COMPARATIVE EFFECT OF INFILL WALL ON RCC TALL BUILDING

Jaiswal Rana Narendra, *Student M.E (Structure), Datta Meghe College of Engineering, Airoli*

Dr. P.A. Dode, Principal and *Professor, Datta Meghe College of Engineering, Airoli*

P. R. Barbude, *Professor, Datta Meghe College of Engineering, Airoli*

Abstract - In the structure design, Masonry infills are normally considered as non-structural elements and their contribution in the stiffness of structure is generally ignored in practice, as in reality the structures also possess masonry infills within most of the frames, but they are ignored in the models so as to minimize the computational work, this assumption can lead to an unsafe design. The infill wall though constructed as a secondary element of a structure behaves as an integral part of the structural system and also determines the behavior and response of the structure, especially when the structure is subjected to lateral loads.

Studies have shown that frames with masonry panels are considerably stiffer than bare frames. Brick infills increase the structure's strength by resisting the lateral deflection of frames subjected to horizontal forces. The infills stiffen the frames, and researchers are still interested in determining the actual shear that occurs at the wall's termination point. The windward side column fails due to the shear force generated at the points where the wall terminates within the frame on the windward side. Equivalent strut width calculations, performed in accordance with Indian Standard Code guidelines, are incorporated into frame analysis for infilled walls.

***Index Terms*— Lateral loads, response spectrum, Equivalent strut, ETABS.**

I. INTRODUCTION

A large number of reinforced concrete and steel buildings are constructed with masonry infills. Masonry infills are often used to fill the voids between the vertical and horizontal resisting elements of the building frames with the assumption that these infills will not take part in resisting any kind of load either axial or lateral; hence its significance in the analysis of frame is generally neglected. Moreover, non-availability of realistic and simple analytical models of infill becomes another hurdle for its consideration in analysis. In fact, an infill wall enhances considerably the strength and rigidity of the structure. It has been recognized that frames with infills have more strength and rigidity in comparison to the bare frames and their ignorance has become the cause of failure of many of the multi-storied buildings.

Recent studies have shown that the use of masonry infill panels significantly affects the strength and stiffness and energy dissipation mechanism of the overall structure. Neglecting the effects of masonry infill can lead to inadequate assessment of structural damage of infill frame structures subjected to intense ground motions.

The use of a masonry infill to brace a frame combines some of the desirable structural characteristics of each, while overcoming some of their deficiencies. As the effect of brick infills on frames, the high in-plane rigidity of the masonry wall significantly stiffens the structure, otherwise the frame becomes relatively flexible. On the other side, the ductile frame contains the brittle masonry, after cracking, up to loads and displacements much larger than it could achieve without the frame. The result is, therefore a relatively stiff and tough bracing system. The wall braces the frame partly by its in-plane shear resistance and partly by its behaviour as a diagonal bracing strut in the frame.

The nature of the forces in the frame can be understood by referring to an analogous braced frame. The windward column or the column facing earthquake load first, is in tension and the leeward column or the other side of the building facing earthquake load last, is in compression. Since the infill bears on the frame not as a concentrated force exactly at the corners, but over short lengths of the beam and column adjacent to each compression corner, the frame members are subjected also to transverse shear and a small amount of bending. Consequently, the frame members or their connections are liable to fail by axial force or shear, and especially by tension at the base of the windward column.

►Infilled Walls

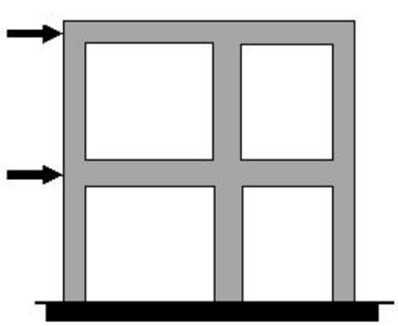
Infill walling is the generic name given to a panel that is built in between the floors of the primary structural frame of a building in other words Infill panel walls are a form of cladding built between the structural members of a building. The structural frame provides support for the cladding system, and the cladding provides separation of the internal and external environments. Infill walls are considered to be non-load bearing, but they resist wind loads.

Functional requirements for infill panel walls include:

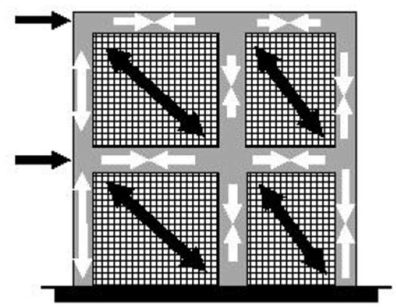
- They are self-supporting between structural framing members.
- They provide weather-resistance.
- They provide thermal and sound insulation.
- The provide fire resistance.
- They provide sufficient openings for natural ventilation and glazing.

►Influence of masonry infill walls:

Infills interfere with the lateral deformations of the RC frame; separation of frame and infill takes place along one diagonal and a compression strut forms along the other. Thus, infills add lateral stiffness to the building. The structural load transfer mechanism is changed from frame action to predominant truss action (Figure 1.1); the frame columns now experience increased axial forces but with reduced bending moments and shear forces.



(a) Frame action in bare frame



(b) Predominant action in infilled frame

Figure 1.1 Change in the lateral load transfer mechanism owing to inclusion of masonry infill walls.

The mode of failure of an infilled building depends on the relative strengths of frame and infill (Table 1.1).

Table 1.1: Modes of failure of masonry infilled RC frames

Description	Weak Infill	Strong Infill
Weak Frame	-	‣ Diagonal cracks in infill ‣ Plastic hinges in columns
Frame with Weak Joints and Strong Members	‣ Corner crushing of infills ‣ Cracks in beam-column joints	‣ Diagonal cracks in infill ‣ Cracks in beam-column joints
Strong Frame	Horizontal sliding in infills	-

In a bare frame, inelastic effects in RC frame members and joints cause energy dissipation, while in an infilled frame, inelastic effects in infills also contribute to it. Thus, energy dissipation in an infilled frame is higher than that in the bare frame. If both frame and infill are detailed to be ductile, then stiffness degradation and strength deterioration under cyclic loading are nominal. However, if inelastic effects are brittle in nature (e.g., cracking of infill, bond slip failure in frame, or shear failure in frame members), the drop in strength and stiffness under repeated loading may be large.

►Macro-modelling of masonry infill

Since the first attempts to model the response of the composite infilled-frame structures, experimental and conceptual observations have indicated that a diagonal strut with appropriate geometrical and mechanical characteristics could possibly provide a solution to the problem (Fig. 1.2).

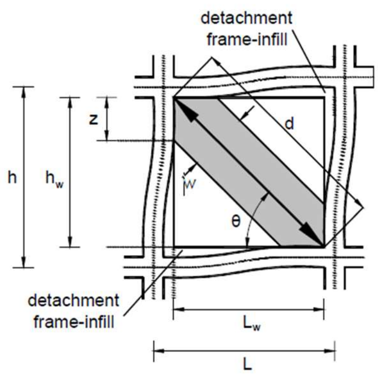


Figure 1.2 Masonry infill frame sub-assemblage

►Single-strut models:

In the early sixties, Polyakov suggested the possibility of considering the effect of the infilling in each panel as equivalent to diagonal bracing, and this suggestion was later adopted by Holmes, who replaced the infill by an equivalent pin-jointed diagonal strut made of the same material and having the same thickness as the infill panel and a width defined by

$$\frac{w}{d} = \frac{1}{3}$$

where d is the diagonal length of the masonry panel. The "one-third" rule was suggested as being applicable irrespective of the relative stiffness of the frame and the infill. One year later, Stafford Smith, based on experimental data from a large series of tests using masonry infilled steel frames, found that the ratio w d varied from 0.10 to 0.25. On the second half of sixties Stafford Smith and his associates using additional experimental data related the width of the equivalent diagonal strut to the infill/frame contact lengths using an analytical equation, which has been adapted from the equation of the length of contact of a free beam on an elastic foundation subjected to a concentrated load [30]. They proposed the evaluation of the equivalent width λh as a function of the relative panel-to-frame-stiffness parameter, in terms of

$$\lambda_h = h\sqrt[4]{\frac{E_w t_w \sin 2\theta}{4EIh_w}}$$

where w E is the modulus of elasticity of the masonry panel, EI is the flexural rigidity of the columns, tw the thickness of the infill panel and equivalent strut, h the column height between centerlines of beams, hw the height of infill panel, and θ the angle, whose tangent is the infill height-to-length aspect ratio, being equal to

$$\theta = \tan^{-1}\left(\frac{h_w}{L_w}\right)$$

in which w L is the length of infill panel (all the above parameters are explained in Fig. 1.2).
Based on experimental and analytical data Mainstone proposed an empirical equation for the calculation of the equivalent strut width, given by

$$\frac{w}{d} = 0.16\lambda_h^{-0.3}$$

Mainstone and Weeks and Mainstone, also based on experimental and analytical data, proposed an empirical equation for the calculation of the equivalent strut width:

$$\frac{w}{d} = 0.175\lambda_h^{-0.4}$$

This formula was included in FEMA-274 (Federal Emergency Management Agency 1997) for the analysis and rehabilitation of buildings as well as in FEMA-306 (Federal Emergency Management Agency 1998), as it has been proven to be the most popular over the years. This equation was accepted from the majority of researchers dealing with the analysis of infilled frames.

►Effect of openings in the lateral stiffness of infill walls:

Infill walls with openings is mostly analytical, restricted to special cases, and as such cannot provide rigorous comparison to actual cases because of its focus on specific materials used and specific types of openings. It is worth noting that the contribution of the infill wall to the frame lateral stiffness is much reduced when the structure is subjected to reversed cyclic loading, as in real structures under earthquake conditions.

II. OBJECTIVE OF THESIS

- A parametric study has been carried out on the RCC multistorey building for its structural responses.
- To study the effect of infilled walls on the overall struture with all IS provision.
- The major objectives of the research work are as follows:
    - To find out the influence of masonry infill wall panel in Reinforced Concrete framed Structures in terms of deformation.
    - To study the behaviour of frame with brick masonry infill by modeling masonry infill as a diagonal strut. The ETABS is to be used for the development of the model.
    - To find the comparative results of experimental models by considering parameters such as Mode shapes, lateral deflection, drift, bending moment, axial force and shear force.

III. THESIS DENITION

The main aim of the thesis is to conduct a study that will involve the finite element analysis of the behaviour of a High-Rise reinforced concrete (R.C.) frame with brick masonry infill.

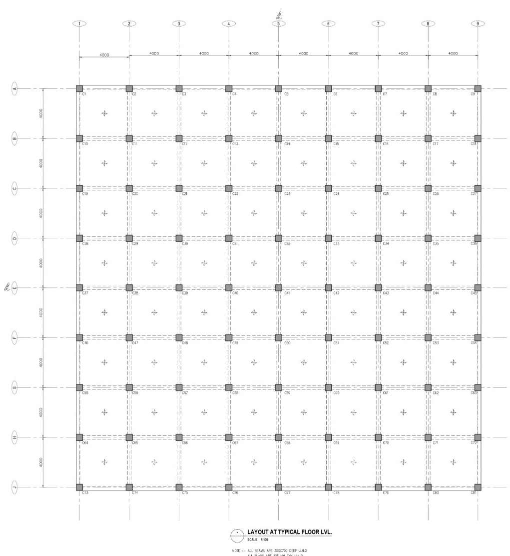


Figure 3.1 Floor plan

Table 3.1 Salient Features of building

Building description	
Building Type	Residential
Length in X direction (b)	32 m
Length in Y direction (d)	32 m
No. of Floors	GL + 30 Typical + Terrace
Height of Building (Foundation to terrace)	97 m

Material	
Concrete	
Floor Description	**Walls**
Ground to 5th floor	M70
5th floor to 10th floor	M60
10th floor to 15th floor	M50
15th floor to 20th floor	M40
20th floor to terrace floor	M30

Floor Description	**Beams / Slabs**
Ground to 5th floor	M50
6th floor to 10th floor	M45
11th floor to 15th floor	M40
16th floor to 20th floor	M30
21st floor to terrace floor	M30

Steel	
Grade in Beam / Slab	Fe 500
Grade in Walls	Fe 500

Seismic Data	
Location	Mumbai
Zone Factor	0.16
Importance factor	1.5
Framing type	SMRF
Response Reduction Factor	4
Soil Type	1 (Hard)

Wind Data	
Location	Mumbai
Basic Wind speed	44
Terrain category	3
Struture class	1
Risk coefficient	1
Topography factor	1

Table 3.2 Load calculation for Beams

BEAMS						
Floor	Height	Beam/SLAB Depth	Thickness of wall	Wall type	Density	Load
Typical floors	3.00 m	0.7 m	150 mm	RCC	25	8.6 Kn/m
Terrace floor	1.6 m	0.0 m	150 mm	RCC PARDI	25	6 Kn/m

Table 3.3 Load description for Slab

SLABS		
Description	SDL	L.L.
Bedroom / Study Living Room / kitchen	1.5	2
Terrace	5	3

Table 3.4 Sizes description

Beam	300x700 U.N.O
Slab	125mm Thk.
Column	500 x 500

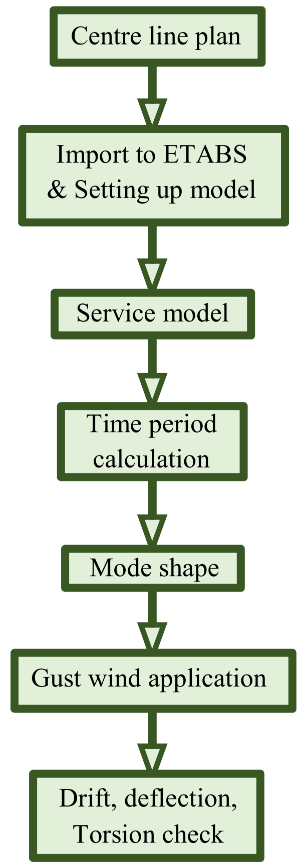
Table 3.5 Load combinations

1	D 1.5 DL + 1.5 LL
2	D 1.5 DL + 1.5 LL ± 1.5 TR
3	D 0.8 DL ± 1.5 RSX / RSY
4	D 0.8 DL ± 1.5 RSZX / RSZY
5	D 0.9 DL ± 1.5 RSX / RSY
6	D 0.9 DL ± 1.5 RSZX / RSZY
7	D 1.2 DL + 1.2 LL ± 1.2 RSX / RSY
8	D 1.2 DL + 1.2 LL ± 1.2 RSZX / RSZY
9	D 1.5 DL ± 1.5 RSX / RSY
10	D 1.5 DL ± 1.5 RSZX / RSZY
11	D 0.9 DL ± 1.5 (WSX+0.75WDX-0.75WCX)
12	D 0.9 DL ± 1.5 (WSX+0.75WDX+0.75WCX)
13	D 0.9 DL ± 1.5 (WSX+WCX)
14	D 0.9 DL ± 1.5 (WSX+WDX)
15	D 0.9 DL ± 1.5 (WSX-WCX)
16	D 0.9 DL ± 1.5 (WSY+0.75WDY-0.75WCY)
17	D 0.9 DL ± 1.5 (WSY+0.75WDY+0.75WCY)
18	D 0.9 DL ± 1.5 (WSY+WCY)
19	D 0.9 DL ± 1.5 (WSY+WDY)
20	D 0.9 DL ± 1.5 (WSY-WCY)
21	D 1.2 DL + 1.2 LL ± 1.2 (WSX+0.75WDX-0.75WCX)
22	D 1.2 DL + 1.2 LL ± 1.2 (WSX+0.75WDX+0.75WCX)
23	D 1.2 DL + 1.2 LL ± 1.2 (WSX+WCX)
24	D 1.2 DL + 1.2 LL ± 1.2 (WSX+WDX)
25	D 1.2 DL + 1.2 LL ± 1.2 (WSX-WCX)
26	D 1.2 DL + 1.2 LL ± 1.2 (WSY+0.75WDY-0.75WCY)
27	D 1.2 DL + 1.2 LL ± 1.2 (WSY+0.75WDY+0.75WCY)
28	D 1.2 DL + 1.2 LL ± 1.2 (WSY+WCY)
29	D 1.2 DL + 1.2 LL ± 1.2 (WSY+WDY)
30	D 1.2 DL + 1.2 LL ± 1.2 (WSY-WCY)
31	D 1.5 DL ± 1.5 (WSX+0.75WDX-0.75WCX)
32	D 1.5 DL ± 1.5 (WSX+0.75WDX+0.75WCX)
33	D 1.5 DL ± 1.5 (WSX+WCX)
34	D 1.5 DL ± 1.5 (WSX+WDX)
35	D 1.5 DL ± 1.5 (WSX-WCX)
36	D 1.5 DL ± 1.5 (WSY+0.75WDY-0.75WCY)
37	D 1.5 DL ± 1.5 (WSY+0.75WDY+0.75WCY)
38	D 1.5 DL ± 1.5 (WSY+WCY)
39	D 1.5 DL ± 1.5 (WSY+WDY)
40	D 1.5 DL ± 1.5 (WSY-WCY)

Table 3.6 Modifiers

Service								
Elements stiffness modifier	Structural Walls	Retaining Walls	Spandrel beam	Slab	Frames stiffness modifier	Columns		Beam
						Frame	Gravity	
F11	1	1	0.7	0.35	Area	1	1	1
F22	0.9	0.9	0.7	0.35	As2	1	1	1
F12	1	1	0.7	0.35	As3	1	1	1
M11	0.9	0.9	0.7	0.35	T	0.001	0.001	0.001
M22	0.9	0.9	0.7	0.35	I22	0.9	0.1	0.7
M12	0.9	0.9	0.7	0.35	I33	0.9	0.1	0.7
V13	1	1	1	1				
V23	1	1	1	1				

IV. MODELLING AND ANALYSIS



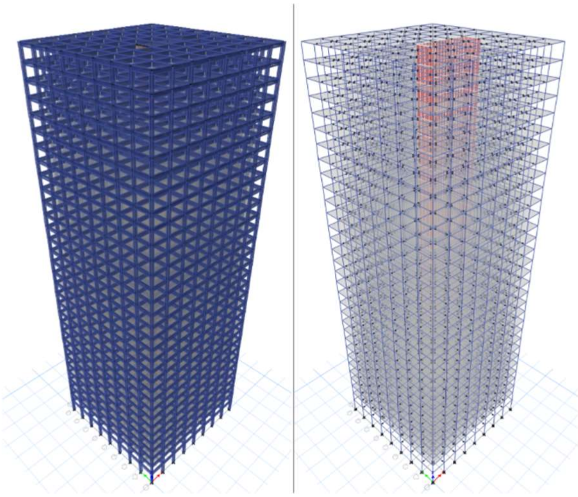


Figure 4.1 3D View of ETABS model

►Analysis data and validation

1.0	**BUILDING DESCRIPTION**				
1.1	Building Type	**RESIDENTIAL**			
1.2	Length in X direction (b)	32.00 m			
1.3	Length in Y direction (d)	32.00 m			
1.4	No. of Floors	**GL+30 TYPICAL+TERRACE**			
1.5	Height of Building	97.00 m	**FOUNDATION TO TERRACE**		
1.1	**TIME PERIOD CALCULATION**				
	Tx = 0.09h/√b	1.54	sec		
	Ty = 0.09h/√b	1.54	sec		
2.0	**GRAVITY LOAD**				
2.10	All floors DL	275219.00	KN		
2.20	All floors SDL	199824.00	KN	Built Up Area	**31744**
2.30	All floors LL	62592.00	KN		
2.40	All floors LL > 3	0.00	KN	Load Intensity	15.458
2.50	All floors NRLL		KN		
2.60	% LLR considered For EQ		%		
2.70	Total seismic weight				
	DL+LL*0.5	490691.00			
	ρb (Building density)	**515.26 Kg/m3**			
3.00	**SEISMIC LOAD**				
3.1	Zone Factor	0.16			
3.2	Importance factor	1.2			
3.3	Framing type	SMRF			
3.4	Response Reduction Factor	4			
3.5	Soil Type	1			
3.6	**EQx**				
	Tx =	**1.543**	sec	FROM TIME PERIOD CALCULATION	
	(Sa/G)x	0.6480			
	Vbx = (z*I* sa / 2* R *G) Wi =	0.0156	*Wi =	7630.975859	
	From ETABS:	7628.5870	<	7630.975859	***SAFE***
				7655.975859	
	Hence use Vbx =	7630.98	KN	10823.7809	0.71
3.7	**EQy**				
	Ty =	**1.543**	sec	FROM TIME PERIOD CALCULATION	
	(Sa/G)y	0.6480			
	Vby = (z*I* sa / 2* R *G) Wi =	0.0156	*Wi =	7630.975859	
	From ETABS:	7628.5870	<	7630.975859	***SAFE***
				7655.975859	
	Hence use Vby =	7630.98	KN	9295.5058	0.82
3.8	Response spectrum bace reaction				
	RSx (Above ground floor) Fx =	3260.34 KN	**Fy**	494.74 KN	
	Scale Factor (Vbx / Fx)	**2.34**			
	Rsy (Above ground floor) Fx =	469.77 KN	**Fy**	2758.08 KN	
	Scale Factor (Vby / Fy)	**2.77**			

3.9	Vertical Seismic Acceleration as per Cl.6.4.6 of IS1893 (Part 1):2016					
		=	(2/3)*(z/2)*(2.5) / (R/I)	= **0.04**		
	RSz	**19627.64**				
	RSz from ETABS	**14983.31**				
	Scale Factor should be max of	RSX	2.3405			
		RSY	2.7668			
		RSZ	1.3100			
		RSZ Scale Factor	2.7668			
3.10	Check ratio of max and min storey deflection at Terrace in Service model in Pure EQX & EQY					
	Ratio of deflection in EQX	**1.0739**	OK	170.00 Max	158.30 Min	
	Ratio of deflection in EQY	**1.0027**	OK	188.00 Max	187.50 Min	
3.11	Modes contribution in Service Model					
		Time Period	**UX**	**UY**	**RZ**	
	Mode 1	4.831	0.0000	0.7517	0.0000	
	Mode 2	4.351	0.6602	0.0000	0.0550	
	Mode 3	3.972	0.0439	0.0000	0.7608	
			70.41	**75.17**	**81.58**	
	Ratio of 1st & 2nd mode time period	1.1103	OK			
	Ratio of 2nd & 3rd mode time period	1.0954	OK			
	First 3 mode summation in X Direction	**86.7**	OK			
	First 3 mode summation in Y Direction	**92**	OK			
	Total mode summation in X Direction	**99.43**	OK			
	Total mode summation in Y Direction	**99.77**	OK			
3.12	Check drift & defection in Strength for unscaled Dynamic EQ					
	max. defl. at roof lvl (δx) =		*FROM SERVICE MODEL*	**77**	mm	
	H/δx			1259.74	>250	***SAFE***
	max. defl. at roof lvl (δy) =		*FROM SERVICE MODEL*	**106**	mm	
	H /δy			915.09	>250	***SAFE***
	max. interstorey drift (dx) =			**0.000968**	<0.004	***SAFE***
	max. interstorey drift (dy) =			**0.001338**	<0.004	***SAFE***
4.0	**WIND LOAD**					
	Check drift & defection in Service model for 0.8 times wind load			Below values are	for 50 year return period	
4.1	Basic Wind speed			44	m/s	
4.2	Terrain category			3		
4.3	Structure class			1		
4.4	Risk coefficient			1		
4.5	Topography factor			1		
4.6	Wind base shear in X dir			5336.1806	KN	
4.7	Wind base shear in Y dir			5336.1806	KN	
4.8	Gust Factor in X Dir			**2.730**	Refer Gust Wind Cal	
4.9	Gust Factor in Y Dir			**2.790**	Refer Gust Wind Cal	
4.10	Max. Deflection at roof lvl (δx)			**86.46**	mm	
	H/δx			1121.91	>500	***SAFE***
4.11	Max. Deflection at roof lvl (δy)			**116**	mm	
	H /δy			836.21	>500	***SAFE***
4.12	max. interstorey drift (dx) =			**0.0011**	<0.0025	***SAFE***
4.13	max. interstorey drift (dy) =			**0.001465**	<0.0025	***SAFE***

►Calculation of equivalent diagonal strut

► Calculation of equivalent diagonal strut

Fck	=	30	mpa		
Ec	=	5000*Sqrt(Fck)	=	27386	mpa
Beam size	=	300mm x 300 mm			
Column size	=	500mm x 500 mm			
fm	=	30	mpa		
f_b	=	10.5	mpa		*Compressive strength of masonary brick*
f_{m0}	=	17.5	mpa		*Compressive strength of motor*
fm	=	$0.433 f_b^{0.64} f_{mo}^{0.36}$	=	5.5	mpa
Em	=	500*fm			*Modulus of elasticity of masonary brick*
	=	2732.2	mpa		
Thickness of infill =			230	mm	
Height of infill =			3000-300		
			2700	mm	
Length of infill =			3000-500		
			2500	mm	
M.I. of column =			$500^4/12$		
			5.21E+09	mm^4	
$\theta = \tan^{-1}\frac{h}{l}$		=	0.823		
Lds		=	h/sinθ		
		=	3682.5	mm	
$\alpha_h = h\left(\sqrt[4]{\frac{E_m t \sin 2\theta}{4E_f I_c h}}\right)$		=	2.16		
$w_{ds} = 0.175\alpha_h^{-0.4} L_{ds}$					*Equivalent diagonal strut*
w_{ds}		=	460.38	mm	= 475.00 mm

So, size of equivalent diagonal strut is 230 mm x 475 mm.

V. RESULTS AND DISCUSSION

A comprehensive three-dimensional structural model has been prepared in ETABS software, encompassing all gravity and lateral force-resisting components. The model incorporates P-Delta effects, stiffness modifiers, and section property adjustments as outlined in Section 4.

► Modal participating mass ratio

Model participation mass ratio indicates the percentage of the structural mass of the model participating in a given direction and mode.

A summary of the periods and mass participation of the first three modes of the building options are provided in Table 5.1 and 5.2 From the summary, it is found that when we model the strut in 3D modal then the modal mass time period decreases. This means that equivalent diagonal structures will increase the stiffness of the building.

Table 5.1 Time Periods and Modal Mass Participation Ratios for Service Model

TABLE: Modal Participating Mass Ratios														
Case	Mode	Period sec	UX	UY	UZ	SumUX	SumUY	SumUZ	RX	RY	RZ	SumRX	SumRY	SumRZ
Modal	1	**4.831**	0.000	**0.752**	0.000	0.000	0.752	0.000	0.227	0.000	0.000	0.227	0.000	0.000
Modal	2	**4.351**	**0.660**	0.000	0.000	0.660	0.752	0.000	0.000	0.250	0.055	0.227	0.250	0.055
Modal	3	**3.972**	0.044	0.000	0.000	0.704	0.752	0.000	0.000	0.021	**0.761**	0.227	0.270	0.816
Modal	4	1.538	0.000	0.122	0.000	0.704	0.874	0.000	0.385	0.000	0.000	0.612	0.270	0.816
Modal	5	1.4	0.008	0.000	0.000	0.712	0.874	0.000	0.000	0.019	0.093	0.612	0.289	0.909
Modal	6	1.187	0.144	0.000	0.000	0.856	0.874	0.000	0.000	0.293	0.007	0.612	0.582	0.916
Modal	7	0.82	0.002	0.000	0.000	0.858	0.874	0.000	0.000	0.003	0.033	0.612	0.585	0.948
Modal	8	0.814	0.000	0.046	0.000	0.858	0.920	0.000	0.078	0.000	0.000	0.689	0.585	0.948
Modal	9	0.559	0.063	0.000	0.000	0.921	0.920	0.000	0.000	0.111	0.001	0.689	0.696	0.949
Modal	10	0.521	0.000	0.026	0.000	0.921	0.946	0.000	0.076	0.000	0.000	0.765	0.696	0.949
Modal	11	0.365	0.000	0.016	0.000	0.921	0.962	0.000	0.032	0.001	0.000	0.797	0.697	0.949
Modal	12	0.327	0.036	0.001	0.001	0.957	0.963	0.001	0.002	0.096	0.000	0.799	0.792	0.950
Modal	13	0.321	0.000	0.000	0.671	0.957	0.963	0.672	0.007	0.000	0.000	0.806	0.792	0.950
Modal	14	0.305	0.000	0.000	0.079	0.957	0.963	0.751	0.063	0.000	0.000	0.869	0.793	0.950
Modal	15	0.259	0.005	0.015	0.001	0.962	0.977	0.751	0.036	0.013	0.000	0.905	0.805	0.950
Modal	16	0.236	0.000	0.000	0.048	0.962	0.977	0.799	0.000	0.000	0.000	0.905	0.805	0.950
Modal	17	0.203	0.030	0.003	0.000	0.992	0.981	0.799	0.010	0.076	0.002	0.914	0.881	0.951
Modal	18	0.15	0.002	0.012	0.017	0.994	0.992	0.816	0.029	0.004	0.000	0.944	0.885	0.951
Modal	19	0.146	0.000	0.006	0.039	0.994	0.998	0.855	0.018	0.001	0.001	0.961	0.885	0.952
Modal	20	0.088	0.000	0.000	0.113	0.994	0.998	0.967	0.000	0.000	0.000	0.961	0.886	0.952

Table 5.2 Time Periods and Modal Mass Participation Ratios for Service Model with STRUT model

TABLE: Modal Participating Mass Ratios														
Case	Mode	Period sec	UX	UY	UZ	SumUX	SumUY	SumUZ	RX	RY	RZ	SumRX	SumRY	SumRZ
Modal	1	**1.683**	0.273	**0.351**	0.000	0.273	0.351	0.000	0.192	0.150	0.018	0.192	0.150	0.018
Modal	2	**1.563**	**0.357**	0.279	0.001	0.629	0.630	0.001	0.152	0.194	0.000	0.344	0.345	0.018
Modal	3	**0.468**	0.057	0.077	0.000	0.686	0.707	0.001	0.042	0.025	**0.424**	0.386	0.369	0.441
Modal	4	0.419	0.102	0.089	0.068	0.788	0.796	0.070	0.138	0.140	0.003	0.524	0.509	0.445
Modal	5	0.344	0.034	0.028	0.000	0.822	0.824	0.070	0.079	0.092	0.339	0.603	0.601	0.783
Modal	6	0.311	0.006	0.006	0.586	0.828	0.831	0.656	0.000	0.003	0.000	0.603	0.604	0.783
Modal	7	0.298	0.001	0.000	0.000	0.829	0.831	0.656	0.000	0.007	0.000	0.603	0.610	0.784
Modal	8	0.291	0.001	0.000	0.048	0.829	0.831	0.704	0.010	0.000	0.001	0.613	0.611	0.784
Modal	9	0.276	0.001	0.000	0.000	0.830	0.831	0.704	0.000	0.002	0.000	0.613	0.613	0.785
Modal	10	0.239	0.000	0.001	0.022	0.830	0.832	0.726	0.000	0.001	0.000	0.614	0.614	0.785
Modal	11	0.232	0.002	0.003	0.046	0.832	0.834	0.772	0.000	0.000	0.000	0.614	0.614	0.785
Modal	12	0.209	0.001	0.064	0.000	0.833	0.898	0.772	0.116	0.000	0.005	0.729	0.614	0.790
Modal	13	0.204	0.064	0.000	0.000	0.897	0.898	0.772	0.000	0.115	0.005	0.730	0.728	0.795
Modal	14	0.182	0.001	0.001	0.014	0.898	0.899	0.786	0.001	0.003	0.000	0.731	0.731	0.795
Modal	15	0.137	0.004	0.049	0.000	0.902	0.948	0.786	0.111	0.008	0.000	0.842	0.739	0.795
Modal	16	0.132	0.054	0.008	0.000	0.956	0.956	0.786	0.018	0.126	0.000	0.860	0.865	0.795
Modal	17	0.119	0.001	0.001	0.102	0.958	0.956	0.887	0.001	0.001	0.000	0.861	0.866	0.795
Modal	18	0.085	0.009	0.024	0.000	0.967	0.981	0.887	0.070	0.027	0.002	0.931	0.893	0.798
Modal	19	0.079	0.020	0.006	0.000	0.987	0.987	0.887	0.016	0.054	0.000	0.947	0.947	0.798
Modal	20	0.062	0.002	0.002	0.081	0.989	0.988	0.968	0.003	0.005	0.000	0.950	0.952	0.798

► Lateral Story drift and deflection

Story drift is the lateral displacement of a floor relative to the floor below. Story drift is the horizontal movement of a building or structure due to the action of external forces, such as wind or earthquake.

Story displacement is the deflection of a single-story relative to the base or ground level of the structure. Intuitively, we can expect higher total displacement values as we move up the structure. So, a graph showing the story displacement vs. the height of the structure looks exactly like the deflected shape.

All the elements comfortably meet the IS acceptance requirements. The drift are within the acceptable range (refer Figure 5.1, 5.2, 5.3, 5.4). As per IS 16700:2017 For earthquake load (factored) combinations the drift shall be limited to hi/250 i.e 0.004. and for wind load (unfactored) combinations the drift shall be limited to hi/400 i.e 0.0025 and The deflection is within the acceptable range (refer Figure 5.6, 5.6, 5.7, 5.8). As per IS 16700:2017 For earthquake load (factored) combinations the deflection shall be limited to hi/250. and for wind load (unfactored) combinations the deflection shall be limited to hi/500.

From the graph, it is found that the model with a strut model has fewer drift/ deflection values compared to the standard 3D model. This is because the strut model's increased stiffness and reduces story drift.

► Lateral Story deflection

Forces refer to the internal forces acting on a column within a structural model, including the axial compressive or tensile force, shear forces, and moments. Here Force and moment distribution for story 8 to story 20 is provided in figure 5.9, 5.10 5.11 and 5.12.

From the comparison (Column C68 at Story 8) it is found that as we model equivalent diagonal strut in analysis model the load transfer mechanism changes from frame action to truss action and there is a definite change in the form in which the frame will resist lateral loads; flexural effects will decrease substantially. There is a drastic change in bending moment, shear force and axial force.

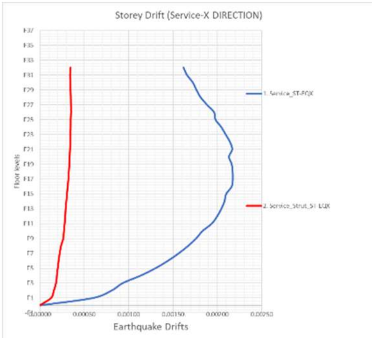


Figure 5.1 Earthquake story drift in X- Direction

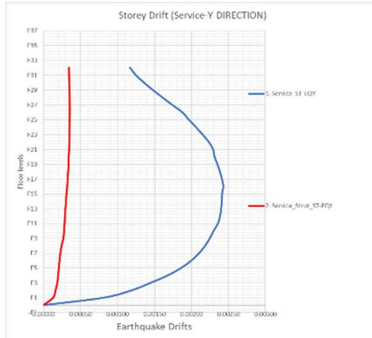


Figure 5.2 Earthquake story drift in Y- Direction

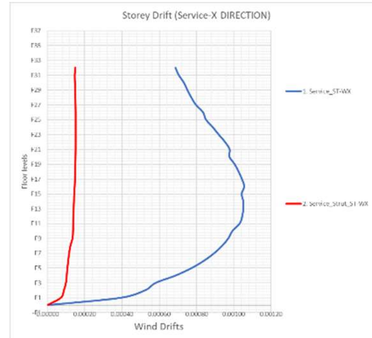


Figure 5.3 Wind story drift in X- Direction

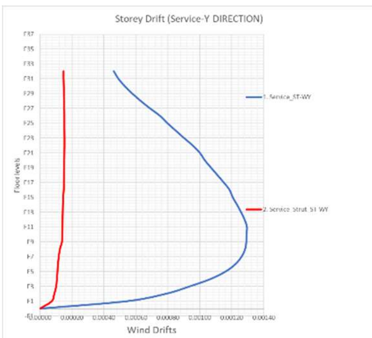


Figure 5.4 Wind story drift in Y- Direction

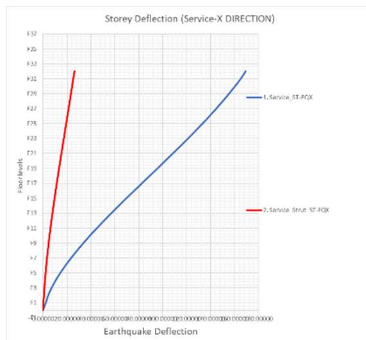


Figure 5.5 Earthquake story deflection in X- Direction

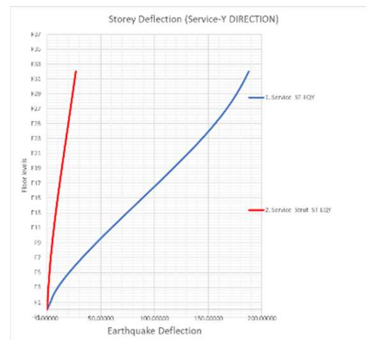


Figure 5.6 Earthquake story deflection in Y- Direction

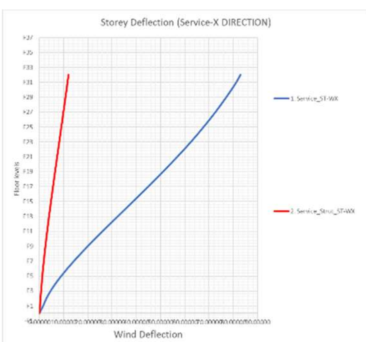


Figure 5.7 Wind story deflection in X- Direction

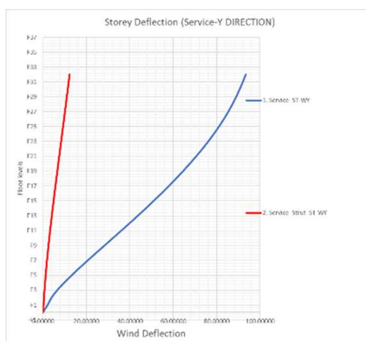
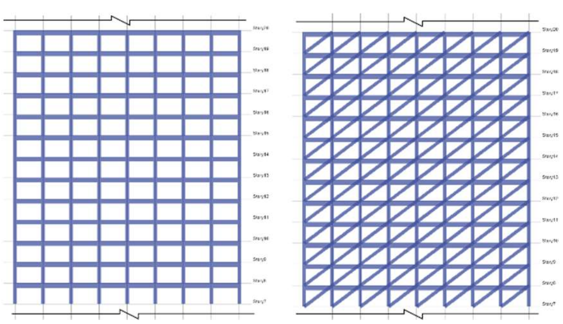


Figure 5.8 Wind story deflection in Y- Direction



(a) (b)

Figure 5.9 Elevation of grid 9

(a) Bare Frame (b) Equivalent diagonal strut Frame

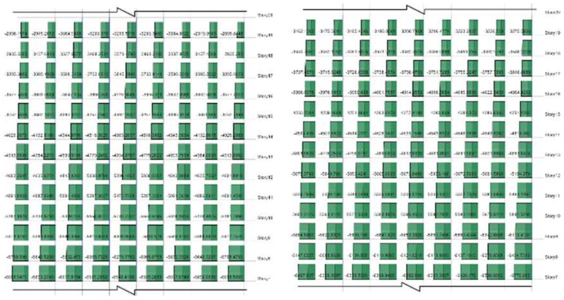


Figure 5.10 Axial Force (Kn) (Elevation of grid 9)

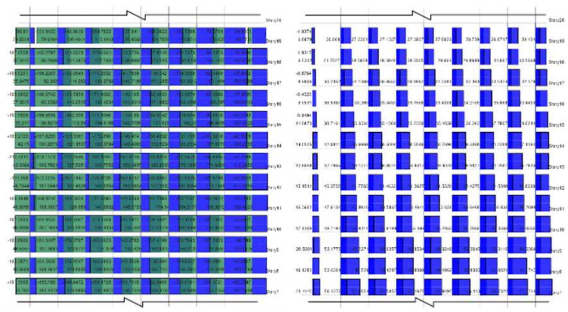


Figure 5.11 Shear Force (Kn) (Elevation of grid 9)

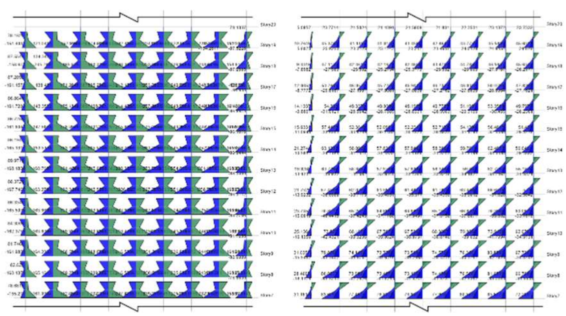


Figure 5.12 Moment (Kn-m) (Elevation of grid 9)

VI. SUMMARY & CONCLUSION

1. All the models were designed according to Indian standards, and the results show that the building drift and deflection are well within the permissible limit.

2. Strut helps to provide more stiffness to the structure.

3. The load transfer mechanism changes from frame action to truss action. The columns now experience increased axial forces but reduced bending moments and shear forces.

The results showed significant effect in the base shear and displacement of the structure. As the stiffness of the structure increased, it started attracting more force on to it thereby increasing the base shear value significantly. As stiffness is inversely proportional to the deflection, the increased stiffness due to equivalent diagonal strut has caused almost decrease in the displacement values. Thus, it is clear from the study that the effect of equivalent diagonal strut cannot be neglected while designing for horizontal forces. Considering equivalent diagonal strut in analysis would influence the seismic behaviour of frame structure to great extent since the strut increases strength and stiffness of the structure.

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