirements are written for the design of new buildings and not for the evaluation of existing buildings, the cost of upgrading an existing building to the current code provisions can be very large, as well as destructive to its heritage value. A set of alternate procedures for evaluating existing buildings was therefore prepared by NRC and published in the Guidelines for Seismic Evaluation of Existing Buildings (thereafter referred to as "Guidelines for Seismic Evaluation. New shear walls may consist of reinforced concrete, reinforced masonry, plywood on studs, or steel. These systems can be placed within the building, as interior or exterior walls or bracing, or outside the building as buttresses.

2.6 International Journal Of Innovative Research In Science, Engineering And Technology By Venkata Sairam Kumar.N, Surendra Babu.R, Usha Kranti.J.

Shear walls are the structural systems that provide lateral stability to structures from lateral loads like wind, earthquake loads. These structural systems are constructed by reinforced concrete, plywood/timber unreinforced masonry, reinforced masonry at which they are sub divided into coupled shear walls, shear wall frames, shear panels and staggered walls. The present paper work was made in the interest of studying various research works involved in modifications of shear walls and their behavior towards lateral loads. As we know shear walls resists major portions of lateral loads in the bottom portion of the structures and the framed element supports the lateral loads in the top portions of structures which is necessary for soft storey medium and high rise buildings, buildings which are similar in nature constructed in India, In India basements are generally used for parking and garages or offices and top floors are used for residential purposes. Vibrations which are caused under the earth's surface generate waves which disturb the earth's surface, termed as earthquakes. It was said that earthquakes will not kill human but structures which are not constructed in considering the earthquake forces do. 60% of India lying in earthquake prone zone at which there is a frequent need of increase of understanding the behavior of earthquake, and methods for constructing and developing earthquake resistant buildings. Shear walls created are used to resist the lateral forces generated during earthquake. Shear walls behavior ultimately depends upon the material used for its construction, shear wall thickness, wall length, wall positioning in building frame and many more factors like reinforcement provided in it is according to code provisions.

2.7 Seismic Evaluation and Strengthening Of Existing Buildings

By Dr. Durgesh C. Rai Department Of Civil Engineering Indian Institute Of Technology Kanpur.

Emphasis is on identifying the most vulnerable components of the buildings at risk, and retrofitting them with their risk level to decrease the risk of partial or complete collapse. However, these steps do not necessarily stop earthquake damage in strengthened buildings, besides these procedures used in strengthening, improves the building performance in earthquakes. In simple words we can say that 'strengthening' means same as 'retrofitting' or improving the seismic performance.

Global strengthening of the structure may be an economical and effective rehabilitation strategy where a seismic Evaluation shows unacceptable performance due to overall structural strengthened. This can be identified when the onset of global inelastic behavior occurs at levels of ground shaking that are substantially less than code design levels. By introducing additional strength to such lateral-force resisting system, it is possible to raise the threshold value of ground motion at which the occurrence of damage occurs. Shear walls and braced frames are the elements used for this purpose, but they should have sufficient stiffness than that of the building to which they are supplemented. This requires their design to provide nearly all of the structure's lateral resistance. This is necessary whenever desirable, as the existing members will probably have very little inelastic strength. Adding of new structural system in an existing building will automatically change the dynamic behavior of the complete building considerably during the earthquake. The choice of the type of element, its number and their size of the added elements depends on the vulnerability of the existing building and the functional layout of the building. Shear walls, because of its greater stiffness and lateral strength, may provide the most significant part of the earthquake resistance of the building structure.

CHAPTER – 3

DETAILED DESCRIPTION OF IS 1893(PART 1):2002

3.1 ASSUMPTIONS

The following assumptions were considered in the earthquake resistant design of structures: Earthquake generates impulsive ground motions, which are not only complex in nature but also irregular in character, changing in the time period and amplitude each lasting for a small duration. Therefore, resonance of the type will not occur as it would need time to buildup such amplitudes.

3.2 SOME IMPORTANT TERMINOLOGY USED FOR EARTHQUAKE ENGINEERING OF BUILDINGS

3.2.1 DAMPING

The effect of internal friction, imperfect elasticity of materials, slipping and sliding etc. in decreasing the amplitude of vibration, is expressed as a percentage of critical damping.

3.2.2 DESIGN BASIS EARTHQUAKE (DBE)

It is the earthquake that is considered ,which can be expected to occur at least once during the design life of the structure.

3.2.3 INTENSITY OF EARTHQUAKE

The intensity of an earthquake at a given place is a measure of the strength of shaking during the Earthquake, and is denoted by a number according to the Modified Mercalli Scale or M.S.K. of seismic intensities (given in annex D of code).

3.2.4 MAXIMUM CONSIDERED EARTHQUAKE (MCE)

The most severe earthquake effects that should be considered in earthquake engineering.

3.2.5 IMPORTANCE FACTOR (I)

This is the factor which is used to get the design seismic forces depending on the functional use of the building, characterized by hazardous consequences of its failure, its post-earthquake functional need, historic value, or economic importance.

TABLE NO. 1 IMPORTANCE FACTOR OF A BUILDING

Sl. no.	Structure	Importance factor
1	**2**	**3**
(i)	Important service and community buildings, such as hospitals, schools, monumental structures, emergency buildings like telephone exchange, television stations, radio stations, railway stations, fire station buildings, large community halls like cinemas, assembly halls and subway stations, power stations	1.5
(ii)	All other buildings	1.0

3.2.6 MAGNITUDE OF EARTHQUAKE (RICHTER% MAGNITUDE)

The magnitude of an earthquake is a number, which is a measure of the amount of energy released by an earthquake. It is defined as logarithm to the base 10 of the maximum trace amplitude, expressed in microns, which the standard short-period torsion seismometer (with a period of 0.8s, magnification 2800 and damping nearly critical) would register due to the earthquake at an epicentral distance of 100 km.

3.2.7 MODAL MASS (M_K)

Modal mass of a structure subjected to horizontal or vertical, as the case maybe, ground motion is a part of the total seismic mass of the structure that is effective in mode k of vibration. The modal mass for a given mode has a unique value irrespective of scaling of the mode shape.

$$M_k = (\sum W_i \phi_{ik})^2 / (g \sum W_i \phi_{ik}^2)$$

Where,

g = acceleration due to gravity

$\boldsymbol{\phi_{ik}}$ = mode shape coefficient at floor i^{th} in mode k

$\mathbf{W_i}$= Seismic weight of floor i^{th}

3.2.8 MODAL PARTICIPATION FACTOR (P_K)

Modal participation factor of mode k of vibration is the amount by which mode k contributes to the overall vibration of the structure under horizontal and vertical earthquake ground motions. Since the amplitudes of 95 percent mode shapes can be scaled arbitrarily, the value of this factor depends on the scaling used for mode shapes.

$$P_k = (\sum W_i \phi_{ik})/(\sum W_i \phi_{ik}^2)$$

3.2.9 MODE SHAPE COEFFICIENT ($Ø_{IK}$)

When a system is vibrating in normal mode k, at any particular instant of time, the amplitude of mass i expressed as a ratio of the amplitude of one of the masses of the system, is known as mode shape coefficient ($ø_{ik}$).

3.2.10 FUNDAMENTAL NATURAL PERIOD (T_1)

This is the first (Longest) modal time period of vibration.

3.2.11 CENTRE OF MASS

This is the point through which the resultant of the masses of a system acts. This point corresponds to the centre of gravity of masses of system.

3.2.12 BASE DIMENSIONS (D)

Base dimension of the building along a direction is the dimension at its base, in meter, along that direction.

3.2.13 SEISMIC WEIGHT (W)

It is the total dead load plus appropriate amounts of specified imposed load.

3.2.14 DESIGN SEISMIC BASE SHEAR (V_B)

It is the total design lateral force at the base of a structure.

3.2.15 RESPONSE REDUCTION FACTOR (R)

It is the factor by which the actual Base Shear Force, that would be generated if the structure were to remain elastic during its response to the Design Basis Earthquake (DBE) shaking, shall be reduced to obtain the design lateral force.

TABLE NO. 2 Response Reduction Factor

S .no.	Lateral Load Resisting System	R
1	**Building Frame Systems**	
2	Ordinary RC moment resisting frame (OMRF)	3
3	Special RC moment-resisting frame (SMRF)	5
4	**Steel frame with**	
5	(a) Concentric braces	4
6	(b) Eccentric braces	5
7	**Steel moment resisting frame designed as per SP 6 (6)**	5
8	**Buildings with Shear Walls**	
9	Ordinary reinforced concrete shear walls	3
10	Ductile shear walls	4
11	**Buildings with Dual Systems**	
12	Ordinary shear wall with OMRF	3
13	Ordinary shear wall with SMRF	4
14	Ductile shear wall with OMRF	4.5
15	Ductile shear wall with SMRF	5

3.2.16 STRUCTURAL RESPONSE FACTORS (S_A/G)

It is a factor denoting the acceleration response spectrum of the structure subjected to earthquake Ground vibrations, and depends on natural period of vibration and damping of the structure.

3.2.17 ZONE FACTOR (Z)

It is a factor to obtain the design spectrum depending on the perceived maximum seismic risk characterized by Maximum Considered Earthquake (MCE) in the zone in which the structure is located. The basic zone factors included in this standard are reasonable estimate of effective peak ground acceleration.

TABLE NO.3 Zone Factor (Z)

SEISMIC ZONE	II	III	IV	V
SEISMIC INTENSITY	LOW	MODERATE	SEVERE	VERY SEVERE
Z VALUE	0.1	0.16	0.24	0.36

3.2.18 DIAPHRAGM

It is a horizontal or we can say that nearly horizontal system, that transmits lateral forces to the vertical resisting elements, for example reinforced concrete floors and horizontal bracing systems.

3.2.19 PRINCIPAL AXES

Principal axes of a building are generally two mutually perpendicular horizontal directions in plan of a building along which the geometry of the building is oriented.

3.2.20 STOREY DRIFT

It is the displacement of one level relative to the other level above or below.

3.2.21 STOREY SHEAR (V_I)

It is the sum of design lateral forces at all levels above the storey under consideration.

Chapter - 4

4.1 DIFFERENT TYPES OF LOADS ACTING ON BUILDING

4.1.1 DEAD LOADS

All the permanent type of constructions of the buildings form the dead loads. The dead load comprises of the weights of walls, false ceilings, partitions floor finishes, false floors and the other permanent constructions in the buildings. The dead load value can be calculated from the dimensions of various members and their unit weights. The unit weights of plain concrete and reinforced concrete made with sand and gravel or crushed natural stone aggregate may be taken as 24 KN/m^3 and 25 KN/m^3 respectively.

4.1.2 IMPOSED LOADS

Imposed load is produced by the intended use or occupancy of a building including the weight of movable partitions, distributed and concentrated loads, load due to impact and vibration loads.

TABLE NO. 4 PERCENTAGE OF IMPOSED LOAD TO BE CONSIDERED IN SEISMIC WEIGHT CALCULATION (CLAUSE 7.3.1)

Imposed Uniformity Distributed Floor Loads ($KN/M^{2)}$	PERCENTAGES OF IMPOSED LOAD
(1)	(2)
Upto And Including 3	25
Above 3	30

***NOTE**: Reduced live loads are considered as per clause 7.3.1 of IS 1893 (part 1): 2002, even though it is proposed to drop this clause in the new edition of the code. For the present case, (live load of 3 KN/m^2) 25% of live load is considered for seismic weight calculations

4.2 SEISMIC LOAD

4.2.1 Design Lateral Force

Buildings and its various portions there of shall be designed and constructed, to resist the effects of design lateral force due to earthquake and wind loading.

It is the horizontal seismic force, that should be used to design a structure or building. The design lateral force should be firstly computed for the building as a whole. Then this value of design lateral force shall be distributed to the various floor levels of the building considered. The overall design seismic force thus obtained at each floor level shall then be distributed to individual lateral load resisting elements depending on the floor diaphragm action.

<u>TABLE NO.5 OF LOAD COMBINATIONS FOR EARTHQUAKE LOADING</u>:

S.No	Load Combination	DL	LL	EQX or EQY
1.	1.5DL	1.5	-	-
2.	1.5DL+1.5LL	1.5	1.5	-
3.	1.2(DL+LL+EQX)	1.2	1.2	+1.2
4.	1.2(DL+LL-EQX)	1.2	1.2	-1.2
5.	1.2(DL+LL+EQY)	1.2	1.2	+1.2
6.	1.2(DL+LL-EQY)	1.2	1.2	-1.2
7.	1.5(DL+EQX)	1.5	-	+1.5
8.	1.5(DL-EQX)	1.5	-	-1.5
9.	1.5(DL+EQY)	1.5	-	+1.5
10.	1.5(DL-EQY)	1.5	-	-1.5
11.	0.9DL+1.5 EQX	0.9	-	+1.5
12.	0.9DL-1.5 EQX	0.9	-	-1.5
13.	0.9DL+1.5 EQY	0.9	-	+1.5
14.	0.9DL-1.5 EQY	0.9	-	-1.5

4.2.2 DYNAMIC ANALYSIS

Dynamic analysis should be performed to get the design seismic forces, and its distribution to different levels along the height of the building and to the various lateral load resisting elements, for the following buildings.

A) Regular Buildings

Those buildings greater than 40 m in height in Zones IV and V and those greater than 90 m in height in Zones II and III.

B) Irregular Buildings

All framed buildings higher than 12 m in Zones IV and V and those greater than 40 m in height in Zones II and III. Buildings with plan irregularities, as defined in Table4 (as per clause7), cannot be modeled for dynamic analysis by the method given in clause 7.8.4.5.

NOTE: For irregular buildings, lesser than 40 m in height in Zones II and III, dynamic analysis, even though not mandatory, is recommended.

Dynamic analysis may be performed (i) by the Time History Method (ii) by the Response Spectrum Method. However, in either of the methods, the design base shear (V_B) shall be compared with a base shear (V_B^*) calculated, using a fundamental period $T_{a.}$ Where V_B is less than V_B^* and all the response quantities (like member forces, displacements, storey forces, storey shears and base reactions) should be multiplied by V_B^* / V_B .

The value of damping for buildings may be taken as 2 and 5 percent of the critical. For the purposes of dynamic analysis of steel and reinforced concrete buildings, respectively.

4.2.3 MODES TO BE CONSIDERED

The number of modes to be used in the analysis should be such that the sum total of modal masses of all modes considered is at least 90 percent of the total seismic mass and missing mass correction beyond 33 percent.

4.2.4 DESIGN SPECTRUM

The purpose of determining seismic forces, our country has been classified into four seismic zone. The design horizontal seismic coefficient A_h for a structure shall be determined by the following expression

$$A_h = Z*I*S_a / 2*R*G$$

Provided that for any structure with $T \leq 0.1$ s, the value of A_h will not be taken less than Z/2 whatever be the value of I/R.

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

I = Importance factor, depending upon the functional use of the buildings, which are characterized by hazardous consequences of its failure, post-earthquake functional needs, historical value, or economic importance (Table 6).

R = Response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. However, the ratio (I/R) shall not be greater than 1.0 (Table 7). The values of R for buildings are given in Table 7.

S_A/g = Average response acceleration coefficient for rock or soil sites and Table 3 based on appropriate natural periods and damping of the structure. These curves Represent free field ground motion.

TABLE NO.6 MULTIPLYING FACTORS FOR OBTAINING VALUES FOR OTHER DAMPING (CLAUSE 6.4.2):

DAMPING PERCENT	0	2	5	7	10	15	20	25	30
FACTOR	3.2	1.4	1	0.9	0.8	0.7	0.6	0.55	0.5

The design acceleration spectrum for vertical motions, when required, may be taken as two-thirds of the design horizontal acceleration spectrum specified in 6.4.2.

For Medium Soil Sites:

$$Sa/g = \begin{cases} 1 + 15T & (0.00 \le T \le 0.10) \\ 2.50 & (0.10 \le T \le 0.55) \\ 1.36/T & (0.55 \le T \le 4.00) \end{cases}$$

(T= time period in seconds)

4.2.5 Design Seismic Base Shear (static analysis, clause 7.5.3)

The total design lateral force or design seismic base shear (V_B) along any principal direction shall be determined by the following expression

$$V_B = A_H * W$$

Where,

A_H = design horizontal acceleration spectrum.

W = seismic weight of all the floors.

4.2.6 Fundamental Natural Period

The approximate fundamental natural period of vibration (T_a), in seconds, of a moment-resisting frame building without brick in the panels may be estimated by the empirical expression

$$T_a = 0.075\ h^{0.75} \text{ for RC frame building}$$
$$T_a = 0.085\ h^{0.75} \text{ for steel frame building}$$

Where,

h = Height of building, in m.

This excludes the basement storey, where basement walls are connected with the ground floor deck or fitted between the building columns. But it includes the basement storey, when they are not so connected.

The approximate fundamental natural period of vibration (T_a), in seconds, of all other buildings, including moment-resisting frame buildings with brick lintel panels, may be estimated by the empirical Expression

$$T = .09H/\sqrt{D}$$

Where, H = Height of building in meter.

D = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force.

4.2.7 Distribution of Design Force

The Vertical distribution of base shear at different floor level. The design base shear (V_B) calculated shall be distributed along the height of the building as

$$Q_i = V_B W_i h_i^2 / \sum W_i h_i^2$$

W_i = Seismic weight of floor i
Q_i = Design lateral force at floor i,
H_i = Height of floor i measured from base, and
N = Number of storey in the building is the number of levels at which the masses are located.

4.3 SHEAR WALLS (AS PER IS 13920:1993)

A wall that is primarily designed to resist lateral forces in its own plane.

GENERAL REQUIREMENTS

1) The thickness of any part of the wall should preferably be adopted, not be less than 150 mm. The effective flange width, used in the design of flanged wall sections, shall be assumed to extend beyond the face of the web for a distance which shall be the smaller of

(a) Half the distance to an adjacent shear wall web, and
(b) $1/10^{th}$ of the total wall height.

Shear walls should be provided with steel reinforcement in the longitudinal as well as in the transverse directions in the plane of the wall. The minimum reinforcement ratio shall be 0.0025 of the gross area in each direction. This reinforcement shall be distributed uniformly across the cross section of the wall as per the code recommends.

When the factored shear stress in the wall exceeds $0.25\sqrt{f_{ck}}$ or if the wall thickness exceeds 200 mm, steel reinforcement should be provided in two curtains, each having bars running in the longitudinal and transverse directions in the plane of the wall.

The diameter of the bars used in any part of the wall shall not exceed ($l/10^{th}$) of the thickness of that part of the wall. The maximum spacing of reinforcement in either direction shall not exceed the smaller of $l_w/5$, $3t_w$, and 450 mm; where l_w is the horizontal length of the wall, and t_w is the thickness of the wall web.

4.3.1 SHEAR STRENGTH:

The nominal shear stress, τ_v shall be computed as

$$T_v = v_u / t_w * d_w$$

V_u = factored shear force,

t_w = thickness of the web, and

d_w = effective depth of wall section. This may by taken as 0.8 l_w, for rectangular sections.

The design shear strength of concrete τ_c shall be calculated as per Table 13 of IS 456: 1978.

The nominal shear stress in the walls τ_v shall not exceed $\tau_{c,max}$ as per Table 14 of IS 456 :1978. When τ_v is less than τ_c shear reinforcement shall be provided and When τ_v is greater than τ_c, the area of horizontal shear reinforcement, A_h to be provided within a vertical spacing. S_v is given by

$$V_{us} = 0.87\, f_y\, A_h\, d_w / s_v$$

Where $V_{us} = (V_u - \tau_c\, t_w\, d_w)$, is the shear force to be resisted by the horizontal reinforcement.

4.3.2 Flexural Strength:

The moment of resistance, M_{uv} of the wall section shall be calculated as done for the columns subjected to combined bending and axial load case as per IS 456 : 1978. The moment of resistance of slender, rectangular shear wall section with uniformly distributed vertical reinforcement is given in Annex A (IS 13920:1993).

4.3.3 Boundary Elements:

These are portions along the wall edges that are strengthened by longitudinal and transverse reinforcement. When the extreme fibre compressive stress in the wall due to factored gravity loads plus factored earthquake force exceeds $0.2f_{ck}$ boundary elements shall be provided along the vertical boundaries of walls. The boundary elements can be in discontinued form where the calculated compressive stress becomes less than $0.15f_{ck}$. The latter may be calculated as

$$M_U - M_{UV} \ / \ C_W$$

Where

M_U = factored design moment on the entire wall section,

M_{UV} = moment of resistance provided by distributed vertical reinforcement across the wall section, and

C_W = center to center distance between the boundary elements along the two vertical edges of the wall.

The percentage of vertical reinforcement in the boundary elements shall not be less than 0.8 percent, nor greater than 6 percent.

4.3.4 Coupled Shear Walls

Coupled shear walls shall be connected by ductile coupling beams. If the earthquake Induced shear stress in the coupling beam exceeds

$$0.1 \ L_s \sqrt{f_{ck}} \ / \ D$$

Where L_s is the clear span of the coupling beam and D is its overall depth, the entire earthquake Induced shear and flexure shall, preferably, be resisted by diagonal reinforcement.

The area of reinforcement to be provided along each diagonal in a diagonally reinforced coupling beam shall be:

$$A_{sd} = \frac{Vu}{1.74 \ f_y \sin\alpha}$$

Where V_u**,** is the factored shear force, and $\boldsymbol{\alpha}$ is the angle made by the diagonal reinforcement with the horizontal.

4.3.5 OPENINGS IN WALLS

The shear strength of a wall with openings should be checked along the critical planes that pass through openings. Reinforcement shall be provided along the edges of openings in walls. The area of the vertical and horizontal bars should be such as to equal that of the respective interrupted bars.

CHAPTER - 5

STRUCTURAL MODELLING AND ANALYSIS

5.1 MATERIAL PROPERTIES

The material properties used in creating the model were as follows:

1. Grade of Concrete – M 25 (For Columns) and M 20 (For Beams)
2. Grade of Reinforcement – Fe 415
3. Poisson Ratio of Concrete – 0.2
4. Poisson Ratio of Reinforcement – 0.3
5. Density of Concrete – 25KN/m^3
6. Density of Reinforcement – 78.5KN/m^3
7. Young's Modulus of concrete – 25000000KN/m^2 (M25)
   – 22360000KN/m^2 (M20)
8. Young's Modulus of reinforcement – 2.1X10^{10} KN/m^2
9. Damping Factor – 0.05 (As per Clause 7.8.2.1 of IS 1893(Part 1):2002

5.2 GEOMETRIC PROPERTIES

The geometrical properties measured and used to create model were as follows:

1. The slab thickness – 125 mm
2. Beam cross sections on all floors – 0.3mX0.50m
3. Column cross section on all floors – 0.3mX0.75m
4. Storey Height – 3.5m on the lower most storey
   – 3.0m on all the above stories
5. Spans – 5.0m x 5.0m
6. Link1 of steel bars = 25mm
7. wall thickness = 200mm

5.3 LOADING

1. Dead load due to self-weight of the structure considered.

2. Live load has been taken as 4 KN/m^2. As per commercial Buildings (i.e. Hospital, office, School)
3. Brick wall load of 115mm consider = 2.75kN/m/m x height (3-.0.5) m= 6.875 KN/m
4. Parapet wall load on top storey is = 3 KN/m

5.4 PROBLEM 1: DETAILS OF MODEL - 1 AND ANALYSIS

A model of (G+8) storey reinforced concrete framed building is located in the seismic zone-IV and on medium soil. The building measures 25 m, each way in plan at all floor levels. Storey heights are about 3m at all elevations and bottom storey is 3.5 m in height. The roof and floors are of concrete slabs consisting of a 125 mm slab thickness. The beams in the building are of (0.3 m x 0.5 m) and columns in the building are (0.3m x 0.75m). The material properties are f_{ck} = 20 mpa, f_{ck} = 25 mpa and f_y = 415 mpa.

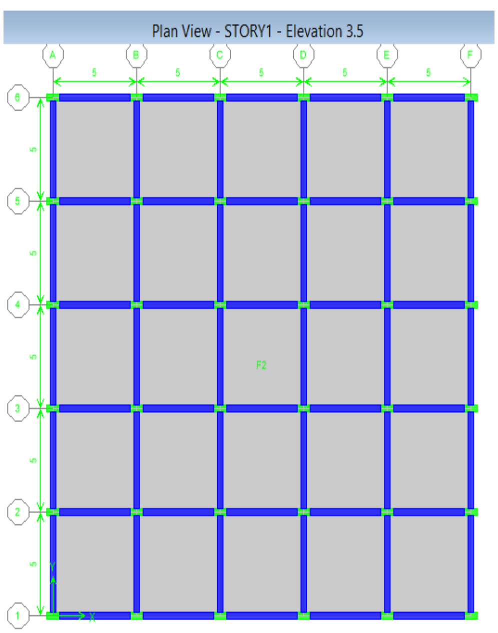


Fig.1 plan view of model -1

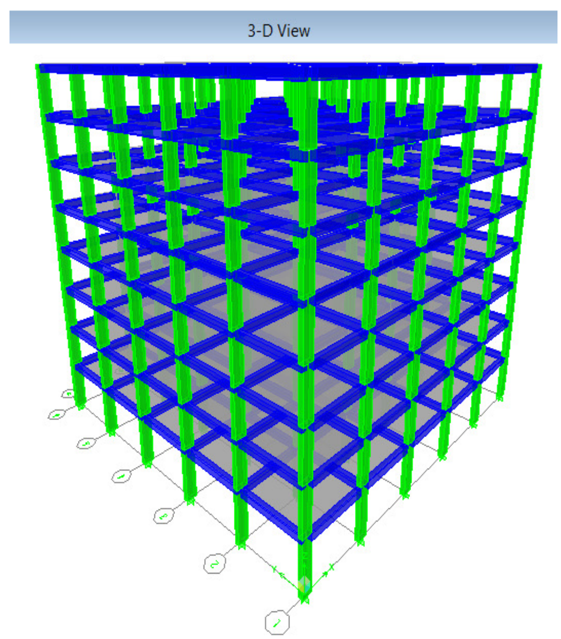


Fig.2 (3-D view of model - 1)

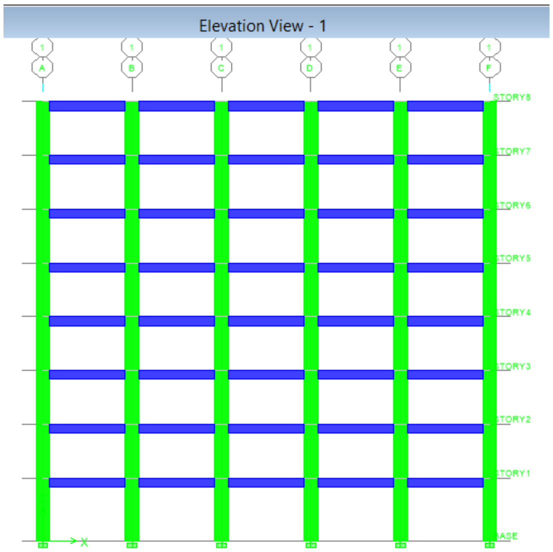


Fig.3 elevation view of model 1

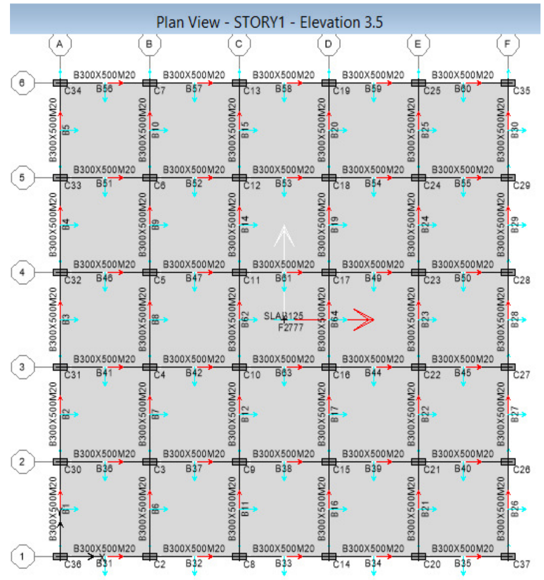


Fig.4 plan view with section dimension model-1

Above Model Is Subjected To Different Load Combinations as per code

S.No	Load Combination	DL	LL	EQ
1.	1.5DL	1.5	-	-
2.	1.5DL+1.5LL	1.5	1.5	-
3.	1.2(DL+LL+EQX)	1.2	1.2	1.2
4.	1.2(DL+LL-EQX)	1.2	1.2	-1.2
5.	1.2(DL+LL+EQY)	1.2	1.2	1.2
6.	1.2(DL+LL-EQY)	1.2	1.2	-1.2
7.	1.5(DL+EQX)	1.5	-	1.5
8.	1.5(DL-EQX)	1.5	-	-1.5
9.	1.5(DL+EQY)	1.5	-	1.5
10.	1.5(DL-EQY)	1.5	-	-1.5
11.	0.9DL+1.5 EQX	0.9	-	1.5
12.	0.9DL-1.5 EQX	0.9	-	-1.5
13.	0.9DL+1.5 EQY	0.9	-	1.5
14.	0.9DL-1.5 EQY	0.9	-	-1.5

After assigning different load combinations and shell area load on slab (that include the self weight that is dead load, live load) and frame line load is also assigned that has been mentioned above with details.

Run the analysis of the framed building has been done.

DETAILS OF RESULTS OBTAINED AS

1) Lateral Displacement Results in (mm)

Table 5.1 Lateral displacement in x - direction for EQX at different storey level

Storey	Diaphragm	Load	lateral displacement in x for model 1
STOREY1	D1	EQX	6.1141
STOREY2	D1	EQX	15.0525
STOREY3	D1	EQX	24.6823
STOREY4	D1	EQX	34.0455
STOREY5	D1	EQX	42.551
STOREY6	D1	EQX	49.6861
STOREY7	D1	EQX	55.0056
STOREY8	D1	EQX	58.3906

Graph 5.1Variation of Lateral displacement in X - Direction Due To EQX

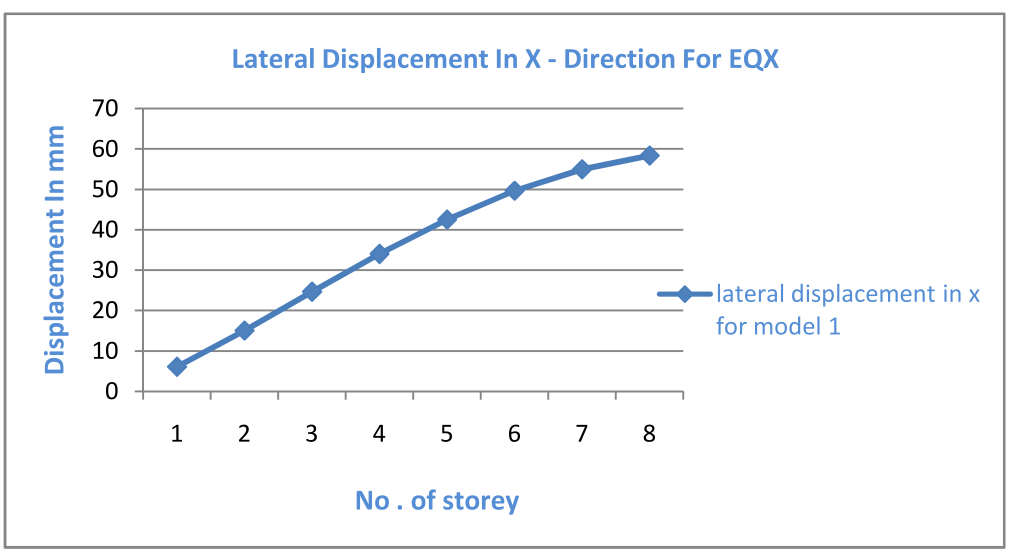
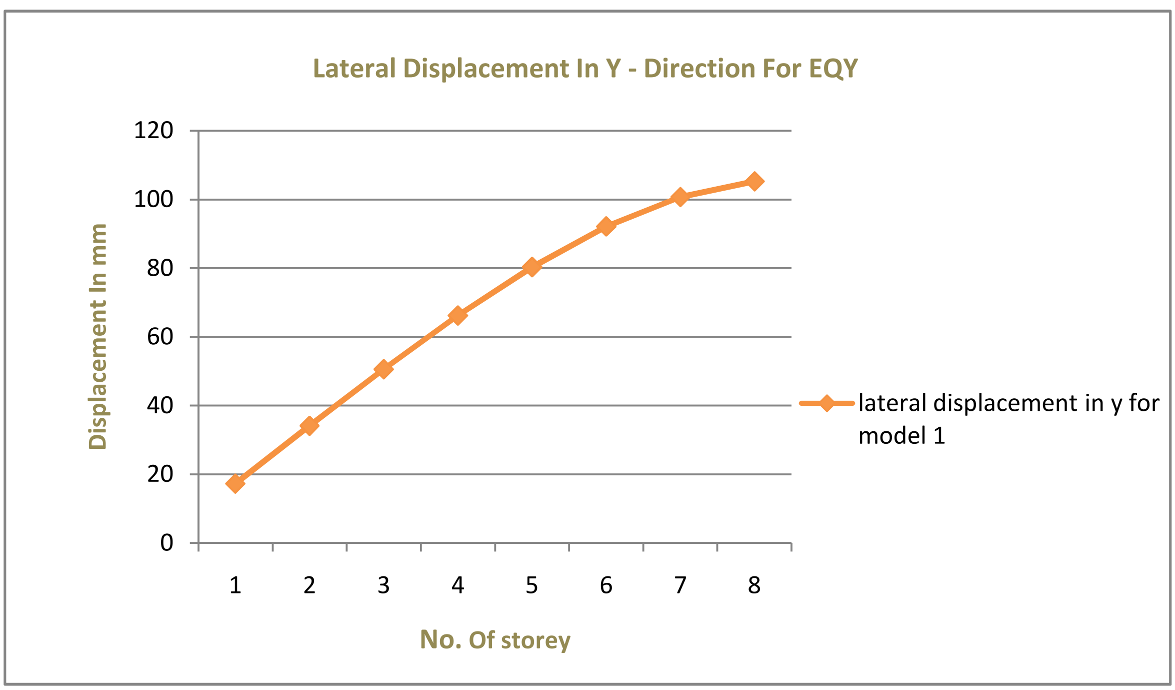


Table 5.2 Lateral displacement in Y - direction for EQY at different storey level

Storey	Diaphragm	Load	lateral displacement in y for model 1
STOREY1	D1	EQY	17.2714
STOREY2	D1	EQY	34.1038
STOREY3	D1	EQY	50.5584
STOREY4	D1	EQY	66.1714
STOREY5	D1	EQY	80.2932
STOREY6	D1	EQY	92.1144
STOREY7	D1	EQY	100.681
STOREY8	D1	EQY	105.2178

Graph 5.2 Variation of Lateral Displacement in Y– Direction Due To EQY



2) Storey Shear Results

Table 5.3 Storey Shear in X - direction for EQX Static Analysis

STOREY	LOAD	STOREY SHEAR IN X (IN KN)
STOREY1	EQX	**4884.69**
STOREY2	EQX	4850.64
STOREY3	EQX	4734.03
STOREY4	EQX	4484.94
STOREY5	EQX	4053.69
STOREY6	EQX	3390.6
STOREY7	EQX	2446
STOREY8	EQX	**1170.19**

Graph no. 5.3 Variation of Storey Shear in X - direction for EQX in Static Analysis

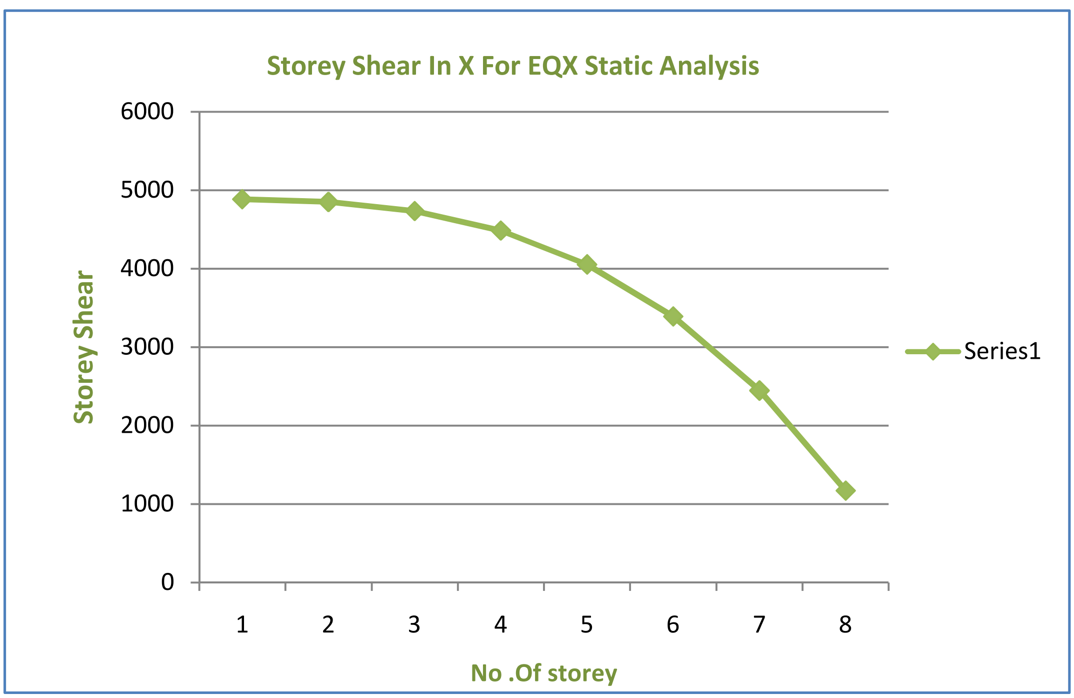


Table 5.4 Storey Shear in X -direction For Spec-1 in Dynamic Analysis

STOREY	LOAD	STOREY SHEAR IN X (IN KN)
STOREY1	SPEC1	1283.59
STOREY2	SPEC1	1235.29
STOREY3	SPEC1	1136.33
STOREY4	SPEC1	1012.43
STOREY5	SPEC1	879.79
STOREY6	SPEC1	722.76
STOREY7	SPEC1	511.4
STOREY8	SPEC1	233.62

Graph no. 5.4

Variation of Storey Shear in X - direction for Spec – 1 Dynamic Analysis

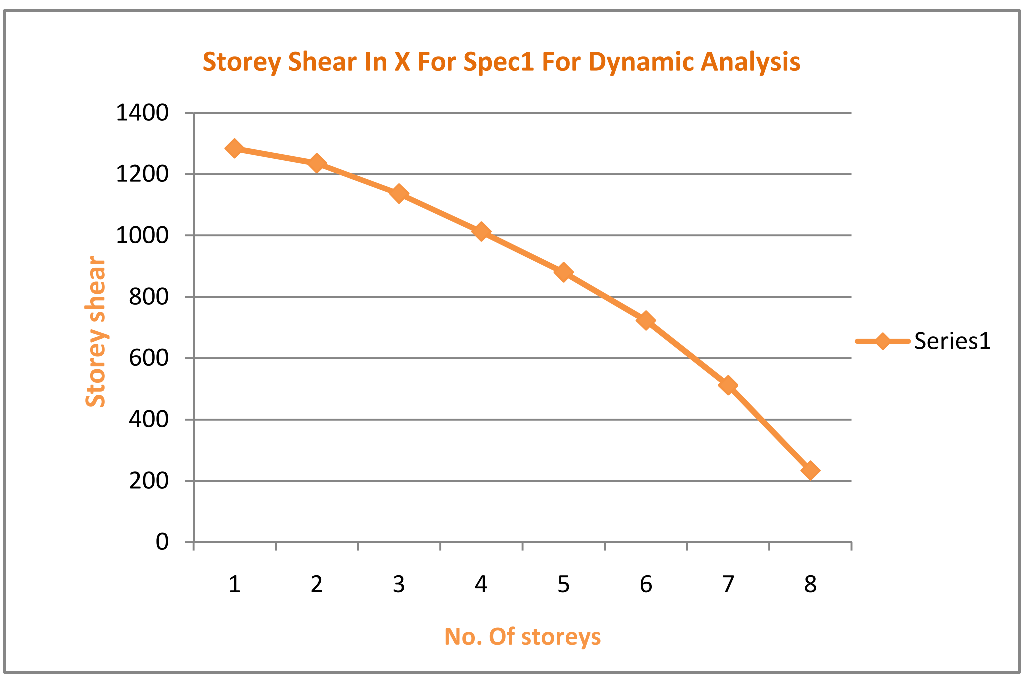


Table 5.5 Storey Shear in Y - direction for EQY in Static Analysis

STOREY	LOAD	STOREY SHEAR IN Y (IN KN)
STOREY1	EQY	4884.69
STOREY2	EQY	4850.64
STOREY3	EQY	4734.03
STOREY4	EQY	4484.94
STOREY5	EQY	4053.69
STOREY6	EQY	3390.6
STOREY7	EQY	2446
STOREY8	EQY	1170.19

Graph 5.5 Variation of Storey Shear in Y - direction for EQY in Static Analysis

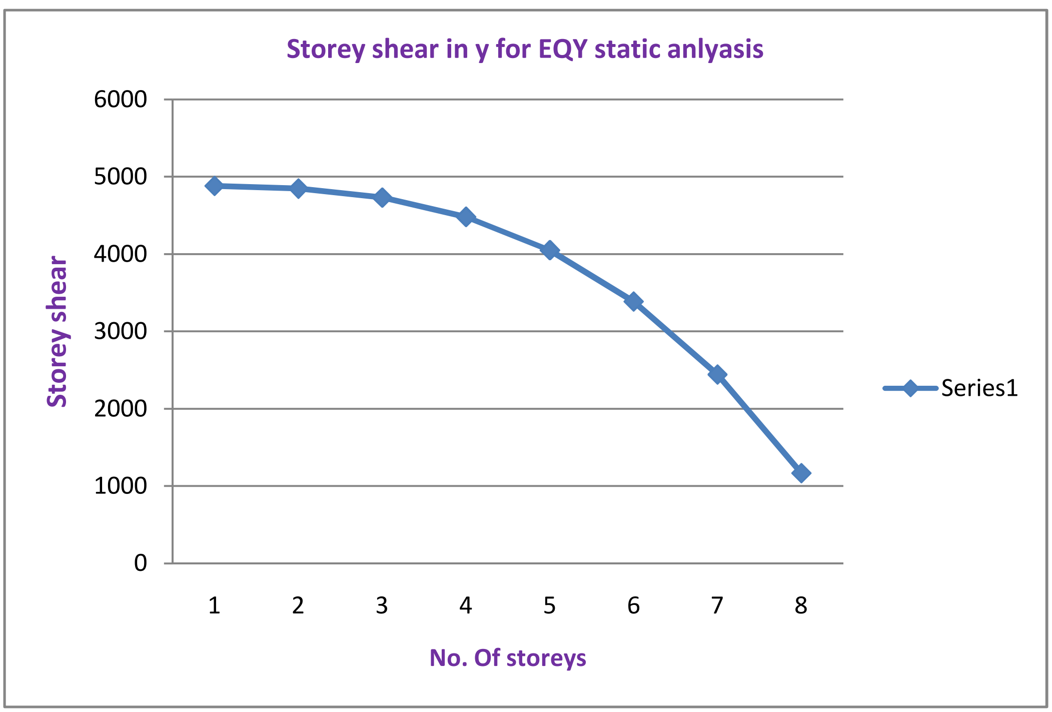
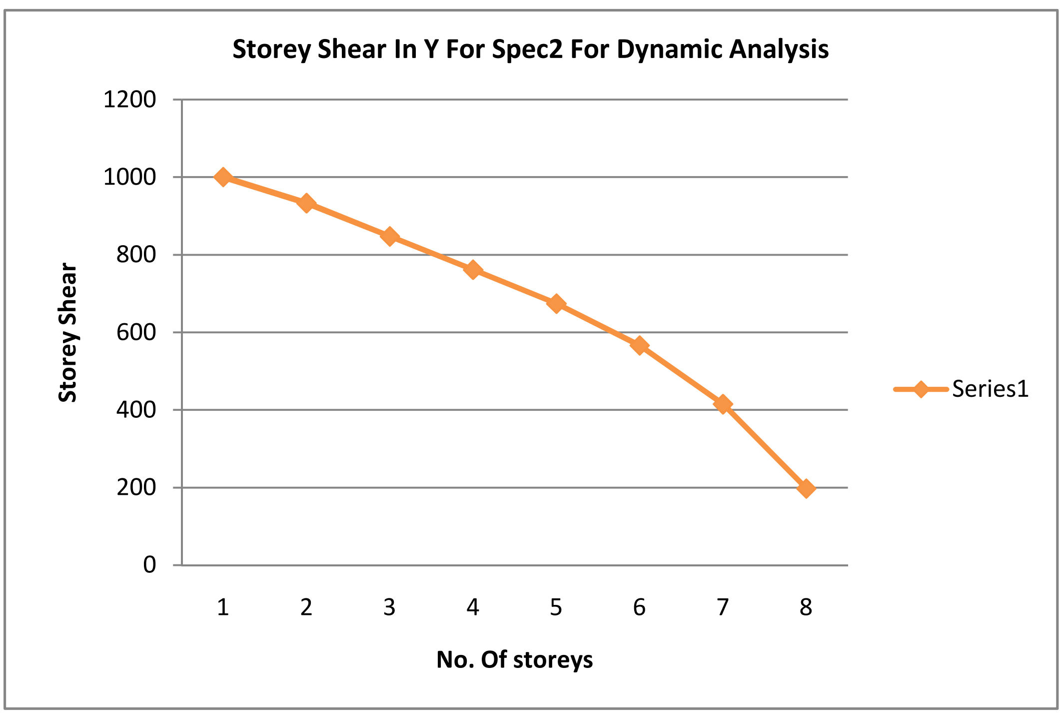


Table 5.6 Storey Shear in Y - direction Due to Spec– 2 in Dynamic Analysis

STOREY	LOAD	STOREY SHEAR IN Y (IN KN)
STOREY1	SPEC2	1000.39
STOREY2	SPEC2	933.15
STOREY3	SPEC2	847.42
STOREY4	SPEC2	761.49
STOREY5	SPEC2	674.27
STOREY6	SPEC2	566.42
STOREY7	SPEC2	415.26
STOREY8	SPEC2	197.86

Graph no. 5.6 Variation of Storey Shear in Y - direction for Spec – 2 in Dynamic Analysis



3) Natural Period Of Vibration For 8 Modes Of Model – 1

Table 5.7 modal information of model 1

Modal Information for MODEL 1						
Mode	Period	UX	UY	UZ	SUMUX	SUMUY
1	1.645377	0	85.6921	0	0	85.6921
2	1.227825	0	0	0	0	85.6921
3	1.191252	81.2602	0	0	81.2602	85.6921
4	0.541219	0	9.2094	0	81.2602	94.9016
5	0.390698	0	0	0	81.2602	94.9016
6	0.371192	10.4395	0	0	91.6997	94.9016
7	0.316476	0	2.9142	0	91.6997	97.8158
8	0.220531	0	1.2359	0	91.6997	99.0517

4) Column Forces At Storey 1 Due To EQX only (column no. 36)

Table 5.8 column forces in model 1

FOR MODEL 1									
Storey	Column	Load	Loc (in m)	P (in KN)	V2 (in KN)	V3 (in KN)	T (in KN-M)	M2 (in KN-M)	M3 (in KN-M)
STOREY1	C36	EQX	0	509.84	117.29	0	0	0	390.99
STOREY1	C36	EQX	1.5	509.84	117.29	0	0	0	215.061
STOREY1	C36	EQX	3	509.84	117.29	0	0	0	39.131
STOREY1	C36	EQY	0	585.83	0	117.14	0	254.18	0
STOREY1	C36	EQY	1.5	585.83	0	117.14	0	78.468	0
STOREY1	C36	EQY	3	585.83	0	117.14	0	-97.244	0

5) Beam Forces At Storey 8 Due To EQX Only (Beam Location As B 51)

Table 5.9 beam forces in model 1

Storey	Beam	Load	Loc	Model 1	
				SHEAR FORCE (V2) IN KN	MOMENT (M3) IN KN-M
STOREY8	B51	EQX	0.375	19.76	43.91
STOREY8	B51	EQX	0.847	19.76	34.577
STOREY8	B51	EQX	1.319	19.76	25.245
STOREY8	B51	EQX	1.792	19.76	15.913
STOREY8	B51	EQX	2.264	19.76	6.581
STOREY8	B51	EQX	2.736	19.76	-2.751
STOREY8	B51	EQX	3.208	19.76	-12.083
STOREY8	B51	EQX	3.681	19.76	-21.415
STOREY8	B51	EQX	4.153	19.76	-30.748
STOREY8	B51	EQX	4.625	19.76	-40.08

Likewise, we can obtain different results for different combination of load at different sections and locations of model – 1 structure.

5.5 PROBLEM 2: RC Framed Structure Of (G+8) Storey, Located In Seismic Zone IV On Medium Soil Strata. Material Properties, Geometrical Properties, And Assigning Of Loading is Same As of Problem – 1, Discussed Above. But In This Problem Addition Of External Shear Wall Of Wall Thickness 200mm Has Been Done In The Model Analysis.

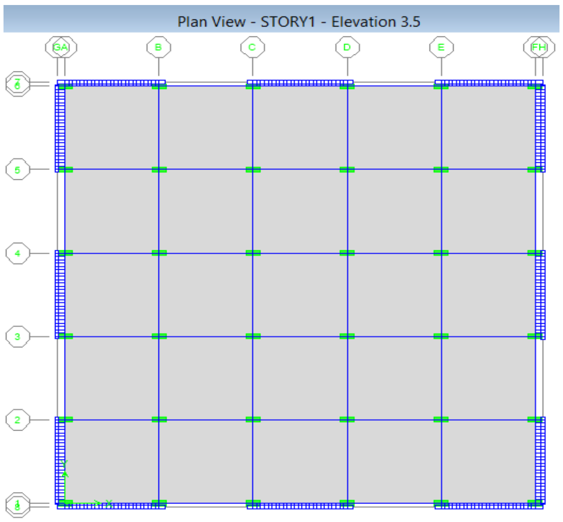


Fig.5 (Plan view of model – 2)

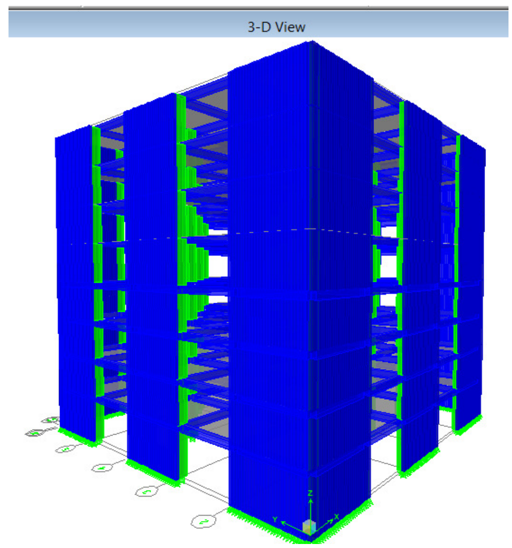


Fig.6 (3 – D view of model -2)

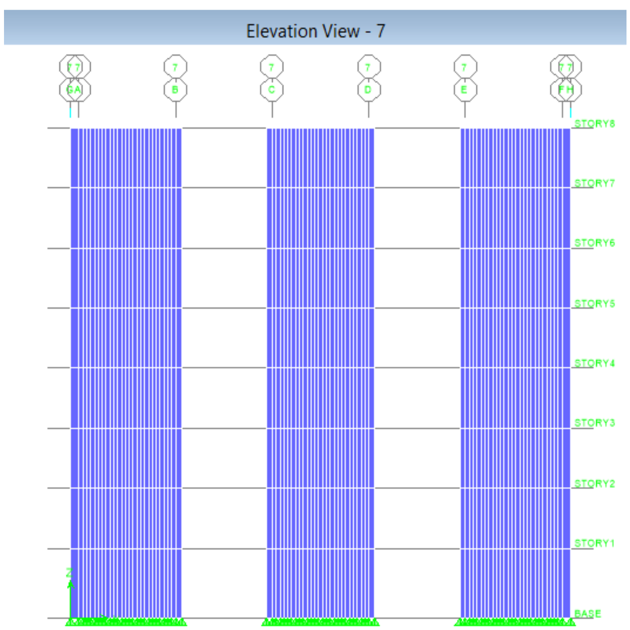


Fig.7 (Elevation View Of Model – 2)

After assigning different load combinations and shell area load on slab (that include the self weight that is dead load, live load) and frame line load is also assigned that has been mentioned above with detail.

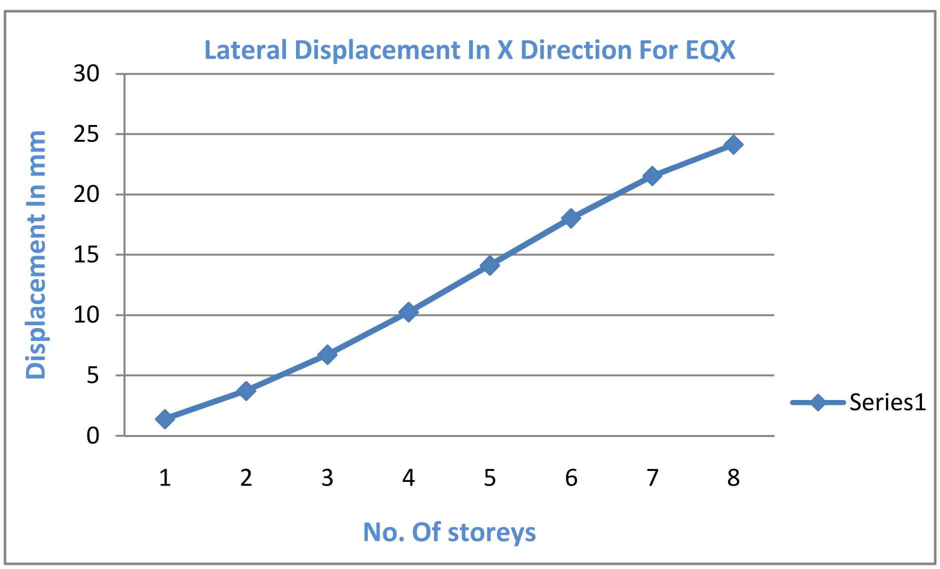
Run the analysis of the framed building has been done.

Details or results obtained as:

1) **Lateral Displcament Results : (In Both X And Y Direction Due To EQX And EQY)**

Table 5.10 Lateral displacement in x- direction for EQ-X at Different Storey Level

Storey	Diaphragm	Load	Lateral Displacement In X Model-2(mm)
STOREY1	D1	EQX	1.4041
STOREY2	D1	EQX	3.7323
STOREY3	D1	EQX	6.7365
STOREY4	D1	EQX	10.2651
STOREY5	D1	EQX	14.1309
STOREY6	D1	EQX	18.0414
STOREY7	D1	EQX	21.544
STOREY8	D1	EQX	24.1426

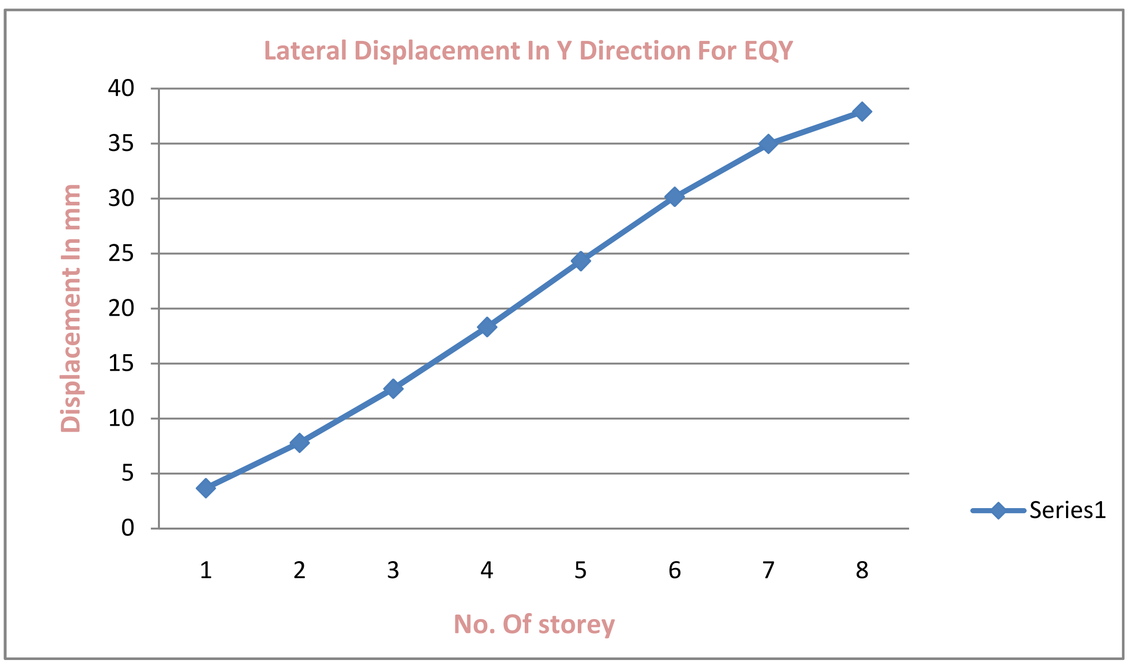


Graph 5.7 Variation of lateral displacement in x – direction due to EQX

Table 5.11 Lateral Displacement In Y- Direction For EQY At Different Storey Level

Storey	Diaphragm	Load	Lateral Displacement in Y
			Model 2
STOREY1	D1	EQY	3.6692
STOREY2	D1	EQY	7.7875
STOREY3	D1	EQY	12.7073
STOREY4	D1	EQY	18.32
STOREY5	D1	EQY	24.3169
STOREY6	D1	EQY	30.1529
STOREY7	D1	EQY	34.966
STOREY8	D1	EQY	37.8912

Graph 5.8 Variation of lateral displacement in y– direction due to EQY



2) Storey Shear Results

Table 5.12 Storey Shear In X - direction For EQX Static Analysis

Storey	Load	STOREY SHEAR IN X (IN KN)
		Model 2
STOREY1	EQX	**5563.5**
STOREY2	EQX	5524
STOREY3	EQX	5389.98
STOREY4	EQX	5103.7
STOREY5	EQX	4608.06
STOREY6	EQX	3845.96
STOREY7	EQX	2760.32
STOREY8	EQX	**1294.02**

Graph 5.9 Variation of Storey Shear In X - direction for EQX In Static Analysis

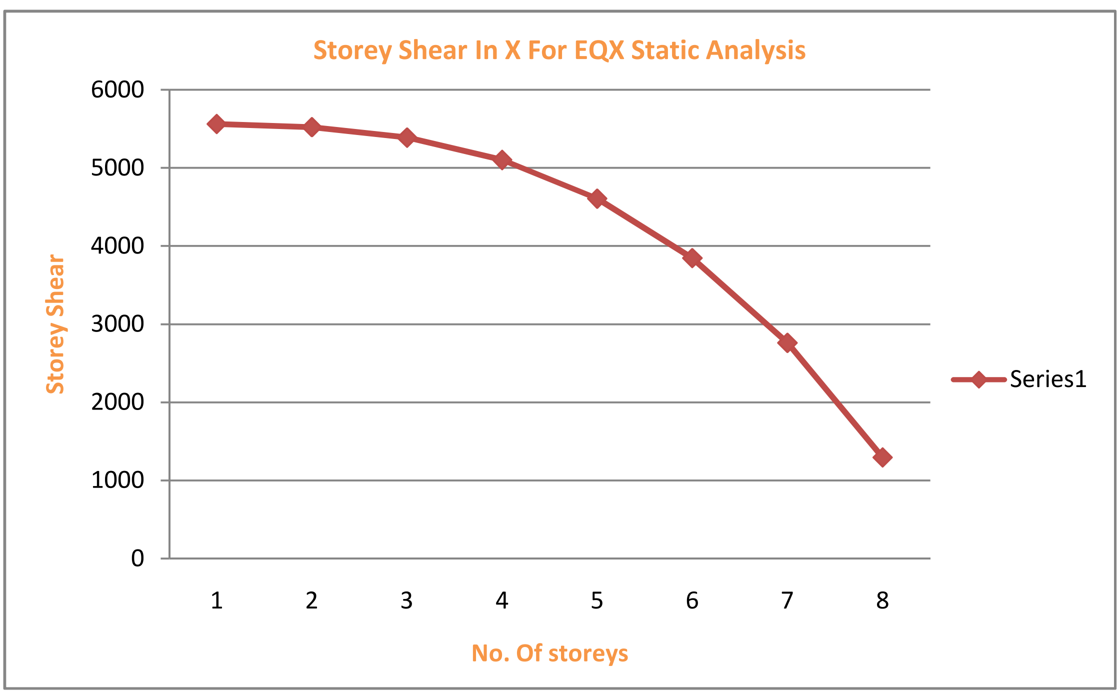


Table 5.13 Storey Shear in X - direction for Spec-1 Dynamic Analysis

Storey	Load	STOREY SHEAR IN X (IN KN)
		Model 2
STOREY1	SPEC1	2301.38
STOREY2	SPEC1	2235.08
STOREY3	SPEC1	2093.57
STOREY4	SPEC1	1898.09
STOREY5	SPEC1	1660.67
STOREY6	SPEC1	1358.2
STOREY7	SPEC1	951.22
STOREY8	SPEC1	424.43

Graph 5.10 Variation Of Storey Shear In X - direction for spec- 1 In dynamic Analysis

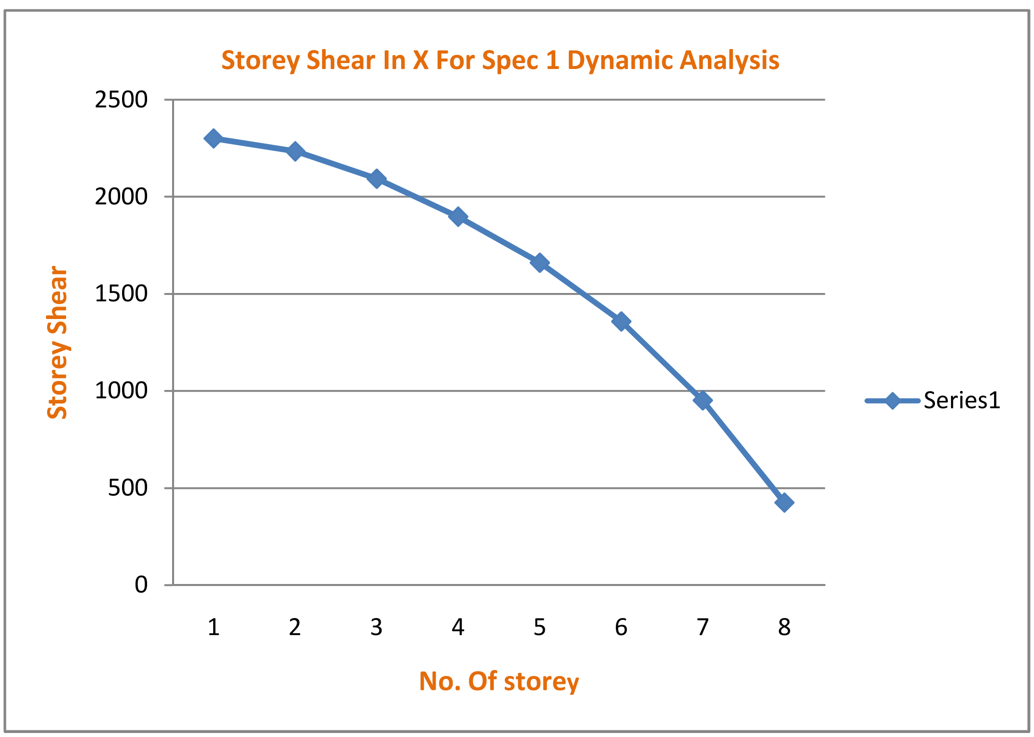


Table 5.14 Storey Shear In Y – direction For EQY Static Analysis

Storey	Load	STOREY SHEAR IN Y (IN KN)
		Model 2
STOREY1	EQY	5563.5
STOREY2	EQY	5524
STOREY3	EQY	5389.98
STOREY4	EQY	5103.7
STOREY5	EQY	4608.06
STOREY6	EQY	3845.96
STOREY7	EQY	2760.32
STOREY8	EQY	1294.02

Graph 5.11 Variation of Storey Shear In Y - Direction For EQY In Static Analysis

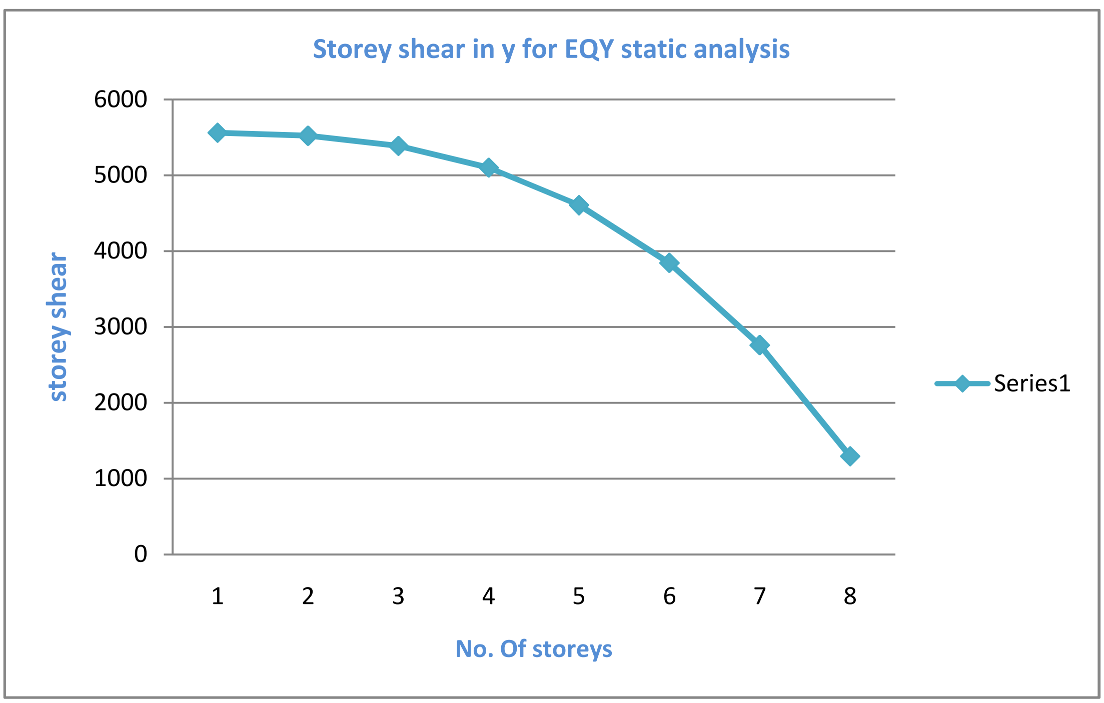
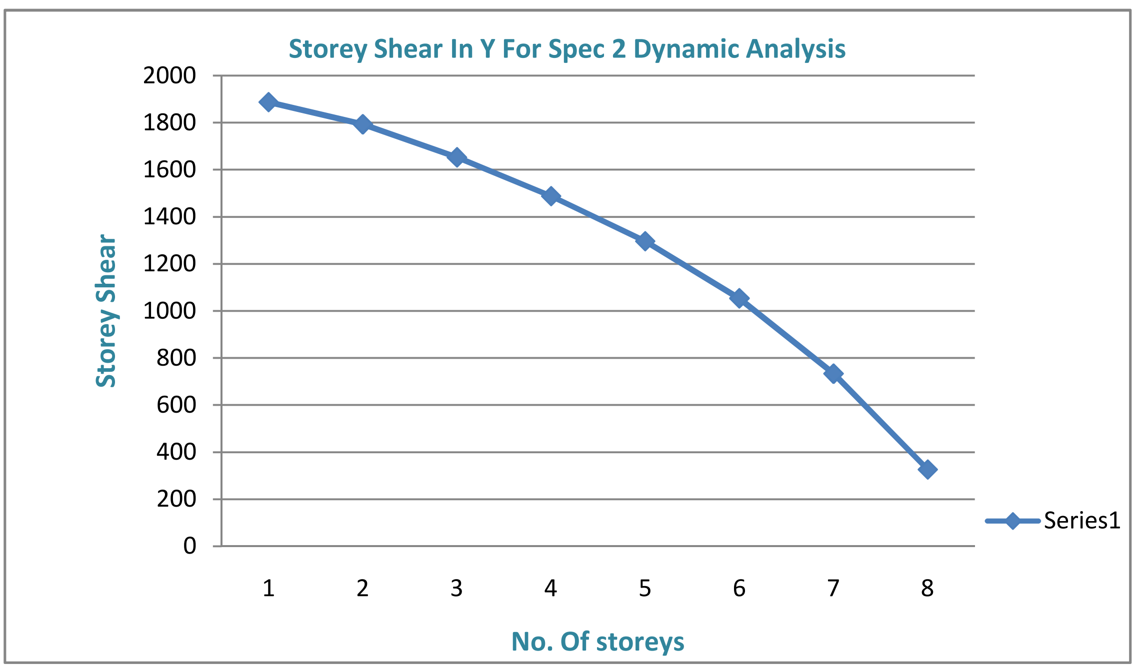


Table 5.15 Storey Shear In Y –direction For Spec-2 Dynamic Analysis

Storey	Load	STOREY SHEAR IN Y (in KN)
		Model 2
STOREY1	SPEC2	1887.58
STOREY2	SPEC2	1793.58
STOREY3	SPEC2	1653.11
STOREY4	SPEC2	1487.99
STOREY5	SPEC2	1296.66
STOREY6	SPEC2	1053.64
STOREY7	SPEC2	733.24
STOREY8	SPEC2	326.05

Graph 5.12Variation Of Storey Shear in Y - direction for spec- 2 In dynamic Analysis



3) Natural Period Of Vibration For 8 Modes Of Model – 2

Table 5.16 modal information of model 2

Modal Information for MODEL 2						
Mode	Period	UX	UY	UZ	Sum UX	Sum UY
1	0.912816	0	78.2485	0	0	78.2485
2	0.705213	72.2588	0	0	72.2588	78.2485
3	0.556027	0.008	0.0006	0	72.2668	78.2491
4	0.404047	0	15.0987	0	72.2668	93.3478
5	0.27753	0	2.5858	0	72.2668	95.9336
6	0.226068	18.2512	0	0	90.5181	95.9336
7	0.203614	0	1.1786	0	90.5181	97.1122
8	0.192209	0.3746	0.0004	0	90.8927	97.1126

4) Column Forces At Storey 1 Due To EQX only (column no. 36)

Table 5.17 column forces in model 2

FOR MODEL 2									
Storey	Column	Load	Loc	P	V2	V3	T	M2	M3
			(In M)	(In KN)	(In KN)	(In KN)	(In KN-M)	(In KN-M)	(In KN-M)
STOREY1	C36	EQX	0	217.25	22.81	0.16	-0.029	0.36	84.244
STOREY1	C36	EQX	1.5	217.25	22.81	0.16	-0.029	0.114	50.034
STOREY1	C36	EQX	3	217.25	22.81	0.16	-0.029	-0.131	15.824
STOREY1	C36	EQY	0	218.91	-0.24	23.86	-0.013	52.884	-0.709
STOREY1	C36	EQY	1.5	218.91	-0.24	23.86	-0.013	17.099	-0.343
STOREY1	C36	EQY	3	218.91	-0.24	23.86	-0.013	-18.686	0.024

5) **Beam Forces At Storey 8 Due To EQX Only (Beam Location As B51)**

Table 5.18 beam forces in model 2

Storey	Beam	Load	Loc	Model 2	
				SHEAR FORCE (V2) IN KN	MOMENT (M3) IN KN-M
STOREY8	B51	EQX	0.375	17.03	37.711
STOREY8	B51	EQX	0.847	17.03	29.67
STOREY8	B51	EQX	1.319	17.03	21.628
STOREY8	B51	EQX	1.792	17.03	13.587
STOREY8	B51	EQX	2.264	17.03	5.545
STOREY8	B51	EQX	2.736	17.03	-2.496
STOREY8	B51	EQX	3.208	17.03	-10.538
STOREY8	B51	EQX	3.681	17.03	-18.579
STOREY8	B51	EQX	4.153	17.03	-26.621
STOREY8	B51	EQX	4.625	17.03	-34.662

5.6 <u>PROBLEM</u> :3 RC Framed Structure Of (G+8) Storey, External Shear Wall Of 200mm Thickness With Opening Provided At Each Storey Level, Material Properties, Geometrical Properties And Assigning Of Load Is Same As That Of Problem- 2 Stated Above.

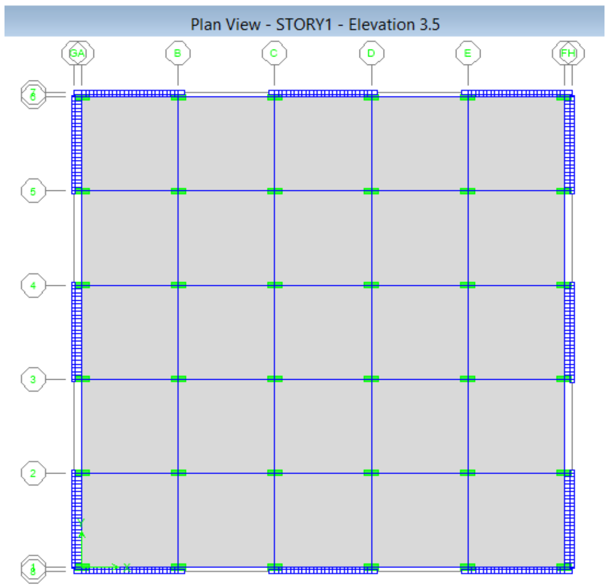


Fig.8 Plan view of model – 3

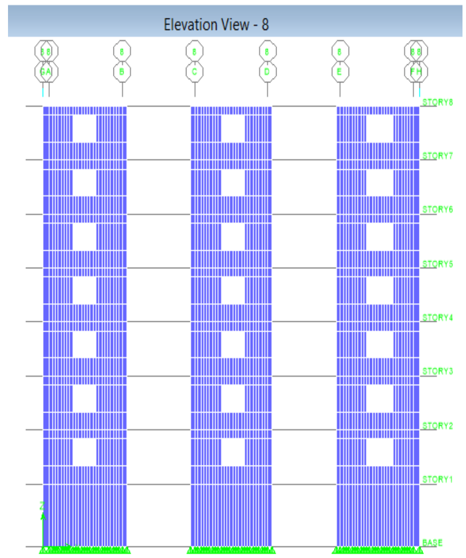


Fig.9 Elevation View of Model - 3

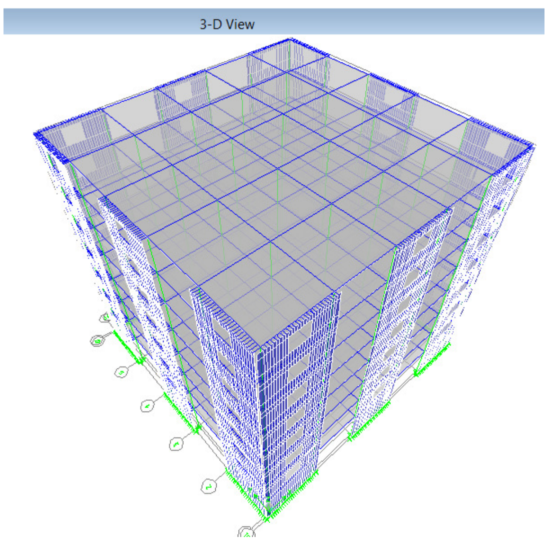


Fig.10 (3 – D view of model – 3)

Details or results obtained as:

1) Lateral Displacement Results

Table 5.19 Lateral displacement in x-direction for EQX At Different Storey Level

Storey	Diaphragm	Load	Lateral Displacement in X
			Model 3
STOREY1	D1	EQX	1.4548
STOREY2	D1	EQX	3.8882
STOREY3	D1	EQX	7.0212
STOREY4	D1	EQX	10.6624
STOREY5	D1	EQX	14.6027
STOREY6	D1	EQX	18.544
STOREY7	D1	EQX	22.0417
STOREY8	D1	EQX	24.6171

Graph 5.13 variation of Lateral Displacement in X - Direction for EQX

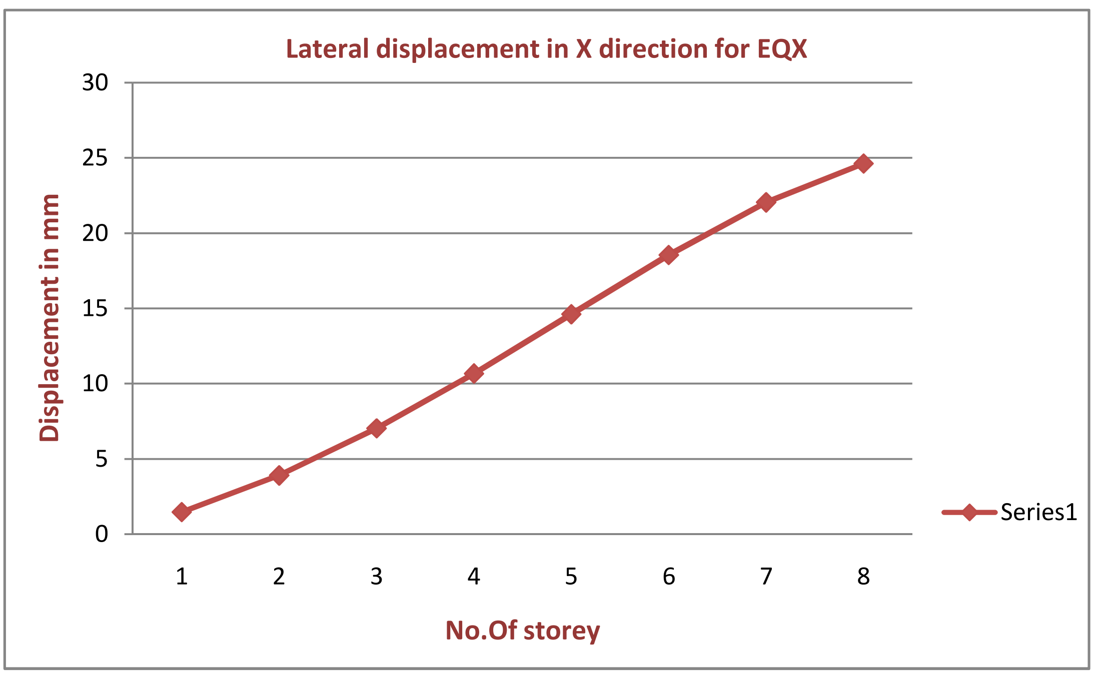
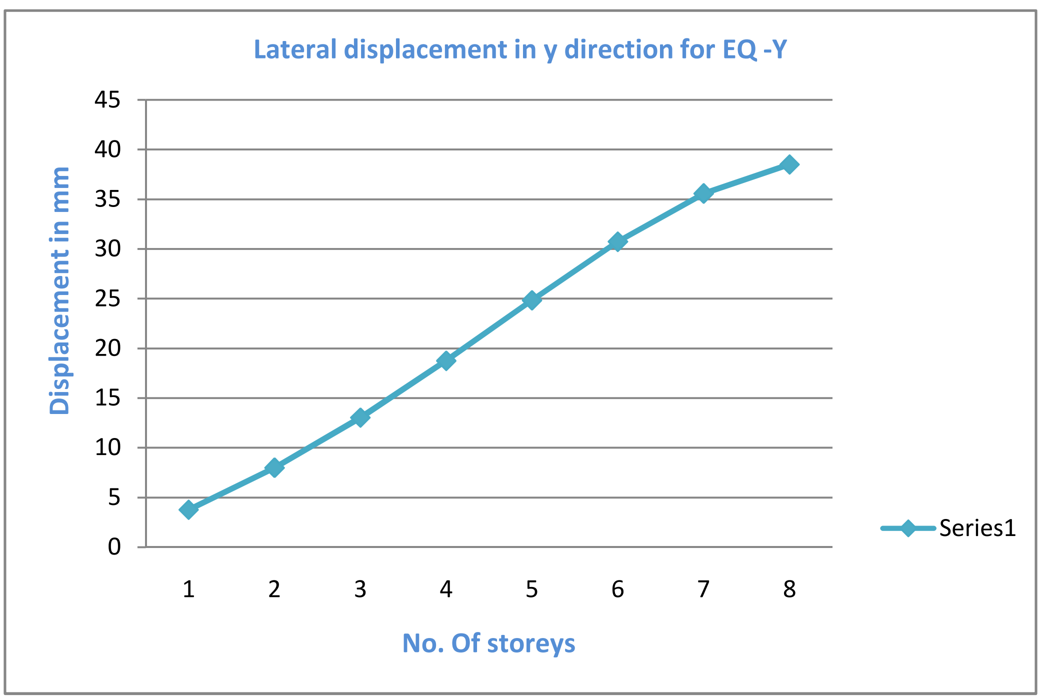


Table 5.20 Lateral Displacement in Y - Direction for EQY

Storey	Diaphragm	Load	Lateral Displacement in Y
			Model 3
STOREY1	D1	EQY	3.7517
STOREY2	D1	EQY	7.9747
STOREY3	D1	EQY	13.0192
STOREY4	D1	EQY	18.7464
STOREY5	D1	EQY	24.8347
STOREY6	D1	EQY	30.7361
STOREY7	D1	EQY	35.5902
STOREY8	D1	EQY	38.5071

Graph 14 Variation of Lateral Displacement in Y – Direction Due To EQY



2) Storey Shear Results

Table 5.21 Storey shear in x - direction due to EQX for static analysis

Storey	Load	STOREY SHEAR IN X (in KN)
		Model 3
STOREY1	EQX	5472.1
STOREY2	EQX	5432.87
STOREY3	EQX	5301.2
STOREY4	EQX	5019.98
STOREY5	EQX	4533.09
STOREY6	EQX	3784.46
STOREY7	EQX	2718.16
STOREY8	EQX	1278.08

Graph 15 Variation Of Storey Shear in X - direction for Different Storey Due To EQX

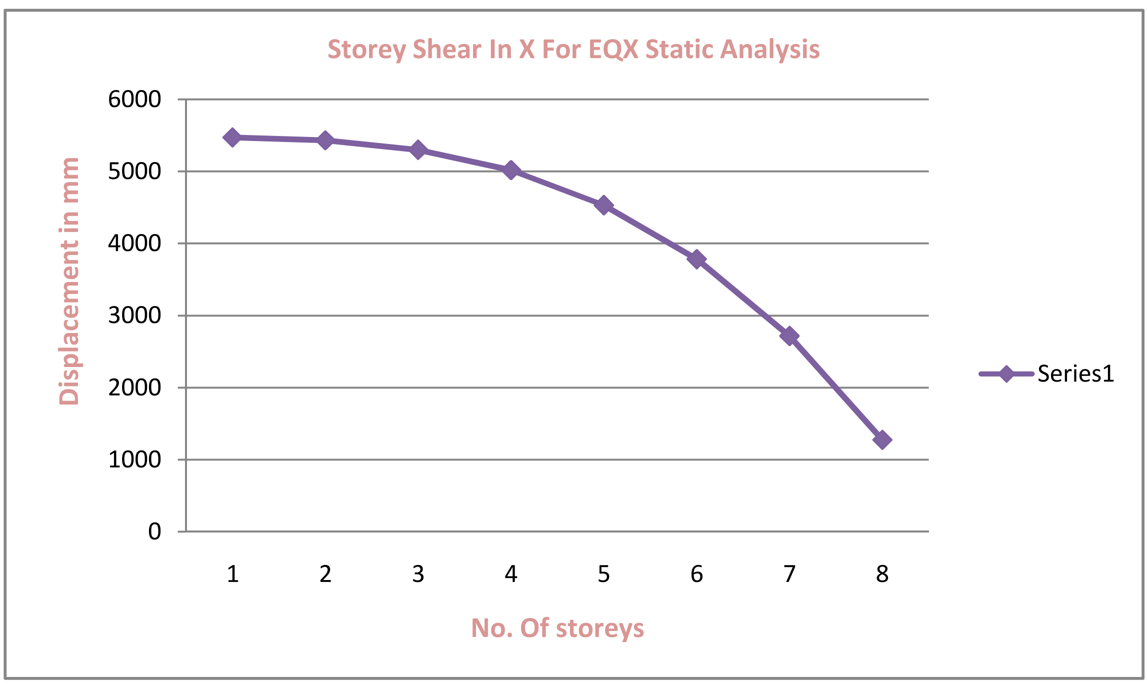


Table 5.22 Storey shear in x - direction due to spec – 1 for dynamic analysis

Storey	Load	STOREY SHEAR IN X (in KN)
		Model 3
STOREY1	SPEC1	2237.12
STOREY2	SPEC1	2173.18
STOREY3	SPEC1	2035.61
STOREY4	SPEC1	1843.83
STOREY5	SPEC1	1611.06
STOREY6	SPEC1	1316.55
STOREY7	SPEC1	922.31
STOREY8	SPEC1	412.71

Graph 16 Variation Of Storey Shear in X -direction Due to Spec – 1 In Dynamic Analysis

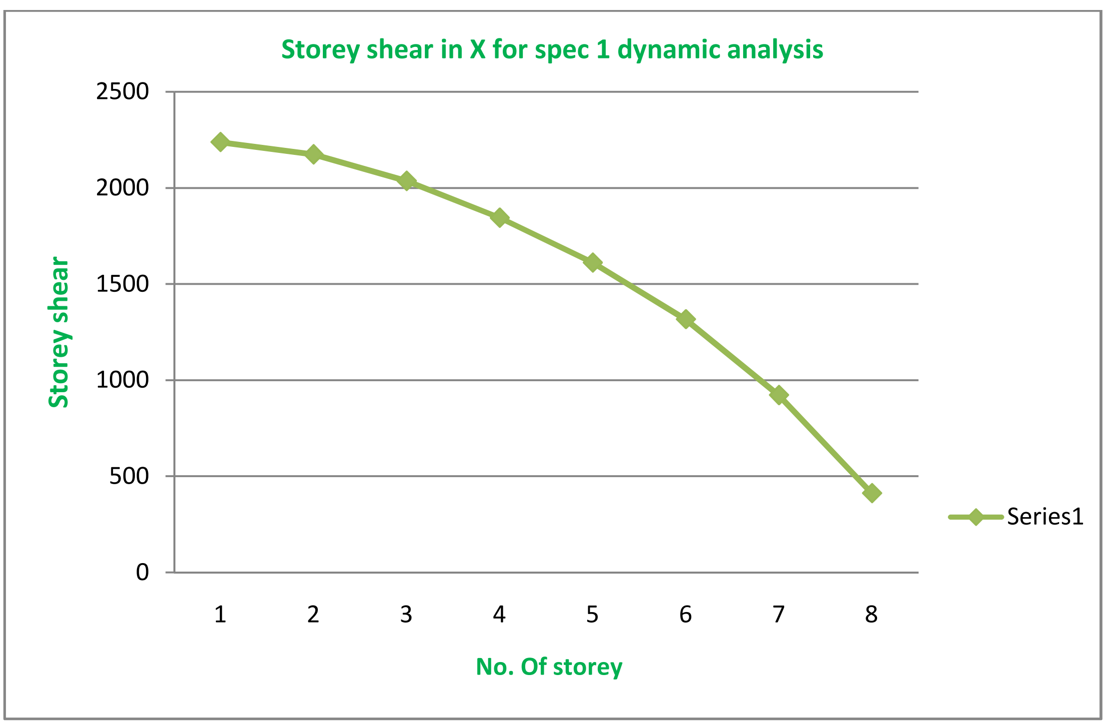


Table 5.23 Storey Shear in Y - direction Due to EQY or Static Analysis

Storey	Load	STOREY SHEAR IN Y (IN KN)
		Model 3
STOREY1	EQY	5472.1
STOREY2	EQY	5432.87
STOREY3	EQY	5301.2
STOREY4	EQY	5019.98
STOREY5	EQY	4533.09
STOREY6	EQY	3784.46
STOREY7	EQY	2718.16
STOREY8	EQY	1278.08

Graph 5.17 Variation Of Storey Shear In Y - direction Due To EQY In Static Analysis

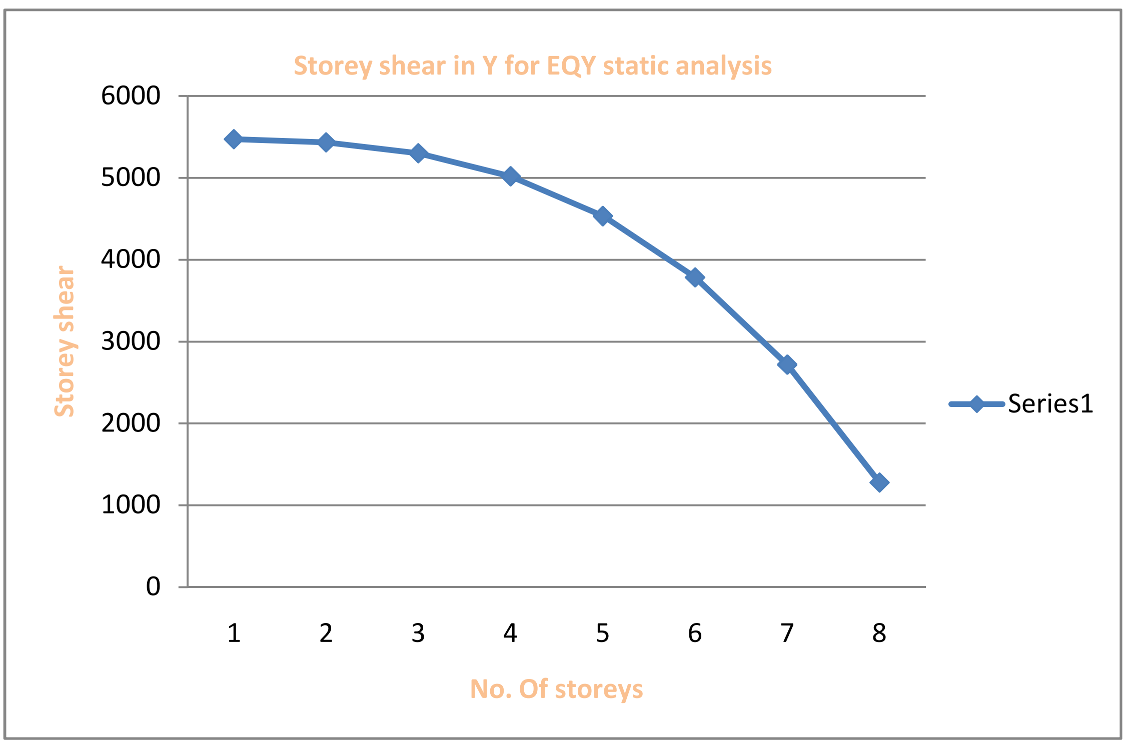
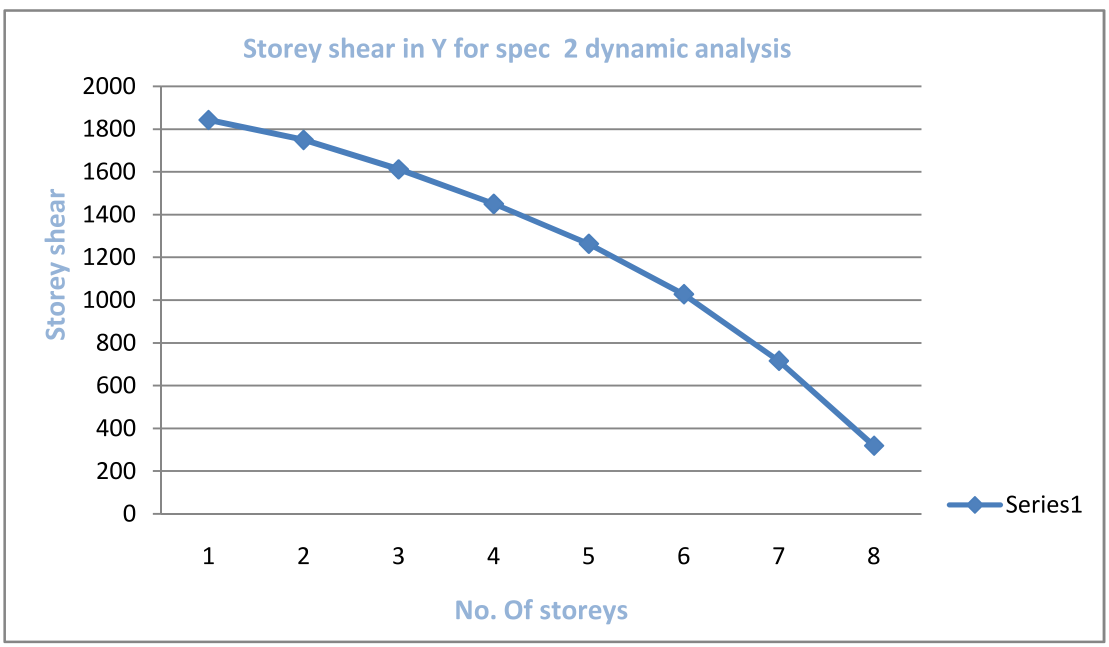


Table 5.24 Storey Shear in Y -direction Due To Spec – 2 In Dynamic Analysis

Storey	Load	STOREY SHEAR IN Y (in KN)
		Model 3
STOREY1	SPEC2	**1842.22**
STOREY2	SPEC2	1749.13
STOREY3	SPEC2	1611.27
STOREY4	SPEC2	1449.79
STOREY5	SPEC2	1263.55
STOREY6	SPEC2	1027.6
STOREY7	SPEC2	716.3
STOREY8	SPEC2	**319.49**

Graph 5.18 Variation of Storey Shear in Y - direction Due To Spec – 2 in Dynamic Analysis



3) Natural Period Of Vibration For 8 Modes Of Model – 3

Table 5.25 modal information for model 3

Modal Information for MODEL 3						
Mode	Period	UX	UY	UZ	Sum UX	Sum UY
1	0.922131	0	78.3849	0	0	78.3849
2	0.714817	72.6453	0	0	72.6453	78.3849
3	0.562441	0.0025	0.0002	0	72.6478	78.3851
4	0.404308	0	15.1176	0	72.6478	93.5027
5	0.277816	0	2.6548	0	72.6478	96.1575
6	0.229986	17.7742	0	0	90.422	96.1576
7	0.204396	0	1.1975	0	90.422	97.355
8	0.195835	0.2716	0.0004	0	90.6936	97.3554

4) Column Forces At Storey 1 Due To EQX only (column no. 36)

Table 5.26 column forces for model 3

FOR MODEL 3									
Storey	Column	Load	Loc	P	V2	V3	T	M2	M3
			(in m)	(in KN)	(in KN)	(in KN)	(in KN-M)	(in KN-M)	(in KN-M)
STOREY1	C36	EQX	0	221.41	23.22	0.17	-0.029	0.368	86.885
STOREY1	C36	EQX	1.5	221.41	23.22	0.17	-0.029	0.108	52.06
STOREY1	C36	EQX	3	221.41	23.22	0.17	-0.029	-0.152	17.235
STOREY1	C36	EQY	0	222.44	-0.24	24.36	-0.012	54.035	-0.683
STOREY1	C36	EQY	1.5	222.44	-0.24	24.36	-0.012	17.488	-0.322
STOREY1	C36	EQY	3	222.44	-0.24	24.36	-0.012	-19.058	0.04

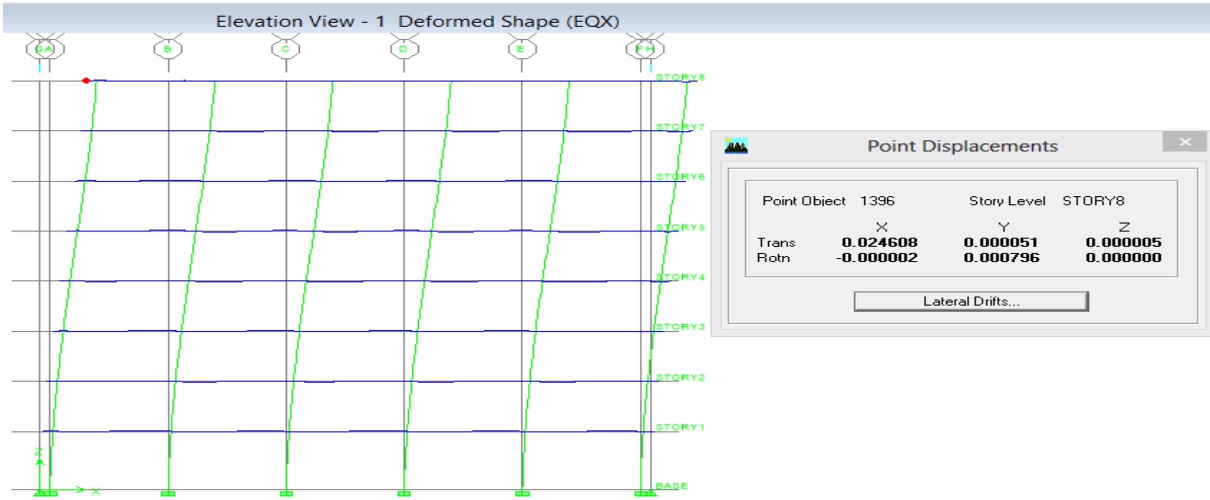


Fig. 11 Deformed shape due to EQX load

5) **Beam Forces At Storey 8 Due To EQX Only (Beam No. 51)**

Table 5.27 beam forces for model 3

Storey	Beam	Load	Loc	Model 3	
				SHEAR FORCE (V2) IN KN	MOMENT (M3) IN KN-M
STOREY8	B51	EQX	0.375	16.79	37.18
STOREY8	B51	EQX	0.847	16.79	29.252
STOREY8	B51	EQX	1.319	16.79	21.325
STOREY8	B51	EQX	1.792	16.79	13.398
STOREY8	B51	EQX	2.264	16.79	5.47
STOREY8	B51	EQX	2.736	16.79	-2.457
STOREY8	B51	EQX	3.208	16.79	-10.384
STOREY8	B51	EQX	3.681	16.79	-18.312
STOREY8	B51	EQX	4.153	16.79	-26.239
STOREY8	B51	EQX	4.625	16.79	-34.166

Chapter 6

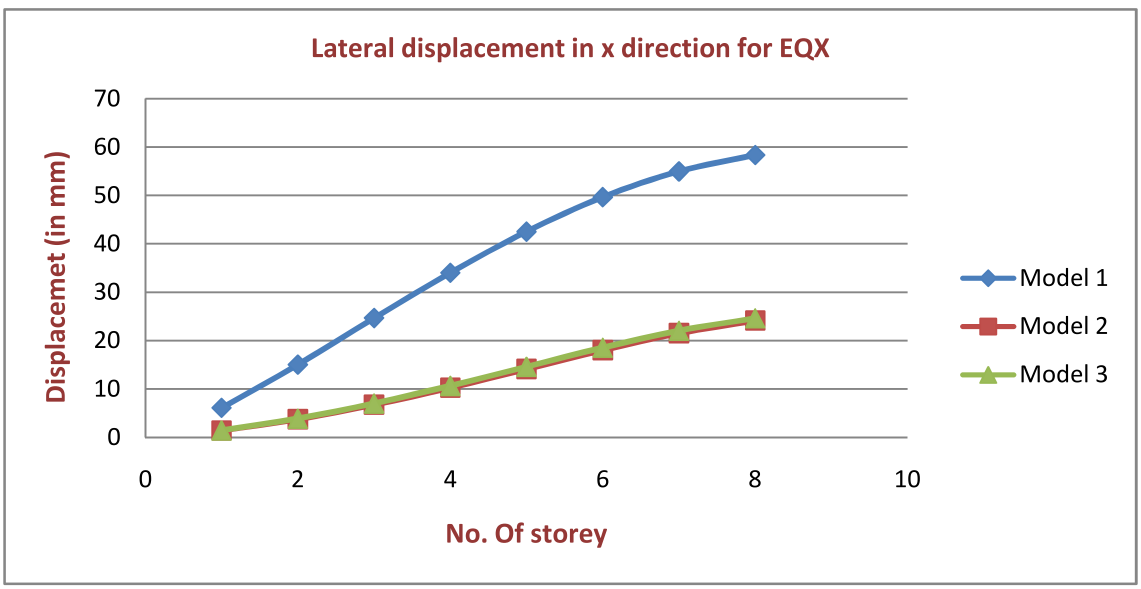
6.1 RESULT COMPARISIONS AND DISCUSSIONS

1) Lateral Displacement Results Of All Three Models Are Compared And Discussed.

A) Lateral Displacement in X - Direction Due To EQX

TABLE 6.1

Storey	Diaphragm	Load	Lateral Displacement in X		
			Model 1	Model 2	Model 3
STOREY1	D1	EQX	6.1141	1.4041	1.4548
STOREY2	D1	EQX	15.0525	3.7323	3.8882
STOREY3	D1	EQX	24.6823	6.7365	7.0212
STOREY4	D1	EQX	34.0455	10.2651	10.6624
STOREY5	D1	EQX	42.551	14.1309	14.6027
STOREY6	D1	EQX	49.6861	18.0414	18.544
STOREY7	D1	EQX	55.0056	21.544	22.0417
STOREY8	D1	EQX	58.3906	24.1426	24.6171



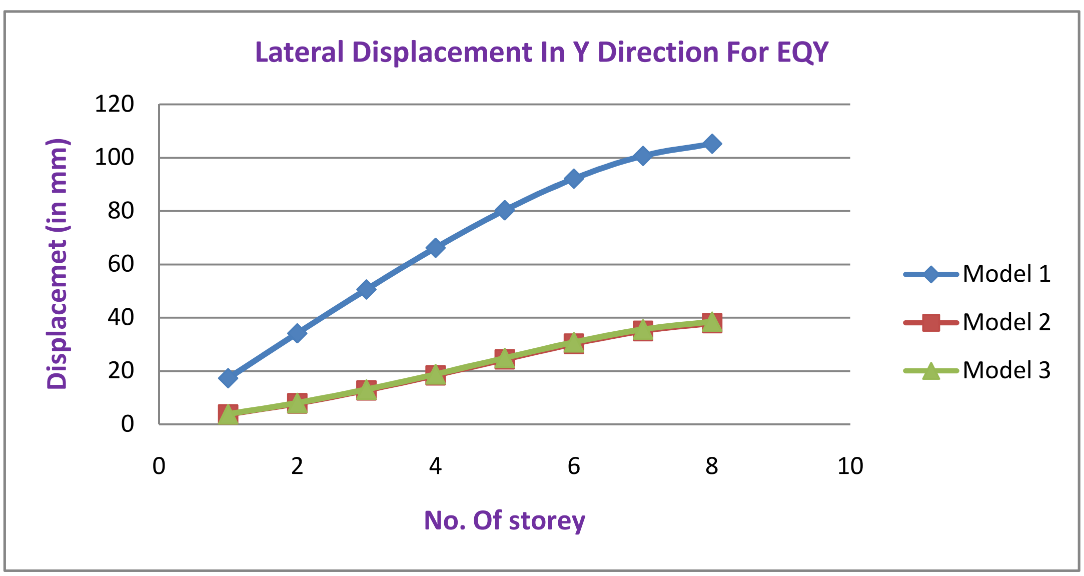
Graph 6.1 variation of lateral displacement in X of all models

Here we can say that by providing external shear wall to the existing building effectively and efficiently we can reduce the lateral displacement of a building up to a reasonable extent, which is in the permissible range which our Indian codes recommends. In above models by providing external shear wall lateral displacement has been reduced to more the 50% which ultimately makes building strong from excessive vibrations.

B) <u>Lateral Displacement In Y - Direction Due To EQY</u>

<u>TABLE 6.2</u>

Storey	Diaphragm	Load	Lateral Displacement in Y		
			Model 1	Model 2	Model 3
STOREY1	D1	EQY	17.2714	3.6692	3.7517
STOREY2	D1	EQY	34.1038	7.7875	7.9747
STOREY3	D1	EQY	50.5584	12.7073	13.0192
STOREY4	D1	EQY	66.1714	18.32	18.7464
STOREY5	D1	EQY	80.2932	24.3169	24.8347
STOREY6	D1	EQY	92.1144	30.1529	30.7361
STOREY7	D1	EQY	100.681	34.966	35.5902
STOREY8	D1	EQY	105.2178	37.8912	38.5071



<u>Graph 6.2 variation of lateral displacement in Y for all models</u>

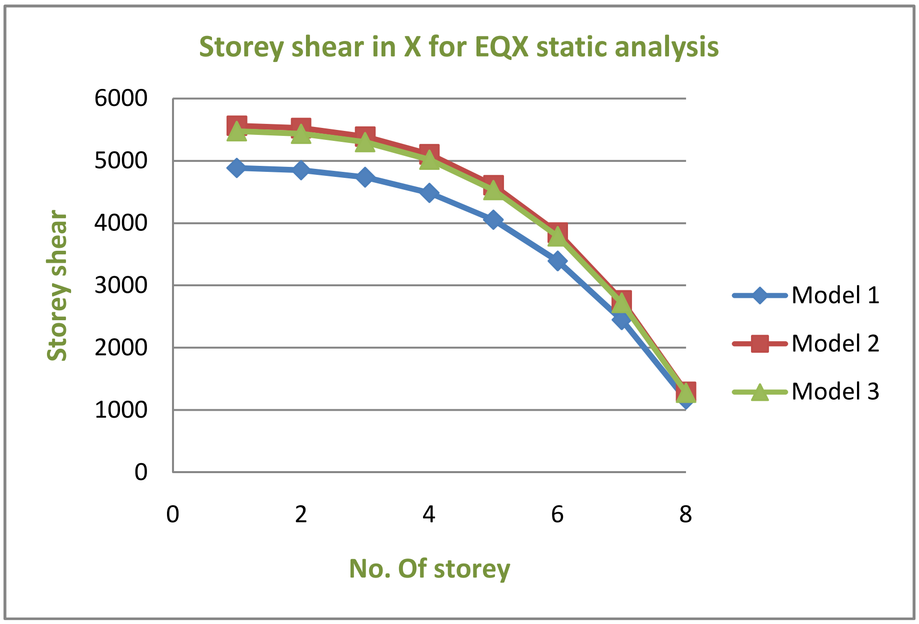
Here also due to external shear wall lateral displacement in Y direction has been also reduced to more than 50%. The external shear wall provides stiffness to the existing building frame ultimately helps in reducing the displacement of Storey.

2) Storey Shear Results

A)Base shear values of all model in x - direction due to EQX in static analysis

TABLE 6.3

Storey	Load	STOREY SHEAR IN X (in KN)		
		Model 1	Model 2	Model 3
STOREY1	EQX	4884.69	5563.5	5472.1
STOREY2	EQX	4850.64	5524	5432.87
STOREY3	EQX	4734.03	5389.98	5301.2
STOREY4	EQX	4484.94	5103.7	5019.98
STOREY5	EQX	4053.69	4608.06	4533.09
STOREY6	EQX	3390.6	3845.96	3784.46
STOREY7	EQX	2446	2760.32	2718.16
STOREY8	EQX	1170.19	1294.02	1278.08



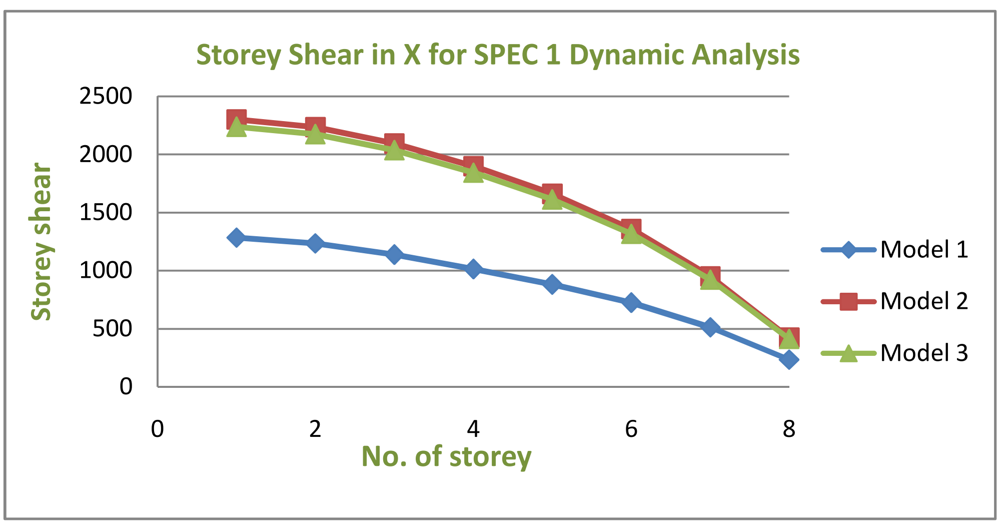
Graph 6.3 variation of storey shear in x- direction of all models

From the above table and graph we can say that by providing external shear wall to the existing building framed structure in both direction symmetrically, we have increased the mass or weight of the building as a whole to act as a monolithic structure during earthquake vibration, so we find that our base shear value at each floor level has been increased to a minor extent but this little increase in mass has reduced our displacement value to much lesser extent. Further in model -3 base shear values again has decreased because by providing opening in the shear wall we have again reduced the mass or weight of the structure as a whole.

B) Base Shear Value In X - direction Due To Spec-1 In Dynamic Analysis

TABLE 6.4

Storey	Load	STOREY SHEAR IN X (in KN)		
		Model 1	Model 2	Model 3
STOREY1	SPEC1	**1283.59**	**2301.38**	**2237.12**
STOREY2	SPEC1	1235.29	2235.08	2173.18
STOREY3	SPEC1	1136.33	2093.57	2035.61
STOREY4	SPEC1	1012.43	1898.09	1843.83
STOREY5	SPEC1	879.79	1660.67	1611.06
STOREY6	SPEC1	722.76	1358.2	1316.55
STOREY7	SPEC1	511.4	951.22	922.31
STOREY8	SPEC1	**233.62**	**424.43**	**412.71**



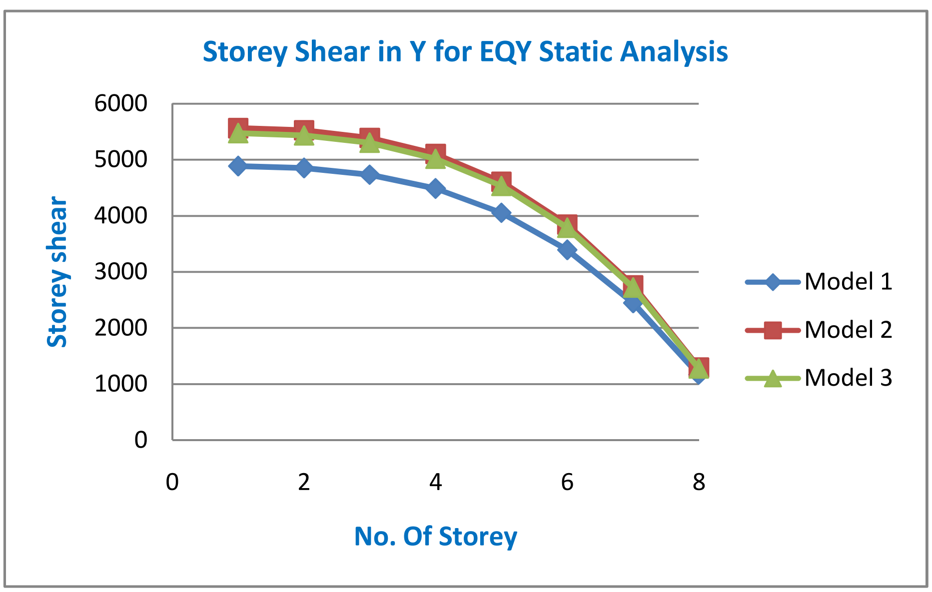
Graph 6.4 variation of storey shear in X direction due to spec – 1

From the above table and graph, we can say that base shear value of spec – 1 case, is less than for EQX case as model – 1 has no external shear wall connection so its time period for different modes is comparatively high as compared to model -2 and model – 3, thus model – 1 has lesser stiffness than other models so it has higher time period. Due to which base shear value comes out to less. But as on the way stiffness of structures when increased in model – 2, and model – 3, time period of different modes reduced and thus we get a higher value of base shear for them.

C) **Base Shear In Y - direction Due To EQY In Static Analysis**

TABLE 6.5

Storey	Load	STOREY SHEAR IN Y (in KN)		
		Model 1	Model 2	Model 3
STOREY1	EQY	4884.69	5563.5	5472.1
STOREY2	EQY	4850.64	5524	5432.87
STOREY3	EQY	4734.03	5389.98	5301.2
STOREY4	EQY	4484.94	5103.7	5019.98
STOREY5	EQY	4053.69	4608.06	4533.09
STOREY6	EQY	3390.6	3845.96	3784.46
STOREY7	EQY	2446	2760.32	2718.16
STOREY8	EQY	1170.19	1294.02	1278.08

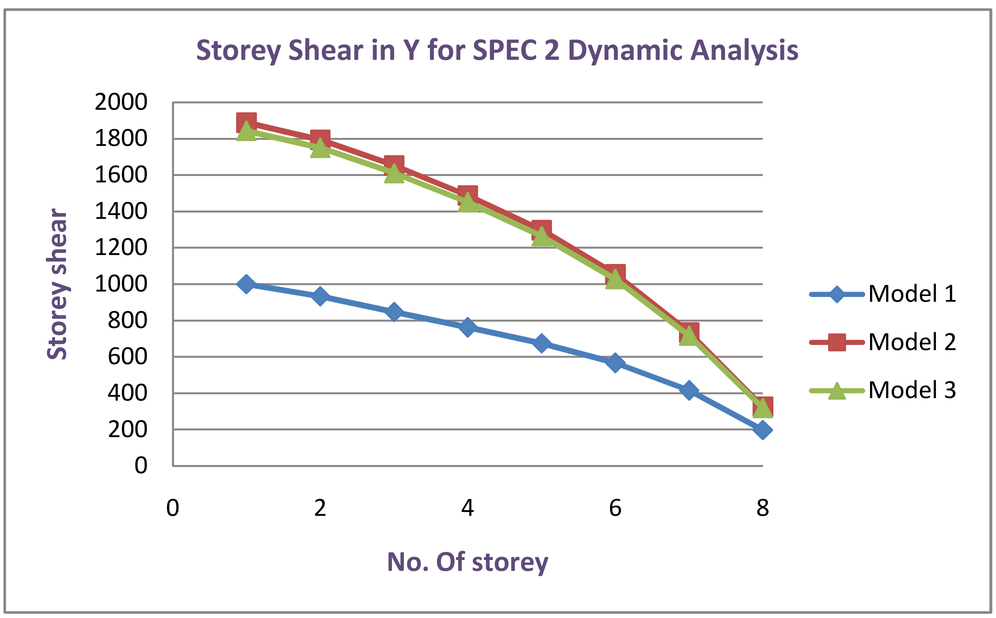


Graph 6.5 variation of storey shear in Y direction for EQY static analysis for all models

D) **Base Shear In Y - direction Due To Spec – 2 Defined In Dynamic Analysis**

TABLE 6.6

Storey	Load	STOREY SHEAR IN Y (in KN)		
		Model 1	Model 2	Model 3
STOREY1	SPEC2	1000.39	1887.58	1842.22
STOREY2	SPEC2	933.15	1793.58	1749.13
STOREY3	SPEC2	847.42	1653.11	1611.27
STOREY4	SPEC2	761.49	1487.99	1449.79
STOREY5	SPEC2	674.27	1296.66	1263.55
STOREY6	SPEC2	566.42	1053.64	1027.6
STOREY7	SPEC2	415.26	733.24	716.3
STOREY8	SPEC2	197.86	326.05	319.49



Graph 6.6 variation of storey shear in Y direction for spec-2 for all models

3) Natural Period Of Vibration For Different Modes

TABLE 6.7

Modal Information for MODEL 1						
Mode	Period	UX	UY	UZ	SUMUX	SUMUY
1	**1.645377**	0	85.6921	0	0	85.6921
2	1.227825	0	0	0	0	85.6921
3	1.191252	81.2602	0	0	81.2602	85.6921
4	0.541219	0	9.2094	0	81.2602	94.9016
5	0.390698	0	0	0	81.2602	94.9016
6	0.371192	10.4395	0	0	91.6997	94.9016
7	0.316476	0	2.9142	0	91.6997	97.8158
8	0.220531	0	1.2359	0	91.6997	99.0517

Modal Information for MODEL 2						
Mode	Period	UX	UY	UZ	SUMUX	SUMUY
1	**0.912816**	0	78.2485	0	0	78.2485
2	0.705213	72.2588	0	0	72.2588	78.2485
3	0.556027	0.008	0.0006	0	72.2668	78.2491
4	0.404047	0	15.0987	0	72.2668	93.3478
5	0.27753	0	2.5858	0	72.2668	95.9336
6	0.226068	18.2512	0	0	90.5181	95.9336
7	0.203614	0	1.1786	0	90.5181	97.1122
8	0.192209	0.3746	0.0004	0	90.8927	97.1126

Modal Information for MODEL 3						
Mode	Period	UX	UY	UZ	SUMUX	SUMUY
1	**0.922131**	0	78.3849	0	0	78.3849
2	0.714817	72.6453	0	0	72.6453	78.3849
3	0.562441	0.0025	0.0002	0	72.6478	78.3851
4	0.404308	0	15.1176	0	72.6478	93.5027
5	0.277816	0	2.6548	0	72.6478	96.1575
6	0.229986	17.7742	0	0	90.422	96.1576
7	0.204396	0	1.1975	0	90.422	97.355
8	0.195835	0.2716	0.0004	0	90.6936	97.3554

From above tables we can confirm that as less stiffness structures or rather flexibility have more time period for different modes, as this can be seen from table of model-1.

As soon as the stiffness is provided to the structure with the help of external shear wall, now time period of different modes comes out to be lesser than that of model – 1.

We can see from table of modal information that for 1st mode time period of model- 1 is 1.645377 seconds and for model – 2 for the same mode time period is 0.912816 seconds. but again in model – 3 time period of mode 1 is increased a little bit to the value 0.922131 seconds, because by providing opening in the shear wall stiffness of the structure as a whole has been reduced to some extent.

4) Column Forces Results Of C36 Located At Storey 1 Due To EQX And EQY Load

TABLE 6.8

A) Building framed structure without external shear wall

Forces comparison of Corner Column No C36 at Story1 between Grid A & 1 for Earthquake Forces

FOR MODEL 1									
Storey	Column	Load	Loc	P	V2	V3	T	M2	M3
			(in m)	(in KN)	(in KN)	(in KN)	(in KN-M)	(in KN-M)	(in KN-M)
STOREY1	C36	EQX	0	**509.84**	**117.29**	**0**	**0**	**0**	**390.99**
STOREY1	C36	EQX	1.5	509.84	117.29	0	0	0	215.061
STOREY1	C36	EQX	3	509.84	117.29	0	0	0	39.131
STOREY1	C36	EQY	0	**585.83**	**0**	**117.14**	**0**	**254.18**	**0**
STOREY1	C36	EQY	1.5	585.83	0	117.14	0	78.468	0
STOREY1	C36	EQY	3	585.83	0	117.14	0	-97.244	0

B) Building framed structure with shear wall without opening

FOR MODEL 2									
Storey	Column	Load	Loc	P	V2	V3	T	M2	M3
			(in m)	(in KN)	(in KN)	(in KN)	(in KN-M)	(in KN-M)	(in KN-M)
STOREY1	C36	EQX	0	**217.25**	**22.81**	**0.16**	**-0.029**	**0.36**	**84.244**
STOREY1	C36	EQX	1.5	217.25	22.81	0.16	-0.029	0.114	50.034
STOREY1	C36	EQX	3	217.25	22.81	0.16	-0.029	-0.131	15.824
STOREY1	C36	EQY	0	**218.91**	**-0.24**	**23.86**	**-0.013**	**52.884**	**-0.709**
STOREY1	C36	EQY	1.5	218.91	-0.24	23.86	-0.013	17.099	-0.343
STOREY1	C36	EQY	3	218.91	-0.24	23.86	-0.013	-18.686	0.024

C) Building framed structure with external shear wall with the openings provided

FOR MODEL 3									
Storey	Column	Load	Loc	P	V2	V3	T	M2	M3
			(in m)	(in KN)	(in KN)	(in KN)	(in KN-M)	(in KN-M)	(in KN-M)
STOREY1	C36	EQX	0	**221.41**	**23.22**	**0.17**	**-0.029**	**0.368**	**86.885**
STOREY1	C36	EQX	1.5	221.41	23.22	0.17	-0.029	0.108	52.06
STOREY1	C36	EQX	3	221.41	23.22	0.17	-0.029	-0.152	17.235
STOREY1	C36	EQY	0	**222.44**	**-0.24**	**24.36**	**-0.012**	**54.035**	**-0.683**
STOREY1	C36	EQY	1.5	222.44	-0.24	24.36	-0.012	17.488	-0.322
STOREY1	C36	EQY	3	222.44	-0.24	24.36	-0.012	-19.058	0.04

Form above table for three models in combined form, we clearly see that in model-1 all the values for forces comes out to be more than that of when compared with model-2 and model-3 , because in model-1 case, for EQX and EQY load acting at the storey level 1, most of the load is taken by framed structure as there is no special lateral load resisting element in the building (like shear wall) but in model-2 and model-3 Now most of the lateral load due to EQX and EQY is taken by the external shear wall which is created, so forces acting on the column members comes out to be less in value, Here we see a clear decrease in forces for the strengthened models with shear wall (Model-2 & Model-3). This clearly shows that the shear walls are playing its part in taking the lateral forces due to earthquake force.

5) **Beam Forces Result Of B51 At Storey 8 Level Due To EQX Load**

TABLE 6.9

Forces comparison of Beam No B51 at Story8 between Grid A to B & 5 for Earthquake Forces									
Storey	Beam	Load	Loc	Model 1		Model 2		Model 3	
				Shear Force (V2) in KN	Moment (M3) in N-m	Shear Force (V2) in KN	Moment (M3) in KN-m	Shear Force (V2) in KN	Moment (M3) in KN-m
STOREY8	B51	EQX	0.375	**19.76**	**43.91**	**17.03**	**37.711**	**16.79**	**37.18**
STOREY8	B51	EQX	0.847	19.76	34.577	17.03	29.67	16.79	29.252
STOREY8	B51	EQX	1.319	19.76	25.245	17.03	21.628	16.79	21.325
STOREY8	B51	EQX	1.792	19.76	15.913	17.03	13.587	16.79	13.398
STOREY8	B51	EQX	2.264	19.76	6.581	17.03	5.545	16.79	5.47
STOREY8	B51	EQX	2.736	19.76	-2.751	17.03	-2.496	16.79	-2.457
STOREY8	B51	EQX	3.208	19.76	-12.083	17.03	-10.538	16.79	-10.384
STOREY8	B51	EQX	3.681	19.76	-21.415	17.03	-18.579	16.79	-18.312
STOREY8	B51	EQX	4.153	19.76	-30.748	17.03	-26.621	16.79	-26.239
STOREY8	B51	EQX	4.625	19.76	-40.08	17.03	-34.662	16.79	-34.166

From the above table results we can say that by providing external shear wall in strengthened models as (model – 2, and model – 3). we have also reduced the load that is acting on the beams because shear wall created are taking most the load thus loads value has been decreased to a reasonable amount which shows that even weak beams do not fails due the above load and remains at stronger side.

6) Rebar percentage details of Model – 1 at grid 1 elevation in columns

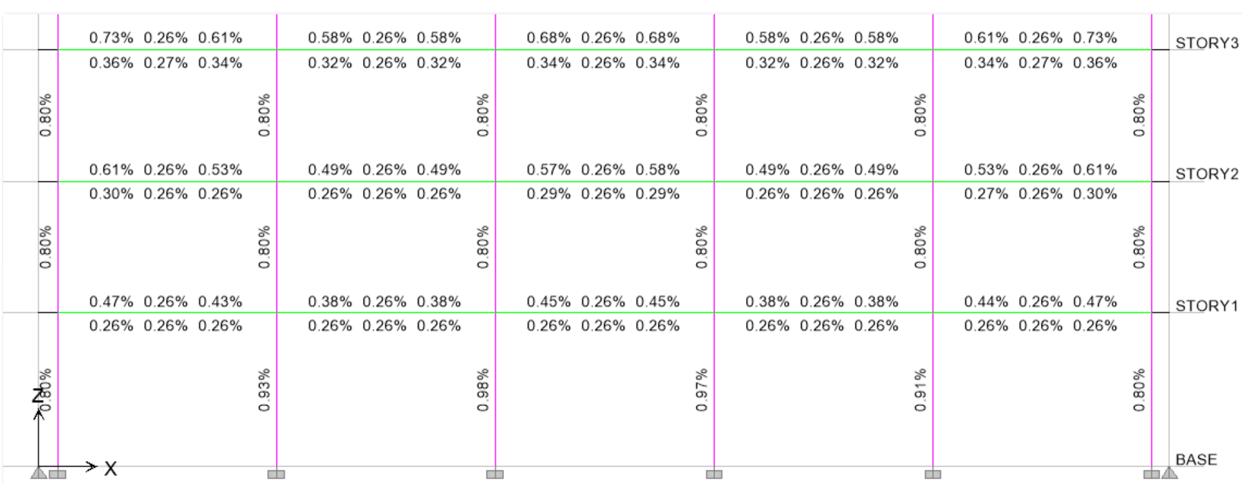
As we can see from the below figure, that the reinforcement required in the bottom columns of the building exceeds the maximum limit which is imposed by our code, so it is necessary to strengthened the building to resist the lateral load.

Fig 12. Rebar % in model - 1

7) Rebar percentage details of Model – 2 at grid 1 elevation in columns

Here we can see that the same columns are now requires less amount of reinforcement when strengthening is done with the help of external shear wall.

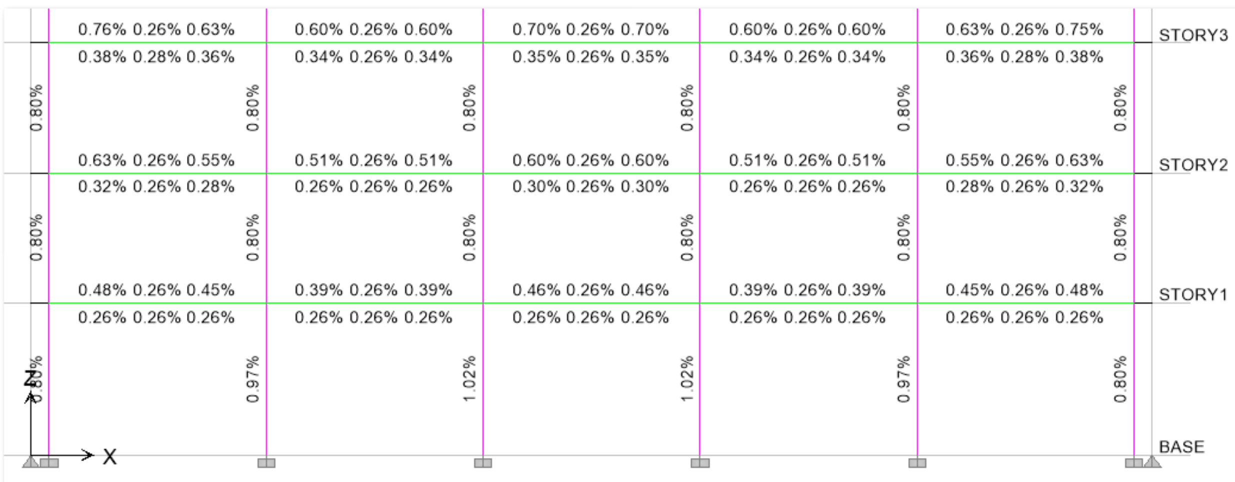
Fig 13. Rebar % in model - 2



8) <u>Rebar percentage details of Model – 3 at grid 1 elevation in columns</u>

When openings are provided in the external shear wall, the reinforcement requirement in the bottom columns has increased but with small value as compared with model -2 because we have decreased the stiffness of the wall so some more force has to be resisted by the columns, which is within the permissible value that our code recommends. So we can say that existing buildings can be effectively strengthened by providing external shear wall to them to resist lateral loads effectively.

Fig 14. Rebar % in model - 3

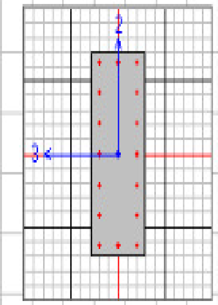


9) Design Sheet Of Column 36 At Storey 1 As Per IS 456 – 2000 Of Model – 1

Concrete Design Information Indian IS 456-2000

File

Units KN-m



Indian IS 456-2000 COLUMN SECTION DESIGN Type: Ductile Frame Units: KN-m (Envelope)

Level : STORY1	L=3.500		
Element : C36	B=0.300	D=0.750	dc=0.040
Section ID : C300X750M25	E=25000000.00	fc=25000.000	Lt.Wt. Fac.=1.000
	fy=415000.000	fys=415000.000	
	RLLF=0.653		

Gamma(Concrete): 1.500
Gamma(Steel) : 1.150

Axial Force & Biaxial Moment Reinforcement For Pu-Mu2-Mu3 Interaction

Column End	Rebar Area	Rebar %
Top	0.007	2.952
Bottom	O/S	O/S

Column End	Design Pu	Design Mu2	Design Mu3	Station Loc	Controlling Combo
Top	2039.707	226.164	63.231	3.000	COMB11
Bottom	2065.019	-469.749	-64.016	0.000	COMB11

Shear Reinforcement For Major Shear (V2)

Column End	Rebar Asv/s	Design Vu	Station Loc	Controlling Combo
Top	3.325E-04	114.532	3.000	COMB13
Bottom	3.325E-04	114.532	0.000	COMB13

Shear Reinforcement For Minor Shear (V3)

Column End	Rebar Asv/s	Design Vu	Station Loc	Controlling Combo
Top	8.313E-04	180.088	3.000	COMB13
Bottom	8.313E-04	180.088	0.000	COMB13

Joint Shear Check/Design

	Joint Shear Ratio	Shear VuTot	Shear Vc	Joint Area	Controlling Combo
Major(V2)	0.595	669.764	1125.000	0.225	COMB2
Minor(V3)	1.408	950.141	675.000	0.135	COMB2

Beam/Column Capacity Ratios

	(1.1)(B/C) Ratio	Column/Beam Ratio	SumBeamCap Moments	SumColCap Moments	Controlling Combo
Major(33)	0.223	4.932	265.877	1311.336	COMB7
Minor(22)	0.824	1.335	418.062	557.951	COMB7

O/S #2 Reinforcing required exceeds maximum allowed

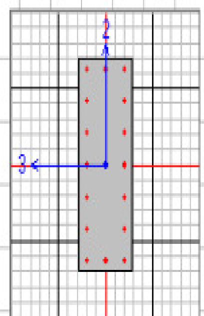
10) Design sheet of column 36 at storey 1 as per IS 456 – 2000 of model – 2

Concrete Design Information Indian IS 456-2000

File

Units KN-m

Indian IS 456-2000 COLUMN SECTION DESIGN Type: Ductile Frame Units: KN-m (Envelope)



Level	: STORY1	L=3.500		
Element	: C36	B=0.300	D=0.750	dc=0.040
Section ID	: C300X750M25	E=25000000.00	fc=25000.000	Lt.Wt. Fac.=1.000
		fy=415000.000	fys=415000.000	
		RLLF=0.653		

Gamma(Concrete): 1.500
Gamma(Steel) : 1.150

Axial Force & Biaxial Moment Reinforcement for Pu-Mu2-Mu3 Interaction

Column End	Rebar Area	Rebar %
Top	0.002	0.800
Bottom	0.002	0.800

Column End	Design Pu	Design Mu2	Design Mu3	Station Loc	Controlling Combo
Top	1026.165	35.760	31.811	3.000	COMB13
Bottom	1041.353	-84.163	-32.282	0.000	COMB13

Shear Reinforcement for Major Shear (V2)

Column End	Rebar Asv/s	Design Vu	Station Loc	Controlling Combo
Top	3.325E-04	5.138	3.000	COMB13
Bottom	3.325E-04	5.138	0.000	COMB13

Shear Reinforcement for Minor Shear (V3)

Column End	Rebar Asv/s	Design Vu	Station Loc	Controlling Combo
Top	8.313E-04	39.974	3.000	COMB13
Bottom	8.313E-04	39.974	0.000	COMB13

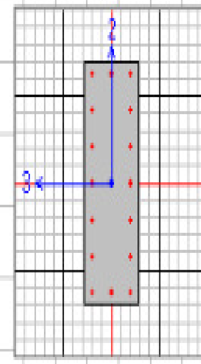
11) Design sheet of column 36 at storey 1 as per IS 456 – 2000 of model – 3

Concrete Design Information Indian IS 456-2000

File

Units KN-m

Indian IS 456-2000 COLUMN SECTION DESIGN Type: Ductile Frame Units: KN-m (Envelope)



Level	: STORY1	L=3.500		
Element	: C36	B=0.300	D=0.750	dc=0.040
Section ID	: C300X750M25	E=25000000.00	fc=25000.000	Lt.Wt. Fac.=1.000
		fy=415000.000	fys=415000.000	
		RLLF=0.653		

Gamma(Concrete): 1.500
Gamma(Steel) : 1.150

Axial Force & Biaxial Moment Reinforcement for Pu-Mu2-Mu3 Interaction

Column End	Rebar Area	Rebar %
Top	0.002	0.800
Bottom	0.002	0.800

Column End	Design Pu	Design Mu2	Design Mu3	Station Loc	Controlling Combo
Top	1031.457	36.318	31.975	3.000	COMB13
Bottom	1046.645	-85.887	-32.446	0.000	COMB13

Shear Reinforcement for Major Shear (V2)

Column End	Rebar Asv/s	Design Vu	Station Loc	Controlling Combo
Top	3.325E-04	5.149	3.000	COMB13
Bottom	3.325E-04	5.149	0.000	COMB13

Shear Reinforcement for Minor Shear (V3)

Column End	Rebar Asv/s	Design Vu	Station Loc	Controlling Combo
Top	8.313E-04	40.735	3.000	COMB13
Bottom	8.313E-04	40.735	0.000	COMB13

12) Design sheet of beam B31 at storey 1 as per IS 456 – 2000 of model – 1

Concrete Design Information Indian IS 456-2000

File

Units KN-m

```
Indian IS 456-2000 BEAM SECTION DESIGN  Type: Ductile Frame  Units: KN-m  (Envelope)

Level       : STORY1      L=5.000
Element     : B31         D=0.500        B=0.300        bf=0.300
Section ID  : B300X500M20 ds=0.000       dct=0.030      dcb=0.030
                          E=22360679.80  fc=20000.000   Lt.Wt. Fac.=1.000
                          fy=415000.000  fys=415000.000

Gamma(Concrete): 1.500
Gamma(Steel)   : 1.150

Flexural Reinforcement for Major Axis Moment
  ------- End-I ---------  --------- Middle --------  --------- End-J ---------
  Rebar Area      Rebar %  Rebar Area      Rebar %    Rebar Area      Rebar %
       0.002        1.296  4.859E-04         0.324         0.002        1.237  Top (+2 Axis)
       0.002        1.066  7.076E-04         0.472         0.002        1.033  Bot (-2 Axis)

   Design Mu  Station Loc   Design Mu  Station Loc   Design Mu  Station Loc
    -277.186        0.375       0.000        3.208    -263.278        4.625  Top (+2 Axis)
     222.539        0.375     107.339        1.792     214.561        4.625  Bot (-2 Axis)

        Controlling Combo        Controlling Combo        Controlling Combo
                   COMB10                   COMB13                    COMB4  Top (+2 Axis)
                    COMB6                    COMB4                   COMB12  Bot (-2 Axis)

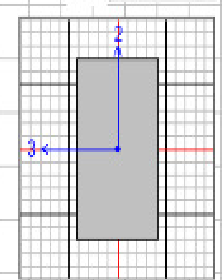
Shear Reinforcement for Major Shear (V2)
  ------- End-I ---------  --------- Middle --------  --------- End-J ---------
 Rebar Asv/s               Rebar Asv/s                Rebar Asv/s
   8.359E-04                 8.392E-04                  8.283E-04

   Design Vu  Station Loc   Design Vu  Station Loc   Design Vu  Station Loc
     227.554        0.375     194.068        1.792     225.256        4.625

        Controlling Combo        Controlling Combo        Controlling Combo
                    COMB2                    COMB8                    COMB8

Torsion Reinforcement
  ------- Shear ---------
       Asvt/s
   4.723E-04

   Design Tu  Station Loc
       0.663        1.792
```

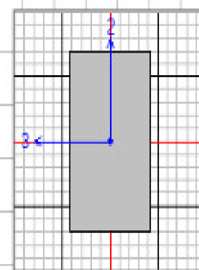


13) Design sheet of beam B31 at storey 1 as per IS 456 – 2000 of Model – 2

Concrete Design Information Indian IS 456-2000

File

Units KN-m




```
Indian IS 456-2000 BEAM SECTION DESIGN  Type: Ductile Frame  Units: KN-m  (Envelope)

Level       : STORY1        L=5.000
Element     : B31           D=0.500         B=0.300         bf=0.300
Section ID  : B300X500M20   ds=0.000        dct=0.030       dcb=0.030
                            E=22360679.80   fc=20000.000    Lt.Wt. Fac.=1.000
                            fy=415000.000   fys=415000.000

Gamma(Concrete): 1.500
Gamma(Steel)   : 1.150

Flexural Reinforcement for Major Axis Moment
  ------- End-I ---------   --------- Middle --------   --------- End-J ---------
  Rebar Area     Rebar %    Rebar Area     Rebar %      Rebar Area     Rebar %
   6.998E-04      0.467      3.879E-04      0.259        6.525E-04      0.435     Top (+2 Axis)
   3.879E-04      0.259      3.879E-04      0.259        3.879E-04      0.259     Bot (-2 Axis)

   Design Mu  Station Loc    Design Mu  Station Loc      Design Mu  Station Loc
    -106.289      0.198       -20.134      3.600          -99.882      4.802     Top (+2 Axis)
       3.530      1.200         8.459      3.600           35.692      4.802     Bot (-2 Axis)

        Controlling Combo          Controlling Combo          Controlling Combo
                   COMB10                     COMB6                      COMB4    Top (+2 Axis)
                   COMB13                     COMB13                     COMB12   Bot (-2 Axis)

Shear Reinforcement for Major Shear (V2)
  ------- End-I ---------   --------- Middle --------   --------- End-J ---------
 Rebar Asv/s                Rebar Asv/s                 Rebar Asv/s
   3.978E-04                  3.325E-04                   3.851E-04

   Design Vu  Station Loc    Design Vu  Station Loc      Design Vu  Station Loc
     114.585      0.198        71.278      3.600          112.285      4.802

        Controlling Combo          Controlling Combo          Controlling Combo
                    COMB2                     COMB13                     COMB8

Torsion Reinforcement
  ------- Shear ---------
      Asvt/s
   2.892E-04

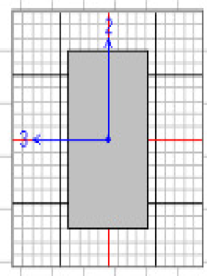
   Design Tu  Station Loc
       0.641      4.802
```

14) Design sheet of beam 31 at storey 1 as per IS 456 – 2000 of Model – 3

Concrete Design Information Indian IS 456-2000

File

Units KN-m

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Indian IS 456-2000 BEAM SECTION DESIGN  Type: Ductile Frame  Units: KN-m  (Envelope)

Level       : STORY1         L=5.000
Element     : B31            D=0.500          B=0.300          bf=0.300
Section ID  : B300X500M20    ds=0.000         dct=0.030        dcb=0.030
                             E=22360679.80    fc=20000.000     Lt.Wt. Fac.=1.000
                             fy=415000.000    fys=415000.000

Gamma(Concrete): 1.500
Gamma(Steel)   : 1.150


Flexural Reinforcement for Major Axis Moment
  ------- End-I ---------   --------- Middle --------   --------- End-J ---------
  Rebar Area       Rebar %   Rebar Area       Rebar %   Rebar Area       Rebar %
   7.203E-04        0.480     3.879E-04        0.259     6.717E-04        0.448   Top (+2 Axis)
   3.879E-04        0.259     3.879E-04        0.259     3.879E-04        0.259   Bot (-2 Axis)

   Design Mu  Station Loc    Design Mu  Station Loc    Design Mu  Station Loc
    -109.031        0.198      -21.368        3.600     -102.508        4.802   Top (+2 Axis)
       3.509        1.200        8.473        3.600       38.332        4.802   Bot (-2 Axis)

        Controlling Combo          Controlling Combo          Controlling Combo
                   COMB10                      COMB6                      COMB4   Top (+2 Axis)
                   COMB13                     COMB13                     COMB12   Bot (-2 Axis)

Shear Reinforcement for Major Shear (V2)
  ------- End-I ---------   --------- Middle --------   --------- End-J ---------
 Rebar Asv/s                Rebar Asv/s                 Rebar Asv/s
   4.032E-04                  3.325E-04                   3.904E-04

   Design Vu  Station Loc    Design Vu  Station Loc    Design Vu  Station Loc
     115.498        0.198       72.184        3.600      113.189        4.802

        Controlling Combo          Controlling Combo          Controlling Combo
                    COMB2                     COMB13                      COMB8

Torsion Reinforcement
  ------- Shear ---------
      Asvt/s
   2.892E-04

   Design Tu  Station Loc
       0.643        4.802
```

CONCLUSION

From this study and The results obtained it is being concluded that the buildings which was constructed before the introduction of modern codes but their importance value is more (such as schools, hospitals, fire stations and power stations etc. Such buildings cannot sustain the lateral load (mainly earthquake load) effectively as these were not designed for these loads or due to various types of faulty construction work.

But now as seismic zones are changing day by day and now these buildings exist in severe to very severe zones we cannot vacate these building for a longer period during renovation work, which causes disturbance to the living life of population.

So this study and its outcomes clearly indicates that we can strengthened these building by inspecting the degree of their vulnerability, by the use of external shear walls along the parallel sides of the building and connection can be made effectively between the exterior shear wall and existing building using dowel bars of sufficient diameter (steel bars) to act as a monolithic structure during lateral forces acting on it. So that stiffness of the structure can be increased as a whole and most of the lateral loads is taken by the shear wall, with the help of which the structures can be damaged to a lesser extent.

This method is also of a great importance that instead of doing elemental strengthening of building by various method of retrofitting we can directly strengthened the building as a whole globally externally, with lesser time for construction and also economical as globally strengthening is cheaper than element to element technique.

Thus various parameters like displacement, storey shear, time period, forces in column and beams have been reduced to a greater percentage which makes the building on stronger side then earlier condition.

FUTURE SCOPE OF THIS STUDY

Rapid visual screening can be used as an effective tool for this study to be enhanced in future. Several old constructions as well as new construction which are not as per what our code recommends can be rapidly visualized and different types of deficiency can be found on faster rate and then we can start the application of this method as early as possible.

Further future work which can be done in this study is Addition of shear walls to a structure will definitely improve its lateral load capacity. This fact has been understood by present study carried out for strengthening of building but On the other hand, exterior shear walls cannot improve the capacity in case of dowel (steel bars) failure which is mainly responsible for making shear wall as monolithic structure. The key point is that exterior shear walls can be successfully applied to existing vulnerable buildings to improve seismic capacity provided that the dowels are well-designed.

So research work can done on design of these dowel (steel bars) used in connection of external shear wall and existing building and some Indian standard code provisions can be made for use of these bars according to the seismic zone.

In India this method of strengthening has greater need, where retrofitting of old construction requires a huge cost, so by introducing this method we can make the old constructions susceptible to lateral load to stronger one with less amount of money as well as time period.

REFERENCES

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2) IS 875 (Part 2) – 1987 Code of Practice for Design Loads (Other than Earthquake) For Buildings and Structures.

3) IS 1893 (Part 1): 2002 Indian Standard Criteria For Earthquake Resistant Design Of Structures.

4) IS 13920: 1993 Ductile Detailing Of Reinforced Concrete Structures Subjected To Seismic Forces -Code Of Practice.

5) CSI Concrete Frame Design Manual As Per IS 456:2000 for E- Tabs.

6) CSI Shear Wall design Manual IS 456:2000 And IS 13920:1993 For E- Tabs.

7) IS 1786:2008 Indian Standard high Strength Deformed Steel Bars and Wires for concrete reinforcement—Specification.

8) Guidelines for epoxy grouted dowels in seismic strengthening.

9) Dynamics of Structure, Theory and Application To Earthquake Engineering, By A.K. Chopra.

10) Earthquake resistant design of structures by S.K. Duggal.

11) Some Concepts in Earthquake Behavior of Buildings by C. V. R. Murty, Rupen Goswami A. R. Vijayanarayanan, Vipul V. Mehta.