7 SOFTWARE FOR WIND ON SLENDER BUILDINGS

7.1 INTRODUCTION

Wind damage to buildings principally manifests in failure of the roof or its components, wall or cladding envelope, and consequent damage to the building contents. Hence, the vulnerability of buildings in wind storms is a function of the strength of building envelope components and their connections. Also, for a given building impacted by winds of a given intensity, the resulting damage is largely dependent upon the nature of its surrounding environment and the geometrical aspects of the building. Once the external and internal factors of demand and capacity have been understood, the engineer can judiciously use retrofitting and damping in order to control the building response to cyclones.

The following sections elaborate on the aspects of wind mitigation and the retrofitting components of a building. Wind loads evaluated from VC++ outputs , are used here to study the inherent structural features such as a shear core, shear walls on the periphery and corners, bracing systems on periphery and at corners, which can be built-in at the time of construction of new buildings in order to resist cyclonic winds. These systems have been described and analyzed on SAP2000. The results are compared and discussed for the various options of mitigating wind effects. Results have also been compared for the current code approach to slender buildings and that in the proposed draft in accordance with other international standards. Virtual Reality implementation has been demonstrated for MRFs, Shear wall frames and Braced Frames.

7.2 RETOROFITTING AND MITIGATION MEASURES

Retrofitting measures are advocated to reduce the risk of damage or failure for all existing structures not equipped adequately for cyclone resistance. Each component of the building is examined for its sensitivity to wind damage [90].

7.2.1 Roof

- In case of light roofs (AC or CGI sheeting) connections near the edges should be strengthened by providing additional U bolts. Mild steel flat ties may be provided to hold down the roof.
- All projections in roofs should be properly checked for strength against uplift and tied down particularly, if longer than 500 mm.
- All metallic connections for different components of roof should preferably be of non-corrosive material, or else must be galvanized or painted and checked before each cyclone season and doubtful ones replaced.
- In case of concrete slabs, 75 mm or 100 mm thick slabs may be subjected to uplift under wind speeds of 60 m/s or more, requiring holding down by anchors at the edges and reinforcement on top face.
- There must be proper bracings, in the plane of rafters, in plan at eaves level, and, in the vertical plane of columns along both axes of the building in sufficient number of panels determined by calculations.
- Flat roofs may be integrated to behave as horizontal diaphragms and either weighted down by dead weights or held down against uplift forces.

7.2.2 Framed Buildings

- The Frame column and shear wall where used shall be properly anchored into the foundation against uplift forces. For RC frame, usually a monolithic footing is provided due to stability against uplift. In case of steel framing too, column posts are properly tied to the footing through anchor bolts.
- In case of a framed structure, the total system is required to be properly braced. If existing lateral strength or bracing is inadequate, braces should be provided to improve the overall stability.
- All roof trusses should be properly connected to posts with the help of anchor bolts or metallic straps.

7.2.3 Load Bearing Walls

- Buttresses should be provided to improve the lateral load resistance of long walls, achieving cross wall spacing to less than 5m, thus reducing the unsupported lengths.

- Undesirable openings in the walls especially near the corners or edges be closed permanently to improve the cross walls.
- If the horizontal bands were not provided during construction, the exterior perimeter may be bolted all round by using Ferro-cement plating in the spandrel wall portion between lintel and eave / roof levels.

7.2.4 Glass Paneling

- The size of large glass panes should be reduced by adding battens at appropriate spacing. Large glass panes should be strengthened by fixing adhesive tapes, along and parallel to diagonals, at 100-150 mm spacing prior to each cyclone season. Alternatively, tin plastic film can be pasted on both faces of the panes to prevent shattering.
- Protective cover in the form of mesh or iron grill should be provided to prevent breakage of glass panels by flying missiles.

7.2.5 Foundations

- Proper drainage around the building should be provided to prevent pooling of water in its vicinity.
- The plinth should be protected against erosion by using pitching of suitable type.

7.3 STRUCTURAL SYSTEMS FOR WIND RESISTANCE

The types of structural systems for buildings, used to resist lateral loads may be categorized into following three general system types: (i) Moment Resisting Frames (ii) Braced Frames (iii) Shear walls and (iv) Dampers.

7.3.1 Moment Resisting Frames

Moment resisting Frames consist of floor members in plane with, and connected to column or pier members with rigid or semi-rigid joints. It is characterized by its flexibility, which is created by flexure of individual beams and columns and the rotation at their joints. An efficient frame action can be developed by providing closely spaced columns and deep beams at the building exterior. The strength and stiffness of a frame is proportional to the beam and column sizes, and inversely proportional to the column's unsupported height and spacing. Cast-in-place concrete or pre-cast concrete with cast in place joints provide the rigid and semi-rigid monolithic joints required. Frames may consist of beams and columns, flat slabs and columns or slabs and bearing walls [90].

The normal behavior of moment resisting frame produces significant bending moments in the beams at the face of the column and in the column at the face of the beam, with inflection points near the mid points of the beams and columns. The lateral sway in such type of framed structure is caused partly due to shear and partly due to column shortening. MRF have an advantage in high rise construction due to its flexibility in architectural planning. Moment resisting frames are normally efficient for buildings up to 20 stories in case of RCC buildings and up to 30 stories in case of buildings made from steel. The lack of efficiency for taller buildings is due to moment resisting frame's reliance derived primarily through flexure of its members. In this system, the members that carry shear and bending moments due to lateral loads often require additional construction depth, necessitating increases in overall height of the building. It has a high degree of redundancy and can continue to perform even if one or more members fail. As, mentioned earlier, moment resisting frame system for building higher than 30 stories is not efficient because the shear racking component of deflection produced by the bending of columns and beams causes the building drift to be too large.

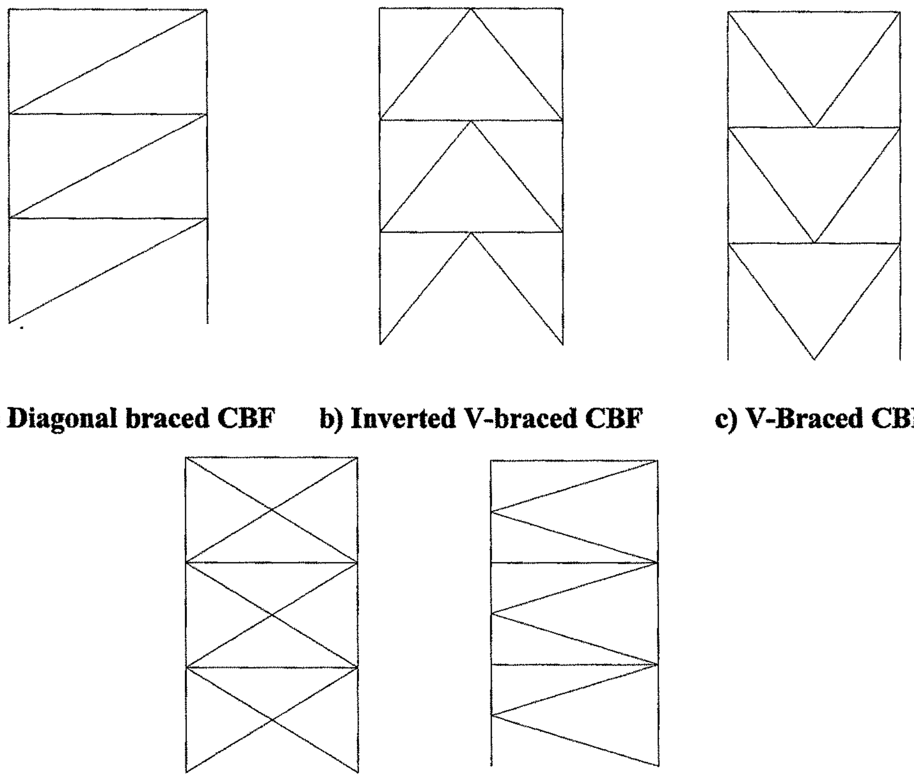
7.3.2 Bracing System

Pure rigid frame systems are not efficient for buildings higher than about 30 stories because the shear-racking component of deflection produced by the bending of column and girder causes the building drift to be too large. Rigid frame's inherent weakness most often lies in the flexibility of girders. A braced frame attempts to improve upon the efficiency of pure rigid frame action virtually eliminating the column and girder bending factors. This is achieved by adding truss members such as diagonals between the floor systems. The shear is now primarily absorbed by diagonal and not by the girders. The diagonals carry the lateral forces directly in predominantly axial action, providing for nearly pure cantilever behavior. All members are subjected to axial loads only, thereby creating an efficient structural system. One of the advantages of braced frame is the reduction in lateral drift compared with moment resisting frames [90].

Any rational configuration of bracing can be used for bracing systems. Bracing types available for incorporation into the structural system range from a concentric simple K or X brace between two columns to knee bracing and eccentric bracing with complicated geometry requiring' computer solutions. In an eccentric bracing system the connection of the diagonal brace is deliberately offset from the connection between the beam and the vertical column. By keeping the beam-to-brace connections close to the columns, the stiffness of the system can be made very close to that of concentric bracing. By shifting the work point away from the column centerline to the column face, connection details can be made simpler.

Concentrically braced frames are defined as those where the centre lines of all intersecting members meet at a point. This traditional form of bracing is widely used for all kinds of construction such as towers, bridges, and buildings, creating stiffness with great economy of materials in two dimensional space frames. The bracing may take the form of either a single diagonal in a bay or double bracing in an X shape.

Unlike the moment-resisting frame, the concentrically braced frame (CBF) is a lateral force-resisting system that is characterized by high elastic stiffness. High stiffness is achieved by the introduction of diagonal bracing members that resist lateral forces on the structural frame by developing internal axial actions and relatively small flexural actions. Diagonal bracing members and their connections to the framing system form the core units of a CBF **(Fig. 7.1)**. Braces can take the form of I-shaped sections, circular or rectangular tubes, double, angles stitched together to form a T-shaped section, solid T-shaped section, single angles, channels and tension-only rods and angles. Brace connections to the framing system are commonly composed of gusset plates with bolted or welded connections to the braces.



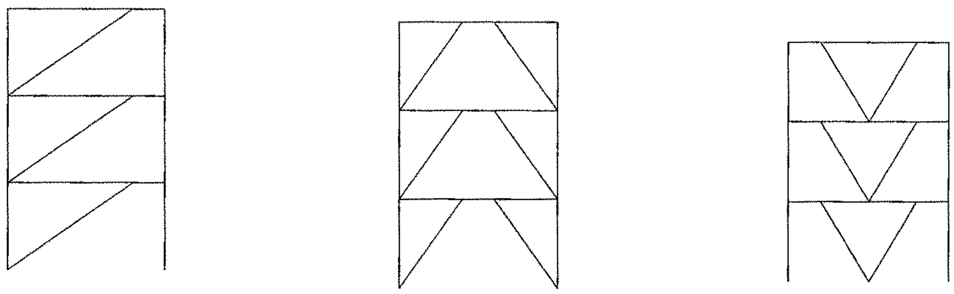
a) Diagonal braced CBF **b) Inverted V-braced CBF** **c) V-Braced CBF**

d) X-braced CBF **e) K-braced CBF**

Fig. 7.1 CBF Configurations of Bracings

In eccentrically braced frames (EBF) the axial forces in the braces are transmitted to the columns through bending and shear in beams, and, if designed correctly, the system possesses more ductility than concentrically braced frames while retaining the advantage of reduced horizontal deflections which braced systems have over moment-resisting frames.

The high elastic stiffness of the concentrically braced steel frame and the ductility and stable energy dissipation capacity of the moment resisting frame are characteristics of the EBF. The key distinguishing feature of an EBF is that at least one end of each brace is connected so as to isolate a segment of beam called a link. Common EBF arrangements are illustrated in **Fig. 7.2**; in each figure the links are identified by the link length e. The three EBF arrangements are termed the D-braced frame (**Fig.7.2a**), the split-K-braced frame (**Fig. 7.2b**), and the V-braced frame (**Fig. 7.2c**).



a) D-braced EBF **b) Split –K-braced EBF** **c)V-braced EBF**

Fig.7.2 Eccentrically Braced Frames

In eccentrically braced frames the axial forces in the braces are transmitted to the columns through bending and shear in beams, and, if designed correctly, the system possesses more ductility than concentrically braced frames while retaining the advantage of reduced horizontal deflections which braced systems have over moment-resisting frames.

The selection of bracing type is a function of the required stiffness, but most often it is influenced by the size of wall opening required for circulation. Because of architectural requirements, sometimes only certain bays around elevator and stair shafts are braced [87].

7.3.3 Shear Wall System

Shear wall system is one of the most popular systems for resisting lateral loads. Earlier applications of this system were limited to buildings in the 30 to 40 story range, but with the advent of super plasticizers and high strength concrete, it is now possible to use this system for taller buildings in the 50 to 60 storey range. Shear walls are planar, generally vertical elements which are relatively long and thin. Individual wall may be subjected to axial, translational, and torsional displacements. The extent to which a wall will contribute to the resistance of overturning moments, storey shear forces, and storey torsion depends on its geometric configuration, orientation, and location within the plane of the building. The major structural consideration for individual structural walls will be aspects of symmetry in stiffness, torsional stability, and available overturning capacity of the foundations. For the best torsional resistance, as many of the walls as possible should be located at the periphery of the building [91].

If two or more shear wall elements are connected together in plane with relatively rigid members, they are called coupled shear walls. When shear walls are compatible with other functional requirements and are of sufficient length, such walls can economically resist lateral force up to 30 to 40 stories. However, planar shear walls are efficient lateral load carriers only in their plane. Therefore, it is necessary to provide walls in two orthogonal directions. However, in long and narrow buildings sometimes it is possible to resist wind loads in the short direction by the frame action of columns and slabs because first, the area of the building exposed to the wind in the short direction is small, and second, because the building is long, usually a sufficiently large number of columns exists in that direction. The shear wall may or may not carry substantial gravity loads, and it may be a simple bearing wall, a wall connecting two or more columns, or a panel wall filling the opening of a beam/column frame. Shear walls may be organized in plan and connected together at their edges to form box like cellular structures to resist wind forces in each direction [90].

A common shear wall system used for tall buildings groups are shear wall around service cores, elevator shafts and stair wells. Majority of the lateral loads are carried by shear walls in the lower portion of the building and mostly by the frame action in the top portion. Although shear walls stiffen the structure and are necessary for the control of the drift due to wind and moderate earthquakes they also lessen its ductility. Shear walls are designed to cantilever from the foundation level. Many tall buildings undergo torsional loading due to non alignment of the building shear center with the location of the horizontal load application. Boxed shear wall system is an efficient in resisting such torsion.

7.3.4 Dampers

The damping in a mechanical or structural system is a measure of the rate at which the energy of motion of the system is dissipated [106]. In many systems, damping is not helpful and it has to be overcome by the system input. In the case of wind sensitive structures such as tall buildings, however, it is beneficial, as damping reduces motion, making the building feel more stable to its occupants. Controlling vibrations by increasing the effective damping can be a cost effective solution. Occasionally, it is the only practical and economical means of reducing resonant vibrations. Types of damping systems that can be implemented include, passive, active and semi-active dampers.

Some examples of ***passive dampers*** are: Tuned Mass Damper (TMD). **Figure 7.3** shows one of the TMDs designed for the skybridge legs of the Petronas Towers, Malayasia. 12 TMDs were installed three in each of the four legs.

Distributed Viscous Dampers: Tuned Liquid Column Dampers (TLCD), also known as *Liquid Column Vibration Absorbers* (LVCA) etc.

Examples of ***active and hybrid dampers*** include: Active Tuned Mass Damper (ATMD) and Active Mass Driver (AMD);

Examples of ***semi-active dampers*** include: Variable Stiffness Dampers, Hydraulic Dampers, Magneto-Rheological (MR) Dampers etc.

While general design philosophy tends to favour passive damping systems due to their lower capital and maintenance costs, active or semi-active dampers may be the ideal solution for certain vibration problems.

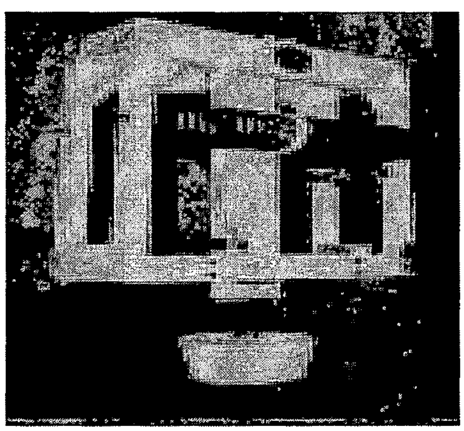


Fig. 7.3 TMD Designed for Skybridge Legs of Petronas Towers

7.4 COMPUTER PROGRAM FOR WIND LOADS

Details of an existing building are taken as a case study for various alternatives for mitigation of wind hazard. Wind load is calculated through VC++ program module and taken as input by SAP2000. Three separate models of Moment Frame, Bracing and Shear Wall are generated in SAP 2000 to get the results of wind load analysis for comparison. Building plan is shown in **Figs. 7.4 -7.5**

7.4.1 Basic Structural Data

Least lateral dimension of the building:	16.74 m
Other lateral dimension of the building:	26.25 m
Height of the building:	33.5 m
Location:	Surat, Gujarat, India
Basic wind speed:	44 m/s
Story Height:	3.35 m
Column sizes:	300 x 750
Beam sizes:	230 x 600
Wall size:	230 mm
Bracing size:	100mm double angle

7.4.2 Basic Wind Data

Height of building (h):	33.5 m (Short For Static Analysis)
Height of building (h):	90.45 m (Slender For Dynamic Analysis)
Width of building (b) :	16.74 m
Length of building (a):	26.25 m
Wind zone:	Zone 3
Basic wind speed:	44 m/s
Terrain category:	3
Effective $Area_{x;}$ = 3.35 x 3.72 =	12.462 m^2
Effective $Area_{y;}$ = 3.35 x 3.69 =	12.362 m^2

7.4.3 Design Factors

Following design factor are considered from draft code using DRF method:

Probability Factor, k_1 = 1.00 (For all general structures)

Topography Factor, k_3 = 1.00 (For upwind slope θ below 3°)

Importance Factor for Cyclone Region, k_4 = 1 (Cl. 5.3.4, Draft Code)

Wind Directionality Factor, K_d = 0.9 (Cl. 5.4.1, Draft Code)

Area Averaging Factor, K_a = 1 (Cl. 5.4.2, Draft Code)

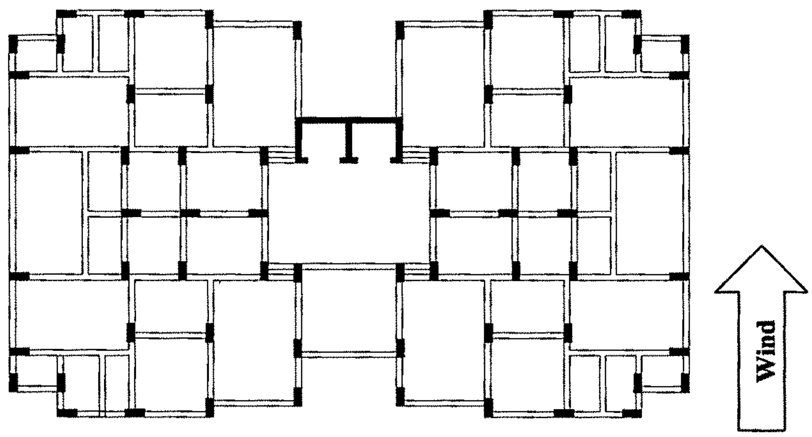


Fig. 7.4 Typical Floor Plan

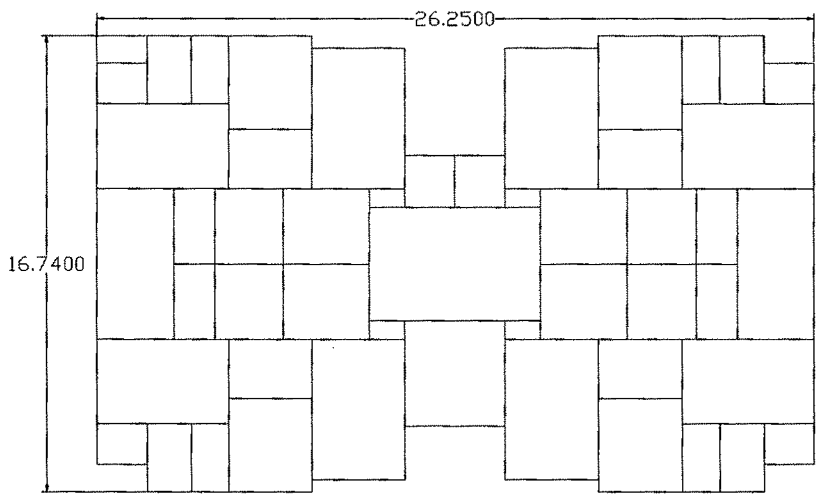


Fig. 7.5 Typical Structural Line Diagram

7.4.4 Load Calculations

Slab thickness = 125 mm.

Floor Finish at floor level = 1 kN / m^2.

Floor finish at terrace level = 1.5 kN / m^2.

Concrete grade = 25 kN / m^3.

(a) Dead Load

(i) Slab load :

Slab load at floor level = Self Weight + Floor Finish

= 25 * 0.125 + 1 = 4.125 kN / m^2

Slab load at floor level = Self Weight + Floor Finish

= 25 * 0.125 + 1.5 = 4.625 kN / m^2

(ii) Wall load :

Wall load on beams at floor level = Thickness * Density of wall * Height

= 0.115 * 19.28 * 3.35 = 7.43 kN / m

Parapet Wall load on beams = Thickness * Density of wall * Height

= 0.115 * 19.28 * 1 = 2.22 kN / m

(b) Live Load

Live Load = 2 kN / m^2 (IS 875 (part 2):– 1987 from table 1 for residential building,

(c) Wind Load

(i) Wind Load For Static Analysis :

Table 7.1 shows the wind load derived from Visual C^{++} program of Static Method.

Table 7.1 Wind Load for Static Analysis

Height	k2	Vz (kN/m)	Pz (kN/m^2)	Pd (kN/m^2)	Fz (kN)
3.35	0.78631	34.5977	0.7182	0.7182	10.6541
6.70	0.8558	37.6552	0.85075	0.85075	12.6204
10.05	0.91072	40.0718	0.96345	0.96345	14.2922
13.40	0.95334	41.9467	1.05572	1.05572	15.6609
16.75	0.98589	43.3792	1.12905	1.12905	16.7488
20.10	1.01065	44.4685	1.18647	1.18647	17.6005
23.45	1.02986	45.3138	1.23201	1.23201	18.2761
26.80	1.04578	46.0145	1.2704	1.2704	18.8456
30.15	1.0705	47.102	1.33116	1.33116	19.747
33.50	1.07971	47.5072	1.35416	1.35416	20.0881

(ii) Wind Load For Dynamic Analysis :

For dynamic analysis height of the same existing building is increased from 33.5m to 90.45m. Table 7.2 shows the wind load derived from the program developed for Gust Factor Method and Table 7.3 shows the wind load derived from program for Dynamic Response Factor Method.

Table 7.2 Wind Load Using Gust Factor Method

Height	k2	Vz (kN/m)	Pz (kN/m^2)	Pd (kN/m^2)	Fz (kN)
13.4	0.53509	23.5439	0.33259	0.33259	12.2385
16.75	0.56514	24.8661	0.37099	0.37099	13.7078
20.1	0.5907	25.9907	0.40531	0.40531	15.0256
23.45	0.61177	26.9179	0.43474	0.43474	16.1589
26.8	0.62835	27.6475	0.45863	0.45863	17.1201
30.15	0.64653	28.4474	0.48555	0.48555	18.2503
33.5	0.65641	28.882	0.5005	0.5005	18.8804
36.85	0.66593	29.3008	0.51512	0.51512	19.4983
46.9	0.69242	30.4667	0.55693	0.55693	21.2732
50.25	0.7006	30.8264	0.57016	0.57016	21.8371
53.6	0.70846	31.1723	0.58303	0.58303	22.3868
56.95	0.71602	31.5048	0.59553	0.59553	22.9218
60.3	0.72328	31.8243	0.60767	0.60767	23.4422
63.65	0.73025	32.1311	0.61945	0.61945	23.9478
67	0.73695	32.4258	0.63086	0.63086	24.4385
70.35	0.74338	32.7087	0.64192	0.64192	24.9145
73.7	0.74955	32.9801	0.65261	0.65261	25.3758
77.05	0.75547	33.2406	0.66296	0.66296	25.8225
80.4	0.76115	33.4904	0.67296	0.67296	26.2548
83.75	0.76659	33.73	0.68263	0.68263	26.6729
87.1	0.77181	33.9598	0.69196	0.69196	27.0771
90.45	0.77682	34.1801	0.70097	0.70097	27.73

Table 7.3 Wind Load for Dynamic Response Factor Method

Height	k2	Vz (kN/m)	Pz (kN/m²)	Pd (kN/m²)	Fz (kN)
3.35	0.78631	34.5977	0.7182	0.64638	8.66049
6.7	0.8558	37.6552	0.85075	0.76568	10.4512
10.05	0.91072	40.0718	0.96345	0.86711	12.0376
13.4	0.95334	41.9467	1.05572	0.95015	13.3909
16.75	0.98589	43.3792	1.12905	1.01615	14.5129
20.1	1.01065	44.4685	1.18647	1.06782	15.4312
23.45	1.02986	45.3138	1.23201	1.10881	16.1934
26.8	1.04578	46.0145	1.2704	1.14336	16.8618
30.15	1.0705	47.102	1.33116	1.19804	17.8559
33.5	1.07971	47.5072	1.35416	1.21874	18.3229
36.85	1.08856	47.8966	1.37645	1.2388	18.787
40.2	1.09706	48.2705	1.39803	1.25822	19.2495
43.55	1.10522	48.6295	1.4189	1.27701	19.71
46.9	1.11304	48.9739	1.43907	1.29516	20.1666
50.25	1.12055	49.3042	1.45854	1.31269	20.6274
53.6	1.12774	49.6206	1.47732	1.32959	21.0671
56.95	1.13463	49.9236	1.49542	1.34588	21.5023
60.3	1.14122	50.2136	1.51284	1.36156	21.9321
63.65	1.14752	50.491	1.52961	1.37665	22.3552
67	1.15355	50.7562	1.54572	1.39114	22.7702
70.35	1.15931	51.0096	1.56119	1.40507	23.1755
73.7	1.16481	51.2516	1.57604	1.41843	23.5694
77.05	1.17006	51.4825	1.59027	1.43124	23.9501
80.4	1.17507	51.7029	1.60391	1.44352	24.3158
83.75	1.17984	51.913	1.61698	1.45528	24.6648
87.1	1.18439	52.1133	1.62948	1.46653	24.996
90.45	1.18873	52.3041	1.64143	1.47729	25.3087

(d) Load Combinations (As per IS 456-2000, Table –18)

1. 1.5(DL + LL)
2. 1.2(DL + LL +WL)
3. 1.2(DL +LL – WL)
4. 1.5(DL + WL)
5. 1.5(DL – WL)
6. 0.9DL +1.5WL
7. 0.9DL – 1.5WL

7.5 STEPS FOR ANALYSIS OF SLENDER BUILDINGS

Models of three structural systems called frame system, bracing system and shear wall system are generated in SAP 2000 software. The step by step method to generate these models is described below.

1. Generate a 3-D frame using .exe file or using commands given in SAP 2000 software.
2. Define the member sections and member properties using, "Frame/Cable Sections" command given in "Define Menu".
3. Assign the respective sections to respective members using, "Frame/cable sections" command given in "Assign Menu".
4. Define different load cases using, "Load Cases" command given in "Define Menu".
5. Define the load combinations using, "Combination" command given in "Define Menu".
6. Assign the loads like dead load, live load, wall load etc. to the respective members.
7. User can select respective floor slabs using, "Set 2D View" command given in "View Menu". Now, select one by one floor and select all joints of that particular floor to assign rigid diaphragm by using, "Joint Constraint" command given in "Assign menu".
8. Define joint restraints at foundation level by using, "Joint Restraint" command given in "Assign Menu".

9. Select the Analysis case like plane frame, space frame etc. by using, "Set Analysis Options" command given in "Analyze" menu.
10. Run Analysis using, "Run Analysis" command given in "Analyze" menu.
11. Design members using "Design menu" and check whether all members pass or not; if any member fails, then first unlock the structure and increase the size of that member and again analyze the structure until all members pass.

7.6 STRUCTURAL SYSTEMS FOR STATIC METHOD

7.6.1 Models Generated for Structural System

Here, the models of the existing building having 33.5 m height are generated using SAP 2000. Models of three different systems are generated. Frame system with shear core (**Fig.7.6**), Bracings at corners (**Fig.7.7**) and Bracings at sides (**Fig.7.9**) and Shear wall at corners (**Fig.7.9**) and Shear Walls at Sides (**Fig.7.10**) are generated as shown below. Wind load is applied on wider face which is shown in **Fig 7.11**.Top Plans (with bracings / shear walls) are shown in **Fig.7.12** and **Fig.7.13**.

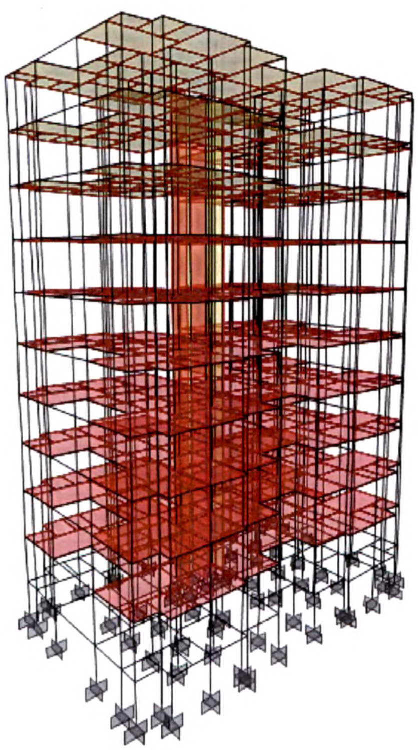


Fig. 7.6 Frame with Shear Core

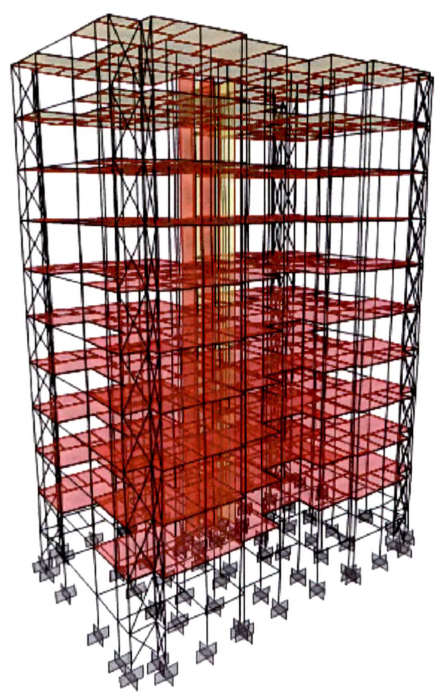


Fig. 7.7 Bracing System (At Corners)

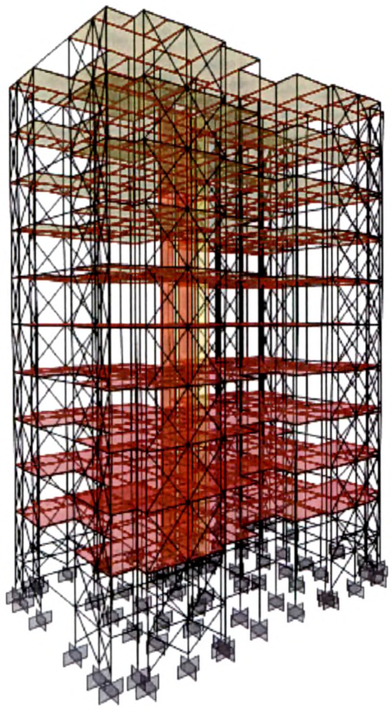


Fig. 7.8 Bracing System (At Sides)

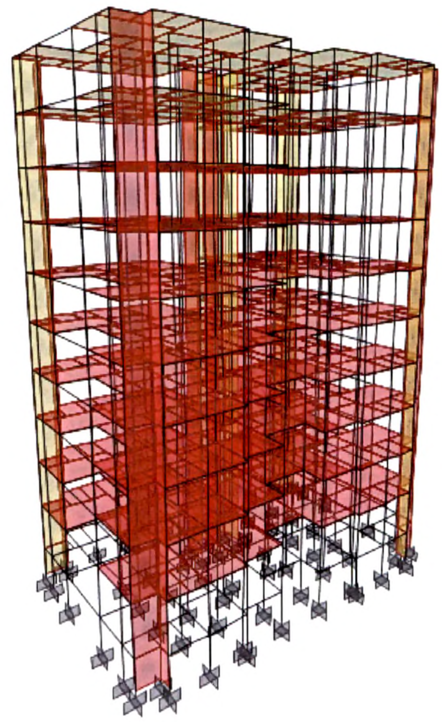


Fig. 7.9 Shear Wall at Corners

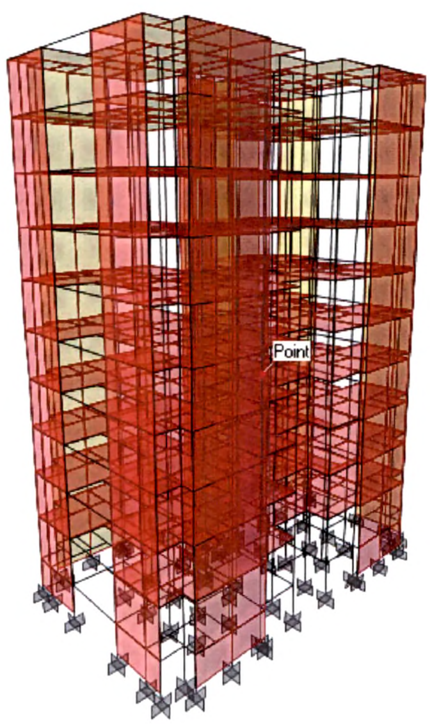


Fig. 7.10 Shear Wall at Sides

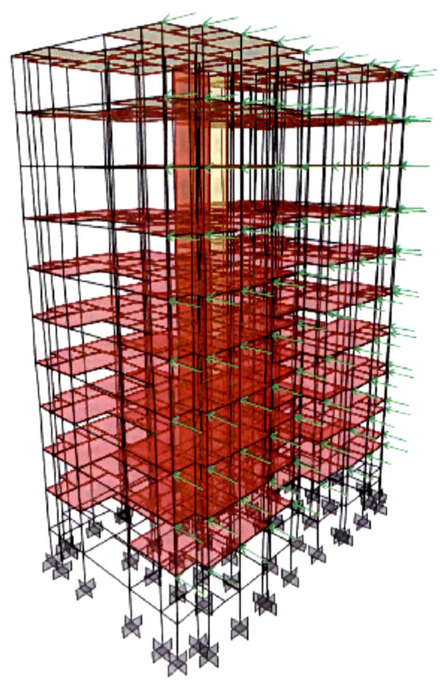


Fig. 7.11 Wind Load on Structure

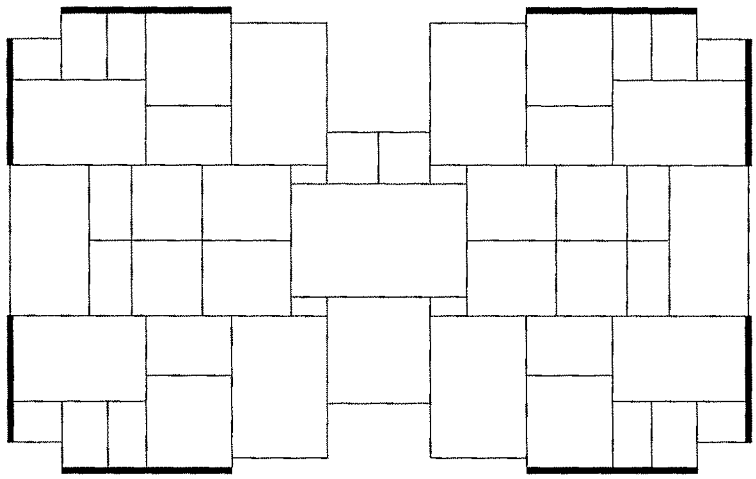


Fig. 7.12 Position of Bracings or Shear Walls Provided at Sides

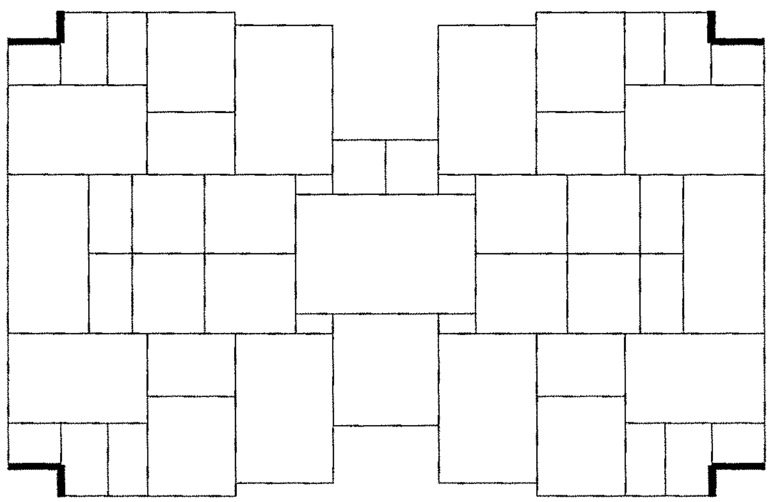


Fig. 7.13 Position of Bracings or Shear Walls Provided at Corners

7.6.2 Comparison of the Structural Systems

Using SAP 2000 software one can compare maximum axial force P (kN) from Table 7.4 and **Fig. 7.14**; Maximum shear force V (kN) given in Table 7.5 and **Fig. 7.15**; Maximum torsional moment T (kN-m) given in Table 7.6 and **Fig. 7.16**; Maximum bending moment M (kN-m) in Table 7.7 and **Fig. 7.17** for different systems under different load combinations.

Table 7.4 Maximum Axial Force P for Static Method

LOAD COMBINATIONS	FRAME SYSTEM	SHEAR WALL-CORNERS	SHEAR WALL-SIDES	BRACING SYSTEM-CORNERS	BRACING SYSTEM-SIDES
1.5(DL+LL)	356.325	324.578	186.907	345.884	294.884
1.2(DL+LL+WL)	533.430	375.305	177.736	511.241	413.434
1.2(DL+LL-WL)	501.655	357.744	180.309	479.308	380.52
1.5(DL+WL)	582.129	507.921	406.757	554.524	459.598
1.5(DL-WL)	711.728	511.648	406.623	683.662	559.438
0.9DL+1.5WL	608.048	440.786	244.081	580.351	458.291
0.9DL-1.5WL	685.808	486.407	243.947	657.835	534.151

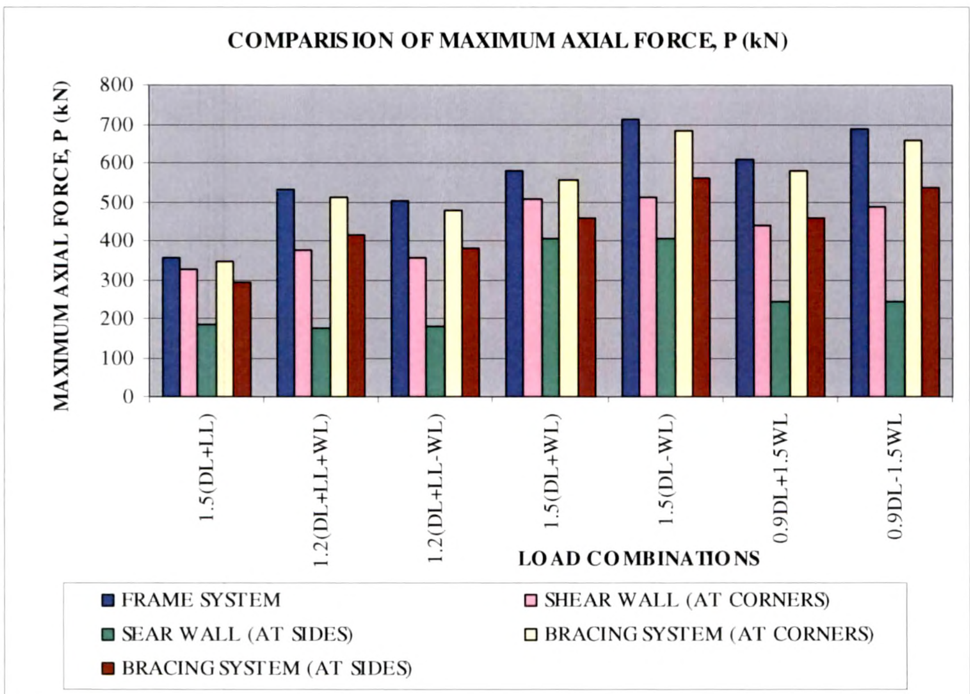


Fig. 7.14 Comparison of Maximum Axial Force for Static Method

From Table 7.4 and figure we can say that, as compared to load combination 1 the axial forces are higher in all the other load combinations having wind load. Minimum axial forces are observed when the shear walls are provided at sides. It is clearly seen that the lateral force resisting systems reduces considerably the axial forces as compared to the forces of frame system.

Table 7.5 Maximum Shear Force V for Static Method

LOAD COMBINATIONS	FRAME SYSTEM	SHEAR WALL (AT CORNERS)	SHEAR WALL (AT SIDES)	BRACING SYSTEM (AT CORNERS)	BRACING SYSTEM (AT SIDES)
1.5(DL+LL)	52.167	51.915	52.521	52.072	52.318
1.2(DL+LL+WL)	56.704	42.048	42.417	52.251	48.405
1.2(DL+LL-WL)	80.606	62.042	41.616	77.93	68.138
1.5(DL+WL)	69.825	52.658	53.404	64.229	59.981
1.5(DL-WL)	94.687	71.297	52.403	91.312	79.566
0.9DL+1.5WL	66.365	46.353	32.242	63.521	52.693
0.9DL-1.5WL	86.573	63.417	31.241	83.251	70.692

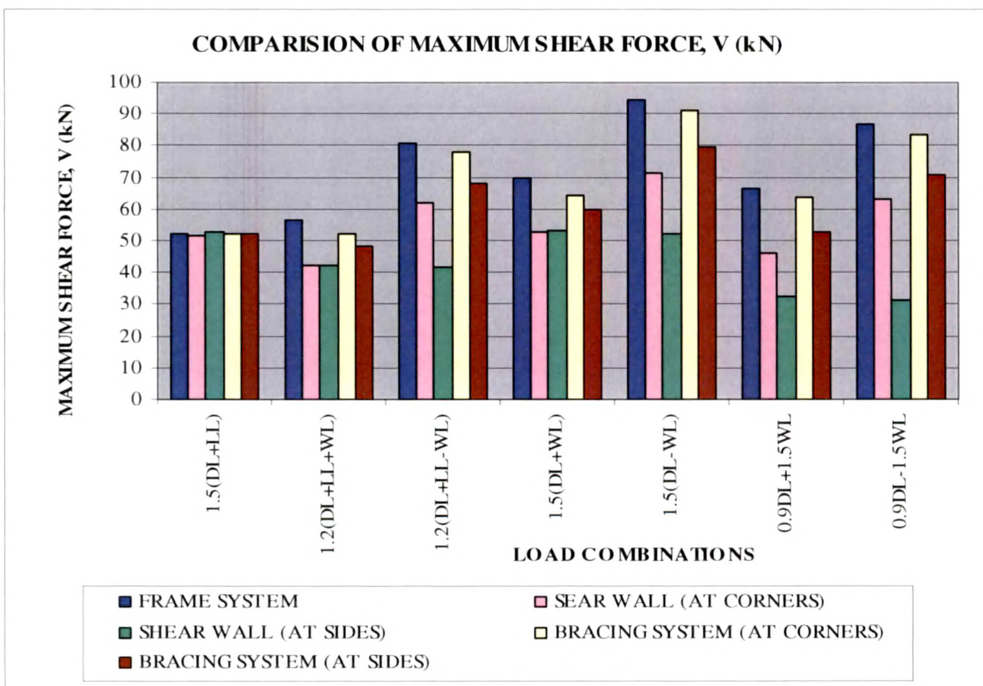


Fig. 7.15 Comparison of Maximum V for Static Method

Table 7.6 Maximum Torsional Moment T for Static Method

LOAD COMBINATIONS	FRAME SYSTEM	SHEAR WALL (AT CORNERS)	SHEAR WALL (AT SIDES)	BRACING SYSTEM (AT CORNERS)	BRACING SYSTEM (AT SIDES)
1.5(DL+LL)	4.6996	4.5689	4.6839	4.6359	4.6695
1.2(DL+LL+WL)	4.006	3.447	3.5737	3.9222	3.5821
1.2(DL+LL-WL)	4.4298	4.4019	3.9206	4.4222	4.2482
1.5(DL+WL)	4.9303	4.4912	4.801	4.8327	4.6791
1.5(DL-WL)	5.5752	5.5144	4.9345	5.5495	5.3335
0.9DL+1.5WL	3.8613	3.1686	2.8815	3.7629	3.3361
0.9DL-1.5WL	3.6802	3.682	3.0474	3.6864	3.4565

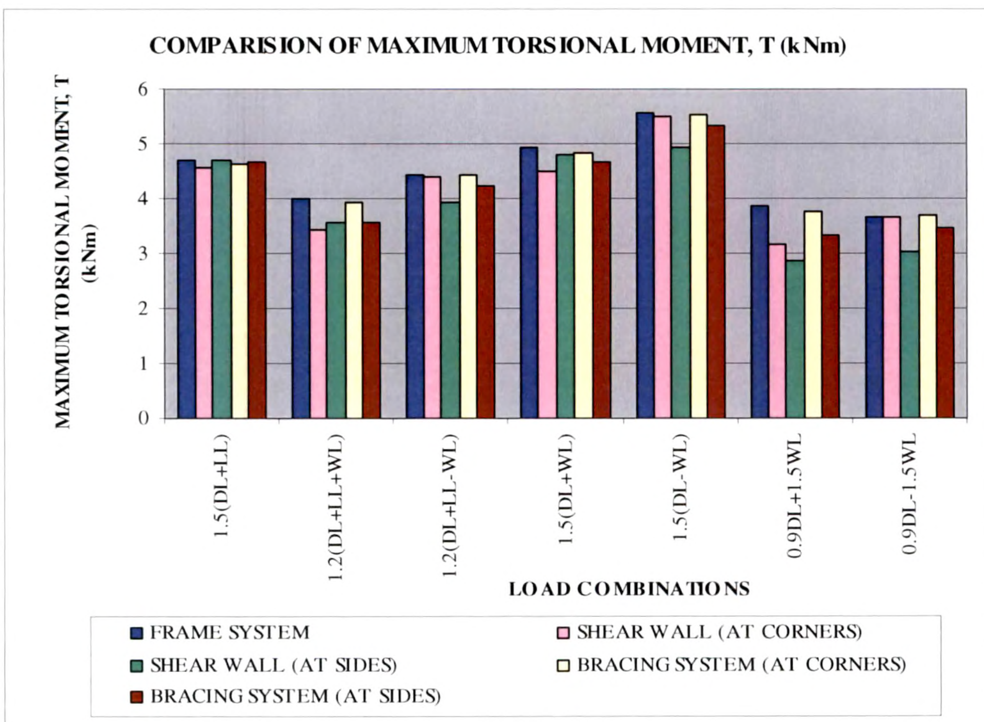


Fig. 7.16 Comparison of Maximum T for Static Method

Table 7.7 Maximum Bending Moment M for Static Method

LOAD COMB.	FRAME SYSTEM	SHEAR WALL (CORNER)	SHEAR WALL (AT SIDES)	BRACING SYSTEM (CORNERS)	BRACING SYSTEM (AT SIDES)
1.5(DL+LL)	48.453	47.9657	48.6637	48.2857	48.3156
1.2(DL+LL+WL)	48.8483	50.1181	41.1787	48.0733	46.755
1.2(DL+LL-WL)	44.7781	43.8854	39.8789	44.7278	43.2525
1.5(DL+WL)	62.3801	63.4673	53.2776	61.3896	59.903
1.5(DL-WL)	55.2338	53.9901	49.0085	55.1328	53.3173
0.9DL+1.5WL	42.8045	45.4739	33.0904	41.851	40.0645
0.9DL-1.5WL	38.4454	40.6161	30.0244	37.1229	34.2904

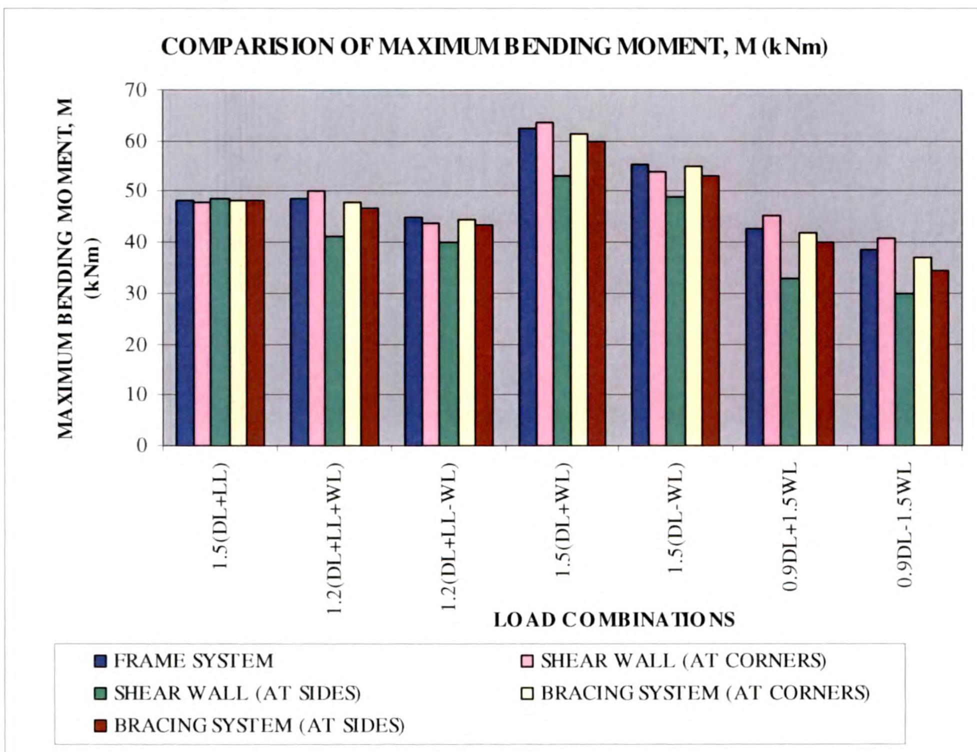


Fig. 7.17 Comparison of Maximum M for Static Method

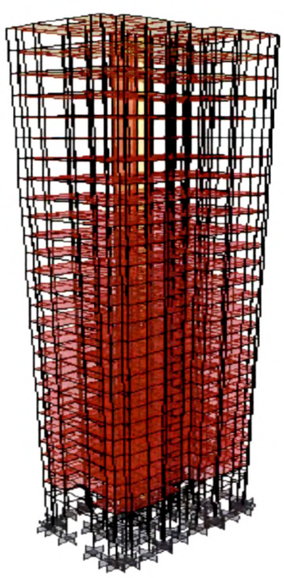
7.6.3 Comments on Results

- From static analysis, it is seen that for simple framed structure with shear core the values of axial force, shear force and bending moment are increased but there is not much change in torsional moment.
- The shear wall at sides makes the structure much stiffer and reduces the forces remarkably for both short and tall structures.

7.7 STRUCTURAL SYSTEMS FOR DYNAMIC METHODS

7.71 Models of Structural Systems for Dynamic Methods

For dynamic analysis of the same existing building, the height of the building is increased up to 90.45 m and accordingly the sizes of columns are also increased to 750 X 750 mm. Then the wind loads obtained by Dynamic Response Factor Method and Gust Factor Methods are applied to the different models generated using SAP 2000 software. Models of three different systems are generated. Frame system with shear core (**Fig. 7.18**), Bracings at corners (**Fig. 7.19**) and Bracings at sides (Fig. **7.20**) and Shear wall at corners (Fig. **7.21**) and Shear Walls at Sides (Fig. **7.22**) are generated as shown below.



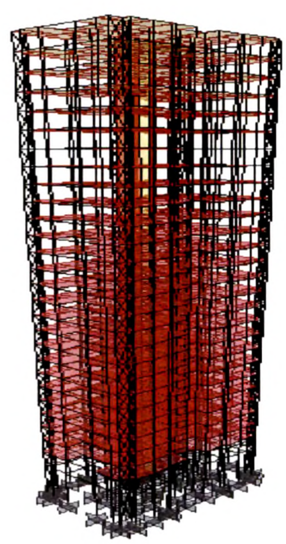


Fig. 7.18 Frame System with Shear Core **Fig. 7.19 Bracing System (At Corners)**

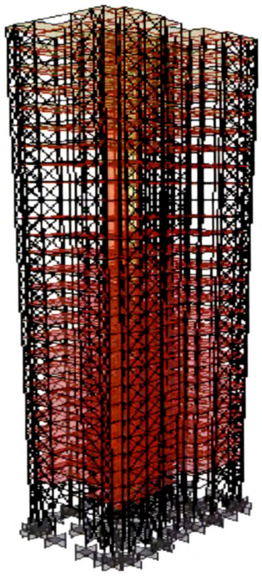


Fig.7.20 Bracing System on Sides

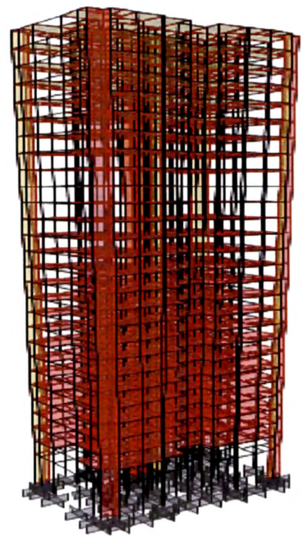


Fig.7.21 Shear Wall at Corners

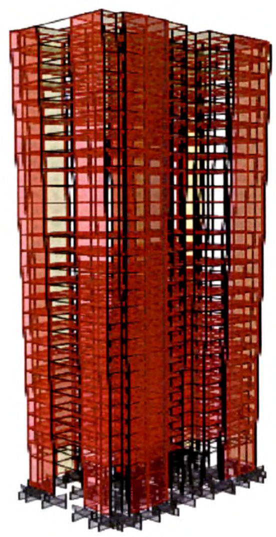


Fig.7.22 Shear Wall on Sides

7.7.2 Comparison of the Structural Systems for GFM

Here, the wind load obtained by Gust Factor Method (GFM) Program is applied to the models generated in SAP 2000 software. So, we can compare maximum axial force P (kN), maximum shear force V (kN), maximum torsional moment T (kN-m), maximum bending moment M (kN-m) of different systems for different load combinations shown in Table 7.8 to 7.11 and Figs. 7.23 to 7.26

Table 7.8 Maximum Axial Force P for Gust Factor Method

LOAD COMBINATIONS	FRAME SYSTEM	SHEAR WALL (CORNER)	SHEAR WALL (SIDE)	BRACING SYSTEM (CORNER)	BRACING SYSTEM (SIDE)
1.5(DL+LL)	747.208	681.729	429.675	690.82	649.724
1.2(DL+LL+WL)	1666.124	1294.852	859.044	1457.058	1265.423
1.2(DL+LL-WL)	1693.745	1338.723	881.228	1481.164	1281.593
1.5(DL+WL)	2504.211	2021.313	1324.497	2241.314	1704.205
1.5(DL-WL)	2548.166	2057.208	1352.019	2280.541	1726.83
0.9DL+1.5WL	2315.811	1835.827	1111.724	2054.494	1598.568
0.9DL-1.5WL	2352.719	1865.43	1128.202	2087.181	1611.825

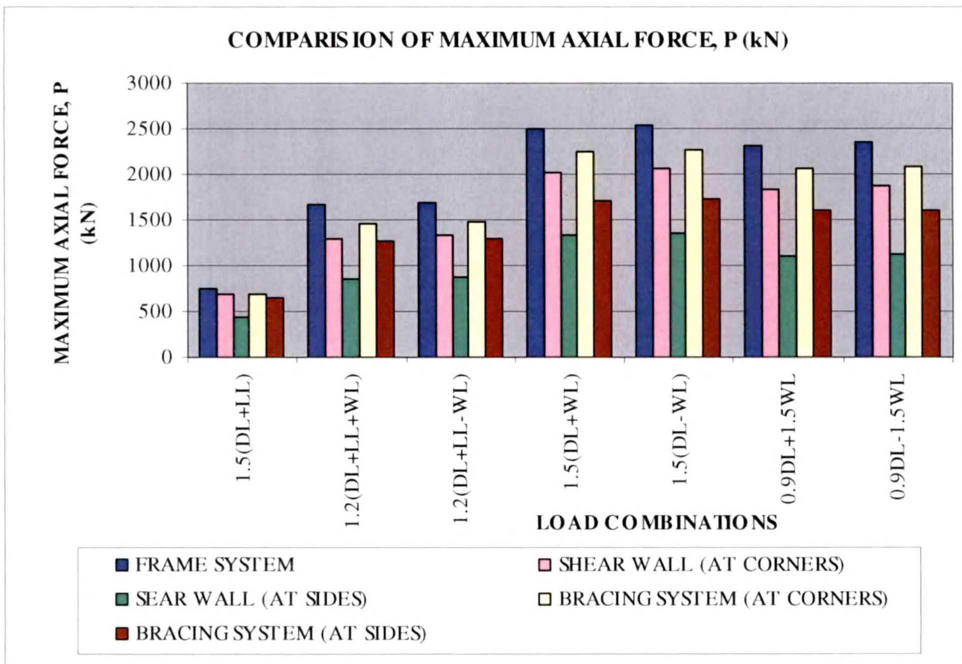


Fig.7.23 Comparison of Maximum Axial Force P - Gust Factor Method

From Table 7.8 and Fig.7.23 it is seen that when building height is increased from 33.5m to 90.45m, there is a remarkable increase in the axial force due to wind effect. Shear walls or bracing system provided at sides help to reduce the axial force rise due to wind effects.

Table 7.9 Maximum Shear Force V for Gust Factor Method

LOAD COMBINATIONS	FRAME SYSTEM	SHEAR WALL (CORNER)	SHEAR WALL (SIDE)	BRACING SYSTEM (CORNER)	BRACING SYSTEM (SIDE)
1.5(DL+LL)	55.23	54.939	55.807	54.253	62.349
1.2(DL+LL+WL)	368.283	281.719	112.213	320.739	236.801
1.2(DL+LL-WL)	368.004	281.193	112.152	319.968	227.784
1.5(DL+WL)	462.592	354.599	143.854	403.299	342.102
1.5(DL-WL)	460.776	353.118	144.39	401.531	329.022
0.9DL+1.5WL	460.624	352.242	139.424	401.066	298.722
0.9DL-1.5WL	457.729	349.91	139.628	398.334	291.006

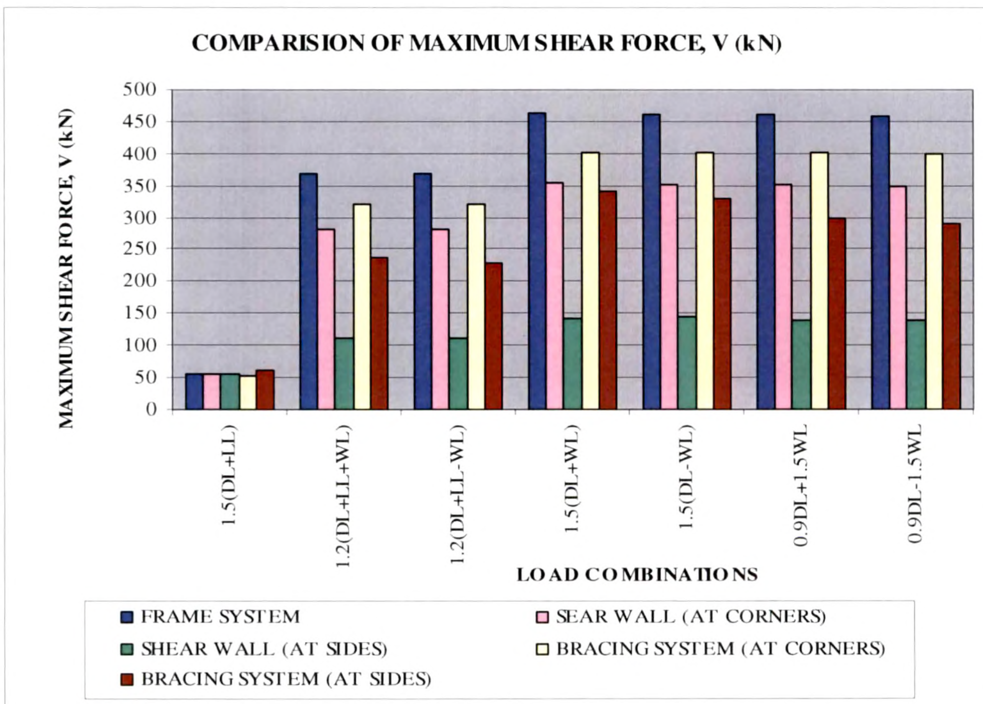


Fig.7.24 Comparison of Maximum Shear Force V- Gust Factor Method

Table 7.10 Maximum Torsional Moment T for Gust Factor Method

LOAD COMBINATIONS	FRAME SYSTEM	SHEAR WALL (CORNERS)	SHEAR WALL (SIDES)	BRACING SYSTEM (CORNERS)	BRACING SYSTEM (SIDES)
1.5(DL+LL)	4.0864	3.9506	4.0034	3.7659	4.1584
1.2(DL+LL+WL)	14.8819	11.1482	3.9082	12.7838	11.1748
1.2(DL+LL-WL)	14.5512	10.7465	4.4432	12.4011	11.2539
1.5(DL+WL)	18.6861	14.0119	4.8158	16.061	13.8986
1.5(DL-WL)	18.1052	13.3564	5.6671	15.4201	14.1755
0.9DL+1.5WL	18.5699	13.8808	4.6863	15.9328	13.9234
0.9DL-1.5WL	18.2214	13.4875	4.2981	15.5483	14.1125

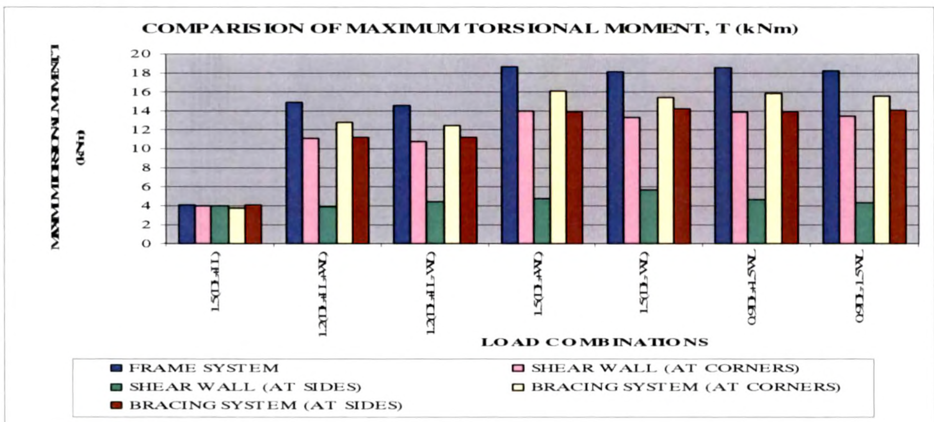


Fig. 7.25 Comparison of Maximum T -Gust Factor Method

Table 7.11 Maximum M for Gust Factor Method

LOAD COMBINATIONS	FRAME SYSTEM	SHEAR WALL (CORNER)	SHEAR WALL SIDES)	BRACING SYSTEM (CORNER)	BRACING SYSTEM (SIDES)
1.5(DL+LL)	62.247	62.1159	64.2225	61.14	66.1046
1.2(DL+LL+WL)	326.1739	243.3141	89.6788	288.3077	269.8956
1.2(DL+LL-WL)	328.7324	244.9181	90.1631	290.8564	271.3044
1.5(DL+WL)	408.2693	304.6984	111.0885	365.3964	340.9878
1.5(DL-WL)	411.6007	306.9347	111.7574	367.2966	342.6843
0.9DL+1.5WL	407.7451	304.1196	110.1912	358.4256	339.7576
0.9DL-1.5WL	409.7898	305.4857	110.5942	360.8214	340.8003

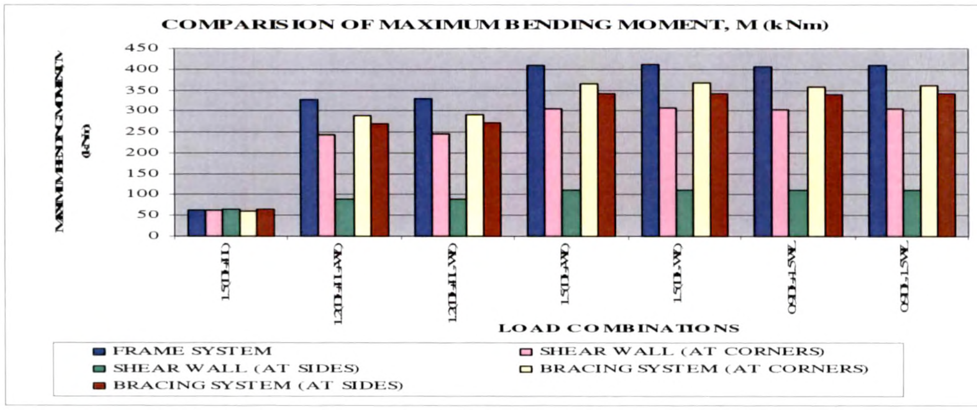


Fig. 7.26 Comparison of Maximum M - Gust Factor Method

7.7.3 Comparison of the Structural Systems for DRFM

Here, the wind load obtained by Dynamic Response Factor Method Program is applied to the models generated in SAP 2000 software. So, we can compare maximum axial force P (kN), maximum shear force V (kN), maximum torsional moment T (kN m), maximum bending moment M (kN m) of different systems for different load combinations as shown in Tables 7.12 -7.15 and Figs. 7.27 -7.30

Table 7.12 Maximum Axial Force P for DRFM

LOAD COMBINATIONS	FRAME SYSTEM	SHEAR WALL (CORNERS)	SHEAR WALL (AT SIDES)	BRACING SYSTEM (AT CORNERS)	BRACING SYSTEM (AT SIDES)
1.5(DL+LL)	747.208	681.729	429.675	690.82	649.724
1.2(DL+LL+WL)	1603.487	1255.94	834.628	1402.398	1221.058
1.2(DL+LL-WL)	1630.296	1299.814	856.814	1425.799	1237.252
1.5(DL+WL)	2425.914	1961.332	1293.976	2172.989	1648.748
1.5(DL-WL)	2468.855	1996.45	1321.501	2211.335	1671.403
0.9DL+1.5WL	2237.515	1775.845	1081.203	1986.169	1543.111
0.9DL-1.5WL	2273.408	1804.673	1097.684	2017.976	1556.399

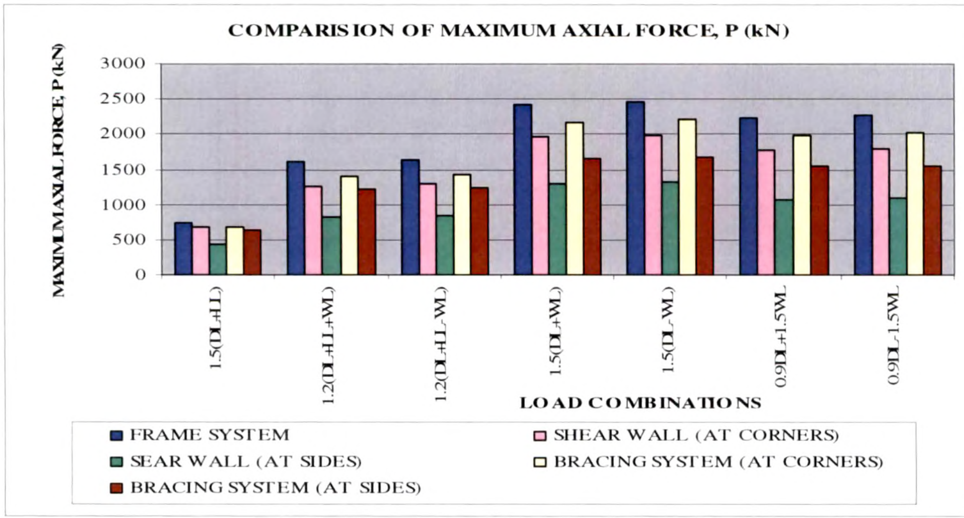


Fig.7.27 Comparison of Maximum Axial Force P - DRFM

Table 7.13 Maximum Shear Force V - DRFM

COMBINATIONS	FRAME SYSTEM	SHEAR WALL (AT CORNERS)	SHEAR WALL (AT SIDES)	BRACING SYSTEM (AT CORNERS)	BRACING SYSTEM (AT SIDES)
1.5(DL+LL)	55.23	54.939	55.807	54.253	62.349
1.2(DL+LL+WL)	354.184	270.977	108.137	308.487	229.603
1.2(DL+LL-WL)	353.904	270.561	108.071	307.845	220.575
1.5(DL+WL)	444.968	341.171	138.741	387.983	333.105
1.5(DL-WL)	443.342	339.828	139.288	386.377	320.011
0.9DL+1.5WL	442.999	338.814	134.311	385.751	289.725
0.9DL-1.5WL	440.131	336.621	134.526	383.179	281.995

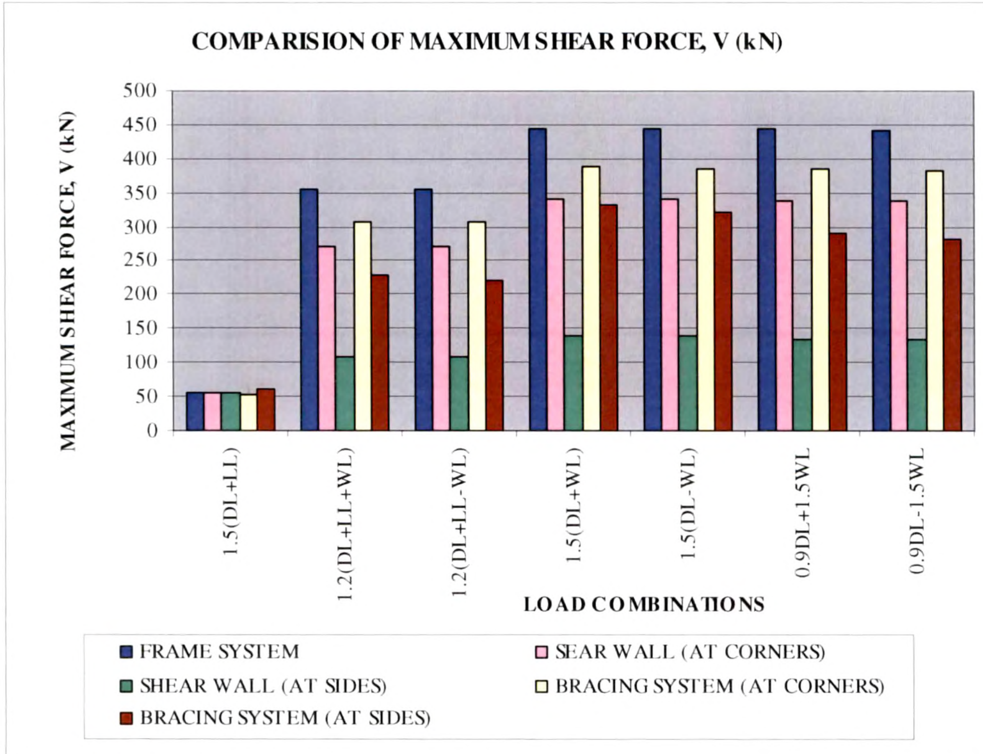


Fig. 7.28 Comparison of Maximum Shear Force V – DRFM

Table 7.14 Maximum Torsional Moment T – DRFM

COMBINATIONS	FRAME SYSTEM	SHEAR WALL (AT CORNERS)	SHEAR WALL (AT SIDES)	BRACING SYSTEM (AT CORNERS)	BRACING SYSTEM (AT SIDES)
1.5(DL+LL)	4.0864	3.9506	4.0034	3.7659	4.1584
1.2(DL+LL+WL)	14.3152	10.7266	3.8056	12.2989	10.743
1.2(DL+LL-WL)	13.9845	10.3249	4.3954	11.9162	10.8221
1.5(DL+WL)	17.9777	13.485	4.684	15.4549	13.366
1.5(DL-WL)	17.3968	12.8294	5.6074	14.814	13.6357
0.9DL+1.5WL	17.8615	13.3538	4.5134	15.3267	13.3836
0.9DL-1.5WL	17.513	12.9605	4.1251	14.9422	13.5727

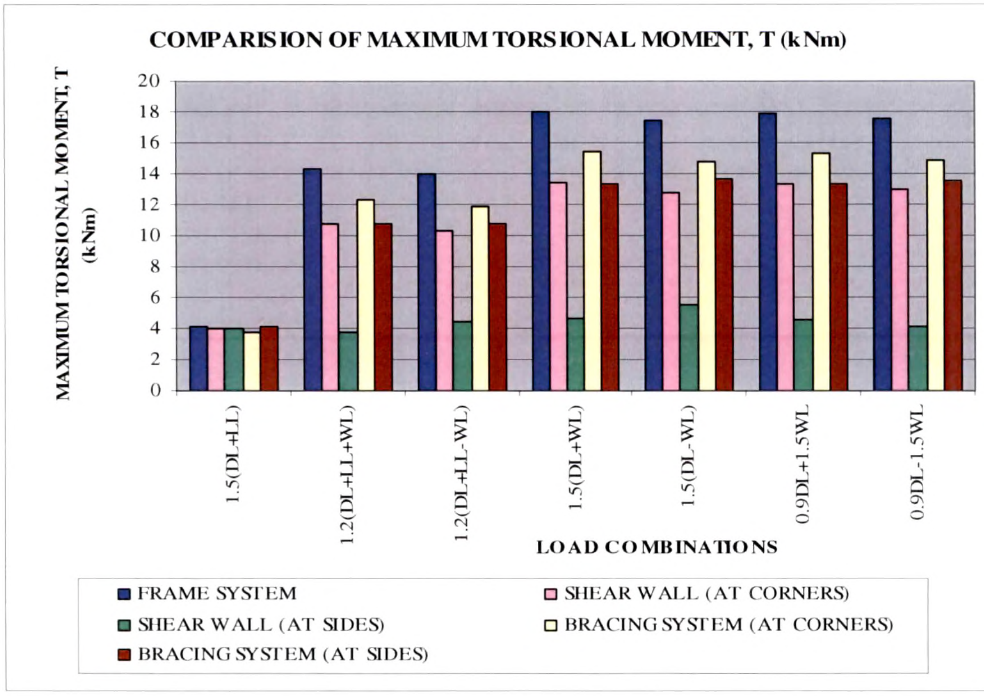


Fig. 7.29 Comparison of Maximum Torsional moment T – DRFM

Table 7.15 Maximum Bending Moment M – DRFM

COMBINATIONS	FRAME SYSTEM	SHEAR WALL (AT CORNERS)	SHEAR WALL (AT SIDES)	BRACING SYSTEM (AT CORNERS)	BRACING SYSTEM (AT SIDES)
1.5(DL+LL)	62.247	62.1159	64.2225	61.14	66.1046
1.2(DL+LL+WL)	313.6368	233.9718	86.3256	277.5878	259.4855
1.2(DL+LL-WL)	316.1917	235.5739	86.8097	280.0398	260.8924
1.5(DL+WL)	392.5979	293.0206	106.897	351.9966	275.4517
1.5(DL-WL)	395.9248	295.2546	107.5657	353.7758	329.6693
0.9DL+1.5WL	392.0736	292.4418	105.9997	345.0258	326.7451
0.9DL-1.5WL	394.1139	293.8055	106.4026	347.3007	327.7854

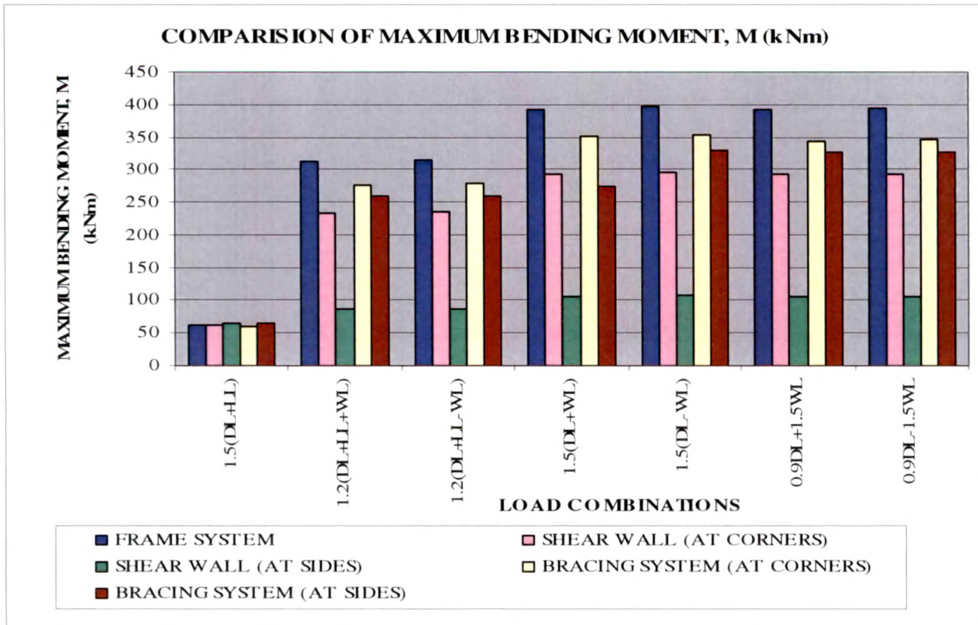


Fig.7.30 Comparison of Maximum Bending moment M – DRFM

7.8 COMPARISON OF RESULTS

7.8.1 General Comments

- The forces are higher for the frame system with shear core than for bracings at corners, shear wall at corners, bracing at sides and shear wall at sides respectively.
- From Dynamic analysis it can be seen that there is not much difference between Gust Factor Method and Dynamic Response Method.
- Due to dynamic effect of wind load the axial force, shear force, bending moment and torsional moment are increased remarkably.
- As compared to static analysis there is a remarkable change in the torsional moment for dynamic analysis. Hence torsion should not be ignored. Both current and proposed draft codes do not mention any provisions for torsion effect.

7.8.2 Comparison of the Element Forces for Dynamic Methods

To see the effects of the different structural systems on individual elements, first all the elements of frame systems, having maximum forces are obtained. Then for the same element the values of forces are checked for different structural systems. For example for the Gust Factor Method, the maximum axial force is in the frame member 442. Then for the same member values of axial force are compared for the shear wall system and the bracing system.

7.8.3 Gust Factor Method

The frame members having maximum forces are shown in **Table 7.16** and they are also compared with other systems as shown in **Figs 7.31 to 7.34**.

Table 7.16 Elements Having Maximum Forces - Gust Factor Method

FORCES	FRAME MEMBER NO.	LOAD COMBI-NATION	FRAME SYSTEM	SHEAR WALL (CORNER)	SHEAR WALL (SIDES)	BRACE SYSTEM (CORNER)	BRACE SYSTEM (SIDES)
Max. Axial Force (kN)	442	Comb. 5	2468.16	2057.208	990.35	2280.54	1459.06
Max. Shear (kN)	201	Comb. 4	462.592	354.59	143.85	403.29	342.10
Max. Torsion	351	Comb. 4	18.68	14.01	4.815	16.06	8.59
Max. Bending	64	Comb. 5	411.60	306.93	69.26	346.10	78.97

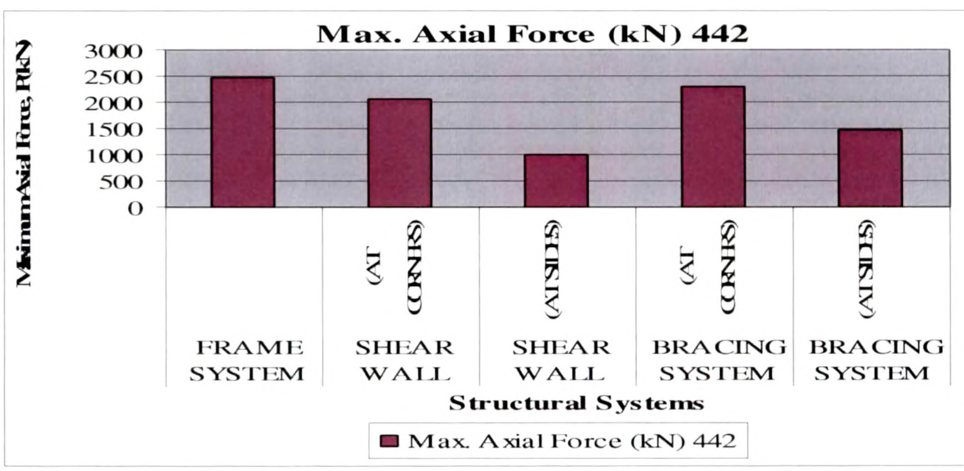


Fig. 7.31 Comparison of Axial Force of Member 442 – GFM

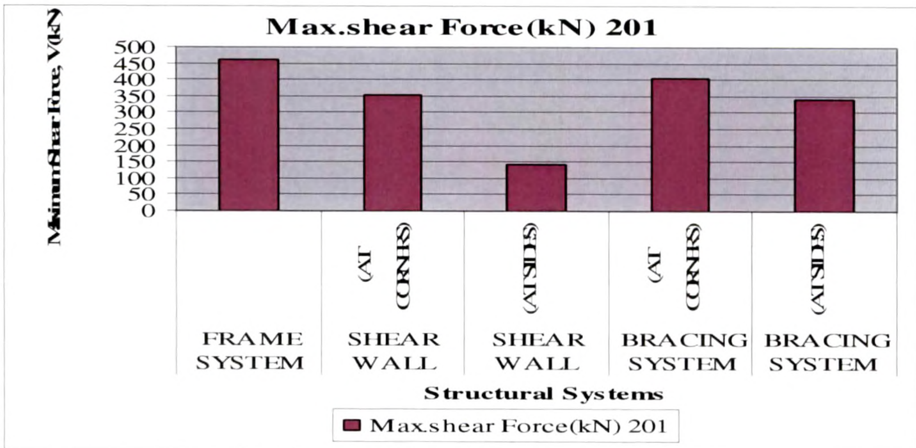


Fig. 7.32 Comparison of Shear Force of Member 201 - GFM

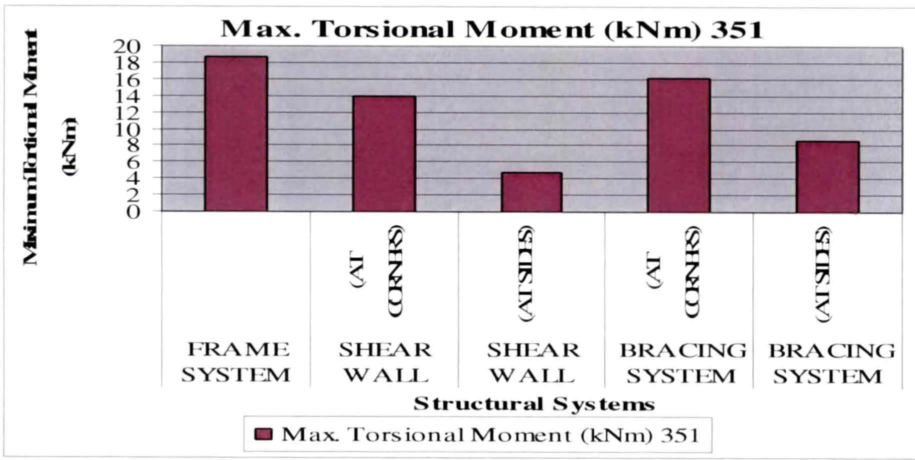


Fig. 7.33 Comparison of Torsional Moment of Member 351 for GFM

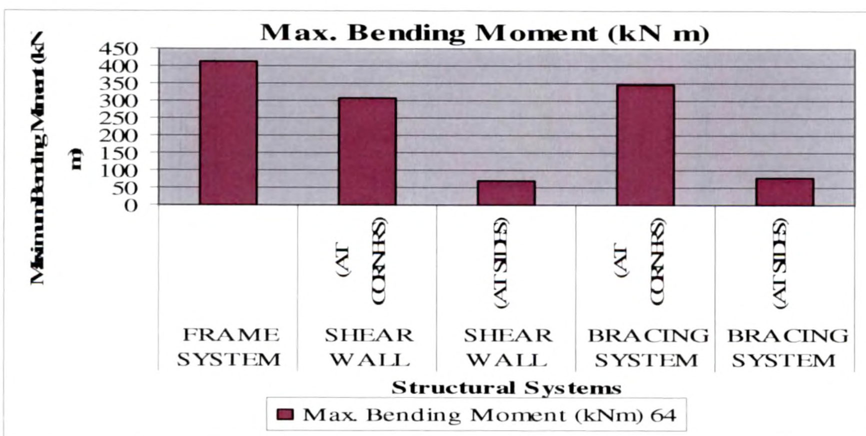


Fig.7.34 Comparison of Bending Moment of Member 64 - GFM

7.8.4 Dynamic Response Factor Method

Table 7.17 shows the members having maximum forces and comparison of these forces for different structural systems and they are compared with other systems. **Figures 7.35 to 7.38** show the comparison of different systems

Table 7.17 Maximum Forces for Dynamic Response Factor Method

Forces	Frame Memb.	Load Comb.	Frame System	Shear Wall (Corners)	Shear Wall (Sides)	Brace System (Corners)	Brace System (Sides)
Max. Axial Force (kN)	442	Comb. 5	2468.85	1996.45	969.32	2211.33	1421.18
Max. shear (kN)	201	Comb. 4	444.96	341.17	138.74	387.98	333.10
Max. Torsion	351	Comb. 4	17.977	13.48	4.64	15.45	8.28
Max. Bending	64	Comb. 5	395.92	295.25	66.69	333.04	76.06

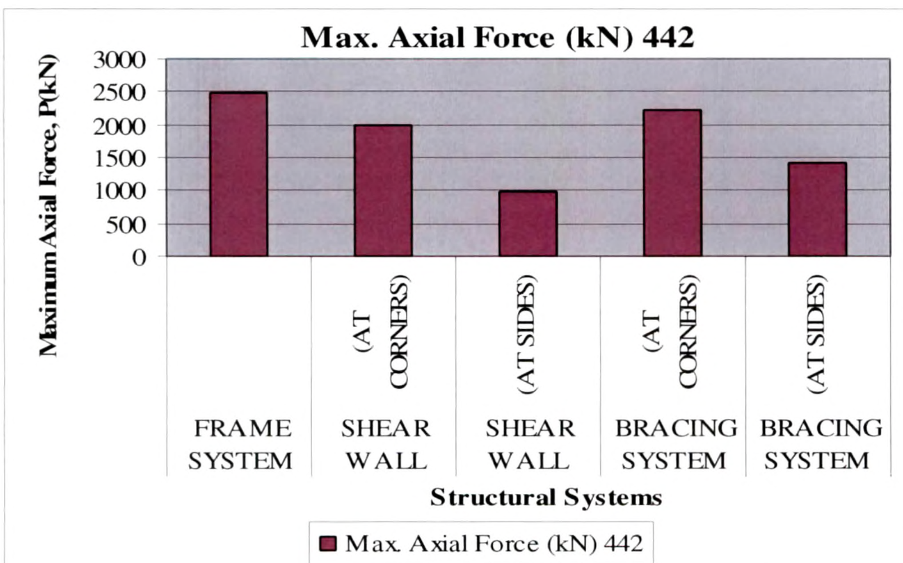


Fig. 7.35 Comparison of P of Member 442 - DRFM

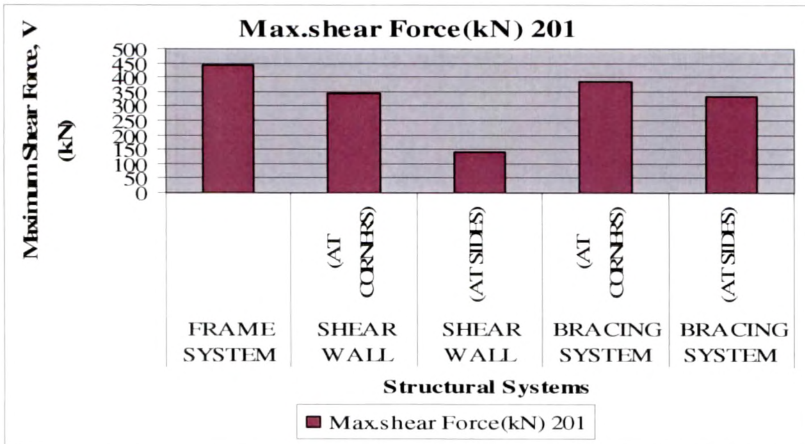


Fig. 7.36 Comparison of V of Member 201 – DRFM

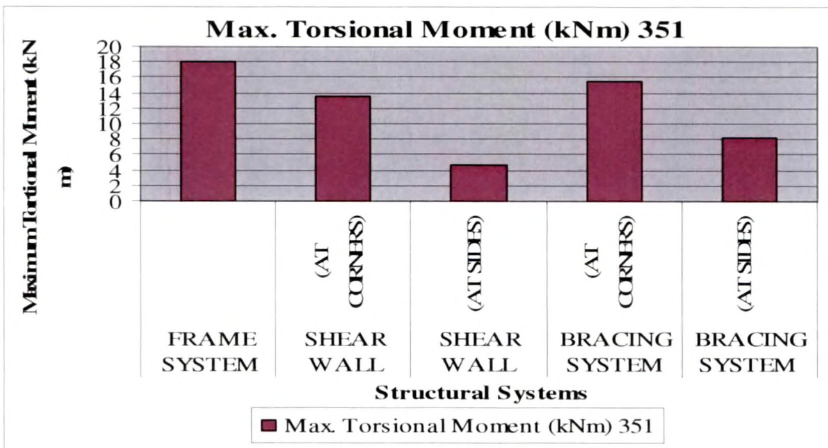


Fig. 7.37 Comparison of T of Member 351 - DRFM

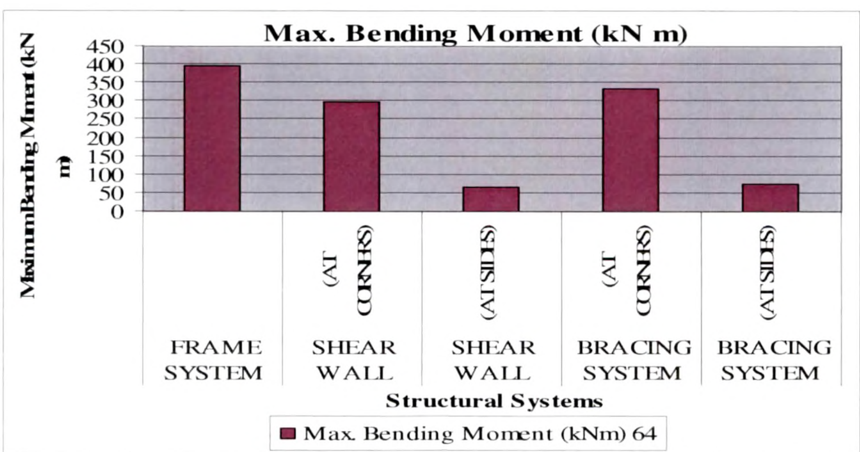


Fig. 7.38 Comparison of BM of Member 64 - DRFM

From the figures 7.31 to 7.38, it can be concluded , we can conclude that, for both the methods the forces are reducing by adding shear walls and bracings to the frame system. Overall the forces are lesser in the shear wall system as compared to the bracing system. The axial force and shear force are much lesser when the shear walls are provided at the sides as compared to the side bracings and the bending moments and the torsional moments are lesser when both the shear walls and the bracings are provided at the sides.

7.9 VIRTUAL REALITY MODULE

Dynamic wind load has been calculated through VC++ program and transferred to SAP where the 93 m high building is then analysed for the static and dynamic loads. As of now the best of commercial softwares are equipped to give output in the form of 2D or 3D wireframe drawings. Thus here too the geometry and outputs can be viewed in the postprocessor of SAP2000 along with animation options for viewing the displacements and story drifts under the effect of high velocity winds. The engineer is thus once again left with no choice but to pour over voluminous numerical tables and switch between the graphical interface for viewing the behaviour of the building.

No doubt such high end graphics facilities have stretched the realm of the design engineer to more precise and real life loads and responses of the built environment. But it is time now to move further and exploit non linearly the capabilities of high end computing and 3D graphics animation to break new ground in structural engineering. An attempt has been made here to interface 3DS MAX with SAP inorder to import the results into a virtual environment where the user is part of the scenario and can venture to touch or change or explore as per his wish certain parameters of the building, specific to his interest.

7.9.1 Virtual Reality Environment

As mentioned earlier 3DS Max is a powerful tool to create 3 dimensional animation of any application. It has state-of-the-art in-built tools to depict real-life like scenarios with the facilities to zoom, pan, rotate any of the lights and camera in the 4 viewports in its working canvas as shown in **Fig. 7.39**. Four Viewports are displayed wherein the top view displays the plan from the top along with the position of the camera focused on the building. The other view ports display the front, back and perspective views as seen from the various cameras focused on it along with the lights. Thus what was trapped in the tables or 2D graphics as output, is available at the touch of a button in any viewport. The geometry has been created through MaxScript wherein a script file is written to read the excel output of SAP or any other software. Within the 3DS Max environment the user has the freedom to animate the frame to observe the displacements.

Figs 7.39 to 7.43 show the various options of retrofitting and strengthening of a building against high velocity wind loads.

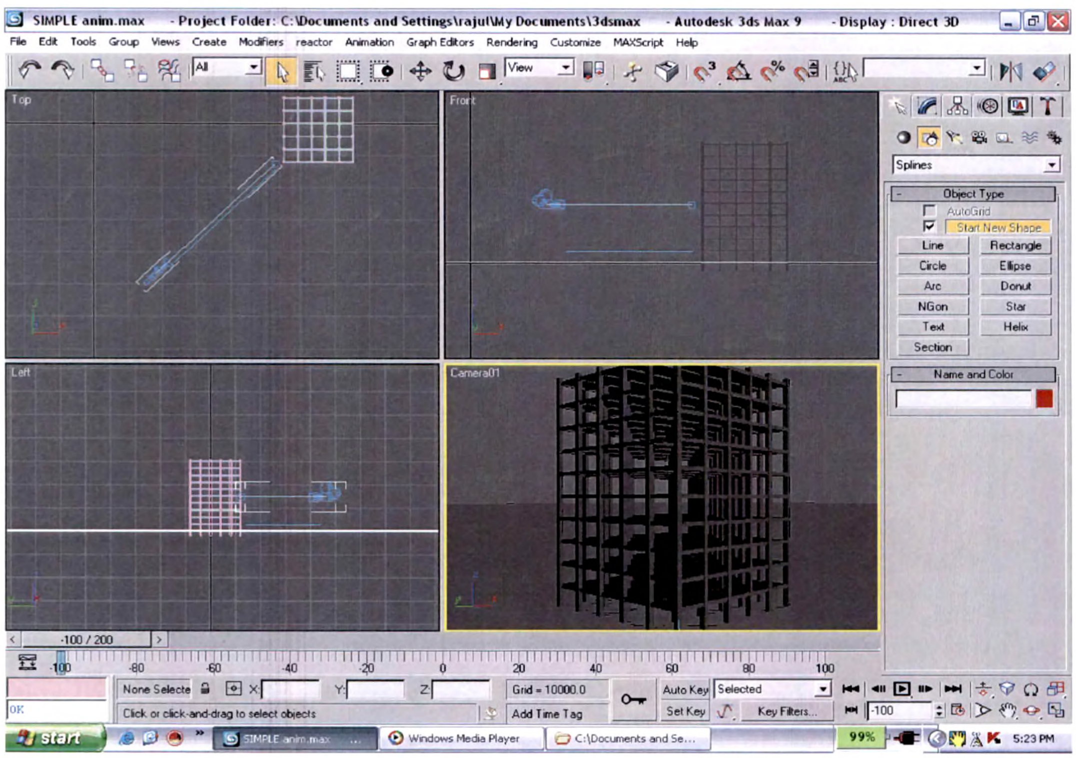


Fig. 7.39 3DS Max Project for Moment Resisting Frame

A tall building subjected to high velocity winds has been analysed as a simple Moment Resisting Frame (**Fig. 7.39**), then as a frame with Shear walls at corners (**Figs. 7.40 - 7.42**) and finally as a frame with Bracing at the corners (**Fig. 7.43**). These are taken as sample case studies to demonstrate the use of VR in a purely structural engineering scenario. The output displacements of each option for various Basic Wind Speeds are built into the backend, so that the user just has to choose which case he wants to study. In short what would have been volumes and volumes of data in the form of numbers and 2-D graphics has been transformed into a totally new avtar which has not been explored so far in the realm of structural engineering.

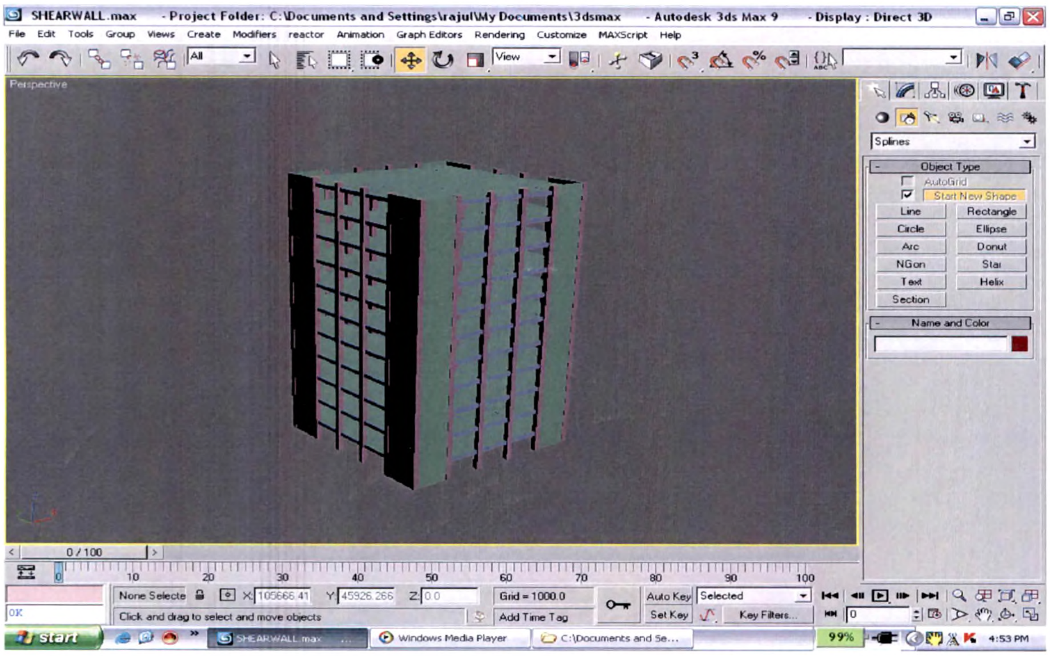


Fig. 7. 40 Frame with Shear Walls at Corners in Single Viewport

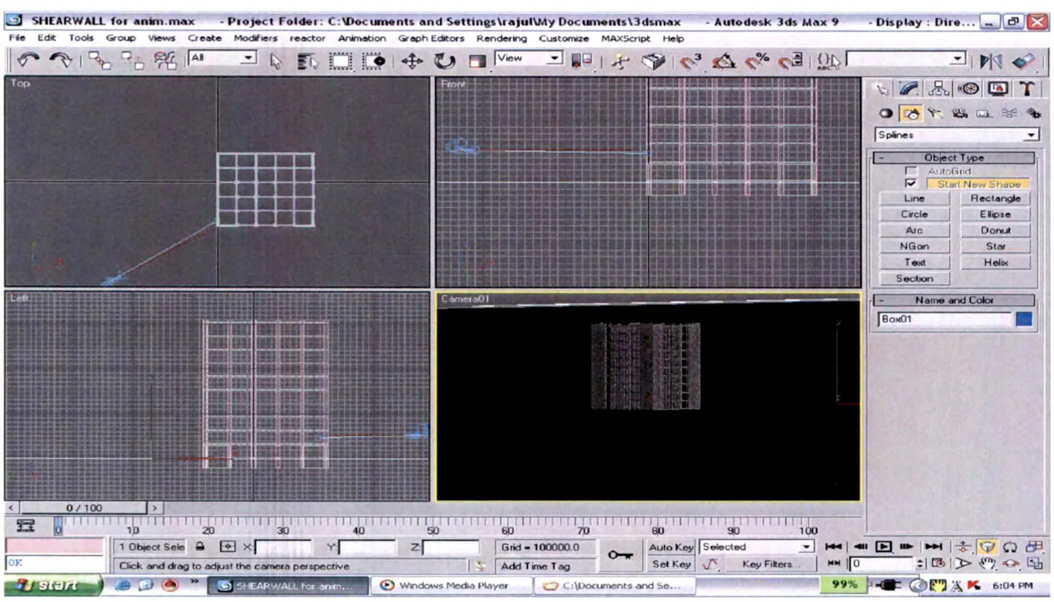


Fig. 7. 41 Frame with Shear Walls at Corners

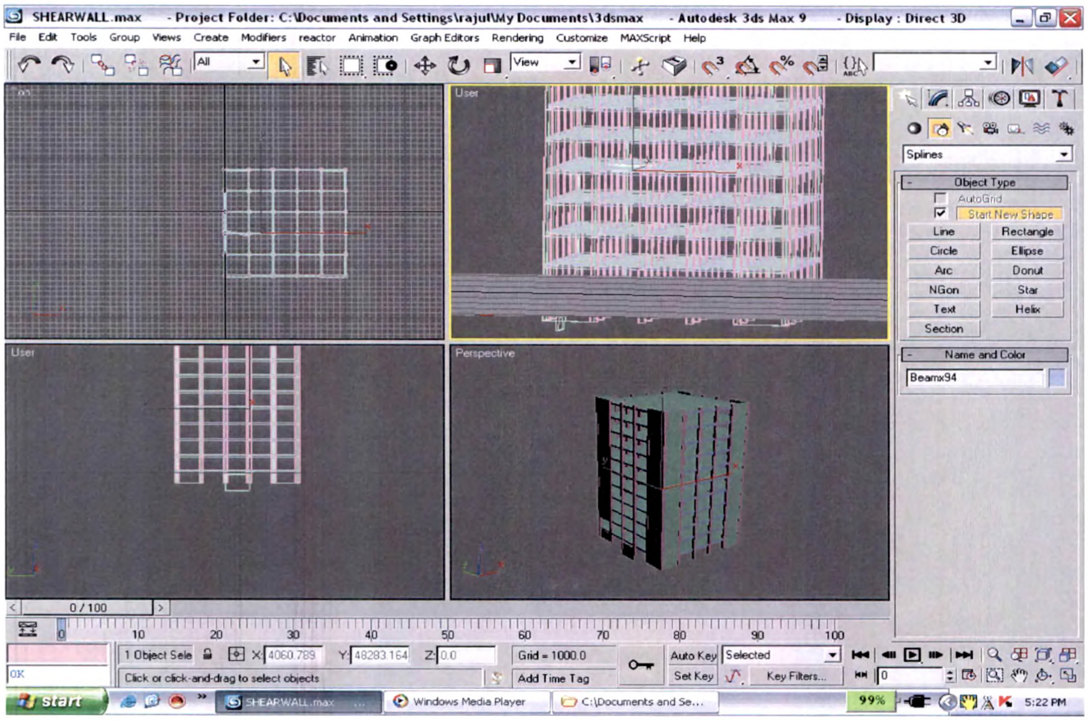


Fig. 7. 42 Frame with Shear Walls at Corners in User Defined Camera

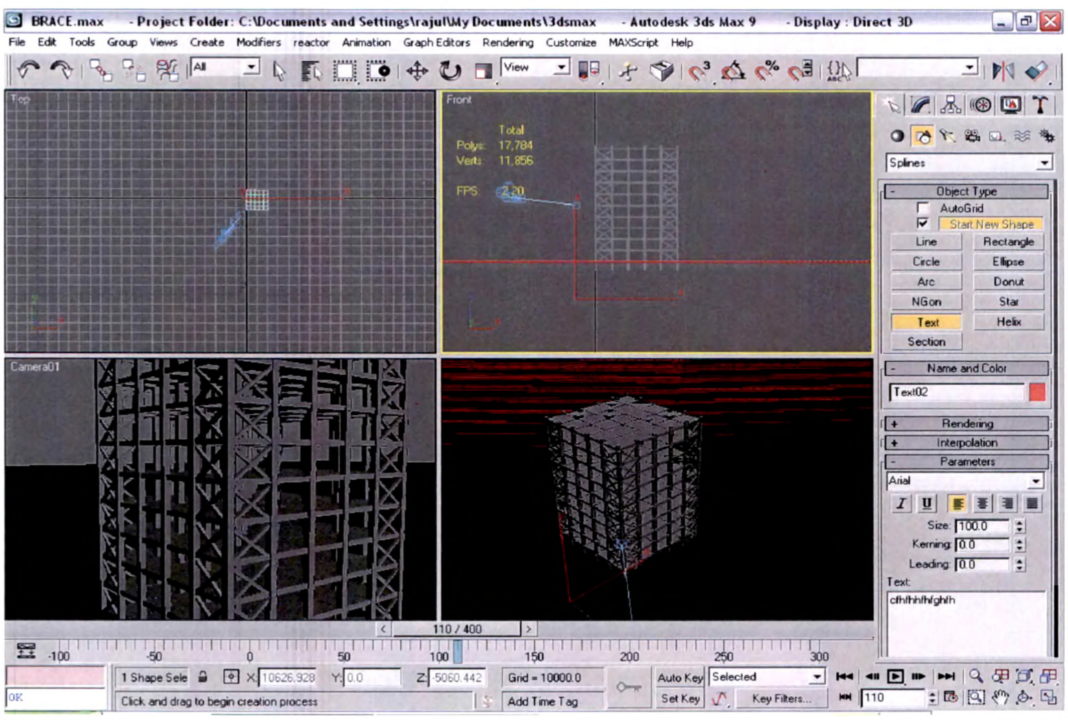


Fig. 7. 43 Frame with Bracing at Corners in 2 Camera Positions

7.9.2 VRML Platform

The 3D max files can either be used as they are or can be converted into avi files in the form of a movie wherein the selected option will be played out in a predefined sequence of camera and light positions. Another option is to convert these to VRML (Virtual Reality Modeling Language) files. VRML plug-ins are freely available on the world wide web for downloading and serve the purpose of converting the 3D Max files onto the virtual reality platform. The option of such conversion is a built-in tool in 3D Max itself and can be fired as a command by pressing a single icon. Once the user has been put on a VRML platform, his ease of manipulation, observation and interpretation of various alternatives take prime importance. As shown in **Fig. 7.44**, the building with bracing on the VRML platform of Cortona can be viewed from any angle and at any height through the commands of zoom, pan, rotate. Besides this, the user has the choice to fly, walk or study any parameter or any member as per his choice. Even if the user had physically visited a real time site such an option would not be available to him due to the constraints of the space and time on a construction site. This added advantage of a Virtual World give a new dimension to the structural analysis and design performed on the most efficient software available in the market.

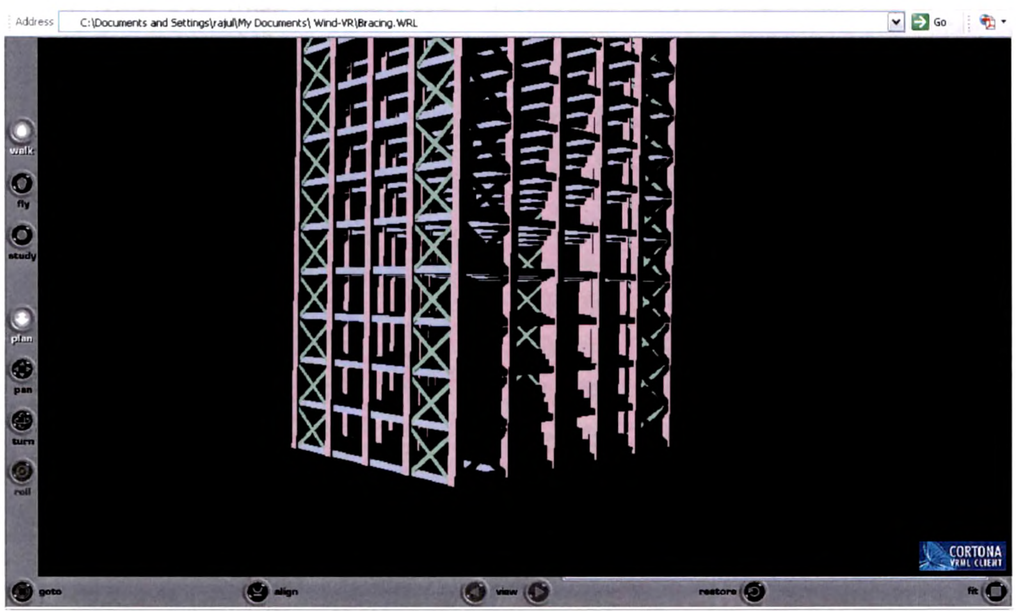


Fig. 7. 44 Frame with Bracing at Corners in VRML Plug-in Module

The Virtual platform can further be exploited to store hundreds of reruns of the various options for future study, or as database for futher development and research. The best use of this powerful tool is that the users can interact across the globe, on details at a micro-level at no cost and saving hundreds of technical man hours as well as construction costs.

7.10 CLOSURE

This chapter was devoted to development of a software at developing a comprehensive software for wind as a static load, as a gust factor load on slender buildings as per IS-875 (1989) and also as per the international standards of Dynamic Response Factor (ATC – 60). The Analysis results are used through SAP implementation, to find the most suitable strengthening option in terms of shear core, shear walls on the edges and bracing systems. The automation has been further expanded beyond the realm of routine software post processors by adding Virtual Reality Modules to the automation. Thus all three aspects of prevention, mitigation and retrofitting post disaster can be reviewed and manipulated in the virtual world without any cost in terms of time, energy and revisions on site.