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Assessment of the Torsion Effect on Buildings under Lateral Seismic Ground Motion.

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ABSTRACT

Reinforced concrete framed buildings are adequate for resisting both the vertical and horizontal loads acting on them. However, when the buildings are in either 'C' or 'L' or 'V' etc., shapes the torsion develops in the building which does not contribute to the safety of buildings during the earthquakes. These practical difficulties call for introduction of shear walls. In this paper, we are discussing about the appropriate location for shear wall to reduce the Torsion forces. It is worth noting that the severe damage occurred during the Michoacán Earthquake, Mexico, 1985, showed the importance of torsional effects and pointed out the need to understand the problem and improve the design requirements [1].

Observation of damage after earthquakes has shown that torsional vibration of buildings, induced by lateral seismic ground motion, may cause serious distress in a structure, sometimes leading to its collapse. Most seismic codes include provisions for design against seismic torsion. The code provisions are specified in terms of design eccentricities that depend on the relative stiffness of the lateral load-resisting elements. Also, the allocation of strength among the lateral load-resisting elements is related to the stiffness of such elements. The current practice is to determine the stiffness of an element from its given geometry. Recent studies have, however, shown that in reinforced concrete structures, the stiffness does not depend on geometry alone, but is strongly related to the strength of the element. Consequently, the conventional method of design of concrete structures for seismic torsion in which the stiffness of an element is considered as being independent of strength needs to be reviewed.

INTRODUCTION

Shear walls in plan, may be deep straight walls or angular, U-shaped or box shaped in plan, around stairs or lifts or toilets, where there will be no architectural difficulty in extending them throughout the height of the building. It is worth noting that the severe damage occurred during the Michoacán Earthquake, Mexico, 1985, showed the importance of torsional effects and pointed out the need to understand the problem and improve the design requirements [1].

Care shall be taken to have a symmetrical configuration of plan in walls should get enough vertical load from floors, for which reason, nearby columns should be omitted and load taken to the shear walls by means long span beams if required. In the last four decades, numerous researchers have conducted extensive studies to investigate the torsional response of asymmetric buildings considering elastic and inelastic behavior [2]. Despite this effort, design provisions include in most of the seismic codes [3] In the last decade, there has been a renovated interest in the evaluation of the torsional effect in building subjected to earthquakes based on the need to revise and improve torsional provisions [4, 5]. Paulay [6, 7, 8, 9] Crisafulli and Formica [10] conducted a large parametric study in which simple asymmetric buildings where analyzed using static linear analysis and applying usual torsional provisions included in seismic codes.
In a number of building structures, the center of rigidity does not coincide with the center of mass. As a consequence, during an earthquake, lateral base motion induces torsional vibrations in the structure. Furthermore, an increase of the final strength eccentricity was observed in some cases, which is obviously an unfavorable condition. Other researchers have continued the investigation of this topic [11, 12, 13].

BASE SHEARS & ADDITIONAL FORCES DUE TO TORSION

Static and Dynamic Properties of the building:

Building Description

The proposed construction to be carried out as a L-Shaped IT building consisting of G+4 floors. The structure is planned to be reinforced concrete1

Structural Modeling

The superstructure will be modeled using standard software, E-Tabs as a space frame with a grid of columns in the vertical direction, interconnected with beam members in the orthogonal directions at each floor level. The nodes (the meeting points of beams with beams and beams with columns) will be treated as rigid joints due to monolithic construction. The end nodes at the bottom of the model will be modeled as fixed supports due to rigid connection of the columns with footings at the foundation level. The columns will also be interconnected at the plinth beams to increase the stability of the structure, wherever necessary. All slabs are modeled as membrane/shell elements wherever necessary.

Geometry of the Building

Minimum width of column (least lateral dimension) : 300mm

Minimum width of main beams : 300mm

Column pedestal will be provided as per the design requirement.

Design Loads

Service Loads

The following service loads are expected to act on the structure during its intended life and they are considered as follows with reference to relevant codes

1) Dead Loads [as per IS 1875 (part-1) 1987]

Dead loads of all the materials used in the construction will be considered as follows:

i) Reinforced concrete members (Slabs, beams and columns)	25 KN/m3
ii) Plain concrete members	24 KN/m3
iii) Brick walls with solid Fly Ash Bricks	22 KN/m3
iv) Floor finishes (on an average; Could vary depending on actual flooring)	1.5 KN/m3
v) Roof finishes (on an average; Could vary depending on actual finishes)	2.5 KN/m3
vi) Built in partitions	1.0 KN/m3
vii) Others	(as required)

2) Imposed loads [as per IS 875 (part-2)-1987]

Refer Annexure-A for details of Imposed loads at different levels as per occupancy requirements)

3) Earthquake loads [as per IS 1893 (part-1)-2002]

Refer Annexure- A for details

3.2 Application of the loads on the model

The dead loads of all the reinforced concrete members will be given as self-weights of the members; all the floor loads expressed as load per square area will be applied as distributed floor loads onto the supporting beams as per Clause 24.5 of IS 456: 2000, all wall loads will be applied as uniformly distributed load per unit length onto the supporting beams; the wind and earthquake loads will be applied as lateral loads at each floor level.

3.3 Design loads and Load combinations

Taking into consideration the probability of various loads acting together during the construction stage and after occupation, and their severity in the internal stresses and deformations, the structure will be analyzed.

As per Clause 18.2.1 of IS 456; 2000 the Limit state method of design will be followed for the design of all the structural members. As per this method, the design loads will be arrived at using partial safety factors for various load combinations as per table 18 of IS 456: 2000

LOAD COMBINATIONS	
Comb 1	1.5 (Dead + Live)
Comb 2	1.5 (Dead + EQX Spectra)
Comb 3	1.5 (Dead - EQX Spectra)
Comb 4	1.5 (Dead + EQY Spectra)
Comb 5	1.5 (Dead - EQY Spectra)
Comb 6	1.2 (Dead + Live + EQX Spectra)
Comb 7	1.2 (Dead + Live - EQX Spectra)
Comb 8	1.2 (Dead + Live + EQY Spectra)
Comb 9	1.2 (Dead + Live - EQY Spectra)
Comb 10	0.9 Dead + 1.5 EQX Spectra
Comb 11	0.9 Dead - 1.5 EQX Spectra
Comb 12	0.9 Dead + 1.5 EQY Spectra
Comb 13	0.9 Dead - 1.5 EQY Spectra
Comb 14	Dead + Live
Comb 15	Dead + EQX Spectra
Comb 16	Dead - EQX Spectra
Comb 17	Dead + EQY Spectra
Comb 18	Dead - EQY Spectra
Comb 19	Dead + 0.8(Live + EQX Spectra)
Comb 20	Dead + 0.8(Live - EQX Spectra)
Comb 21	Dead + 0.8(Live + EQY Spectra)
Comb 22	Dead + 0.8(Live - EQY Spectra)
Comb 23	Dead Static Load
Comb 24	EQX Spectra
Comb 25	EQY Spectra
Comb 26	Live Static Load

Note: DL stand for dead load; EL for-Earthquake load.

Earth pressure, Hydrostatic & surcharge will also be considering wherever necessary.

3.4 Material Specifications:

a) Concrete

i) Grade M25

ii) Static modulus of elasticity $5000\sqrt{f_{ck}}$ ($2.5x10^7$ KN/m^2)

iii) Poisson's ratio 0.2

b) Reinforcement Steel

i) Specification High strength deformed steel bars conforming to IS 1786-1985

ii) Yield strength, Fy 415 N/mm^2

iii) Modulus of elasticity, Es $2x10^{5\ N}$/mm^2

2) Nominal cover to all reinforcements including links:

Columns/Pedestals 40mm

Beams	30mm
Floor and roof slabs	25mm
RC walls	25mm

Structural analysis:

The 3D space frame analysis with "Response Spectrum" method will be carried out using the modeling software's, which utilize finite element technique to carry out the analysis. The in-built solver processes the member and element properties, load combinations and support conditions to arrive at the stiffness and load matrices. The solver further uses matrix method to determine the nodal displacements, member forces/stresses and support reactions, which will be further utilized in the design of the structural members.

Loads:

Grade of concrete	=	M25
Grade of steel	=	Fe415
Dead loads		
Density of concrete	=	25 KN /m^3
Floor finish		
Basement	=	1.5 KN/m^3
Typical	=	1.5 KN/m^3
Terrace	=	2.5 KN/m^3
Partition loads (location as per arch drawing)	=	1.0 KN/m^3
125 mm thick slab		
Thickness	=	125mm
Dead weight of the slab	=	3.125
Floor finishes	=	1.5
Partitions	=	1

		5.625 KN/m^2

Live loads		
Typical	=	2 KN/m^2
Corridors/staircase/Balcony	=	3 KN/m^2
Vehicular loading (General parking)	=	5 KN/m^2
Fire tender (on marked drive ways)	=	25 KN/m^2
Sunken loads (Min 300 mm depth)		
Density of filling (Brick bats)	=	19 KN/m^2
Floor Height As per Arch drwg	=	3 (say)
Beam depth	=	0.45

Dynamic Properties		
Zone	=	II
Seismic Zone Coefficient	=	0.1
Importance Factor	=	1
Response Reduction Factor	=	3
Rock or soil site factor SS	=	1

Type of structure ST	=	1
Damping ratio	=	0.05
Time period		=Calculate Optional time period of structure in X-direction
Height of the building	=	h= ly = 15 m
Fundamental Natural period Ta	=	$0.075h^{0.75}$

Stair case loads

Floor finishes	=	1.5 KN/m^3
Live load	=	3 KN/m^3

MODELING, STATIC & DYNAMIC ANALYSIS, DESIGN & PUSHOVER ANALYSIS THROUGH E-TABS

Step by step procedure:

1. Define Plan grids and story data
2. Define material properties
3. Define frame sections
4. Define slab sections
5. Define load cases
6. Define load Combinations
7. Define mass source
8. Define diaphragms
9. Draw Beam objects (Frame members)
10. Draw Column objects (Frame members)
11. Draw slab sections
12. Assign beams
13. Assign columns
14. Assign slabs
15. Assign beam loads
16. Assign slab loads
17. Assign diaphragms
18. Increase story data
19. Edit story data
20. Run analysis
21. View analysis results graphically
22. Unlock the model
23. Define response spectrum functions
24. Define response spectrum cases
25. Run analysis
26. View analysis results
27. Define preferences of concrete frame design
28. Start design/check of structure
29. Verify analysis vs design section
30. Define static nonlinear/pushover cases
31. Run static nonlinear analysis

Structure without Shear Wall plans and details

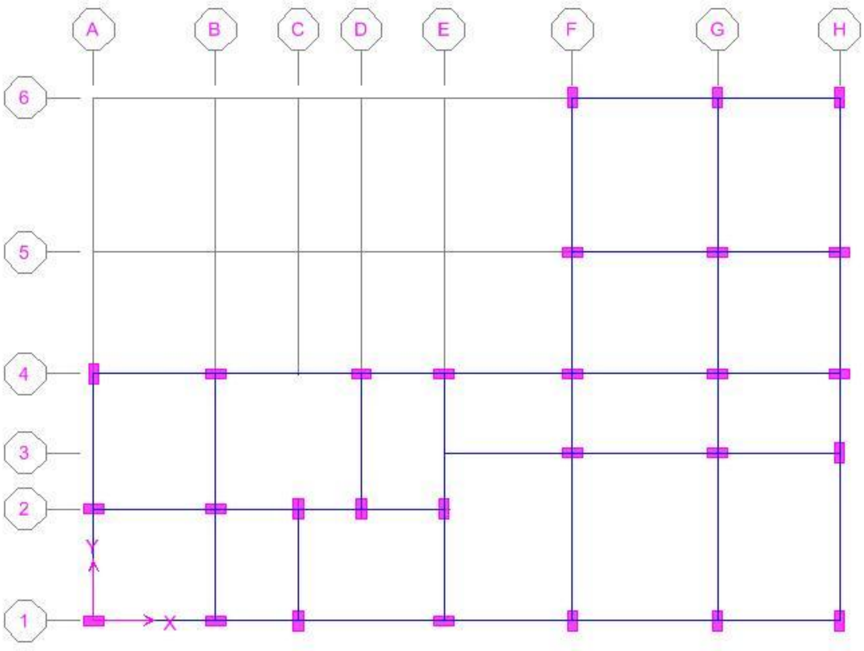

Fig1: Plan

The base dimensions of the plan are 22.70m x 15.97m. Floor to floor height is 3m.

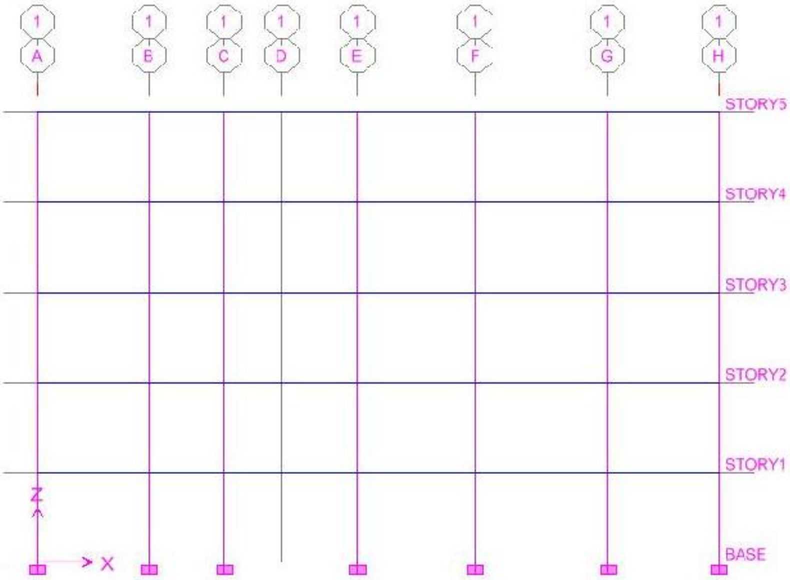

Fig2: Elevation

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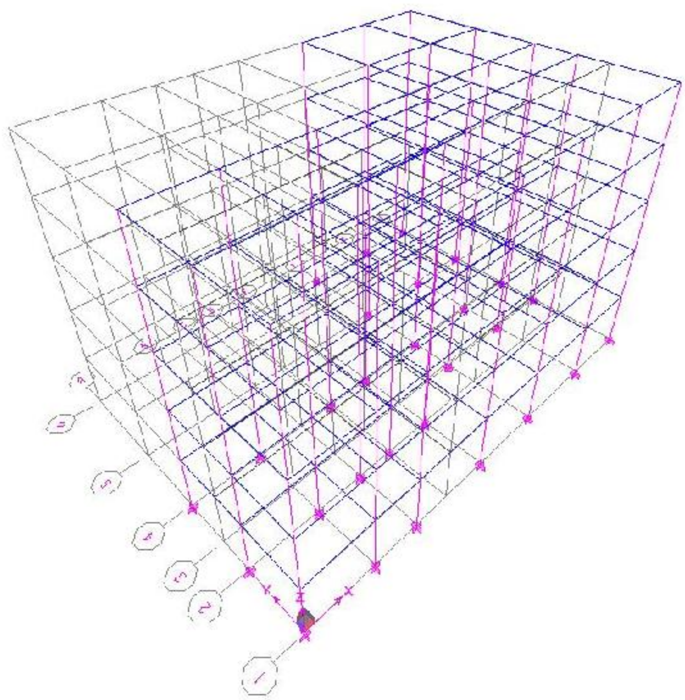

Fig3: 3D View

Base shear Calculation from ETABS

From IS 1893:2002 Clause 7.5.3. The total design lateral force or design seismic base shear (VB) along any principal direction shall be determined by the following expression.

$Vb = A_h W$

Where W = seismic weight of the structure

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

Z = Zone factor = 0.1 (Consider Zone 2)
I = Importance factor = 1
S_a/g = Spectral acceleration

From IS 1893:2002 using response spectrum method we have calculated the base shear from ETABS

Base shear of Structure without shear wall

Spec	Mode	Dir	F1	F2	F3	M1	M2	M3
EQX	1	U1	0.1	-5.91	0	63.672	1.078	-78.461
EQX	2	U1	39.62	3.82	0	-39.578	426.81	-193.71
EQX	3	U1	363.16	2.79	0	-30.585	3897.45	-2199.69
EQX	4	U1	0.02	-1.43	0	-1.662	0.002	-18.904
EQX	5	U1	7.76	0.88	0	0.78	-8.705	-36.025
EQX	All	All	388.44	5.02	0	53.264	4168.361	2354.019

Spec	Mode	Dir	F1	F2	F3	M1	M2	M3
EQY	1	U2	-5.91	352.87	0	-3799.38	-64.313	4681.838
EQY	2	U2	3.82	0.37	0	-3.815	41.138	-18.671
EQY	3	U2	2.79	0.02	0	-0.235	29.891	-16.87
EQY	4	U2	-1.43	95.73	0	110.867	-0.123	1261.266
EQY	5	U2	0.88	0.1	0	0.088	-0.987	-4.083
EQY	All	All	5.02	366.45	0	3803.357	53.4	4835.466

From the table, we can compute the total base shear in X and Y- Direction

Base shear along X- direction = 388.44 KN/m

Base shear along Y -direction = 366.45 KN/m

Base shear Distribution to floors

From IS 1893:2002 Clause 7.7 Distribution of Design Force is Vertical Distribution of Base Shear to Different Floor Levels The design base shear (V~) computed in 7.5.3 shall be distributed along the height of the building as per the following expression:

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$$

Story	Height	Σ (Wj * Hj2)	from IS 1893
			Qi = Vb * (Wi*Hi2)/Σ (Wj * Hj2)
STORY5	15	141193.89	162.88
STORY4	12	104278.9248	120.30
STORY3	9	58656.8952	67.67
STORY2	6	26069.7312	30.07
STORY1	3	6517.4328	7.52
		336716.874	388.44

Stiffness is computed for each column in one particular frame, by adding the all column stiffness in the corresponding frame will give the frame stiffness.

Lateral load distribution to frames

Let us consider Eqx force which is acting on the 1st floor will be distributed to all frames in x- direction considering the corresponding stiffness. By using the formula Eqx* K/ ∑K, earthquake force can be distributed along all the frames. From the above results, we can observe that distributed force is maximum for some frames. We can reduce the distributed forces by adding the shear walls where the force will be maximum. Frame A, B and M are having more distribution force along the X-direction.So, we can reduce the force by adding the shear wall in frame E and G since the lateral forces were less in those frames.

Structure without wall _ Calculation of lateral load distribution on frames in X direction

FRAME	A	B	C	D	E	F	G	H	SUM
TOTAL STIFFNESS (K frame)	**90000**	**45000**	**120000**	**75000**	**90000**	**165000**	**165000**	**210000**	**960000**
1st floor Eqx	**7.52**								
1st floor Distribution of force KA/ΣK * Eqx (KN)	0.71	0.35	0.94	0.59	0.71	1.29	1.29	1.65	7.52
2nd floor Eqx	30.07								
2 nd floor Distribution of force KA/ΣK * Eqx (KN)	2.82	1.41	3.76	2.35	2.82	5.17	5.17	6.58	30.07
3rd floor Eqx	**67.67**								
3 rd floor Distribution of force KA/ΣK * Eqx (KN)	6.34	3.17	8.46	5.29	6.34	11.63	11.63	14.80	67.67
4 th floor Eqx	**120.30**								
4 th floor Distribution of force KA/ΣK * Eqx (KN)	11.28	5.64	15.04	9.40	11.28	20.68	20.68	26.32	120.30
5 th floor Eqx	**162.88**								
5th floor Distribution of force KA/ΣK * Eqx (KN)	15.27	7.64	20.36	12.73	15.27	28.00	28.00	35.63	162.88

Calculation of Center of mass and Center of stiffness *(Center of mass from ETABS)*

Center of mass = $\sum M_i \times a_i \times Y_i / \sum M_i \times a_i$

Where M = mass

A = area

Y = centroidal distance.

Story	Diaphragm	Mass X	Mass Y	XCM	YCM
STORY5	D1	627.5284	627.5284	13.223	6.179
STORY4	D1	724.1592	724.1592	13.142	6.15
STORY3	D1	724.1592	724.1592	13.142	6.15
STORY2	D1	724.1592	724.1592	13.142	6.15
STORY1	D1	724.1592	724.1592	13.142	6.15
		3524.165 TONNS			
		35241.65 NEWTONS			

Location of center of mass = (13.142, 6.15)

Center of stiffness: STIFFNESS IN X-DIRECTION

h=3 *E=5000*sqrt(f_{ck})=25000*

SOFTCOPY GRID LINE DATA	COLUMN TYPE	COLUMN SIZE		I = bd^3/12 mm4	K = 12 * E* I / h 3
		BREADTH(b)	LENGTH(d)		
1A	C1	600	300	1350000000	15000
1B	C1	600	300	1350000000	15000
1C	C2	300	600	5400000000	60000
1E	C1	600	300	1350000000	15000
1F	C2	300	600	5400000000	60000
1G	C2	300	600	5400000000	60000
1H	C2	300	600	5400000000	60000
					285000
2A	C1	600	300	1350000000	15000
2B	C1	600	300	1350000000	15000
2C	C2	300	600	5400000000	60000
2D	C2	300	600	5400000000	60000
2E	C2	300	600	5400000000	60000
					210000
3F	C1	600	300	1350000000	15000
3G	C1	600	300	1350000000	15000
3H	C2	300	600	5400000000	60000
					90000
4A	C2	300	600	5400000000	60000
4B	C1	600	300	1350000000	15000
4D	C1	600	300	1350000000	15000
4E	C1	600	300	1350000000	15000
4F	C1	600	300	1350000000	15000
4G	C1	600	300	1350000000	15000
4H	C1	600	300	1350000000	15000
					150000
5F	C1	600	300	1350000000	15000
5G	C1	600	300	1350000000	15000
5H	C1	600	300	1350000000	15000
					45000
6F	C2	300	600	5400000000	60000
6G	C2	300	600	5400000000	60000
6H	C2	300	600	5400000000	60000
					180000

STIFFNESS IN Y-DIRECTION

				h=3	*E=5000*sqrt(fck)=25000*
SOFTCOPY GRID LINE DATA	**COLUMN TYPE**	**COLUMN SIZE**		**I = bd3/12 mm4**	**K = 12 * E* I / h 3**
		BREADTH(b)	**LENGTH(d)**		
1A	C1	600	300	1350000000	15000
2A	C1	600	300	1350000000	15000
4A	C2	300	600	5400000000	60000
					90000
1B	C1	600	300	1350000000	15000
2B	C1	600	300	1350000000	15000
4B	C1	600	300	1350000000	15000
					45000
1C	C2	300	600	5400000000	60000
2C	C2	300	600	5400000000	60000
					120000
2D	C2	300	600	5400000000	60000
4D	C1	600	300	1350000000	15000
					75000
1E	C1	600	300	1350000000	15000
2E	C2	300	600	5400000000	60000
4E	C1	600	300	1350000000	15000
					90000
1F	C2	300	600	5400000000	60000
3F	C1	600	300	1350000000	15000
4F	C1	600	300	1350000000	15000
5F	C1	600	300	1350000000	15000
6F	C2	300	600	5400000000	60000
					165000
1G	C2	300	600	5400000000	60000
3G	C1	600	300	1350000000	15000
4G	C1	600	300	1350000000	15000
5G	C1	600	300	1350000000	15000
6G	C2	300	600	5400000000	60000
					165000
1H	C2	300	600	5400000000	60000
3H	C2	300	600	5400000000	60000
4H	C1	600	300	1350000000	15000
5H	C1	600	300	1350000000	15000
6H	C2	300	600	5400000000	60000
					210000

Center of stiffness = $Ki * x_i / \sum K_i$

Where K_i = stiffness at the I^{th} frame.

$\sum K_i$ = sum of all stiffness.

x_I = distance

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Location of Center of stiffness is (13.324, 5.257)

Identification of Center of mass and Center of stiffness

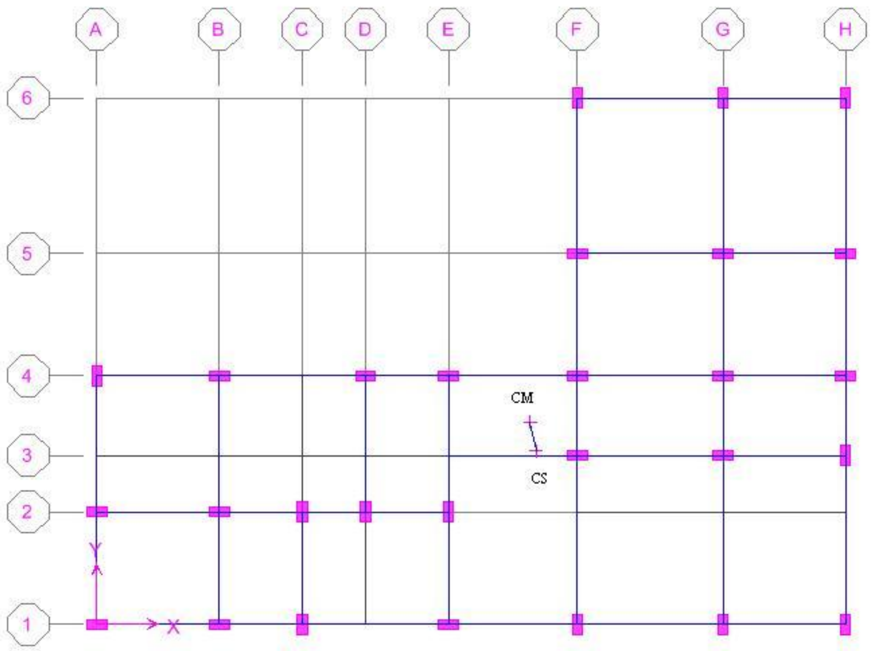

Fig: location of center of mass & center of stiffness

Center of mass (CM) will be the geometric center of the floor slab, ie. (13.142, 6.150)

And Center of stiffness (CR) will be at (13.324, 5.257)

From Clause 7.9.2. From IS 1893:2002 Design eccentricity is computed as follows

Calculation of Static and Design eccentricity

The difference between the Center of mass and Center of stiffness will be the eccentricity.

Eccentricity in X- direction (e_x) =| CMx – CSx | = | 13.142 – 13.324 | = 0.182 m

Eccentricity in Y- direction (e_y) =| CMy – CSy | = | 6.150 – 5.257 | = 0.893 m

From IS 1893:2002 Clause 7.9.2 the design eccentricity, e_{di} to be used at for shall be taken as

e_{di} = $1.5\ e_{si} + 0.05\ b_i$ (or)

= $e_{si} - 0.05\ b_i$

Where, e_{di} = Design eccentricity

e_{si} = static eccentricity at floor I defined as the distance between center of mass and center of stiffness.

b_i = Floor plan dimension of floor I perpendicular to direction of force.

In X- Direction

e_{di} = 1.5 * 0.182 + 0.05 *22.705 = 1.408

e_{di} = 0.182 – 0.05 * 22.705 = -0.953

In Y- Direction

e_{di} = 1.5 * 0.893 + 0.05 * 15.975 = 2.137

e_{di} = 0.893 – 0.05 * 15.975 = 0.095

Calculation of Torsional and Direct forces

Torsional force = $Q^* e_{di} * K_i * r_i / \sum(K_i * r_i * r_i)$

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Direct force = $Q_x * K_i / \sum K_i$

Additional forces due to Torsion considering edi = 1.408

Frame	Ki	ri	Ki*ri	Ki*ri*ri	Ki*ri/∑ (Ki*ri*ri)	Additional force in X- direction		
						Torsion force	Direct force	total force
1	285000	5.257	1498245	7876274	0.02	0.19	1.12	
2	210000	1.842	386820	712522.4	0.00	0.05	0.82	
3	90000	0.127	11430	1451.61	0.00	0.00	0.35	
4	150000	-2.288	-343200	785241.6	0.00	-0.04	0.59	
5	45000	-6.033	-271485	1637869	0.00	-0.03	0.18	
6	180000	10.718	1929240	20677594	0.02	0.24	0.71	
A	90000	13.324	1199160	15977608	0.01	0.15		
B	45000	9.609	432405	4154980	0.01	0.05		
C	120000	7.094	851280	6038980	0.01	0.11		
D	75000	5.179	388425	2011653	0.00	0.05		
E	90000	2.664	239760	638720.6	0.00	0.03		
F	165000	-1.251	-206415	258225.2	0.00	-0.03		
G	165000	-5.666	-934890	5297087	-0.01	-0.12		
H	210000	-9.381	-1970010	18480664	-0.02	-0.25		
	1920000		3210765	84548870				

Additional forces due to Torsion considering edi = -0.953

Frame	Ki	ri	Ki*ri	Ki*ri*ri	Ki*ri/∑ (Ki*ri*ri)	Additional force in X- direction		
						Torsion force	Direct force	total force
1	285000	5.257	1498245	7876274	0.02	-0.13	1.12	
2	210000	1.842	386820	712522.4	0.00	-0.03	0.82	
3	90000	0.127	11430	1451.61	0.00	0.00	0.35	
4	150000	-2.288	-343200	785241.6	0.00	0.03	0.59	
5	45000	-6.033	-271485	1637869	0.00	0.02	0.18	
6	180000	10.718	1929240	20677594	0.02	-0.16	0.71	
A	90000	13.324	1199160	15977608	0.01	-0.10		
B	45000	9.609	432405	4154980	0.01	-0.04		
C	120000	7.094	851280	6038980	0.01	-0.07		
D	75000	5.179	388425	2011653	0.00	-0.03		
E	90000	2.664	239760	638720.6	0.00	-0.02		
F	165000	-1.251	-206415	258225.2	0.00	0.02		
G	165000	-5.666	-934890	5297087	-0.01	0.08		
H	210000	-9.381	-1970010	18480664	-0.02	0.17		
	1920000			84548870				

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SHEAR WALLS

Shear walls are vertical elements of the horizontal force resisting system. When shear walls are designed and constructed properly, they will have the strength and stiffness to resist the horizontal forces. Reinforced concrete framed buildings are adequate for resisting the both the vertical and horizontal loads acting on them.

TYPES OF SHEAR WALLS

Deep Straight walls or angular, U shaped and Box shaped walls.

To form an effective box structure, equal length shear walls should be placed symmetrically on all four exterior walls of the building. Shear walls should be added to the building interior when the exterior walls cannot provide sufficient strength and stiffness or when the allowable span-width ratio for the floor or roof diaphragm is exceeded. For subfloors with conventional diagonal sheathing, the span-width ratio is 3:1. This means that a 25-foot wide building with this subfloor will not require interior shear walls until its length exceeds 75 feet unless the strength or stiffness of the exterior shear walls are inadequate.

WHAT TYPES OF FORCES DO SHEAR WALLS RESIST?

Shear walls resist two types of forces: shear forces and uplift forces. Connections to the structure above transfer horizontal forces to the shear wall. This transfer creates shear forces throughout the height of the wall between the top and bottom shear wall connections.

Initial location for shear wall to reduce torsion

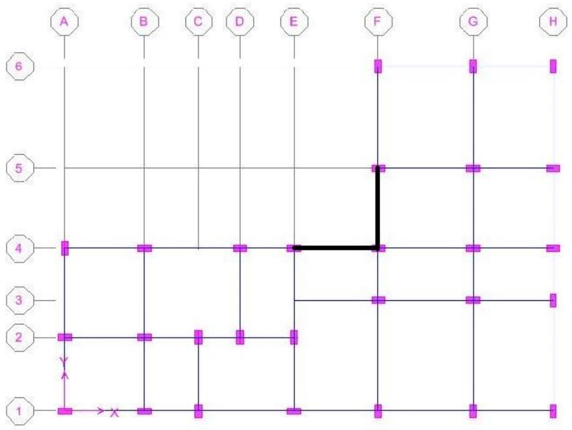


Fig : Shear wall location and plans

Lateral force reduction in shear wall building

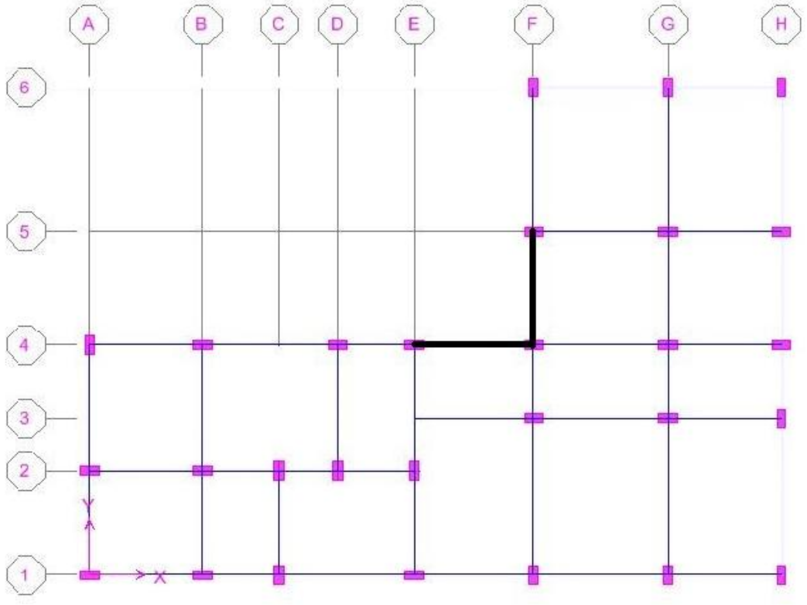
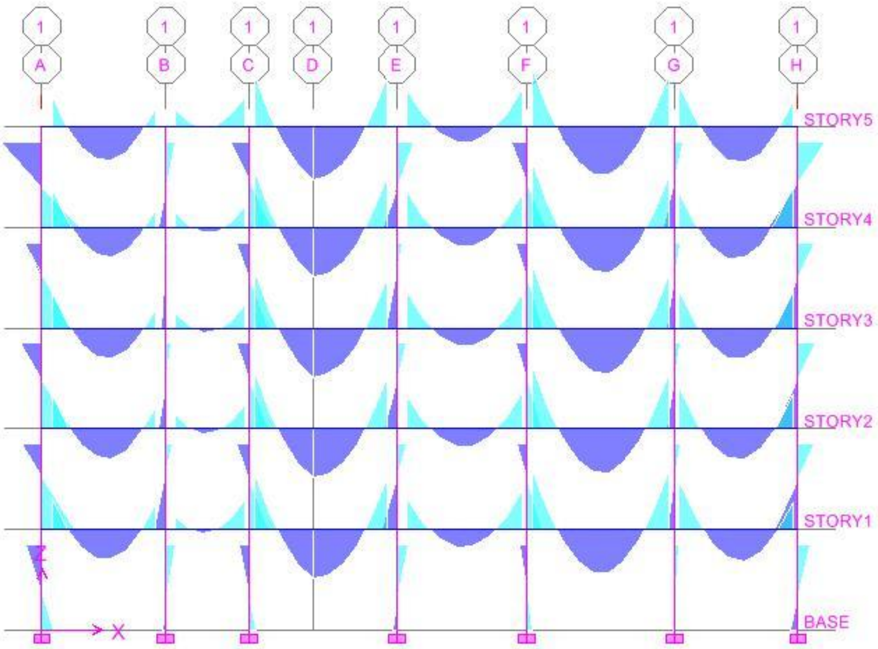


Fig5: Plan of the structure with shear wall

RESULTS

Bending Moment & Shear force diagrams:

BMD



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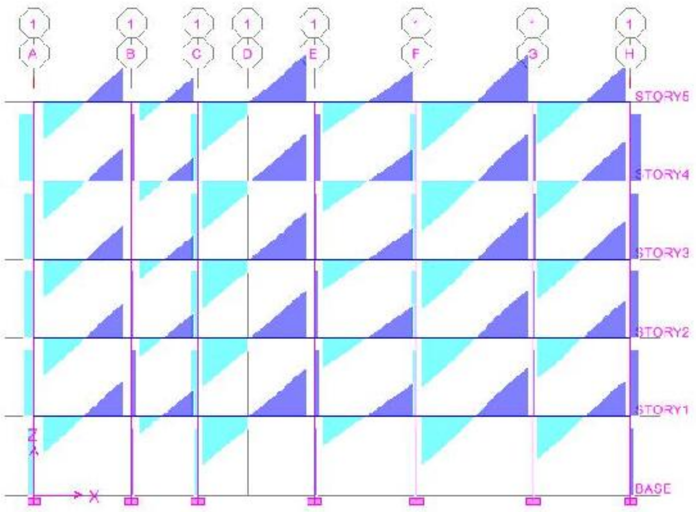
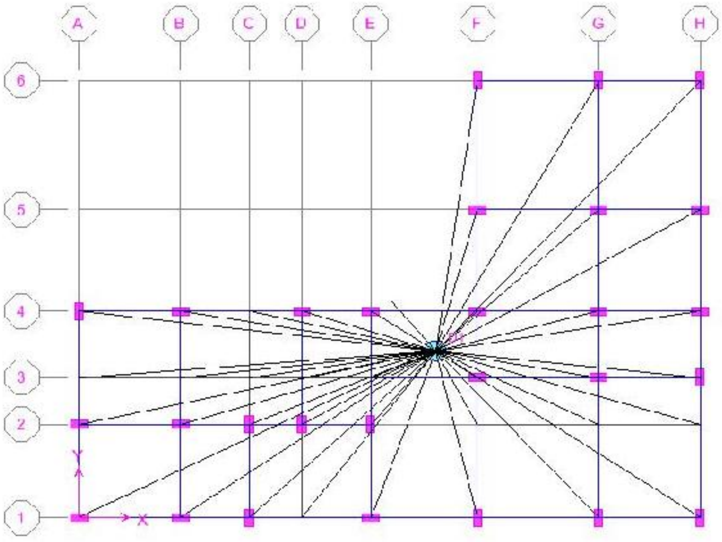
SFD



CENTER OF MASS



SHEAR WALL DESIGN PROCEDURE USING ETABS

Following is a typical shear wall design process that might occur for a new building. Note that the sequence of steps you may take in any particular design may vary from this but the basic process will be essentially the same.

1. Use the Options menu > Preferences > shear wall design command to review the shear wall design preferences and revise them if necessary. Note that there are default values provided for all shear wall design preferences so it is not actually necessary for you to define any preferences unless you want to change some of the default preference values.
2. Run the building analysis using the Analyze menu > Run Analysis command.

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3. Assign the wall pier and wall spandrel labels. Use the Assign menu > Frame/Line > Pier Label, the Assign menu > Shell/Area > Pier Label, the Assign menu > Frame/Line > Spandrel Label, and the Assign menu > Shell/Area > Spandrel Label commands to do this. Note that the labels can be assigned before or after the analysis is run.
5. Assign shear wall overwrites, if needed, using the Design menu > shear wall design > View/Revise Pier Overwrites and the Design menu > shear wall design > View/Revise Spandrel Overwrites commands. Note that you must select piers or spandrels first before using these commands. Also note that there are default values provided for all pier and spandrel design overwrites so it is not actually necessary for you to define any overwrites unless you want to change some of the default overwrite values. Note that the overwrites can be assigned before or after the analysis is run.
6. If you want to use any design load combinations other than the default ones created by ETABS for your shear wall design, click the Design menu > shear Wall Design > Select Design Combo command. Note that you must have already created your own design combos by clicking the Define menu > Load Combinations command.
7. Click the Design menu >shear Wall Design > Start Design/Check of Structure command to run the shear wall design.
8. Review the shear wall design results. To do this you might do one of the following:

a. Click the Design menu > Shear Wall Design > Display Design Info command to display design information on the model.

b. Right click on a pier or spandrel while the design results are displayed on it to enter the interactive wall design mode. Note that while you are in this mode you can revise overwrites and immediately see the new design results.

If you are not currently displaying design results, click the Design menu > shear Wall Design > Interactive Wall Design command and then right click a pier or spandrel to enter the interactive design mode for that element.

c. Use the File menu > Print Tables > Shear Wall Design command to print shear wall design data. If you select a few piers or spandrels before using this command, data is printed only for the selected elements.

9. If desired, revise the wall pier and/or spandrel overwrites, rerun the shear wall design, and review the results again. Repeat this step as many times as needed.
10. If desired, create wall pier check sections with user-defined (actual) reinforcing specified for the wall piers using the Section Designer utility. Use the Design menu > Shear Wall Design > Define General Pier Sections command to define the sections in Section Designer. Use the Design menu > Shear Wall Design > Assign Pier Section Type command to assign the sections to the piers; be sure to indicate that the reinforcing is to be checked by checking the Reinforcement to Be Checked check box. Rerun the design and verify that the actual flexural reinforcing provided is adequate.
11. Assign these check sections to the piers, change the pier mode from Design to Check, and rerun the design. Verify that the actual flexural reinforcing provided is adequate.
12. If necessary, revise the geometry or reinforcing and rerun the design.
13. Print or display selected shear wall design results if desired.

Note: Shear wall design is performed as an iterative process. You can change your wall design dimensions and reinforcing during the design process without rerunning the analysis. However, you always want to be sure that your final design is based on analysis properties (wall dimensions) that are consistent with your design (actual) wall dimensions.

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FIGURES SHOWING THE LOCATION OF SHEAR WALL

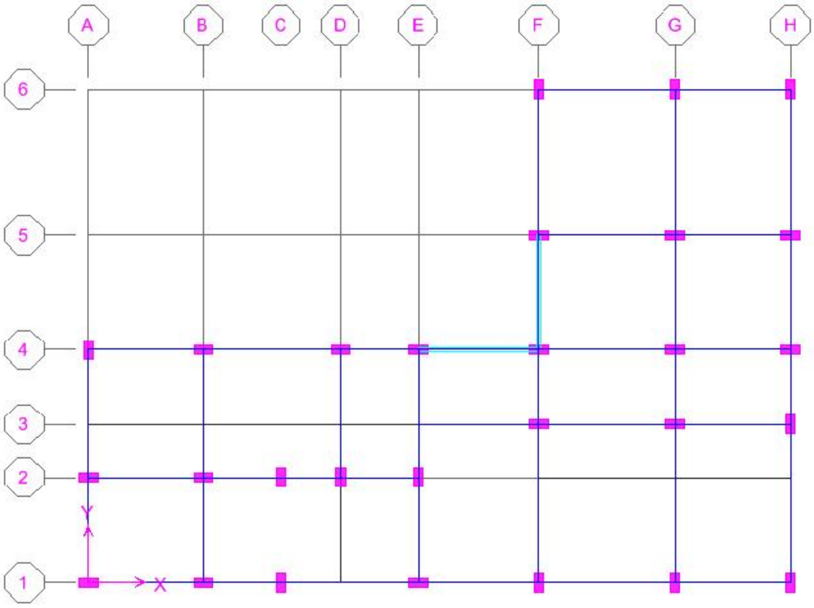


Fig: Shear wall location in plan

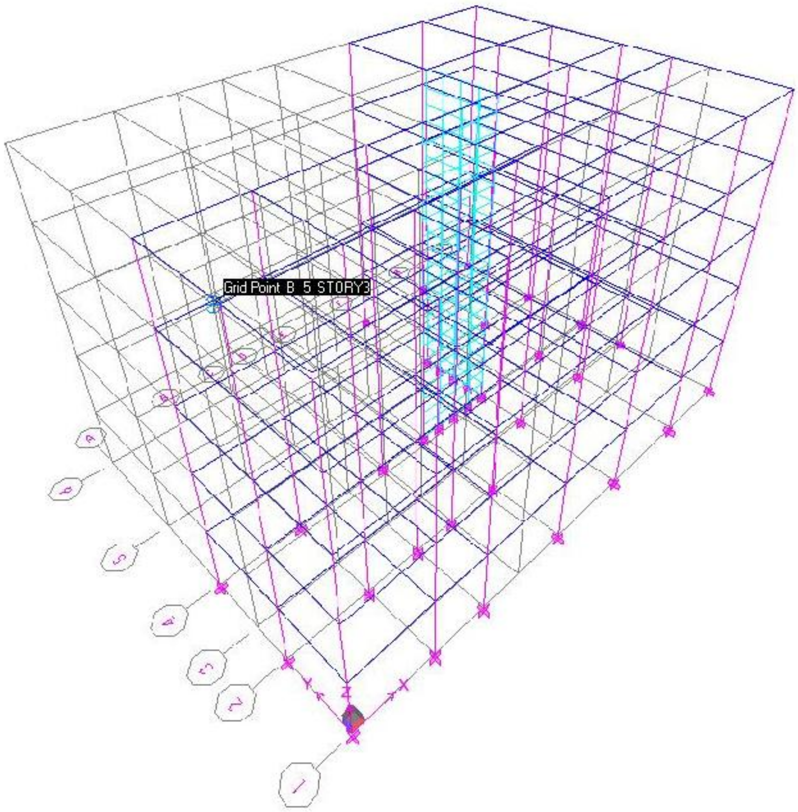
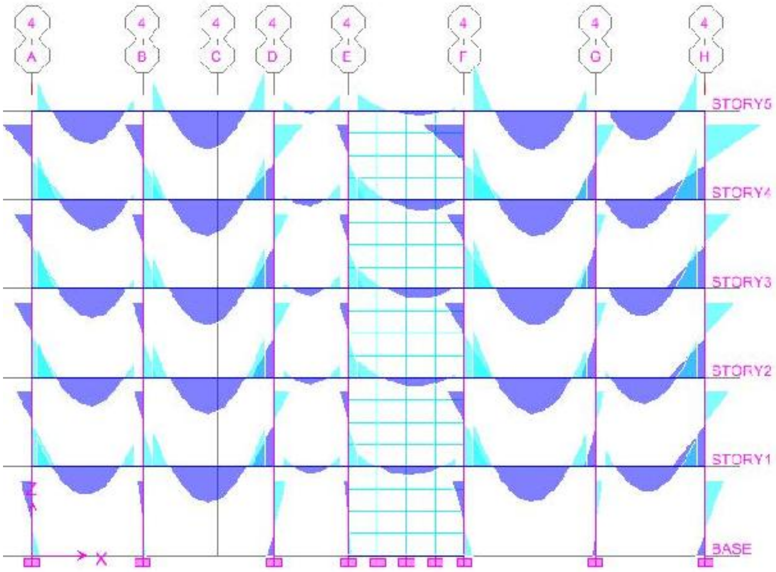


Fig: Shear wall location in 3D

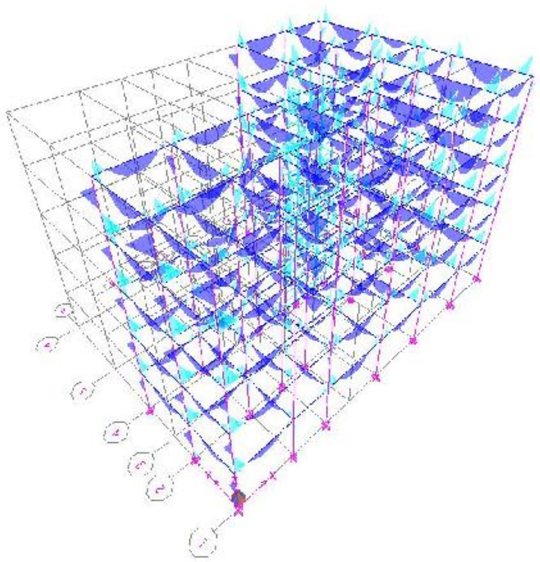
At each floor
16d@0.25EF(D)
16d@corners(0.031)

DESIGN OF THE STRUCTURE

The design has been carried out in the software itself. ETABS has been used for design of the structure. Also verification for the design and analysis has been checked. Then the rebar percentage is checked. Bending moment of the structure is shown in the figure.



Bending Moment Diagram _ Elevation



Bending Moment Diagram _ 3D view

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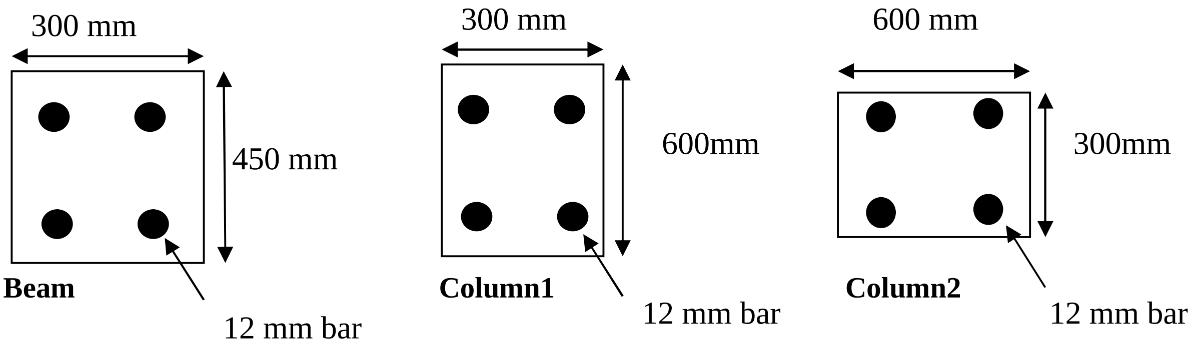
Element	Beam	Column
Longitudinal Reinforcement	**4-12mm Dia bars**	**4-16mm dia**
Transverse Reinforcement	**10 mm dia bars @ 150 c/c**	**10 mm dia bars @ 150 c/c**



PUSHOVER ANALYSIS

Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern. Static pushover analysis is an attempt by the structural engineering profession to evaluate the real strength of the structure and it promises to be a useful and effective tool for performance based design. The ATC-40 (1996) documents have developed modeling procedures, acceptance criteria and analysis procedures for pushover analysis. These documents define force-deformation criteria for hinges used in pushover analysis. As shown in Figure 1, five points labeled A, B, C, D, and E are used to define the force-displacement behavior of the hinge and three points labeled IO, LS, and CP are used to define the acceptance criteria for the hinge. The IO, the LS and the CP stand for Immediate Occupancy, Life safety and Collapse Prevention, respectively. These are informational measures that are reported in the analysis results and used for performance-based design. Figure 2 illustrates a typical representation of capacity curve of RC structure containing shear wall. It is clear to be observed that the pattern of the experimental cure is continuous and is different from the general Skelton model shown in Fig. 1. As shown in that figure, the response is linear to an effective crack point, B, followed by cracking (with concrete cracking) to yield point C, followed by yielding (possibly with strain hardening) to ultimate point D, followed by final collapse and loss of gravity load capacity at point E.

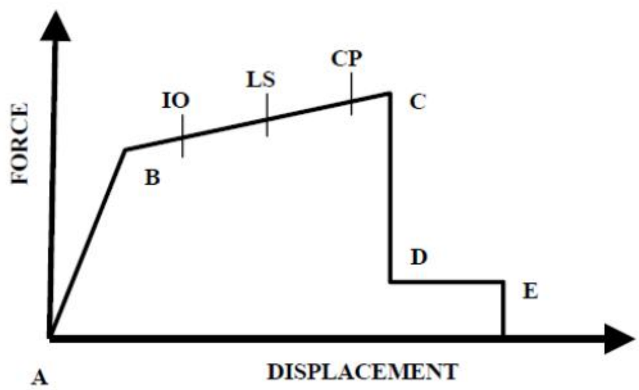
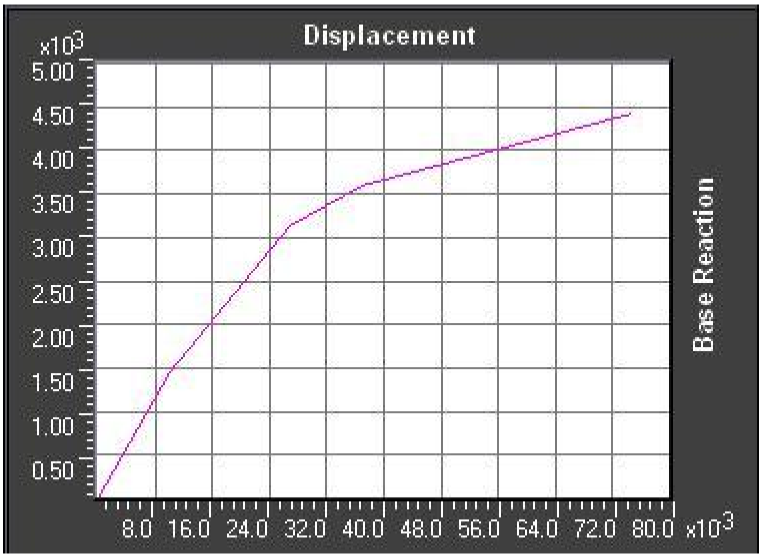


Fig: Force-displacement curve defined for the plastic hinge in the pushover analysis

Static pushover curve:



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ETABS Hinges Results											
STEP	DISPLACEMENT	BASE FORCE	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	TOTAL
0	0	0.0000	571	1	0	0	0	0	0	0	572
1	0.0091	1516.7551	531	35	6	0	0	0	0	0	572
2	0.0239	3190.8799	497	42	33	0	0	0	0	0	572
3	0.0347	3712.8330	468	37	16	46	0	5	0	0	572
4	0.0677	4479.9106	468	37	16	46	0	2	3	0	572
5	0.0528	1989.9036	572	0	0	0	0	0	0	0	572

CONCLUSIONS

In the structure without shear wall the lateral forces are high. To reduce the lateral forces, shear walls are added which results reduction of lateral forces in all frames except shear wall frames. And the appropriate location for the shear walls where the lateral forces are maximum in the frames there we can add the shear walls to reduce the forces in all frames. Therefore, we can choose the appropriate location for the shear walls in the less lateral forces frames.

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