LOADS ON TALL BUILDINGS

GENERAL

The loading on tall buildings is quite different from that on low - rise buildings. The collection of gravity loading over a large number of stories in a tall building can produce column loads of higher order. Wind loading on a tall building acts not only over a large building surface, but also with greater intensity at greater heights and with a larger moment arm about the base. Wind on a tall building can have a dominant influence on its structural arrangement and design. In earthquake regions, inertial loads resulting from ground shaking may well exceed the loading due to wind and therefore be dominant in influencing the buildings structural form, design and cost. The gravity loading can be assessed with reasonable accuracy while wind and earthquake loadings are difficult to predict with same accuracy. The application of probabilistic theory has helped to simplify the approaches for estimating wind and earthquake loading.

DEAD LOADS

The most important dead load is the self weight of the structure. D.L. can be calculated from the designed member sizes and estimated material densities. This is pone to minor inaccuracies such as differences between the real and designed sizes, and between the actual and assumed densities. The weight of finish depends on the type of floor finish.

IMPOSED LOADS

LIVE LOAD

L.L. is specified as UDL according to the occupancy or use of the space. In parking areas, offices and plant rooms, the floors should be considered for the alternative worst possibility of specified concentrated loads. In bridge structures, they are considered as specified moving concentrated loads. They may also include a dynamic coefficient depending on the type of structure. The magnitude of LL are estimates based on a combination of experience and the results of typical field surveys. Pattern distribution of LL over adjacent and alternate spans should be considered for estimating the local maxima of member forces while LL reductions may be allowed to account for the improbability of total loading being applied simultaneously over larger areas.

No. of floors supported by columns	Reduction in LL on all floors (%)
1	0
2	10
3	20
4	30
5 to 10	40
Over 10	50

CLIMATIC LOADS

Loads due to wind, snow, etc. are known as climatic loads. The wind loads that are prescribed for the given region should be used for the design of structures in a given location. The loads induced in a restrained structure due to temperature changes also need to be considered in design. This depends on the maximum and minimum temperatures experienced by that region.

EXCEPTIONAL OVERLOADS

Loads due to cyclones, hurricanes, tornados, earthquakes, etc. come under this category. Values assessed from the past records and available in codes of practice are to be used for the purpose of design.

DYNAMIC SUPERIMPOSED LOADS

Loads produced on structures by the dynamic effects, vibration and impact caused by machinery or mobile equipment come under this category. The values given in codes of practice are to be used.

CONSTRUCTION LOADS

Construction loads need special attention in tall buildings. Many failures occur in buildings under construction than those that are complete. Construction load is the weight of the floor forms and a newly placed slab, which in total, may equal twice the floor dead load. This load is supported by props that transfer to the previously constructed floors below. The problem is more severe with the possibility of 3 - day cycle or 2 - day cycle storey construction and especially with pumped concrete. This is because, the newly released slab, rather than contributing to supporting the construction loads, is still in need of support itself.

The climbing crane is another common construction load. This is usually supported by connecting it to a number of floors below with additional shoring in stories further below.

SEQUENTIAL LOADING

For loads that are applied after completion of the building, such as live, wind or seismic loading, the analysis is independent of the construction sequence. For dead loads, which are applied to the building frame as construction proceeds, the effects of sequential loading should be considered to assess the worst condition to which any component may be subjected, and also to determine the true behavior of the frame.

In multistory RC construction, the usual practice is to shore the freshly placed floor on several previously cast floors. The construction loads in the supporting floors may appreciably exceed the loads under service conditions. Such loads depend on the sequence and rate of erection.

If the column axial deformations are calculated as though the dead loads are applied to the completed structure, bending moments in the horizontal components will result from any differential column shortening that is shown to result. Because of the cumulative effects over the height of the building, the effects are greater in the highest levels of the building. The effects of such differential movements would be greatly overestimated because in reality, during the construction sequence, a particular horizontal member is constructed on column in which the initial axial deformation due to the dead weight of the structure up to that particular level have already taken place. The deformation of that particular floor will then be caused by the loads that are applied subsequent to its construction. Such sequential effects must be considered if an accurate assessment of the structural actions due to dead loads is to be achieved.

LOADS DUE TO IMPOSED DEFORMATIONS

These loads may be due to one or more of the following effects:

- Shrinkage
- Temperature
- Prestressing
- Movement of supports

These loads are estimated from the knowledge of materials, climate, soil properties, erection methods, etc. used for the structure under consideration.

WIND LOADS

Important Design Considerations

Strength

(i) Main Wind Force Resisting System

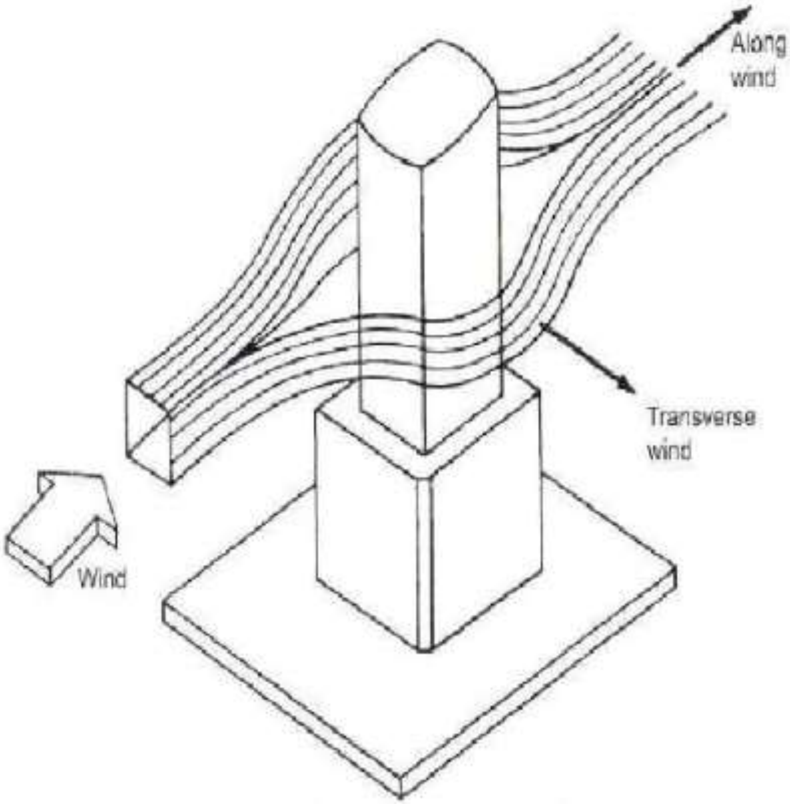
(ii) Cladding & Architectural Features

Stiffness

(i) Drift

(ii) Dynamic Response

2D Flow of Wind around a Building



GENERAL

The lateral loading due to wind is a major factor that causes the design of tall buildings to differ from those of low - to medium - rise buildings. For buildings of up to about 10 stories and of typical proportions, the design is not much affected by wind loads. With innovations in architectural treatment, increases in material strengths and advances in analysis methods, tall buildings have become more efficient and lighter and hence more prone to deflect and even to sway under wind loading. This has triggered significant advances in understanding the nature of wind loading and in developing methods for its estimation. These developments have been mainly in experimental and theoretical techniques for determining the increase in wind loading due to gusting and the dynamic interaction of structures with gust forces.

In IS:875 (Part 3), a single map of India gives the basic maximum wind speed as peak gust velocity averaged over a short time interval of 3 sec. duration. The map of India is divided into 6 zones with basic wind speeds of 33, 39, 44, 47, 50 and 55 m/sec. corresponding to 120, 140, 160, 170, 180 and 200 km/hr.

The highest peak wind speed of 55 m/sec. is assigned to the hilly regions around Silchar and Ladakh. For most northern parts of India, the assigned wind speed is 47 m/sec. The wind speed assigned to the east coast of India and the coastal regions of Gujarat where cyclones are possible is 50 m/sec.


INDIA

DESIGN WIND SPEED

The designed wind speed for a place is obtained from the basic wind speed as

$$V_z = V_b\, k_1\, k_2\, k_3$$

where V_z = design wind speed at height z

V_b = basic wind speed for the site as shown on the map of India. It is the peak gust speed averaged over 3 sec. for terrain category 2 at 10 m above ground level on 50 year mean return period.

k_1 = probability factor or risk coefficient with return periods

k_2 = factor for the combined effects of terrain height and size of the component on structure

k_3 = factor for local topography (hills, valley, cliffs, etc.)

K1 Factor

As the basic wind speeds are given for terrain category 2 based on a return period of 50 years and the return period that is taken for different types of structures will be different, a corresponding reduction in wind speeds has to be made for lesser return periods and an increase in basic wind speeds for higher return period. This probability also depends on the basic wind speed. The values of k_1 recommended are given in IS : 875 (Part 3) - Table 1 - Page 11.

S.No.	Class of Structure	Design life in years	Basic Wind Speed in m/sec.					
			33	39	44	47	50	55
1	General Buildings	50	1.0	1.0	1.0	1.0	1.0	1.0
2	Temporary Sheds	05	0.82	0.76	0.73	0.71	0.70	0.67
3	Structures with hazards	25	0.94	0.92	0.91	0.90	0.90	0.89
4	Important Buildings	100	1.05	1.06	1.07	1.07	1.08	1.08

K2 Factor

The terrain category used in design shall be made with due regard to the effect of obstructions which constitute the ground surface roughness. The terrains are classified into the following four categories.

- Category 1 - Exposed open terrain with few or no obstructions and in which the average height of any object surrounding the structure is less than 1.5 m.
- Category 2 - Open terrain with well scattered obstructions having heights generally between 1.5 to 10 m.
- Category 3 - Terrain with numerous closely spaced obstructions having the size of building structures upto 10 m in height with or without a few isolated tall structures.
- Category 4 - Terrain with numerous large high closely spaced obstructions.
- The buildings/structures are classified into the following three categories depending upon their size.

- **Class A** - Structures and/or their components such as cladding, glazing, roofing, etc. having maximum dimension less than 20 m.
- **Class B** - Structures and/or their components such as cladding, glazing, roofing, etc. having maximum dimension between 20 m and 50 m.
- **Class C** - Structures and/or their components such as cladding, glazing, roofing, etc. having maximum dimension greater than 50 m.

- Table 2.3 (Table 2 - Page 12) gives multiplying factors by which the basic wind speed shall be multiplied to obtain the wind speed at different heights, in each terrain category for different sizes of buildings/structures.

Height (m)	Terrain 1			Terrain 2			Terrain 3			Terrain 4		
	A	B	C	A	B	C	A	B	C	A	B	C
10	1.05	1.03	0.99	1.00	0.98	0.93	0.91	0.88	0.82	0.80	0.76	0.67
15	1.09	1.07	1.03	1.05	1.02	0.97	0.97	0.94	0.87	0.80	0.76	0.67
20	1.12	1.10	1.06	1.07	1.05	1.00	1.01	0.98	0.91	0.80	0.76	0.67
30	1.15	1.13	1.09	1.12	1.10	1.04	1.06	1.03	0.96	0.97	0.93	0.83
50	1.20	1.18	1.14	1.17	1.15	1.10	1.12	1.09	1.02	1.10	1.05	0.95
100	1.26	1.24	1.20	1.24	1.22	1.17	1.20	1.17	1.10	1.20	1.15	1.05
150	1.30	1.28	1.24	1.28	1.25	1.21	1.24	1.21	1.15	1.24	1.20	1.10
200	1.32	1.30	1.26	1.30	1.28	1.24	1.27	1.24	1.18	1.27	1.22	1.13
250	1.34	1.32	1.28	1.32	1.31	1.26	1.29	1.26	1.20	1.28	1.24	1.16
300	1.35	1.34	1.30	1.34	1.32	1.28	1.31	1.28	1.22	1.30	1.26	1.17
350	1.37	1.35	1.31	1.36	1.34	1.29	1.32	1.30	1.24	1.31	1.27	1.19
400	1.38	1.36	1.32	1.37	1.35	1.30	1.34	1.31	1.25	1.32	1.28	1.20
450	1.39	1.37	1.33	1.38	1.36	1.31	1.35	1.32	1.26	1.33	1.29	1.21
500	1.40	1.38	1.34	1.39	1.37	1.32	1.36	1.33	1.28	1.34	1.30	1.22

K3 Factor

The local topographical features such as hills, valleys, cliffs significantly alter the wind speed. It is a general observation that winds are higher near the summit and lower in valleys. This effect becomes important only on slopes higher than 3^0 below which the value of k_3 is to be taken as unity. Above 3^0, the value of k_3 can vary from 1.00 to 1.36. This factor becomes important in hilly regions in South India and the northern boundary states of India. Appendix C of IS 875 (Part 3) (Pages 55-57) gives the details of the method of computation of k_3 factor.

GENERAL WIND EFFECTS ON STRUCTURES

Most of the man-made structures stand bluff (perpendicular) and the wind blows horizontally. As the structures are not generally made streamlined or smooth, wind-flow over these obstructions tends to be turbulent. In addition, the velocity of wind itself varies with time. The stiffness also varies from structure to structure. Only in recent times, a unified approach to wind loads on all types of structures has been attempted.

The wind speed can be considered as fluctuating with a steady component and a fluctuating component superimposed on it. The structure stiffness can vary from a very stiff structure to a flexible structure. With a very stiff structure, the movement of the structure with wind will be negligible and the wind effect can be analysed by a quasi-static approach. As the structure stiffness decreases, the deflection, natural frequency and damping characteristics of the building have to be considered in the calculation of the effect of wind on the structure. The wind forces on a deflected structure will be different from that of a stiff structure. In addition, if the natural frequency of the structure is low, there is a chance for the frequency of the fluctuating part of wind to coincide with the natural frequency of the structure and resonance can result. The deflection can become very large and the forces related to mass and acceleration of the structure can be much greater than the wind force itself. Under these circumstances, natural frequency, structural damping, etc. are of great importance.

In addition to static pressures we need to consider the dynamic effects in structures. The response of the structure in the direction of wind is called along - wind response and that normal to the direction of wind is called across - wind response.

When a bluff body is exposed to wind, vortices tend to be shed from the sides of the body creating a pattern in its wake called the vortex shedding. The frequency of shedding depends mainly on the shape and size of the body and the velocity of flow. This can cause across-wind effects such as transverse vibrations in lightly damped structures such as chimneys and towers.

In addition to the static and dynamic effects of wind on individual members, we need to consider the effects of adjacent structures on the pattern of flow of the wind itself and on each other. These are known as aerodynamic interference effects. The venture effect and reverse - flow effect belong to this category. If one structure is located in the wake of another, the vortices shed from the upstream structure may cause oscillations of the downstream structure. This is called wake buffeting. These interference effects are best studied by wind tunnel tests than by calculations.

Wind Effects on Structures

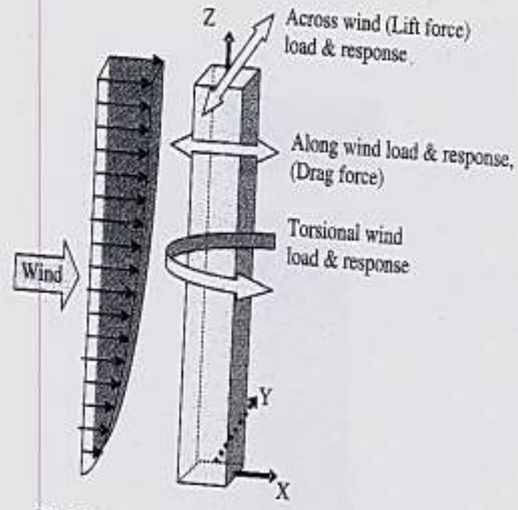


Fig.1 Primary Wind Load Effects on Tower-Like Structures

Peak Acceleration

Along-wind Response

$$a_D = 4\pi^2 n_D^2 g_p \sqrt{\frac{KsF}{C_e \beta_D}} \frac{\Delta}{C_g}$$

a	= Acceleration (ft/s^2)
n	= Fundamental Frequency (Hz)
g_p	= Peak Factor
K	= Surface Roughness Coefficient
s	= Size Reduction Factor
F	= Gust Energy Ratio
C_e	= Exposure Factor
β	= Critical Damping Ratio
Δ	= Top Displacement (ft)
C_g	= Gust Response Factor

Peak Acceleration

Across-wind Response

$$a_W = n_W^2 g_p \sqrt{WD} \left(\frac{a_r}{\gamma g \sqrt{\beta_W}} \right)$$

a = Acceleration (ft/s^2)
n = Fundamental Frequency (Hz)
g_p = Peak Factor
W = Cross-wind Plan Dimension (ft)
D = Along-wind Plan Dimension (ft)
a_r = Acceleration Factor (ft/s^2)
γ = Average Building Density (lbs/ft^3)
g = Acceleration Due to Gravity (ft/s^2)
β = Critical Damping Ratio

Vortex Shedding

Cross-wind Response Usually Controls When

$$\sqrt{\frac{WD}{H}} \leq \frac{1}{3}$$

Slender

W = Cross-wind Plan Dimension (ft)
D = Along-wind Plan Dimension (ft)
H = Building Height (ft)

- ***The rough guide given in IS 875 (Part 3) (Pages 47- 48) states that only buildings or structures which has either of the following conditions needs to be examined for the dynamic effects of wind.***

- ***Buildings and closed structures with a height to*** **minimum lateral dimension ratio of more than about 5.0.**
- **Buildings and closed structures whose natural frequency in the first mode is less than 1.0 Hertz.**

- **Any building or structure which does not satisfy either of the above two criteria shall be examined for the dynamic effects of wind.**

- **Note 1**: The time period T for MS buildings may be determined
- as

$$T = 0.1n \quad \text{(for MRF without bracing or shear walls)}$$

- where n is the number of storeys including basement storeys
- and
- $T = 0.09H/d^{1/2}$ (for all others)
- where H = total height of the main structure in metres and
- d = maximum base dimension in metres

- **Note 2 :** The design engineer must be aware of the following wind - induced motions.

- **Galloping** is transverse oscillation due to aerodynamic forces which are in phase with the motion. Non-circular sections are prone to this type of excitation. Twisted cables and cables with ice encrustations are also prone to this type of excitation.

- **Flutter** is unstable oscillatory motion due to coupling between aerodynamic force and elastic deformation of the structure. Long span suspension bridge decks or any member of a structure with large values of d/t (d is the depth of the structure or structural member parallel to wind stream and t is the least lateral dimension of the member) are prone to low speed flutter.

- **Ovaling** is associated with thin walled structures with open ends such as oil storage tanks and natural draught cooling towers. These oscillations are characterized by periodic radial deformation of the hollow structure.

Conductor Galloping

FORCE COEFFICIENTS FOR STRUCTURE AS A WHOLE

- The force coefficients given in the wind codes are used for determining the overall wind load on the structure taken as a whole. The total wind force is given as

- $F = C_f \; A_e \; p_a$

- where C_f = force coefficient (Table 23 of IS: 875 - part 3)

- A_e = effective frontal area of the structure

- p_a = design wind pressure

WIND TUNNEL EXPERIMENTAL METHOD

- If the building is very tall or if it is located in extremely severe exposure conditions, the effective wind loading on the building may be increased by dynamic interaction between the motion of the building and the gusting of the wind. The best method for assessing such dynamic effects is by wind tunnel testing.

- The pressure or force coefficients so developed are then used in calculations.

If the slenderness or flexibility is such that its response to excitation by gusting may significantly influence effective wind loading, the test should be fully dynamic. In this case, the elastic structural properties and the mass distribution of the building as well as the relevant wind characteristics should be modeled.

- For this test, the models are constructed to scales varying from 1/100 to 1/1000 depending on the structure size and the size of the wind tunnel. Tall buildings typically exhibit a combination of shear and bending behaviour that has a fundamental sway mode comprising a flexurally shaped lower region and a relatively linear upper region. This can be represented by a rigid model with a flexurally sprung base. When additional modes of oscillation are possible, complex models are used. These models consist of lumped masses, springs and flexible rods designed to simulate the stiffness and mass properties of the prototype.

Wind pressure measurements are made by flush surface pressure taps on the faces of the models and pressure transducers are used to obtain the mean, root mean square and peak pressures. The wind characteristics that need to be generated in a wind tunnel are the vertical profile of the horizontal velocity, turbulence intensity and the power spectral density of the longitudinal component. Boundary layer wind tunnels have been used for this purpose. Some use long working sections in which the boundary layer develops naturally over a rough floor and some use short working sections comprising grids, fences or spires while some others activate the boundary layer by jets.

TYPES OF WIND TUNNEL

1. Aeronautical Type and Boundary Layer Type
2. Open Circuit and Closed Circuit
3. Suction Type and Blower Type
4. High Speed and Low Speed
5. Open Jet and Closed Jet
6. Long Test Section and Short Test Section

WIND TUNNEL MODELS

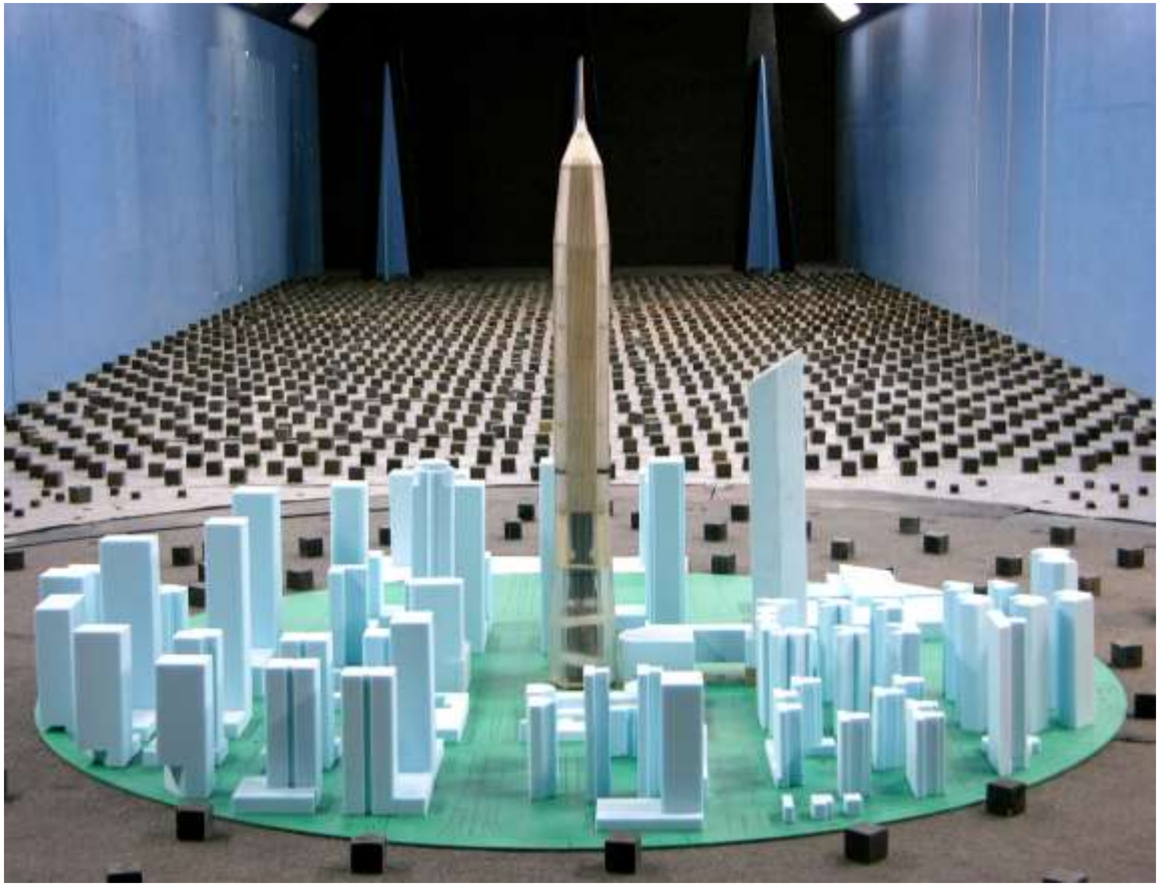
1. Rigid Models

2. Aeroelastic Models

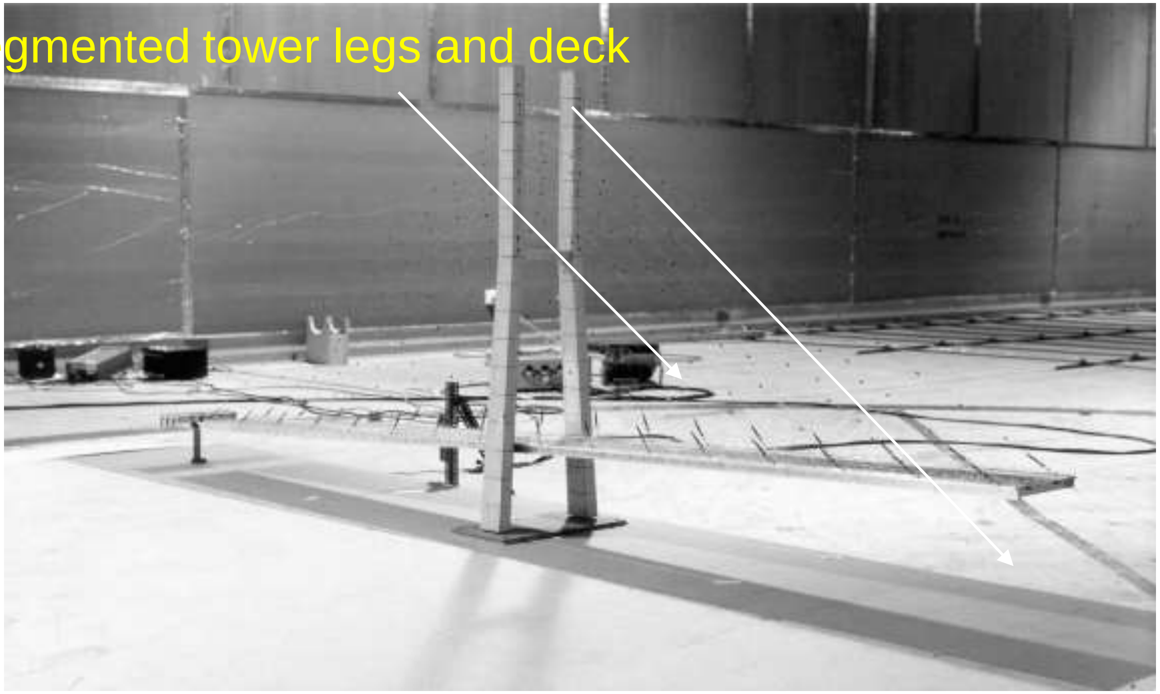
3. Linear Models

4. Sectional Models

5. Aerodynamic Models



CABLE STAYED SUSPENSION BRIDGE



SEISMIC LOAD

GENERAL

- Earthquake loading consists of the inertial forces of the building mass that result from the shaking of its foundation by a seismic disturbance. The seismic resistant design hinges more on the translational inertia forces than the vertical or shaking components. The earthquakes may be severe, moderate or minor. Severe earthquakes are rare, moderate earthquakes occur often and minor earthquakes are frequent. The general philosophy of EQR design is based on the principles that they should resist minor earthquakes without damage; resist moderate earthquakes without structural damage; resist severe earthquakes without significant damage.

SEISMIC ZONING

- **The earthquake activity in different parts of the country is not of the same severity. Hence the country has been classified into 4 zones so that the forces for which the structures are to be designed at any site are varied according to the severity of the probable earthquake intensities.**

- **Zone V - Risk of widespread collapse and destruction (MSK IX or greater)**
- **Zone IV - Risk of partial collapse and heavy damage (MSK VIII likely)**
- **Zone III - Risk of moderate damage (MSK VII likely)**
- **Zone II - Risk of minor damage (MSK VI likely)**

- **It is to be noted that the above zoning is directly applicable for the design of average structures. But for the design of important structures, it is necessary to conduct geological and seismological studies for the site in question for arriving at site-dependent design parameters and seismic forces to cater for future earthquakes.**

EARTHQUAKE MAGNITUDE AND INTENSITY

- **The magnitude of earthquake is a measure of the energy released and is measured by the Richter scale. Richter's magnitude is defined as the logarithm to the base ten of the maximum trace amplitude recorded in mms by a seismometer located at 100 km from the epicenter of an earthquake.**

- **The intensity of an earthquake is an estimate of the perceived local effects which depends on the peak acceleration velocity and duration. The scales used for this purpose include MMI scale and MSK scale.**

EQUIVALENT LATERAL FORCE

The design lateral force shall be first computed for the building as a whole and then shall be distributed to various floor levels.

The design seismic base shear can be determined as $V_B = A_h W$

where A_h **= design horizontal seismic coefficient =** $ZIS_a/2Rg$

W **= seismic weight of the building**

Z **= Zone factor (Table 2 - IS:1893 (part 1))**

I **= Importance factor (Table 6)**

R **= Response reduction factor (Table 7)**

S_a/g **=Average response acceleration coefficient**

The seismic weight of the building is the sum of the seismic weights of all the floors. The seismic weight of each floor is its full dead load plus appropriate amount of imposed load.

PERCENTAGE OF IMPOSED LOAD TO BE CONSIDERED IN SEISMIC WEIGHT CALCULATION

Imposed Loads	Percentage of imposed load
Upto and including 3.0	25
Above 3.0	50

Table 2 Zone Factor, Z

(*Clause* 6.4.2)

Seismic Zone	II	III	IV	V
Seismic Intensity	Low	Moderate	Severe	Very Severe
Z	0.10	0.16	0.24	0.36

Table 6 Importance Factors, I

(*Clause* 6.4.2)

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

NOTES

1 The design engineer may choose values of importance factor I greater than those mentioned above.

2 Buildings not covered in Sl No. (i) and (ii) above may be designed for higher value of I, depending on economy, strategy considerations like multi-storey buildings having several residential units.

3 This does not apply to temporary structures like excavations, scaffolding etc of short duration.

Table 7 Response Reduction Factor [1], *R*, for Building Systems

(*Clause* 6.4.2)

Sl No.	Lateral Load Resisting System	*R*
(1)	(2)	(3)
	Building Frame Systems	
i)	Ordinary RC moment-resisting frame (OMRF)[2]	3.0
ii)	Special RC moment-resisting frame (SMRF)[3]	5.0
iii)	Steel frame with	
	a) Concentric braces	4.0
	b) Eccentric braces	5.0
iv)	Steel moment resisting frame designed as per SP 6 (6)	5.0
	Building with Shear Walls[4]	
v)	Load bearing masonry wall buildings[5]	
	a) Unreinforced	1.5
	b) Reinforced with horizontal RC bands	2.5
	c) Reinforced with horizontal RC bands and vertical bars at corners of rooms and jambs of openings	3.0
vi)	Ordinary reinforced concrete shear walls[6]	3.0
vii)	Ductile shear walls[7]	4.0
	Buildings with Dual Systems[8]	

viii)	Ordinary shear wall with OMRF	3.0
ix)	Ordinary shear wall with SMRF	4.0
x)	Ductile shear wall with OMRF	4.5
xi)	Ductile shear wall with SMRF	5.0

[1] The values of response reduction factors are to be used for buildings with lateral load resisting elements, and not just for the lateral load resisting elements built in isolation.

[2] OMRF are those designed and detailed as per IS 456 or IS 800 but not meeting ductile detailing requirement as per IS 13920 or SP 6 (6) respectively.

[3] SMRF defined in **4.15.2**.

[4] Buildings with shear walls also include buildings having shear walls and frames, but where:

a) frames are not designed to carry lateral loads, or

b) frames are designed to carry lateral loads but do not fulfil the requirements of 'dual systems'.

[5] Reinforcement should be as per IS 4326.

[6] Prohibited in zones IV and V.

[7] Ductile shear walls are those designed and detailed as per IS 13920.

[8] Buildings with dual systems consist of shear walls (or braced frames) and moment resisting frames such that:

a) the two systems are designed to resist the total design force in proportion to their lateral stiffness considering the interaction of the dual system at all floor levels; and

b) the moment resisting frames are designed to independently resist at least 25 percent of the design seismic base shear.

DISTRIBUTION OF TOTAL BASE SHEAR

- A reasonable distribution of the total base shear throughout the height would be in accordance with a linear acceleration distribution, as given by

- $Q_i = V_B \; W_i h_i^2 / \sum W_j h_j^2$

Where

- Q_i = Design lateral force at floor i
- W_i = Seismic weight of floor i
- h_i = Height of floor i measured from base
- n = Number of storeys in the building = Number of levels at which the masses are located

FUNDAMENTAL NATURAL PERIOD

- For MRF without in-fills, $T_a = 0.075\, h^{0.75}$ - RC frame buildings
  $= 0.085\, h^{0.75}$ - Steel frame buildings

- For MRF with in-fills, $T_a = 0.09/d^{0.50}$

where

h = height of building in 'm'

- d = base dimension of the building at the plinth level in 'm' along the considered direction of the lateral force

MODAL ANALYSIS PROCEDURE

- The equivalent static analysis is suitable for the majority of high-rise buildings. If the vertical distribution of mass is irregular over the height of the building, as in buildings with large floor-to-floor variations on internal configuration or with setbacks, a modal analysis would be appropriate. In this method, a lumped mass model of the building with horizontal degrees of freedom at each floor is analyzed to determine the modal shapes and modal frequencies of vibration. The results are then used in conjunction with an earthquake design response spectrum, and estimates of the modal damping, to determine the probable maximum response of the structure from the combined effect of its various modes of oscillation. Buildings in which the mass at the floor levels is highly eccentric from the corresponding centres of resistance will be subjected to torque, causing the possibility of significant torsional vibrations and of coupling between the lateral and torsional mode. In such cases, the modal method can be applied by adding the third rotational degree of freedom at each floor level. This method is applicable to linear elastic systems. Hence the results for a building structure's response are only an approximate estimate as it is designed to undergo significant inelastic deformations only in moderate earthquakes. However accurate values of response may be obtained by using modified design spectra for inelastic systems.

Seismic Evaluation Procedure

Linear (Static & Dynamic)

Non - Linear (Static & Dynamic)

Linear Procedures

- **When the linear static or dynamic procedures are used for seismic evaluation, the design seismic forces, the distribution of applied loads over the height of the buildings and the corresponding displacements are determined using a linearly elastic analysis.**

- **It is difficult to obtain accurate results for structures that undergo nonlinear response through linear procedures.**

- **Therefore, linear procedures may not be used for irregular structures.**

Non - Linear Procedures

- **Also known as a push-over analysis, consists of laterally pushing the structure in one direction with a certain lateral force or displacement distribution until either a specified drift is attained or a numerical instability has occurred.**

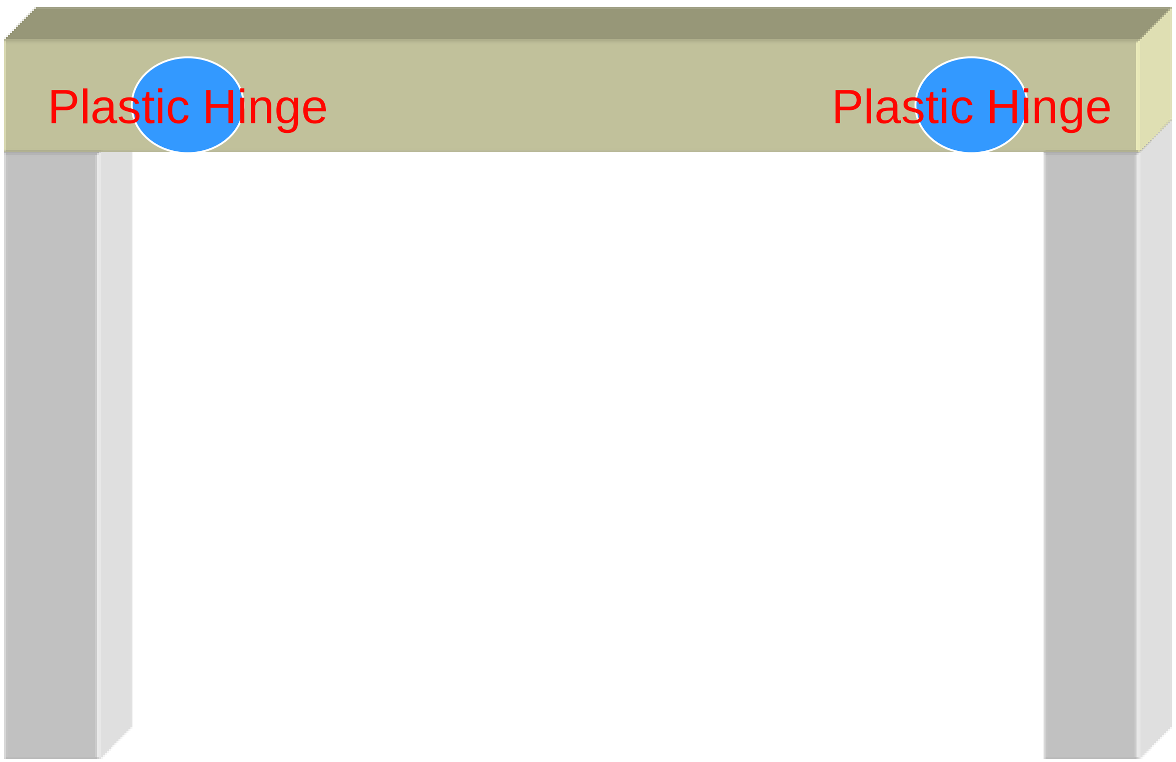
- **Because linear procedures have limitations and nonlinear dynamic procedures are complicated, nonlinear static analysis is commonly used.**

PUSHOVER ANALYSIS

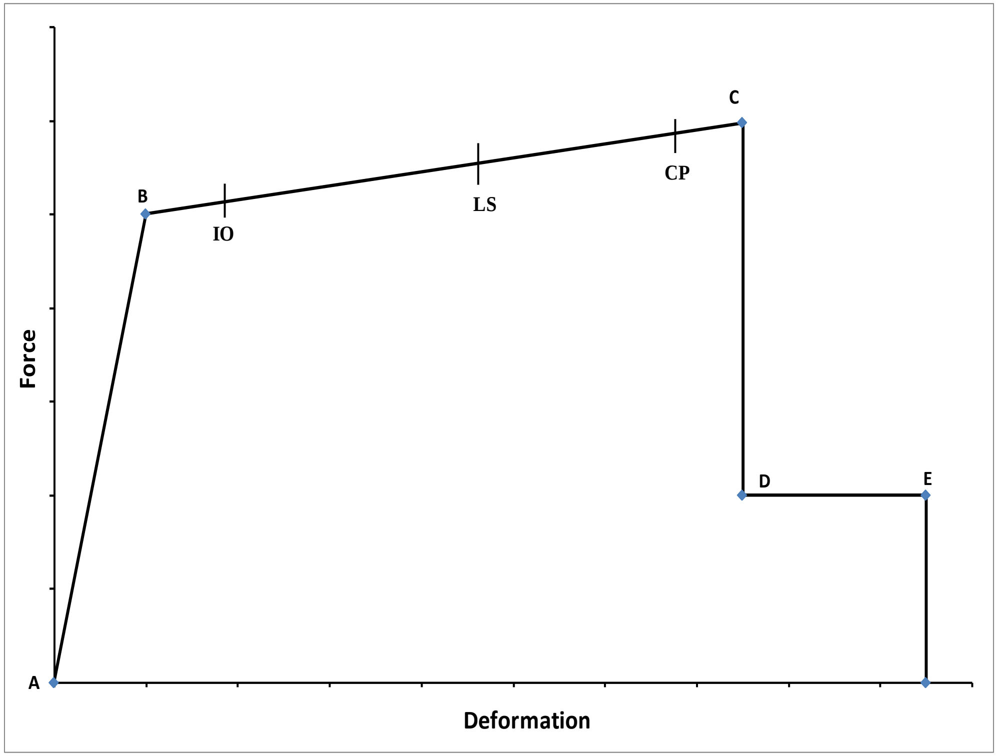
What is Pushover Analysis?

- Pushover analysis is a static analysis procedure to substitute rigorous dynamic analysis of seismic performance of structures
- Pushover assesses the capacity of the structure against the demand posed by the earthquake
- Pushover analysis applies monotonically increasing loads on structures, but estimates the performance under cyclic and repeated loads using mathematical transformations
- Guidelines for steel structures presented in FEMA-273
- Guidelines for concrete structures presented in ATC-40
- SAP2000 software has built in facility for carrying out pushover analysis

Formation of Plastic Hinge

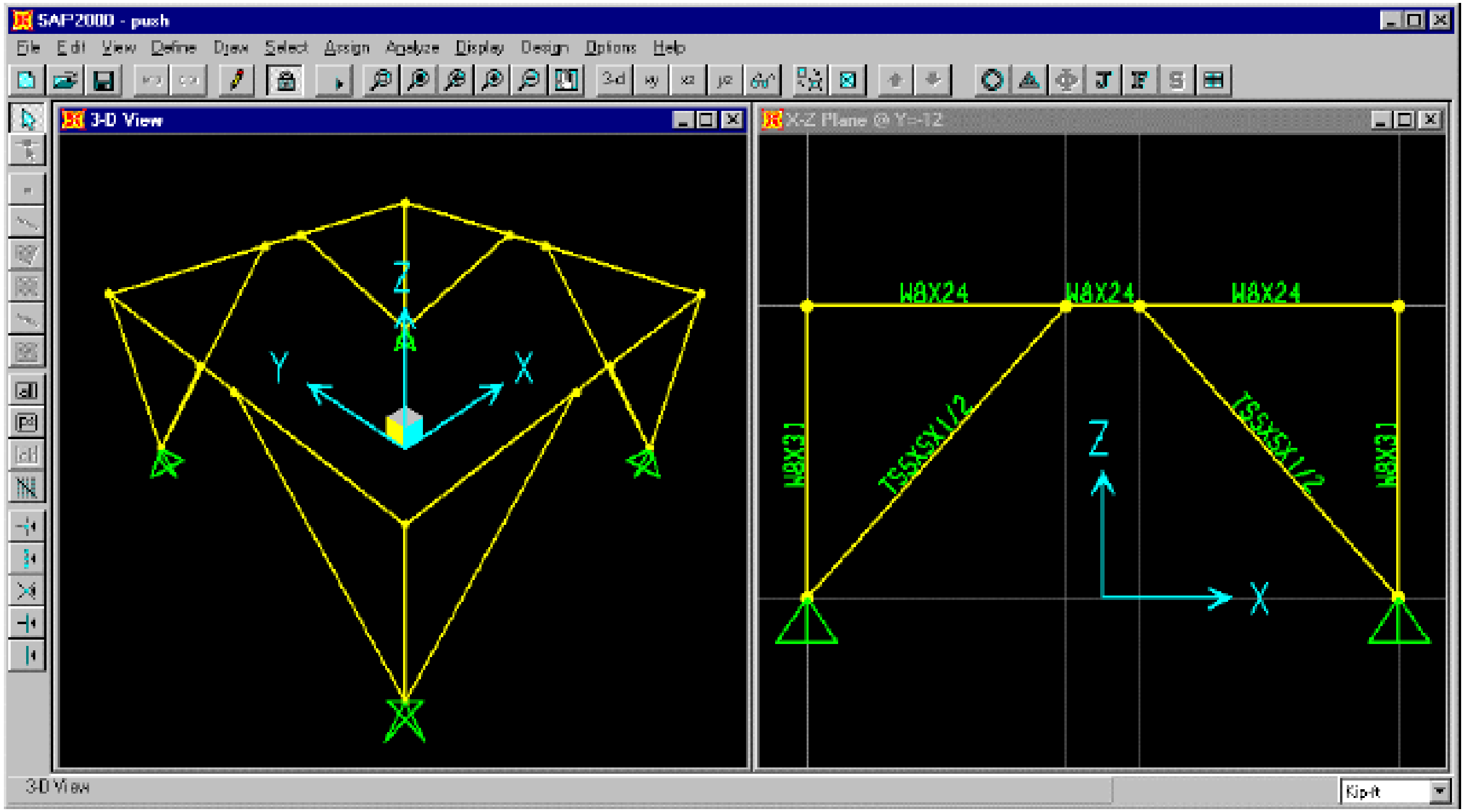


How the Plastic Hinge Behaves?



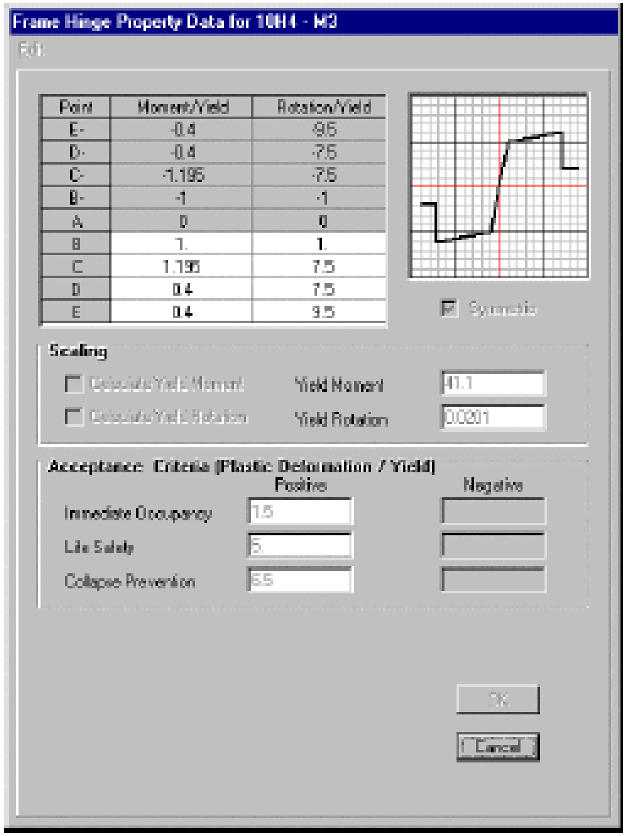
Basic Steps in Pushover Analysis

1. Create the basic structural model



2. Frame hinge properties and acceptance criteria for the model are to be specified

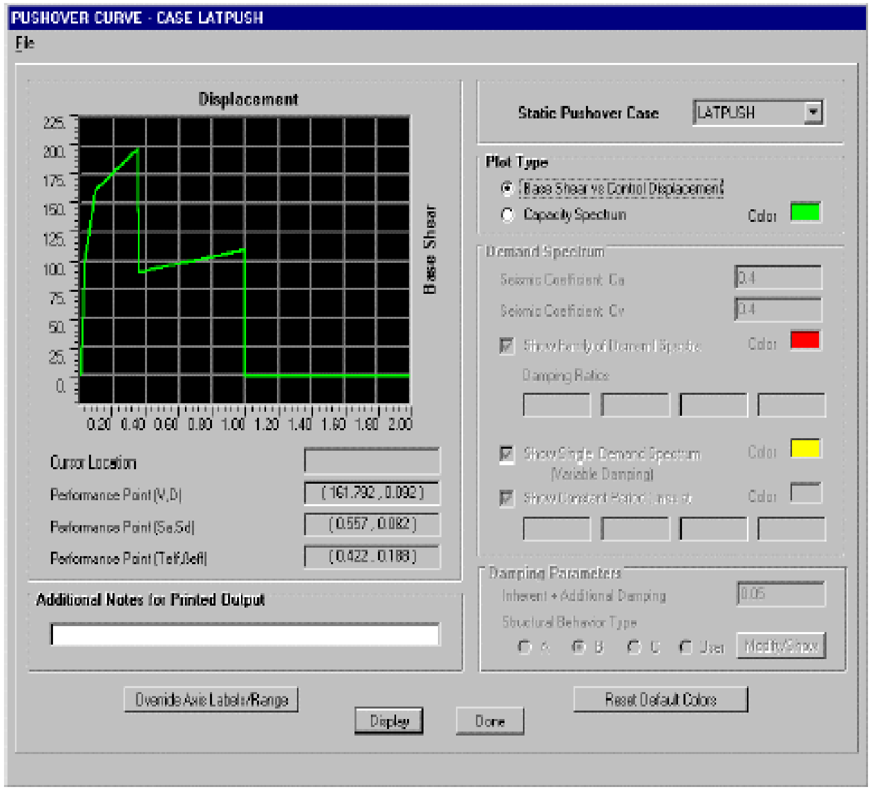
E-	-0.4	-9.5
D-	-0.4	-7.5
C-	-1.195	-7.5
B-	-1	-1
A	0	0
B	1	1
C	1.195	7.5
D	0.4	7.5
E	0.4	9.5



3. Assign hinge points to the structure

4. Define Load Cases for Pushover Analysis (Gravity + Lateral Deformation)

5. Run static Analysis, followed by Pushover Analysis. Figure shows pushover curve obtained after the analysis



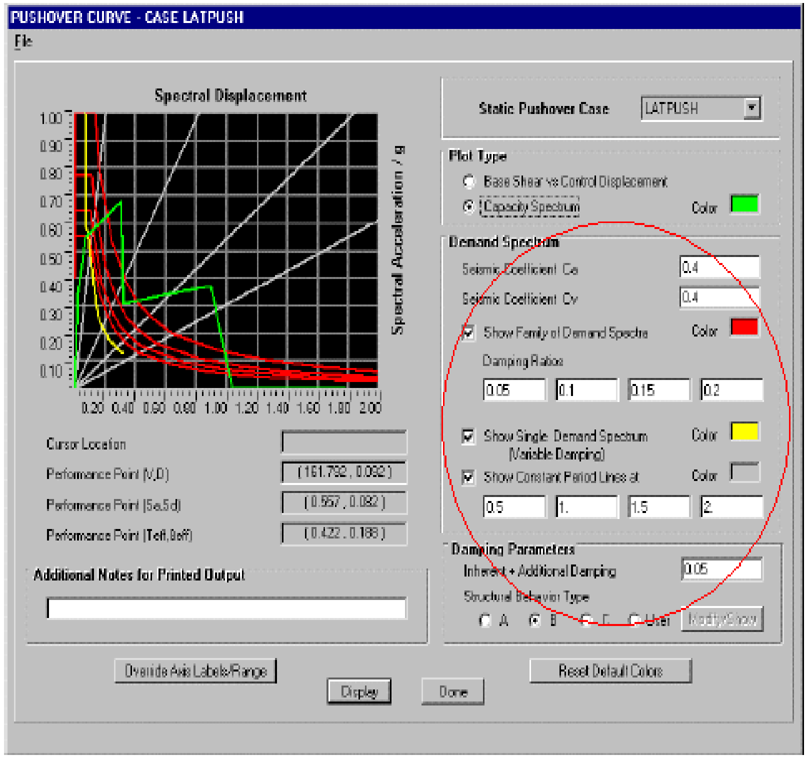
6. Summary Table for Pushover Analysis

PUSHOVER CURVE

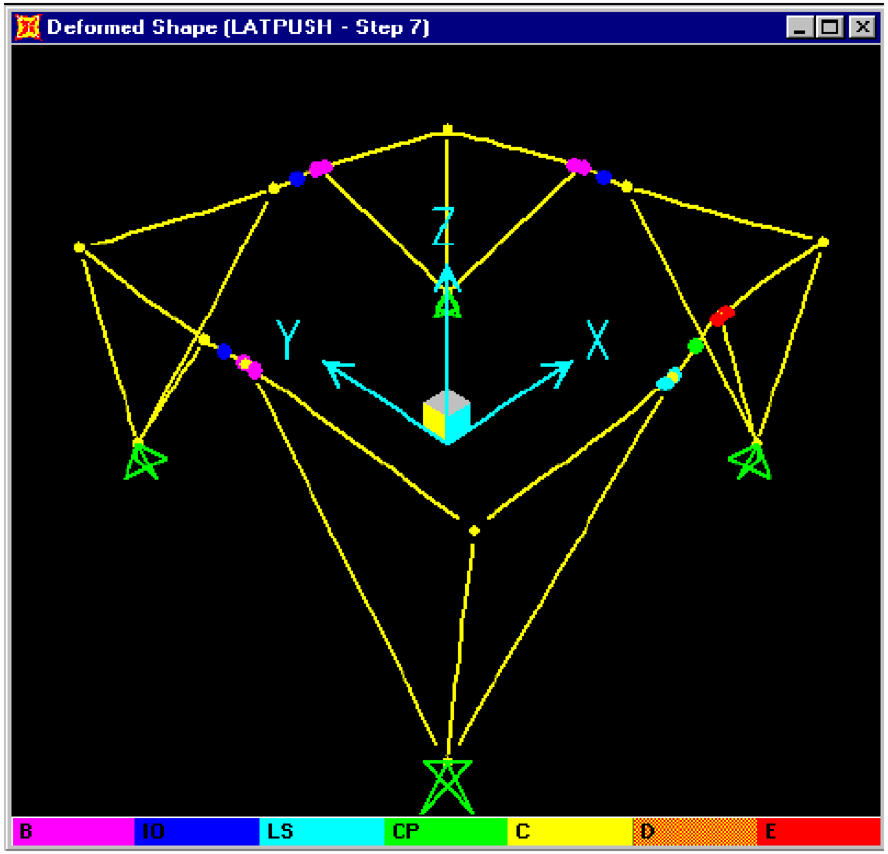
File

Step	Displacement	Base Shear	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	TOTAL
0	0.0000	0.0000	60	0	0	0	0	0	0	0	60
1	0.0274	100.4879	57	3	0	0	0	0	0	0	60
2	0.0821	157.4939	51	8	1	0	0	0	0	0	60
3	0.0878	161.1401	48	11	1	0	0	0	0	0	60
4	0.3481	198.9614	46	6	3	2	1	2	0	0	60
5	0.3481	160.7877	46	6	3	2	1	0	2	0	60
6	0.3513	161.2326	46	6	3	2	1	0	0	2	60
7	0.3513	142.7613	46	6	3	2	1	0	0	2	60
8	0.3530	144.4506	46	6	3	2	1	0	0	2	60
9	0.3564	144.9255	46	6	3	0	1	2	0	2	60
10	0.3565	108.0511	46	6	3	0	1	0	2	2	60
11	0.3596	108.2019	46	6	3	0	1	0	0	4	60
12	0.3596	89.6470	46	6	3	0	1	0	0	4	60
13	0.4538	93.9015	40	12	3	0	1	0	0	4	60
14	0.6538	99.5164	40	0	15	0	1	0	0	4	60
15	0.8538	105.1312	40	0	10	5	1	0	0	4	60
16	1.0031	109.3216	40	0	0	13	1	2	0	4	60
17	1.0031	-0.4843	40	0	0	11	1	0	0	8	60
18	1.2031	-0.4404	40	0	0	11	1	0	0	8	60
19	1.4031	-0.3965	40	0	0	11	1	0	0	8	60
20	1.6031	-0.3526	40	0	0	11	1	0	0	8	60
21	1.8031	-0.3086	40	0	0	11	1	0	0	8	60
22	2.0000	-0.2654	40	0	0	11	1	0	0	8	60

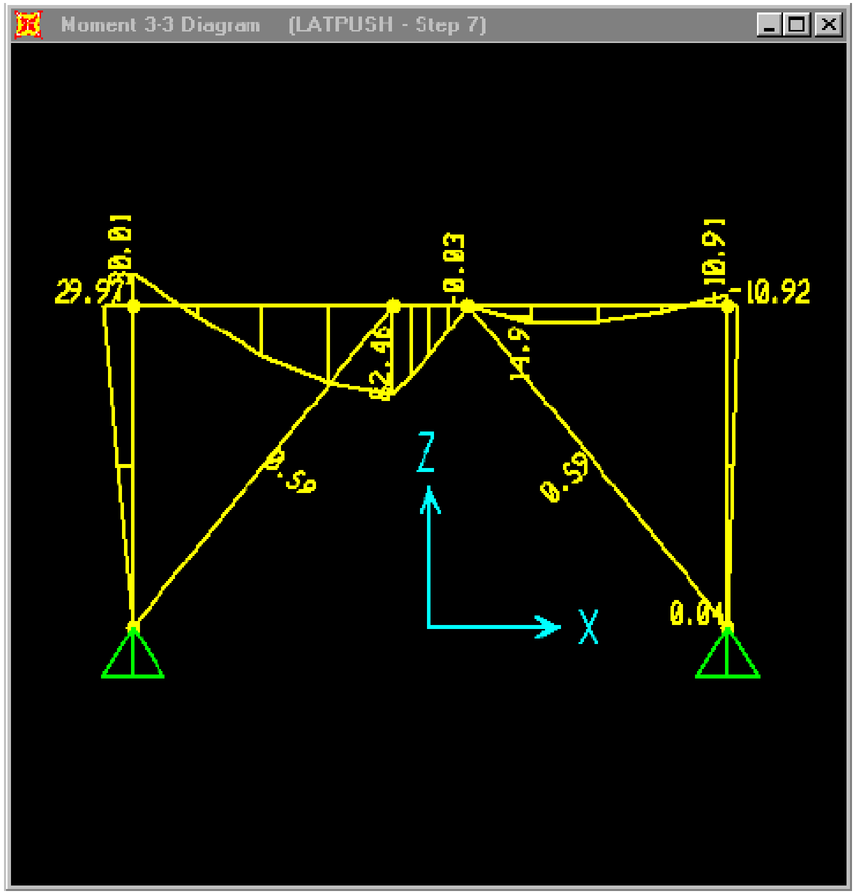
7. Capacity Spectrum Curve Displayed after Pushover Analysis. Demand spectrum is obtained by filling Seismic coefficients and Damping ratios.



8. Displaced Shape and Sequence of Hinge formations after Pushover Analysis



9. Member forces after Pushover Analysis



SHAKE TABLE TESTING

Shake Table

Testing of a Building Model