4. SOFTWARE FOR EARTHQUAKE ANALYSIS

4.1 GENERAL

The analysis process can be categorized on the basis of three factors: the type of external applied load, behavior of the structure or materials, and type of the structural model selected (**Fig.4.1**)

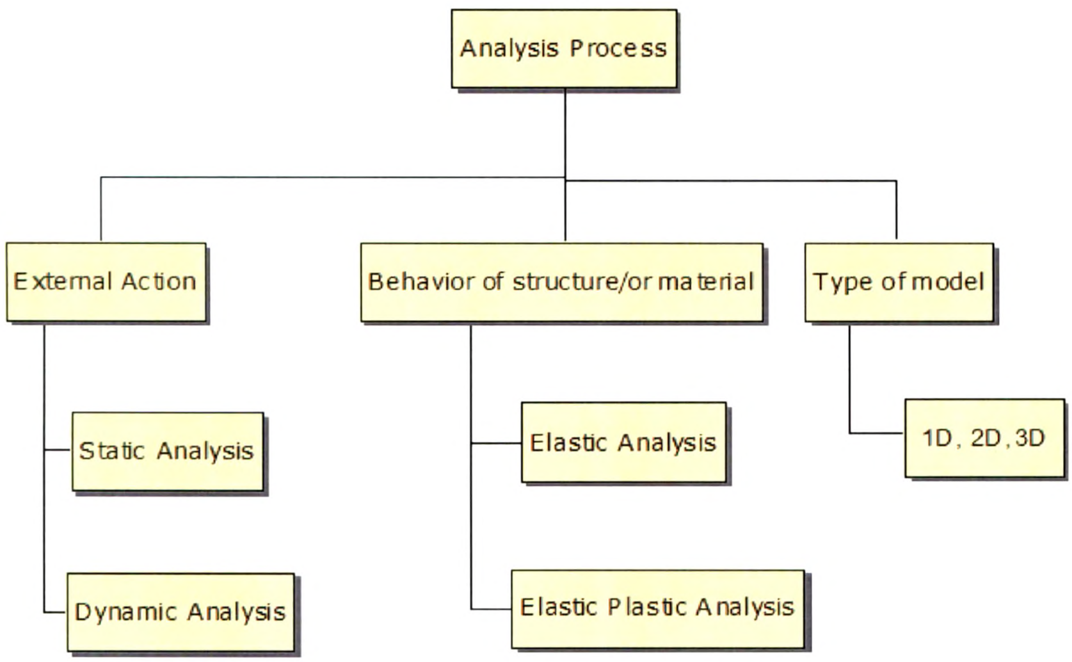


Fig. 4.1 Types of Analysis Processes

Based on the external actions the analysis process is divided into two basic groups – linear procedures and nonlinear procedures.

Linear procedures include static analysis (LS) formulae, such as those used for routine design of most new buildings, or dynamic analysis (LD) methods such as response spectrum analysis commonly used for design of buildings that are high, irregular or otherwise deemed inappropriate for static analysis.

Nonlinear static (NLS) analysis procedures include the pushover analysis method of FEMA -274 [21], ATC-40 [96] and code formulas for isolation system displacement that are based on nonlinear (secant) stiffness and effective damping of isolators.

Nonlinear dynamic (NLD) procedures include time history analysis (of a nonlinear model) and response spectrum analysis when such analysis is based on secant stiffness of yielded elements.

The current chapter examines the various methods of analysis for earthquake forces and gives details of the software prepared. The highly complex performance based approach of Pushover Analysis has been adopted for preparing the software which is used to get results of an NLS analysis, as propounded by all international literature. The complex nature of a dynamic problem coupled with performance based approach makes the programming an extremely tough job. Following modules have been implemented in the software prepared:

- Response Spectrum Analysis,
- Iterations for a displacement driven increment of Pushover,
- Hinge Locations and Hinge Properties for beams and columns
- Corresponding reduction in stiffness of structure
- Capacity and Demand Spectrum and finally
- Performance Index based on the performance point

The results are further used on a VR platform to explore the hitherto unexplored domain of the virtual world in structural engineering, as discussed in the next chapter.

4.2 METHODS OF ANALYSIS FOR EARTHQUAKE FORCES

The application of any of the four methods of analysis based on external forces, depends on following three major structural criteria as indicated in **Table 4.1**.

- Are inelastic displacements approximately proportional to elastic displacements i.e. is the structure essentially elastic?
- Is the structural system sufficiently regular to limit torsion?

◈ Is response of the building dominated by the fundamental mode?

Table 4.1 Choice of Analysis Methods

Types of Analysis Methods	Structural Criteria		
	Nonlinear Linear Response?	Regular Configuration?	Limited Higher-Mode Effects?
Linear Static (Code Formula)	*Yes*	*Yes*	*Yes*
Linear Dynamic (Response Spectrum)	*Yes*	*No*	*No*
Nonlinear Static (Pushover)	*No*	*No*	*Yes*
Nonlinear Dynamic (Time History)	*No*	*No*	*No*

If all three structural criteria are satisfied, then linear static analysis is deemed valid. If the building meets the linearity criterion, but fails either regularity or dynamic criteria, then response spectrum analysis would be more appropriate. If the building has limited higher mode effects then nonlinear static analysis is appropriate.

Of the four available techniques, linear static, linear dynamic, nonlinear static and nonlinear dynamic, the linear static analysis method provides the least amount of information on the behavior of a building under load. The information is limited to the amount of force and displacement that each of the elements of the structures undergoes, presuming that the structure remains linear.

The linear dynamic method provides data on the dynamic characteristics of the structure, including its natural period of vibration and mode shapes in addition to the information on the forces and displacements of the structural members. For irregular structures, it provides a better estimate of the response of the structure than does the static method. However, it is still fundamentally limited by the basic assumption that the structure remains elastic.

The nonlinear static method which includes the Pushover Analysis method is a relatively simple technique which allows estimation of the amount of deformation that the members of a structure will experience, once it goes nonlinear. It provides detailed information on the forces and member deformations at various stages of nonlinearity. However, it does not account for the effect of higher modes of response very accurately.

The nonlinear dynamic method provides full information on the forces and deformations in each member of the structure, throughout the period during which ground shaking is occurring. It allows determination of the sequence of damage events, assuming that the ground motion is known, and also directly accounts for all important dynamic modes of the structure. However, the method is very sensitive to both modeling and ground motion assumptions.

4.3 RESPONSE SPECTRUM METHOD

The seismic coefficient, used in the design of any structure, depends on many variable factors. Hence, the guidelines specified in IS: 1893-2002 [4] indicate only broadly the seismic coefficients that could be adopted, after considering all the factors as close to the actual conditions using sound engineering judgement. The code specifies that the design lateral seismic force, and its distribution to different levels along the height of the building and to various lateral load resisting elements should be obtained, depending on the various factors such as height and shape of building and seismic zone.

Thus, the seismic coefficient method recommended in the previous version of IS: 1893-2002 [4] which was a pseudo static method for finding design shear forces, is now limited only to preliminary calculations for small regular buildings. While time-history analysis using a number of earthquake records provides a nearly exact dynamic analysis, its use in routine structural analysis is limited due to the massive computational requirements. Response Spectrum Analysis on the other hand provides a rational and practical approximation based on fundamental principles of dynamics and vibration.

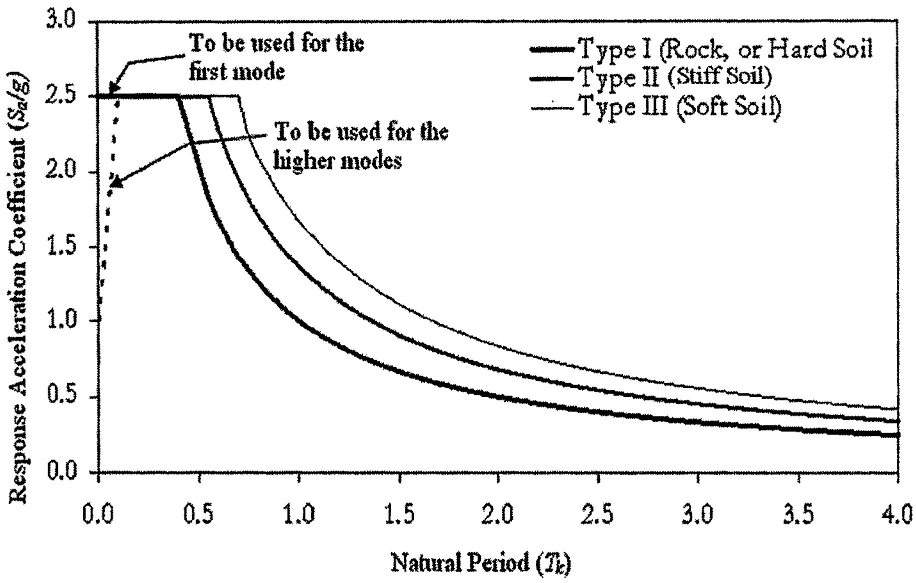


Fig. 4.2 Response Acceleration Coefficient

The design lateral force is first computed for the building as a whole and then distributed along the height at the floor levels. This total lateral force, also called the Base Shear Vb is calculated from the design acceleration spectrum values Ah.

$$A_h = \frac{ZIS_a}{2Rg} \qquad \text{.... (4.1)}$$

Where, Z = Zone factor, the country is divided into IV seismic zones, I = Importance factor, depending on the functional use of the building, R= Response reduction factor, depending on damage performance of the building and Sa/g = Average response acceleration coefficient (**Fig.4.2**).

The total design lateral force at the base of a building Vb along any principal direction is then derived from Ah and the seismic weight W of building:

The design base shear is then distributed along the height of the building at each floor and given by:

$$V_b = A_h W \qquad \text{.... (4.2)}$$

$$Q_i = V_b \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2} \qquad \text{.... (4.3)}$$

Where, Wi = Seismic weight of floor i, hi = Height of floor i measured from base, n = Number of story where masses are located.

The response spectrum analysis basically involves the evaluation of maximum value of structural response such as displacements and member forces for each mode of vibration using spectra of earthquake records. The earthquake spectrum is nothing but an average number of earthquake records modified for site specific conditions and smoothened for the design purpose.

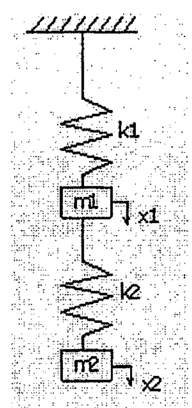


Fig. 4.3 Spring –Mass Model

Buildings with regular plan configuration are modeled as a system of lumped masses at floor levels with each mass having one degree of freedom that is the lateral displacement in the direction under consideration. Undamped free vibration analysis, of the building as spring-mass model as shown in **Fig. 4.3**, is performed using appropriate masses and stiffness of the structural system to obtain natural periods (T) and mode shape coefficients (ϕ) of those modes of vibration that are required to be considered. The number of modes to be used should be such that the sum of modal masses of all the modes considered is at least 90% of the total seismic mass.

The displacement at each floor is obtained by :

Displacement = shear force/ stiffness

The roof displacement v/s the base shear are plotted. Due to yielding of the structure the model is modified and the stiffness is reduced. The structure with reduced stiffness is again analyzed by increasing the lateral force. The process is continued till the target displacement is reached which is taken as 0.04H (H = height of the structure). Thus at various stages base shear (V) and roof displacement (d) are obtained and plotted on Y-axis and X-axis respectively. The graph so plotted is known as capacity curve which is converted into the capacity spectrum. The response spectra for various damping is plotted according to IS: 1893-2000 [4].

Thus the Response Spectrum Method requires dynamic analysis of a mathematical model of a building to establish fundamental period, modal frequencies and mode shapes. The peak modal responses are determined for sufficient modes to capture at least 90% of the participating mass of the building in each of two orthogonal principal horizontal directions of the building. Modal damping ratios will reflect the damping in the building at deformation levels less than the yield deformation. Peak member forces, displacements, story forces, story shears, and base reactions for each mode of response are combined by either the square root sum of squares(SRSS) rule or the complete quadratic combination (CQC) rule.

4.4 PERFORMANCE BASED ANALYSIS

Traditional analysis and design procedures developed primarily for new construction are not adequate to handle the inherent complexity and uncertainties of concrete buildings and their components during the event of an earthquake. Thus in recent years there has been an extensive examination of performance of structures during an earthquake using performance based techniques. The commonly used method to evaluate performance of structures is pushover analysis, also known as performance based analysis. Performance based engineering is not a totally new concept. Automobiles, airplanes, turbines have been designed and manufactured using this approach since many decades. But the applications of the same for buildings were limited. Generally in such applications one or more full-scale prototypes of structure are built and subjected to extensive testing (pushing in case of buildings). The design and manufacturing process is then revised to incorporate the lessons learned from the experimental evaluations. What makes performance based analysis of buildings different is that each building designed by this process is virtually unique and the experience obtained is not directly transferable to the buildings of other types, sizes and performance objectives.

Engineers have been using unrealistic simplified static lateral force procedures to design buildings to resist seismic forces and displacements. While traditional methods can result in adequate designs, they obscure a basic understanding of actual structural behavior and performance during earthquakes. Pushover analysis presents a relatively new technology that allows engineers to gain a more realistic picture of the potential seismic performance characteristics of buildings. These nonlinear static procedures constitute an inelastic analysis that considers what happens to buildings after they begin to crack and yield in response to realistic earthquake motions. In order to utilize performance-based analysis efficiently, one needs to be aware of the uncertainties involved in both structural performance and seismic hazard estimation.

In traditional linear static analysis procedure, engineers reduce seismic forces to levels at which building can be designed under the assumption that they remain undamaged. Although unrealistic and potentially misleading, this simplistic approach can work well for new buildings and for smaller, simpler, and regular existing buildings. The advantage of the newer, nonlinear static procedures, when applied to existing buildings, is that they credit the good features of buildings while identifying deficiencies at the same time.

At present, this approach has limited applications to small sized regular and highly redundant lateral force-resisting systems, which have small fundamental time period. However, despite the fact that the advantages of using nonlinear static analysis for design purposes have been recognized since the early seventies, the code drafting committees have been reluctant to adopt this type of analysis, even as an alternative to the standard (static or dynamic) elastic analysis procedure. They are associated with high computational costs that may be one of the main reasons why these methods have not been adopted by existing codes.

4.5 PUSHOVER ANALYSIS

The basic purpose of using nonlinear static pushover analysis procedure is to calculate strength requirements in yielding members, or assess inelastic demands and capacity actions. Every structural system is designed to have a seismic capacity that exceeds the anticipated seismic demand. Nonlinear Static Analysis (Pushover Analysis) provides a particularly rigorous treatment to explicitly address the nonlinear behavior of the structure.

It provides data on the strength and ductility of the structure, which cannot be obtained by elastic analysis. At the same time, it is inaccurate for structures whose higher mode effects are significant. It may not detect structural weaknesses, which may be generated when the structure's dynamic characteristics change after the formation of the first local plastic mechanism. Pushover analysis can be used as a viable tool for the seismic performance evaluation of new and existing structures. Nevertheless, many a times, it turns out to be too complicated and due to lack of awareness and knowledge, the results turn out to be erroneous.

In Performance Based Analysis a performance objective [80] specifies the desired seismic performance level of the building for a given earthquake ground motion. A performance level describes a limiting damage condition which may be considered satisfactory for a given building and a given ground motion. A key requirement of any meaningful performance based analysis such as Pushover

Analysis, is its ability to assess seismic demands and capacities with a reasonable degree of certainty.

This procedure uses a series of sequential elastic analysis, superimposed to approximate a force-displacement capacity diagram of the overall structure. The mathematical model of the structure is modified to account for reduced resistance of yielding components. Then in the next sequence a lateral force distribution is again applied until additional components yield. This process is continued until the structure becomes unstable or until a predetermined limit is reached (ATC-40,1996) [80] . Hence it is also known as the Capacity Spectrum Method.

The fundamental elements of a performance-based design procedure are demand, capacity and performance. Demand is a representative of the earthquake ground motion. Capacity is a representation of the structure's ability to resist the seismic demand. Performance is dependent on the manner that the capacity is able to handle the demand. Determination of these three primary elements of capacity, demand and performance are required for nonlinear static pushover analysis.

The Capacity Spectrum Method is a nonlinear static procedure that provides a graphical representation of the global force-displacement capacity curve of the structure and compares it to the response spectra representation of the earthquake demands. It is a very useful tool in the evaluation and retrofit design of existing concrete buildings. The graphical representation provides a clear picture of how a building responds to earthquake ground motion, and how various retrofit strategies such as adding stiffness or strength, will impact the building's response to earthquake.

4.5.1 Capacity Curve of a Building

In Nonlinear Static Procedure, the basic demand and capacity parameter for the analysis, is the lateral displacement of the building. The generation of a capacity curve (base shear Vs roof displacement (**Fig.4.4**) defines the capacity of the building uniquely for an assumed force distribution and displacement pattern. It is independent of any specific seismic shaking demand and replaces the base shear capacity of conventional design procedures. If the building displaces laterally, its response must lie on this capacity curve. Any point on the curve defines a specific damage state for the structure, since the deformation for all components can be related to the global displacement of the structure.

By correlating this capacity curve to the seismic demand generated by a specific earthquake or ground shaking intensity, a point can be found on the capacity curve that estimates the maximum displacement of the building the earthquake will cause. This defines the performance point or target displacement.

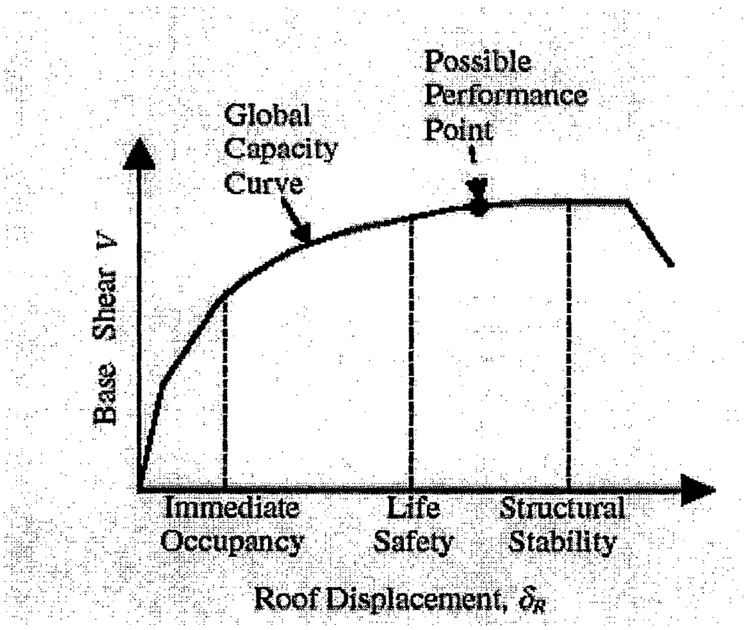


Fig. 4.4 Building Capacity Curve

The location of this performance point relative to the performance levels defined by the capacity curve indicates whether or not the performance objective is met. The performance point is found using the capacity spectrum procedure described later. Thus simplified nonlinear analysis procedures using pushover methods (also called the capacity spectrum method), require determination of three primary elements: capacity, demand (displacement) and performance.

4.5.2 Capacity

Capacity is defined by the expected ultimate strength (in flexure, shear, or axial loading) of a structural component excluding the reduction factors commonly used in design of concrete members. Capacity usually refers to the strength at the yield point of the element or structure's capacity curve. For deformation-controlled components, capacity beyond the elastic limit generally includes the effects of strain hardening. The overall capacity of a structure depends on the strength and deformation capacities of the individual components of the structure. In order to determine capacities beyond the elastic limits, some form of nonlinear analysis, such as the pushover procedure, is required. This procedure uses a series of sequential elastic analyses, as described below and these sequential analyses are superimposed to approximate a force-displacement capacity diagram of the overall structure.

- Apply lateral forces to each story in proportion to the standard code (IS:1893-2002) [4].
- Record the base shear and the roof displacement.
- Revise the model using zero (or very small) stiffness for the yielding elements.
- Apply new increment of lateral load to the revised structure such that another element yields.
- Add the increment of lateral load and the corresponding increment of roof displacement to the previous totals to give the accumulated values of base shear and roof displacement.
- Repeat above three steps until structure reaches an ultimate limit such as distortions considerably beyond the desired performance level, an element (or a group of elements) reaching a lateral deformation level at which significant strength degradation begins.

4.5.3 Demand (Displacement)

It is the representation of the earthquake ground motion or shaking that the building is subjected to. Ground motions during an earthquake produce complex horizontal displacement patterns in structures that may vary with time. Tracking this motion at every time-step to determine structural design requirements is impractical. Traditional linear analysis methods use lateral forces to represent a design condition. For nonlinear methods it is easier and more direct to use a set of lateral displacements as a design condition. For a given structure and ground motion, the displacement demand is an estimate of the maximum expected response of the building in terms of drift or member displacements during the ground motion.

4.5.4 Performance

Once a capacity curve and demand displacements are defined, a performance check can be done. A performance check verifies that structural and nonstructural components are not damaged beyond the acceptable limits of the performance objective for the forces and displacements implied by the displacement demand. The performance point is the intersection of the capacity spectrum with the appropriate demand spectrum. This performance point represents the condition for which the seismic capacity of the structure is equal to the seismic demand imposed on the structure by the specified ground motion.

The location of the performance point must satisfy following two relationships:

- The point must lie on the capacity spectrum curve in order to represent the structure at a given displacement, and
- The point must lie on a spectral demand curve which represents the nonlinear demand at the same structural displacement.

In the general case, determination of the performance point requires a trial and error search for satisfaction of the two criteria specified above.

The main steps involved in this method are:

- Drawing capacity curve for the given building.
- Finding out the demand i.e. an estimate of the maximum expected response of the building during the ground motion to get the response curve.

- ◈ Carry out the performance check, by locating the performance point (**Fig. 4.5**).

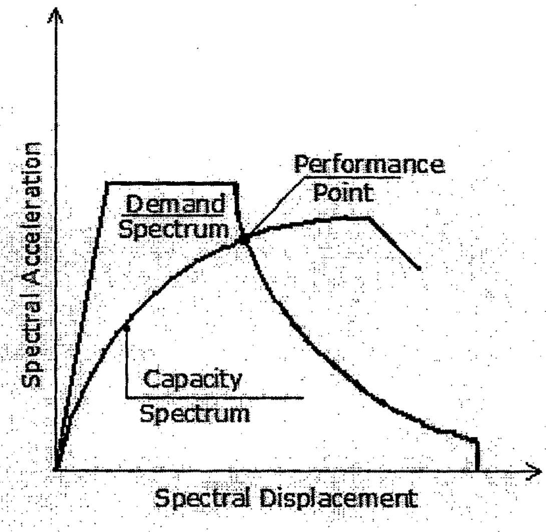


Fig. 4.5 Graphical Representation of Capacity Spectrum

The capacity of a particular building and the demand imposed upon it by a given earthquake motion are not independent. One source of this mutual dependence is evident from the capacity curve itself. As the demand increases, the structure eventually yields and, as its stiffness decreases, its period lengthens. Conversion of the capacity curve to spectral ordinates makes this concept easier to visualize. The following section shows the conversion of the capacity curve to the capacity spectrum.

4.5.5 Capacity Spectrum

The Capacity Spectrum Method is also known as Acceleration- Displacement Response Spectra Method. The Capacity Spectrum Method requires that both the capacity curve and the demand curve be represented in response spectral ordinates, in order to relate their intersection with each other to get the performance point. Hence it is necessary to convert the capacity curve, which is in terms of base shear and roof displacement to what is called capacity spectrum, which is a representation of the capacity curve in Acceleration-Displacement

Response Spectra (ADRS) [6] format (i.e., Sa versus Sd) as shown in **Fig. 4.6** and **4.7**.

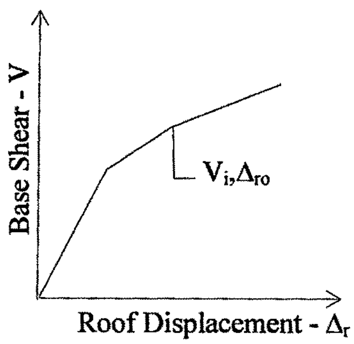


Fig. 4.6 Capacity Curve

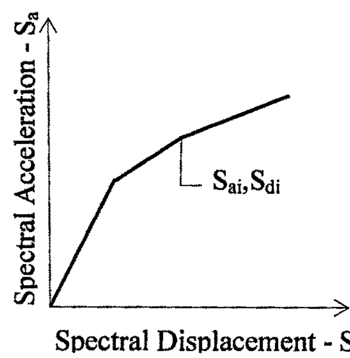


Fig. 4.7 Capacity Spectrum in ADRS Format

The equations required to convert the capacity curve to a spectrum so that it can be matched with the response spectrum are given below:

$$PF_1 = \left[\frac{\sum_{i=1}^{N} (w_i \phi_{i1})/g}{\sum_{i=1}^{N} (w_i \phi_{i1}^2)/g} \right] \quad \text{.... (4.4)}$$

$$\alpha_1 = \frac{\left[\sum_{i=1}^{N} (w_i \phi_{i1})/g \right]^2}{\left[\sum_{i=1}^{N} w_i / g \right] \left[\sum_{i=1}^{N} (w_i \phi_{i1}{}^2)/g \right]} \quad \text{.... (4.5)}$$

$$S_a = \frac{V/W}{\alpha_1} \quad \text{.... (4.6)}$$

$$S_d = \frac{\Delta_{roof}}{PF_1 \phi_{roof,1}} \quad \text{.... (4.7)}$$

Where,

PF1 = modal participation factor for the first natural mode.

$\alpha 1$ = modal mass coefficient for the first natural mode.

wi/g = mass assigned to level i.

$\phi i1$ = amplitude of mode 1 at level i.

N = level N, level which is uppermost in the main portion of structure.

V = base shear.

W = building dead weight plus likely live loads.

Δroof = roof displacement (V and the associated Δroof make up points on the capacity curve).

Sa = spectral acceleration.

Sd = spectral displacement (Sa and the associated Sd make up points on the capacity spectrum).

The general process for converting the capacity curve to the capacity spectrum, that is, converting the capacity curve into the ADRS format, is to first calculate the modal participation factor PF1 and the modal mass coefficient $\alpha 1$ using Eqns. (4.4) and (4.5). Then for each point on the capacity curve, V vs. Δroof (**Fig. 4.4**), calculate the associated point Sa, Sd (**Fig. 4.5**) on the capacity spectrum using Eqns. (4.3) and (4.4).

The Sa versus Sd (ADRS) representation of the Response Spectra is less familiar compared to the traditional Sa versus T representation. **Figure 4.8** shows the same spectrum in each format. In the ADRS format, lines radiating from the origin have constant period. For any point on the ADRS spectrum, the period, T, can be computed using the relationship T=2π(Sd/Sa)1/2. Similarly, for any point on the traditional spectrum, the spectral displacement, Sd, can be computed using the relationship Sd=SaT2/4π2. These two relationships are the same formula arranged in different ways. **Figure 4.9** shows the same capacity spectrum is shown superimposed on each of the response spectra plots shown in **Fig.4.8**. Along the capacity spectrum, the period is constant, at T1. When the next point is reached, the period is T2. This indicates that as a structure undergoes inelastic displacement, the period lengthens. The lengthening period is most apparent on the traditional spectrum plot, but it is also clear on the ADRS plot, remembering that lines of constant period radiate from the origin.

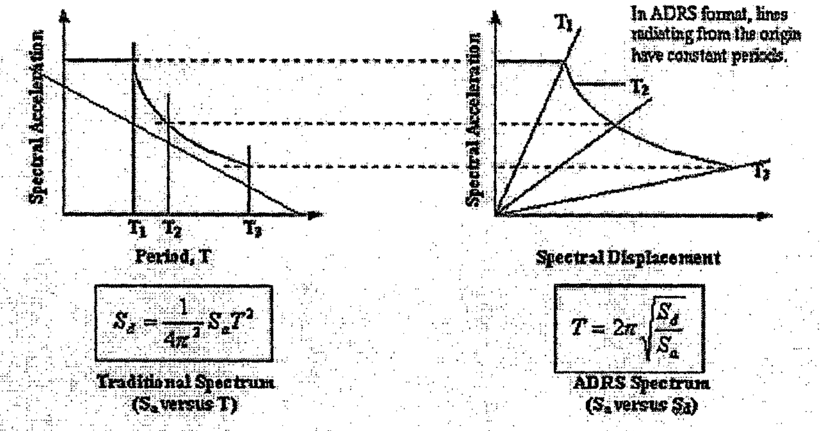


Fig. 4.8 Response Spectra in Traditional and ADRS Formats

4.5.6 Performance Point

Performance point is the intersection of the Demand Spectrum with the Capacity Spectrum. When the displacement at the intersection of the Demand Spectrum and the Capacity Spectrum, di, is within 5 percent (0.95dpi ≤ di ≤ 1.05dpi) of the displacement of the trial performance point (api, dpi), di becomes the performance point. If the intersection of the demand spectrum and the capacity spectrum is not within the acceptable tolerance, then a new api, dpi point is selected and the process is repeated. **Figure 4.9** illustrates the concept. The performance point represents the maximum structural displacement expected for the demand earthquake ground motion.

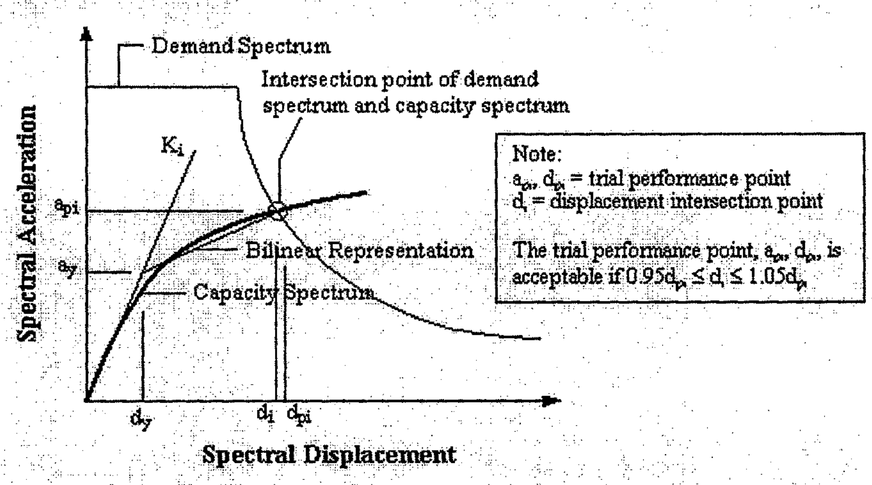


Fig. 4.9 Intersection Point of Demand and Capacity Spectrum

4.5.7 Performance Level

The performance criterion for pushover analysis is generally established as the desired state of displacement of the building rooftop or spectral displacement amplitude. During its displacement the building will undergo gradual degradation due to hinge formation in its members.

As shown in **Fig. 4.10**, five points labeled A, B, C, D and E are used to define the force deflection behavior of the hinge. The range A to B is elastic range. B to IO is the range of immediate occupancy. IO to LS is the range of life safety, and LS to CP is the range of collapse prevention. If all the hinges at full gravity and full earthquake load stages are within CP range i.e., collapse prevention, the pushover criteria are satisfied. But if the hinges cross CP, it is unsafe and that element will need retrofit. If all the hinges are within CP limit, the structure is said to be safe. However depending upon the importance of the structure, the hinges after IO range also need to be retrofitted.

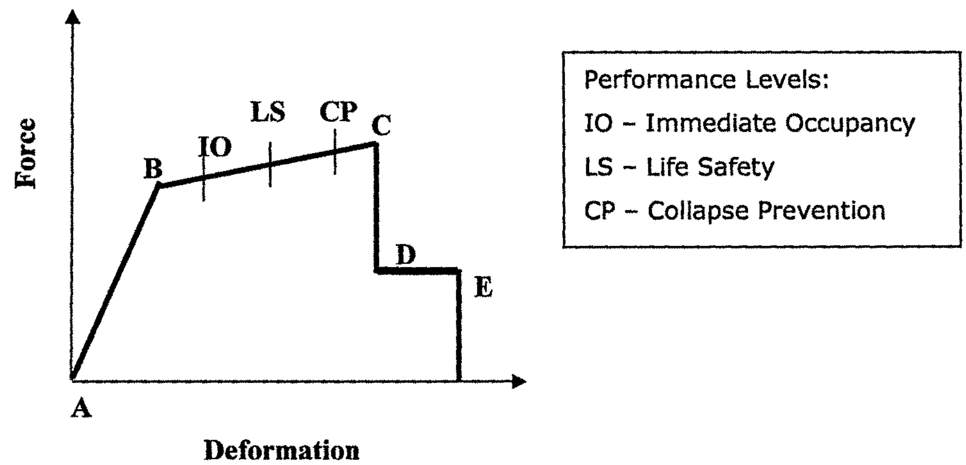


Fig. 4.10 Force-Deformation for Pushover Hinge

4.5.7.1 Immediate occupancy performance level

This Performance Level, indicates a post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral-force-resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness. The risk of life-threatening injury as a result of structural damage is very low, and although some minor structural repairs may be appropriate, these would generally not be required prior to reoccupancy.

4.5.7.2 Damage control level

It is a state of damage somewhere between Immediate Occupancy and Life Safety. It provides a place holder for the many situations where it may be desirable to limit structural damage beyond the Life Safety level, but occupancy is not the issue. The range is also called Limited Damage. The expected performance of most new buildings for the 10 percent/50-year event fall in this range.

4.5.7.3 Life safety performance level

It indicates a post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, it is expected that the overall risk of life-threatening injury as a result of structural damage is low. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to re-occupancy.

4.5.7.4 Collapse prevention performance level

It indicates that the building is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, including significant degradation in the stiffness and strength of the lateral-force-resisting system, large permanent lateral deformation of the structure, and to a limited extent degradation in vertical-load-carrying capacity. However, all significant components of the gravity-load-resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. The structure may not be technically practical to repair and is not safe for reoccupancy, as aftershock activity could induce collapse. ATC-40(Vol.I) [80] provides the guidelines, considerations, and assumptions required to assess the acceptability of the seismic response of the various components and elements of the structural and nonstructural systems. Response limits based on story drift considering the Immediate Occupancy, Life Safety, and Collapse Prevention performance levels are given in **Table 4.2**.

Table 4.2 Maximum Performance Limits

Storey Drift	Performance Level		
Interstory Drift Limit	*Immediate Occupancy*	*Damage Control*	*Life Safety*
Maximum total drift	*0.01*	*0.01-0.02*	*0.02*
Maximum inelastic drift	*0.005*	*0.005-0.015*	*No limit*

4.5.8 Pushover Analysis Procedure

Pushover analysis is used to get the base shear versus the top displacement curve of the structure, usually called capacity curve or pushover curve (**Fig. 4.11**). The basic demand and capacity parameter for the analysis is the lateral displacement of the building. The generation of capacity curve defines the capacity of the building uniquely for an assumed force distribution and displacement pattern.

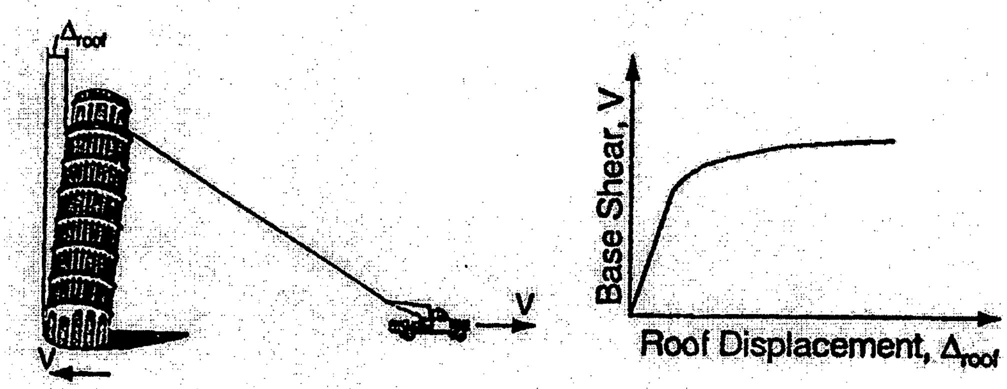


Fig. 4.11 Procedure to Determine Capacity

As described earlier the most important parts of this method are the construction of the Capacity Spectrum and the design Response Spectra and finding of the point of intersection of the capacity and the response spectra. The intersection defines the performance level of the structure for the design earthquake. The following procedure is adopted in the current software based on the ATC 40 procedure.

1. Form the analytical model of the structure without pushover data.

2. Apply the gravity load and the equivalent static seismic lateral load to the structure incrementally. Distribution of lateral loads between different floor levels is based on the current code specified load distribution or equivalent static load computed based on modal analysis procedure. Analyze the structure for the internal forces and carry out design of the members in the usual way.

3. Assign the plastic hinges with specific properties to various members at the plastic hinge locations. Set the performance criteria, like drift at specific floor levels, limiting plastic hinge rotation at specific plastic hinge spots.

4. Define static nonlinear load cases. Typically first pushover load case is used to apply gravity load and then subsequent lateral pushover load cases (**Fig. 4.11**) are specified to start from the final conditions of the gravity pushover.

5. Pushover load cases can be force controlled or deformation controlled. Typically a gravity load pushover is force controlled and lateral pushovers are displacement controlled. The distribution of lateral force used in the pushover can be based on a uniform acceleration in a specified direction, a specified mode shape, or a user-defined static load case.

6. Run the basic static analysis and, if desired, dynamic analysis. Then run the nonlinear static pushover analysis. Draw the "Base Shear vs. Controlled Displacement" curve from the results similar to (**Fig. 4.11**). This is called "pushover curve".

7. Convert the pushover curve to the Acceleration-Displacement Response-Spectra (ADRS) format. This is called capacity spectrum.

8. Obtain the equivalent damping based on the expected performance level.

9. Get the design Response Spectra for different levels of damping and adjust the spectra for the nonlinearity based on the damping in the Capacity Spectrum.

10. The capacity spectrum and the design response spectra can be plotted together when they are expressed in the ADRS format.

11. The intersection of the capacity spectrum and the response spectra defines the performance level. If the performance level satisfies the performance objective, the design is okay, otherwise adjustment to the structure is required (increased capacity in case of new; retrofitting in case of existing structure).

12. Review the pushover displaced shape and sequence of hinge formation on a step-by-step basis. Hinges appear when members yield. Similarly review member forces on a step-by-step basis.

4.6 COMPUTER IMPLEMENTATION

The software package Structural Aspects of Disasters in Virtual Reality (SA-DVR) facilitates analysis of plane frame for static loads, beam and column design, plotting M-Phi (Bending) and P-M (Axial with Bending) interaction curves and pushover analysis of 2-D plane frames. Static Analysis, Design and Pushover analysis modules have been developed using VC++ while M-Phi curve and P-M interaction curves are developed using readymade software Xtract. To illustrate the strength of the developed package, solution is obtained for a 2 storey frame problem. Flowchart to illustrate the hierarchy of procedure is shown in **Fig. 4.12**. The package is divided into 2 parts comprising of response spectrum analysis to get the lateral loads due to earthquake. These lateral forces are then read into the second module of Plane frame analysis. The cycle is repeated by incrementing force and observing the drift. An attempt has been made in the present work to program pushover analysis method inspite of the inherent complexity of the procedure on the lines of SAP as shown in Table 4.3 The effort is worthwhile as this package can then be used further for any other application such as design, retrofit, CAD output, or for that matter for performance based analysis for any other disaster load such as for wind or fire. Results of SA-DVR have been compared and found to match substantially with those of SAP, thus validating the prepared software for further independent use with other modules as mentioned above. As shown in the flow chart this procedure uses a series of sequential elastic analyses, as described below and this sequential analysis are superimposed to approximate a force-displacement capacity diagram of the overall structure:

1. Apply lateral forces to each storey as obtained from Response Spectrum Analysis as per IS:1893 (2002) [4].
2. Record the base shear and the roof displacement.
3. Revise the model using zero stiffness for the yielding elements.
4. Apply new increment of lateral load to the revised structure such that another element yields.
5. Add the increment of lateral load and the corresponding increment of roof displacement to the previous totals to give the cumulative values of base shear and roof displacement.
6. Repeat above three steps until structure reaches an ultimate limit beyond the desired performance level, such as an element reaching a lateral deformation level at which significant strength degradation begins.
7. The mathematical model of the structure is modified to account for reduced resistance of yielding components.
8. A lateral force distribution is again applied until additional components yield. This process is continued until the structure becomes unstable or until a predetermined limit is reached. The pushover capacity curve approximates how structures behave after exceeding their elastic limit [80].

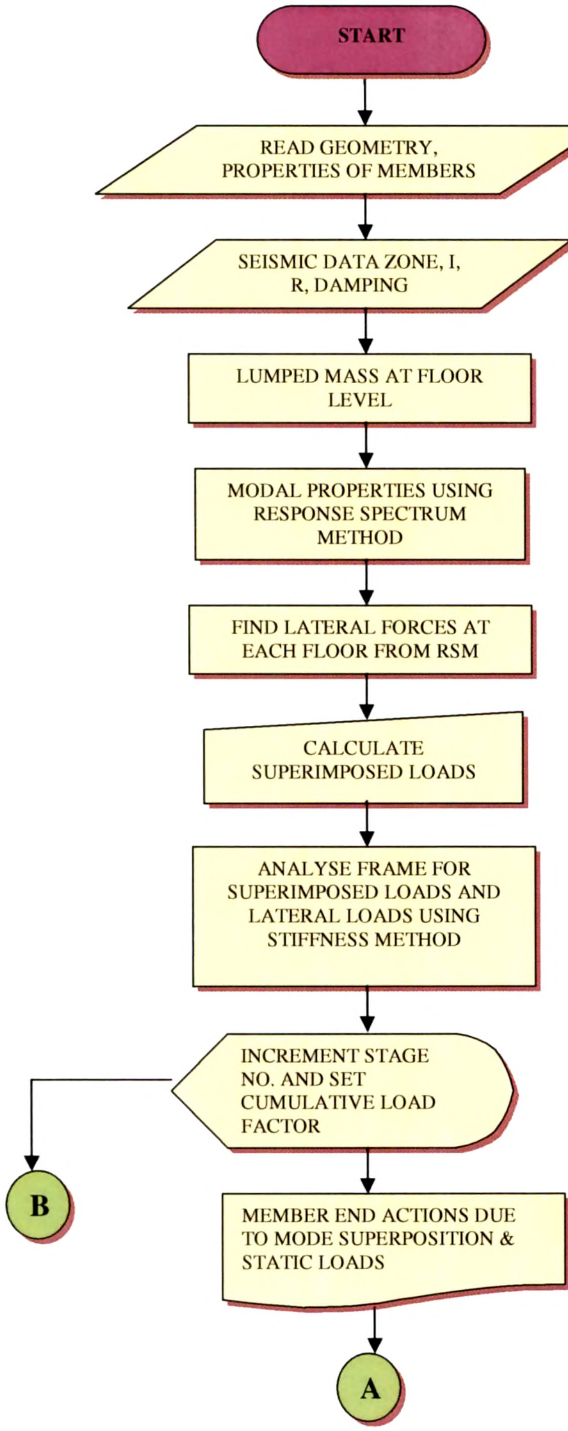


Fig. 4.12 (a) Flowchart for Pushover

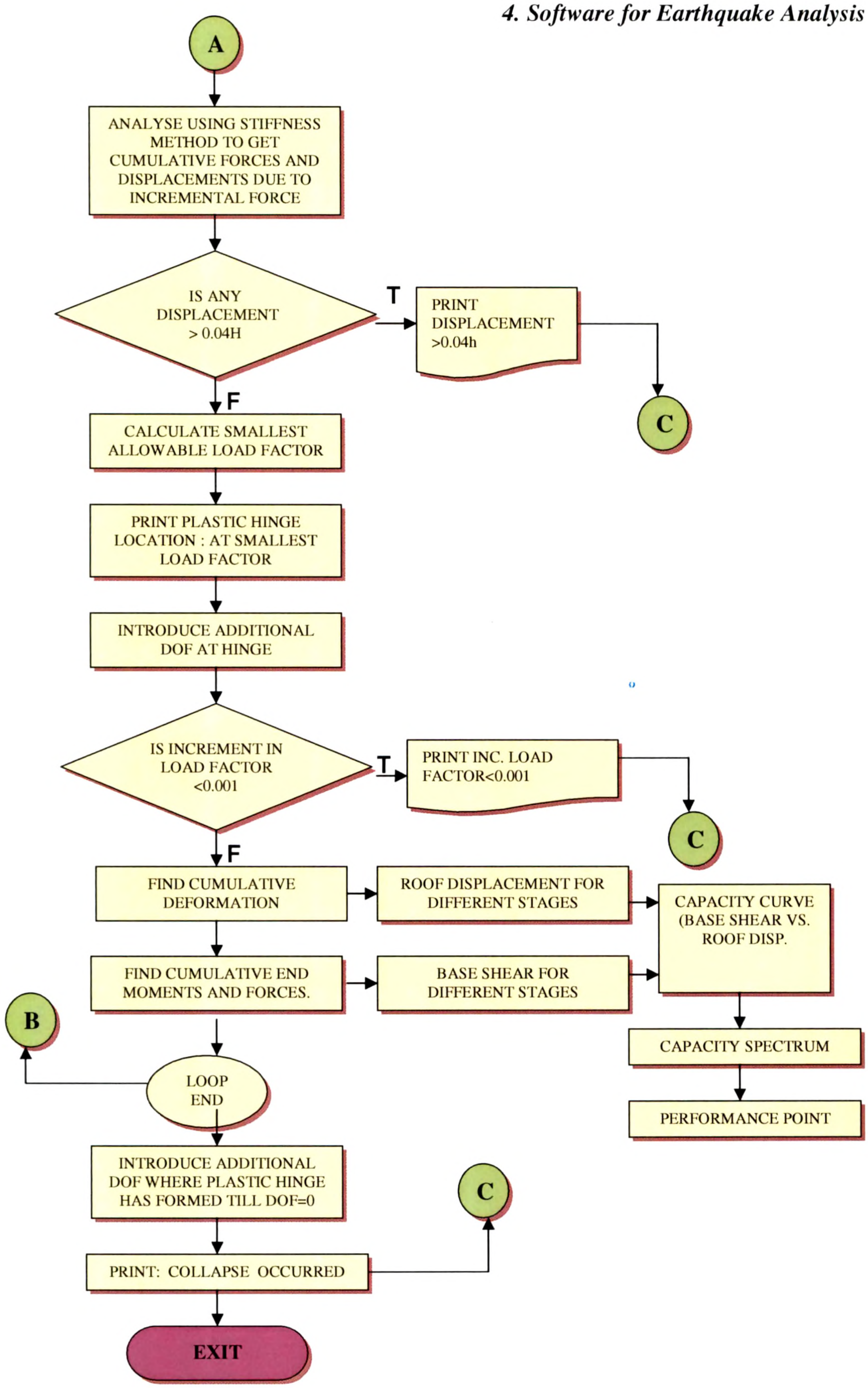


Fig. 4.12 (b) Flowchart for Pushover Analysis (Continued)

Table 4.3 Procedural Hierarchy in SAP2000 and SA-DVR

SAP2000	SA-DVR
1. Define Geometry ➢ No. of bays, No. of storey	1. Enter the Geometry through dialog box ➢ No. of bays, No. of storey
2. Section properties of beam and column	2. Section properties of beam and column through the dialog box
3. Define material property	3. Modulus of elasticity is entered through dialog box.
4. Define hinge properties	4. Hinge Properties defined through Xtract software
5. Generate mathematical model	5. Generating mathematical model and finding out lumped masses at each storey.
6. Find out lateral load for each storey using Response spectrum method.	6. Find out lateral load for each storey using Response spectrum method.
7. Gradually increase the lateral load and apply to the structure and find out cumulative member end action and roof displacement.	7. Gradually increase the lateral load and apply to the structure and find out cumulative member end action and roof displacement.
8. Plot a point (V,D) on capacity curve	8. Plot a point (V,D) on capacity curve
9. If roof displacement > target displacement stop the process.	9. If roof displacement > target displacement (0.04H) stop the process. Go to step 10.
10. If roof displacement < target displacement compare the member end action with ultimate capacity.	10. If displacement < target displacement find out member forces (M) and find out smallest load factor (slf = Mr/M).
11. If M>Mu for any hinge location revise the structure by inserting hinge or revising the stiffness of yield member to almost zero. Go-to step 8	11. Hinge will be formed at a point where there is smallest load factor. Introduce additional degree of freedom(DOF) where hinge has formed.
12.If M<Mu for any hinge location goto step 8.	12. If increment in load factor less than 0.001 then collapse has occurred.

4.7 STEPS FOR USING THE PACKAGE SA-DVR

This package is simple to use due to graphical environment. However, to understand available features of the package, solution of one example is illustrated below:

- **Input Data:** To give the input of the structure such as geometry of the structure, properties of the members click "Input" (Alt+I) on the main menu bar as shown in **Fig.4.13** Dialog box as shown in **Fig.4.14** will be displayed on the screen.

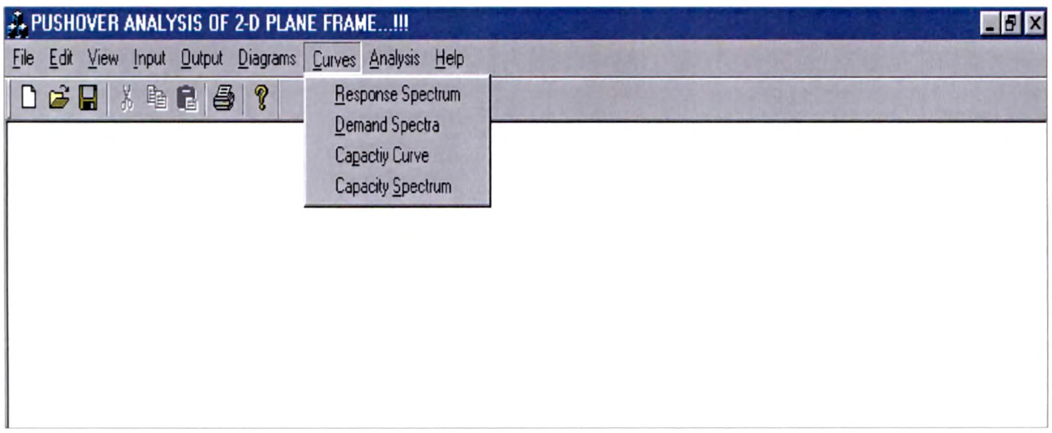


Fig. 4.13 Main Menu Bar

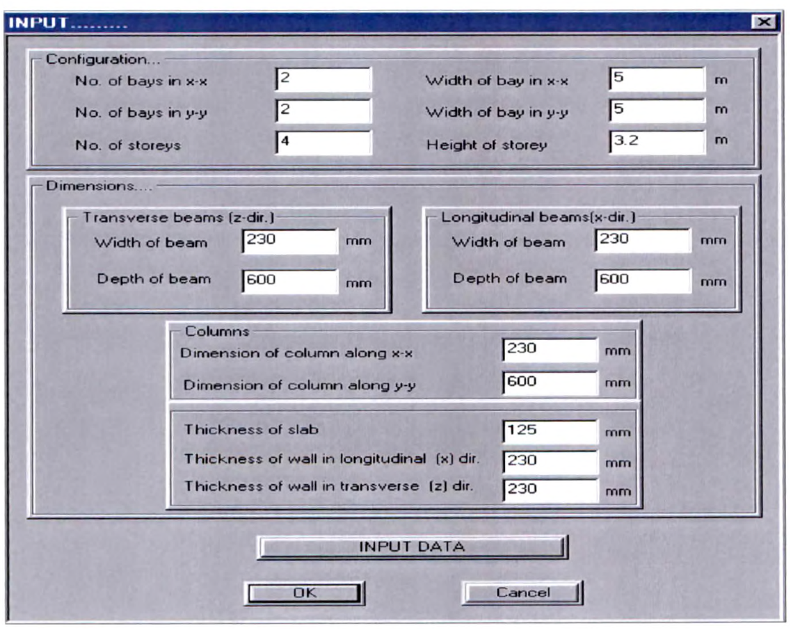


Fig. 4.14 Dialog Box for Assigning Geometry and Properties

On pressing "OK" button geometry will be displayed on screen. On clicking "Geometry" tab, of the "Diagram" pop up menu option, the geometry of the frame will be displayed on the screen with the joint numbering as shown in Fig. 4.15.

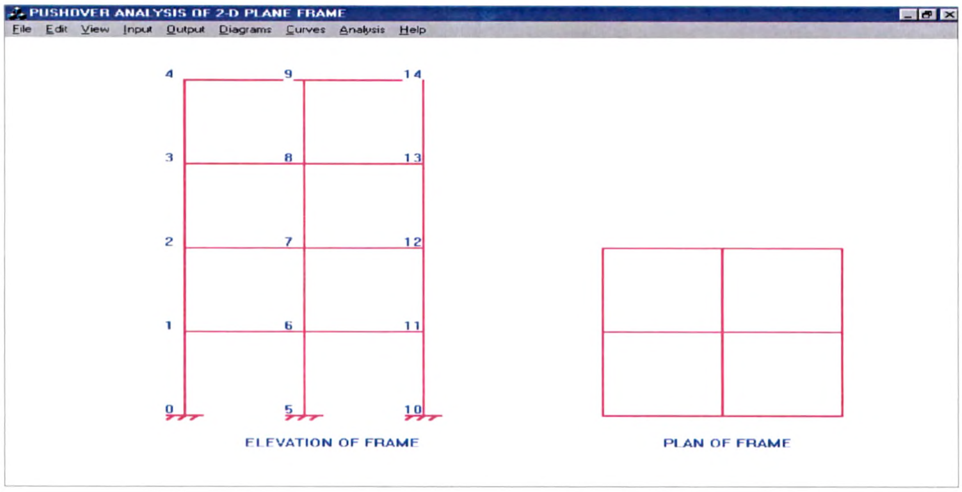


Fig. 4.15 Geometry Display of Regular Plane Frame

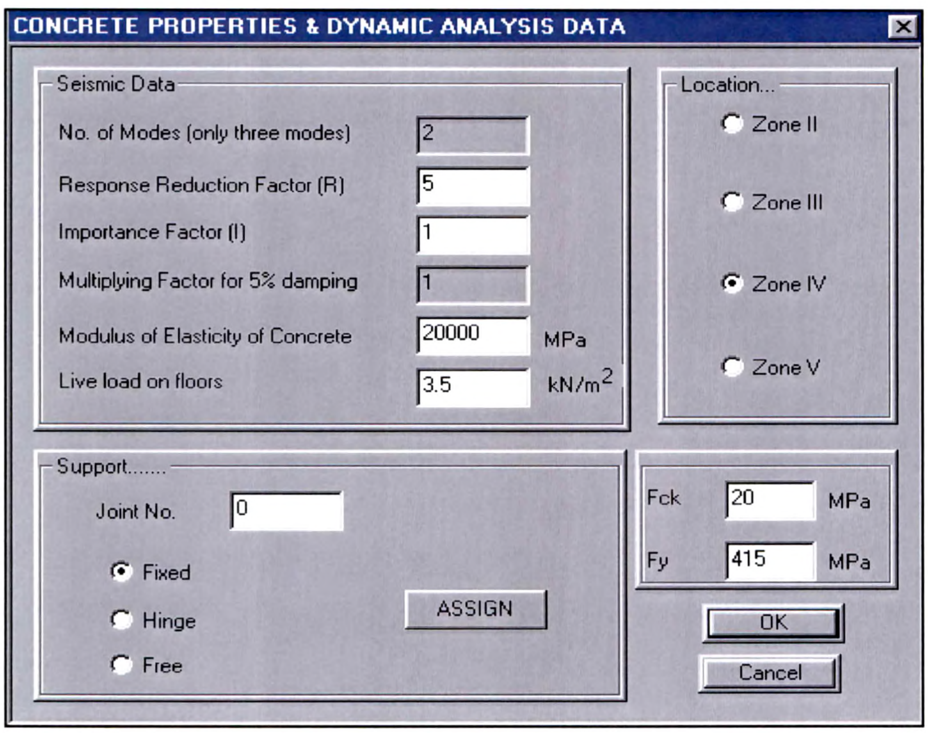


Fig. 4.16 Seismic Parameters and Concrete Properties

The seismic parameters required for performing analysis i.e. response reduction factor, importance factor, location of zone, damping factor, number of modes, etc.

as well as support conditions and live load on floors are assigned through sub-dialog box which appears by clicking "INPUT DATA" as shown in **Fig. 4.16**. The values of these parameters are entered by referring IS-1893 (2002) [4]. Support conditions can be assigned at each base point of the frame by selecting the type of support such as fixed, hinge or free.

- **Analysis:** On clicking "Analysis" the analysis for the assigned structure will be carried out. After understanding the input file format, user can modify the input data according to the changes required and can analyze the modified structure again.

- **Post-processor:** To get the graphs such as response spectra for various damping, demand spectra, capacity curve and capacity spectrum select "Response spectrum" (Alt + R) or "Demand Spectra" (Alt + S) or "Capacity Curve" (Alt + p) or "Pushover Curve", (Alt + P) from the "Curves" (Alt + C) pull down menu. On pressing any of the curve buttons from pull down menu the curve will be displayed on the screen, as shown in **Fig. 4.17** to **4.20** for the selected example.

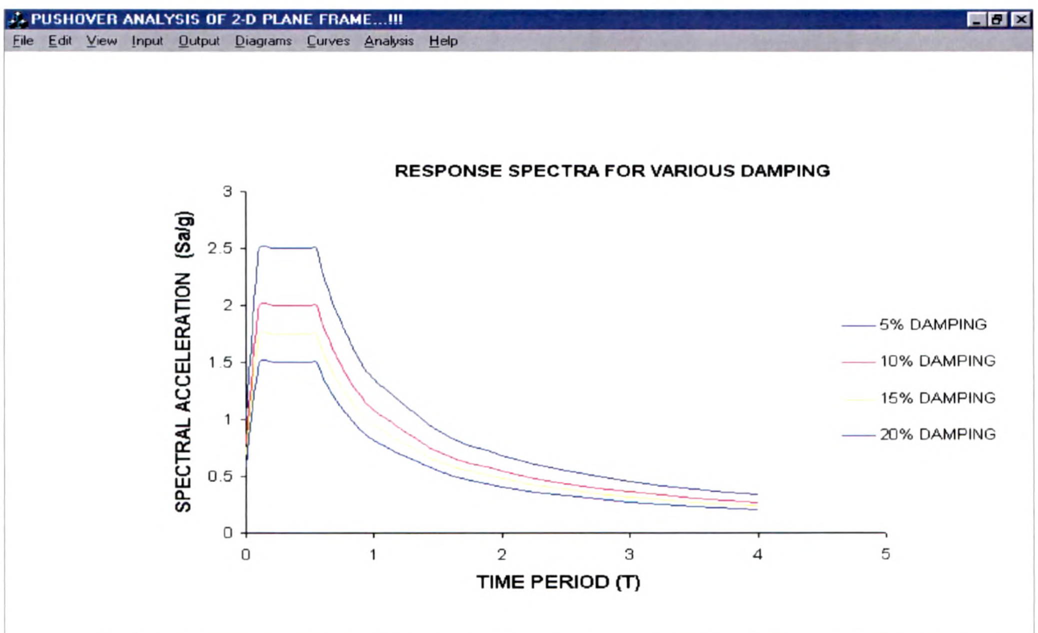


Fig. 4.17 Response spectra

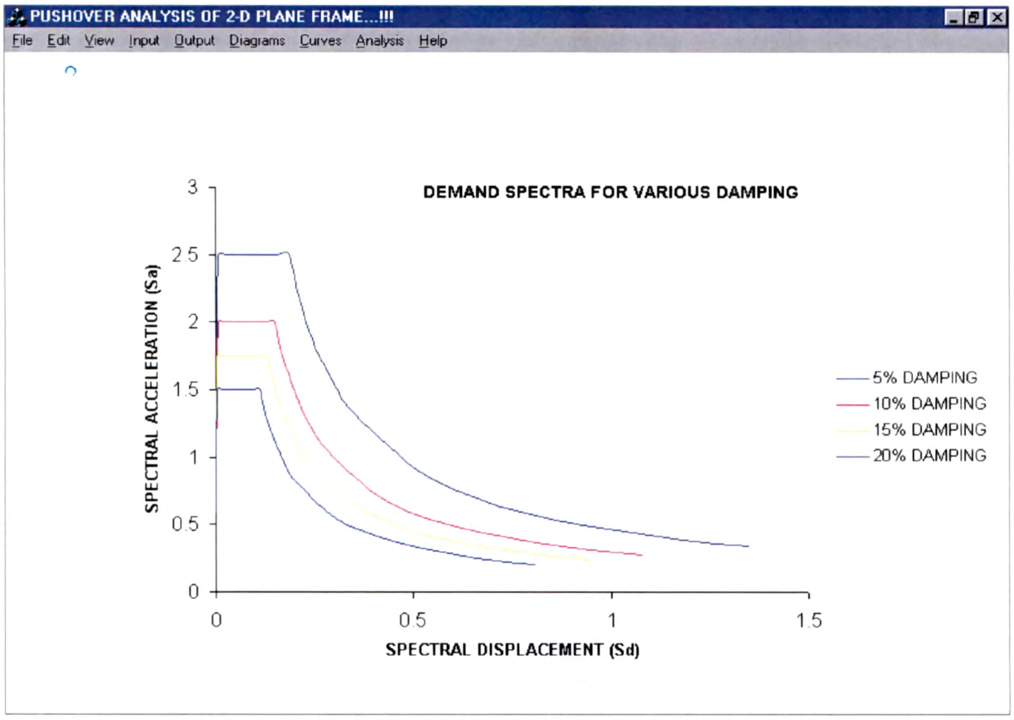


Fig. 4.18 Demand spectra

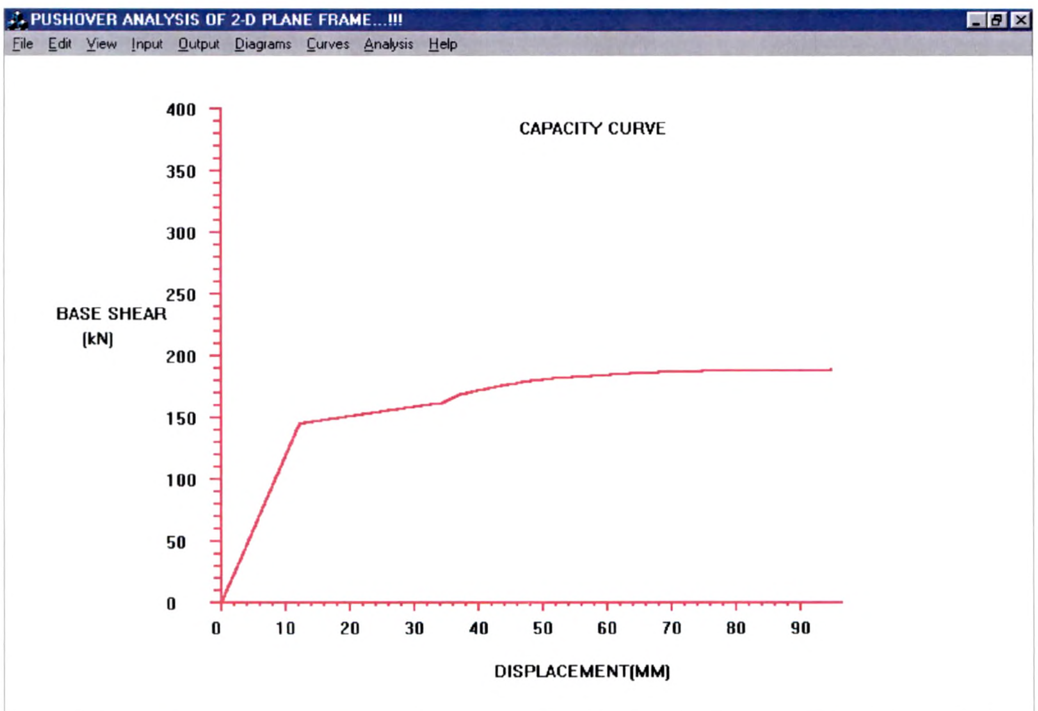


Fig. 4.19 Capacity Curve

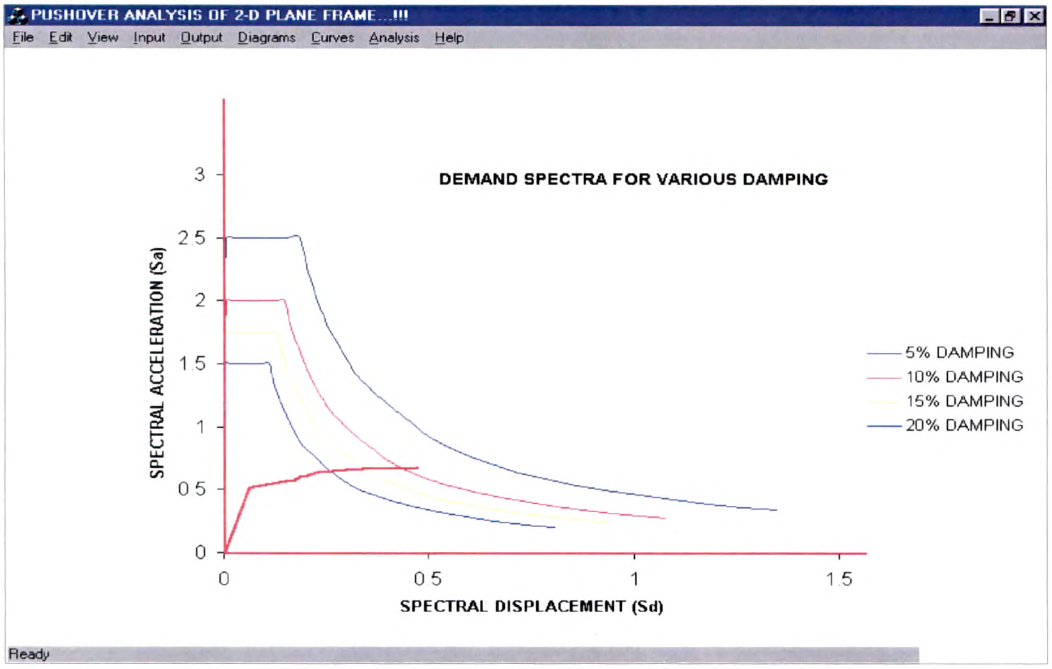


Fig. 4.20 Capacity Spectrum Overlapped on Demand Spectra

The demand spectra are available for various damping from IS: 1893 [4]. In order to get the performance point, the capacity curve should also be converted into a dimensionless form of Capacity Spectrum. Hence the capacity curve as shown in **Fig. 4.19** is converted into the capacity spectrum and superimposed over the demand spectrum as shown in **Fig. 4.20**. The intersection of Capacity Spectrum and Demand Spectra gives the performance point. The performance point so obtained represents the condition for which the seismic capacity of the structure is equal to the seismic demand imposed on the structure by the specified ground motion. If in any case the capacity spectrum does not intersect the demand spectrum, then it can be inferred that the capacity of the structure is less than the demand and hence major retrofitting is required.

The next section demonstrates the Pushover Analysis of plane frame as a case study for using SA-DVR. The results shall be compared with those from SAP, inorder to validate the strength of SA-DVR for independent use with other modules.

4.8 PLANE FRAME EXAMPLE

- Location of building : Zone IV
- Height of building : 6.2 m
- Typical storey height : 3.1 m
- Bay width : 4 m
- No. of Storeis : 2

Member Properties

- Beam : 230 mm X 337.5 mm
- Column : 230 mm X 350 mm
- Slab thickness : 125 mm
- Wall thickness : 230 mm

Material Properties

- fck : 20 MPa
- fy : 415 MPa
- Modulus of Elasticity : 2.0 E07 kN/m^2
- Density : 25 kN/m^3

On executing the program, the Mainframe of VC++ package (Mainframe.cpp is the VC++ file that allows execution of all classes and resources) will be displayed as shown in **Fig. 4.21**. The Mainframe menu bar consists of four main menus of Analysis, Design, Hinge Properties and Pushover.

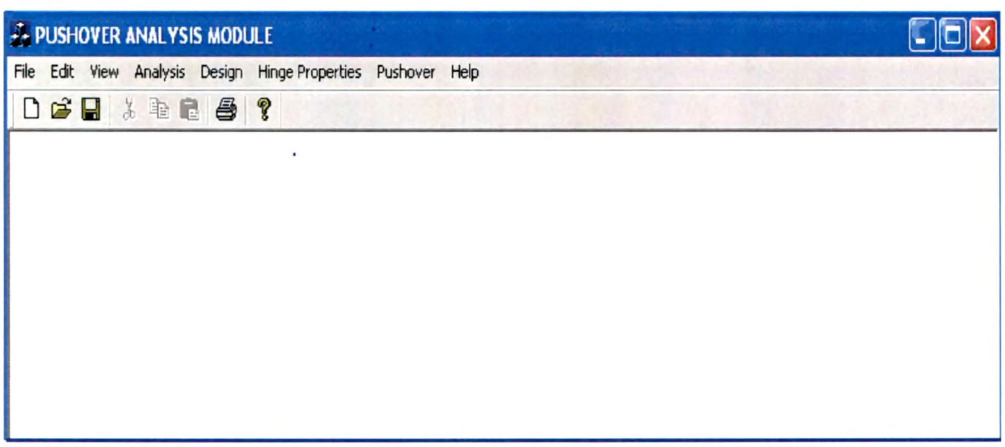


Fig. 4.21 Mainframe of SA-DVR

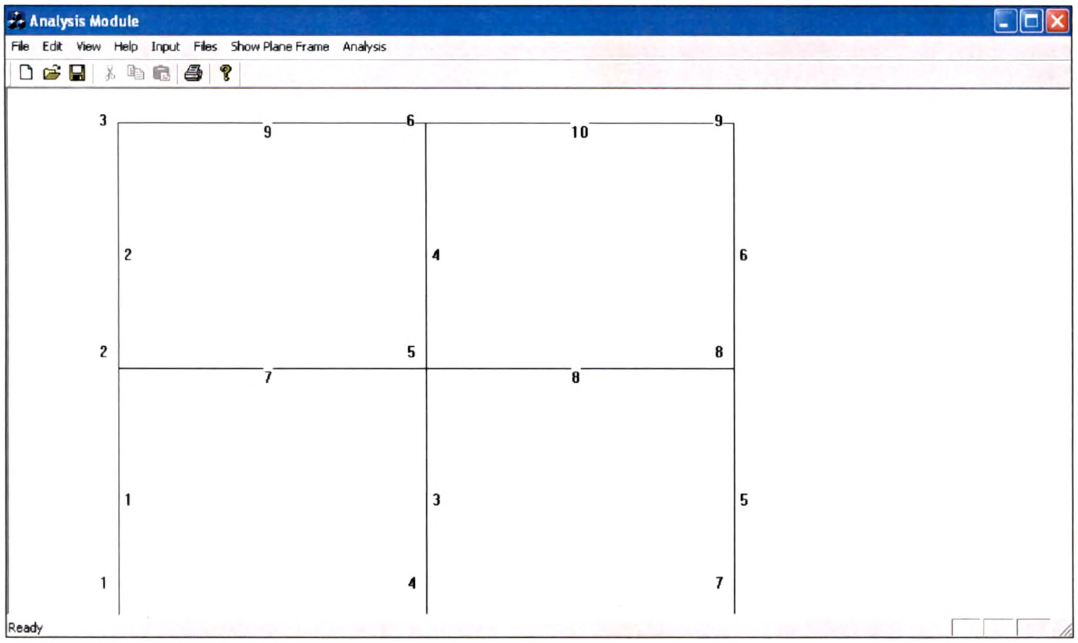


Fig. 4.22 Problem Configuration Displayed in Analysis Module

4.8.1 Analysis Module

Input: On clicking the Analysis option in the main menu bar (Fig. 4.21), Analysis Module will appear on the screen as shown in Fig. 4.22. Input menu of Analysis Module has three dialog boxes of "Geometry", "Supports" and "Loading" as pop-ups. On selecting Geometry and Properties menu, document class will be invoked and dialog box shown in Fig. 4.23 will appear on the mainframe screen. Similarly on selecting "support" menu and "loading" menu respective document class is invoked and the dialog boxes shown in Fig. 4.24 and Fig.4.25 respectively are displayed.

Analysis: By clicking "Analysis" button from the main menu the plane frame will be analyzed for the assigned data using the input data file. After understanding the input file format, user can alter or modify the input parameters into the file directly (bypassing the dialog box) if so required before analyzing.

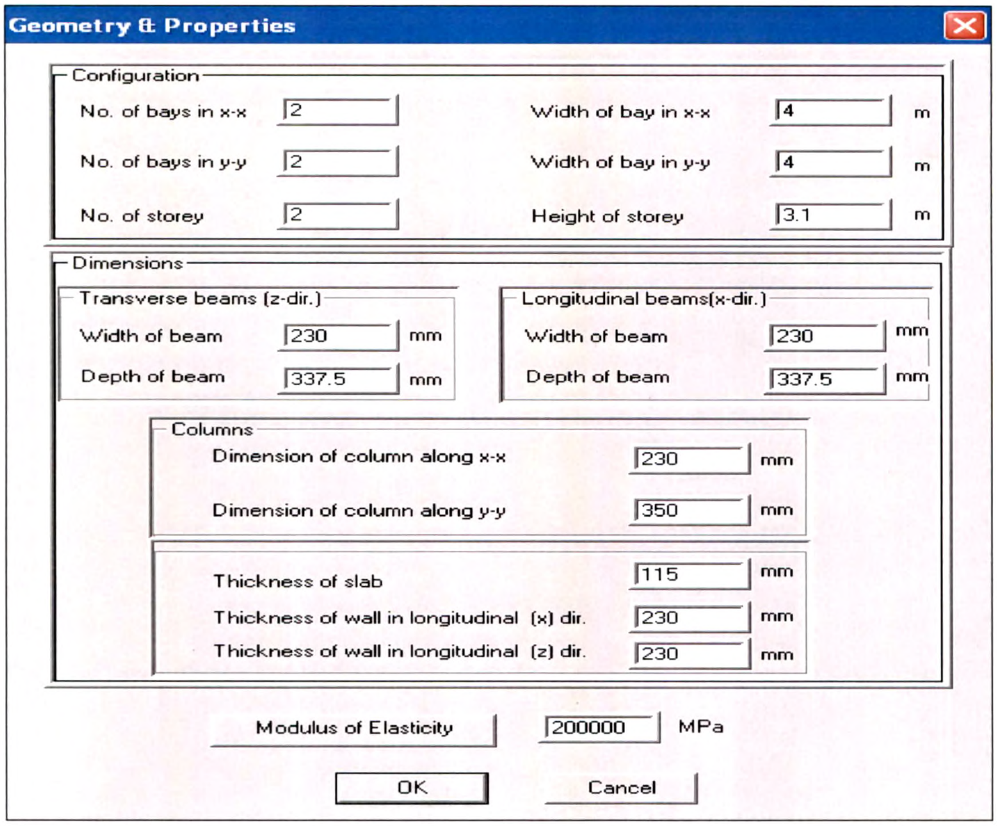


Fig. 4.23 Geometry and Properties Dialog Box

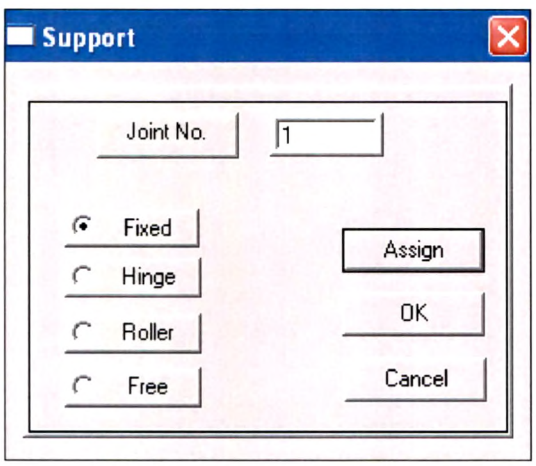


Fig. 4.24 Support Dialog Box

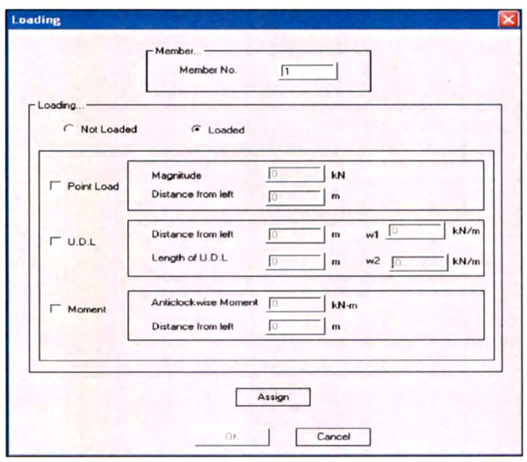


Fig. 4.25 Loading Dialog Box

Post-Processor: Output results from the Analysis module of the package can be extracted in report form as well as graphical form. Text output can be extracted by clicking “Output” from main menu bar as shown in **Table 4.4.**

Table 4.4 Analysis Output

---------------------JOINT DISPLACEMENTS------------------------			
JOINT	X-DISP. (m)	Y-DISP. (m)	Z-ROTATION (Rad.)
1	0	0	0
2	0.00256	-0.0162	-0.0032
3	0.0322	-0.0316	-0.0265
4	0	0	0
5	0.00544	-0.0514	-0.00823
6	0.0288	-0.0892	-0.0192
7	0	0	0
8	0.0108	-0.0202	-0.000675
9	0.0215	-0.0329	0.0168

--------------------MEMBER END ACTIONS----------------------						
	JEND			KEND		
MEM. NO.	AXIAL (kN)	SHEAR (kN)	MOMENT (kN.m)	AXIAL (kN)	SHEAR (kN)	MOMENT (kN.m)
1	84.10	-1.90	-0.914	-84.10	1.90	-4.98
2	80.00	-13.10	-5.44	-80.00	13.10	-35.10
3	367.00	-5.81	-3.77	-367.00	5.81	-14.20
4	196.00	-15.30	-16.70	-196.00	15.30	-30.70
5	105.00	7.72	12.40	-105.00	-7.72	11.50
6	65.90	28.30	32.80	-65.90	-28.30	55.00
7	28.30	70.90	4.31	-28.30	43.10	-39.80
8	13.10	80.00	35.1	-13.10	34.00	-34.40
9	-20.60	74.90	25.00	20.60	39.10	-44.40
10	28.30	70.90	4.31	-28.30	43.10	-39.80

4.8.2 Design Module

4.8.2.1 Beam design

After completing the analysis, the next step is beam and column design. The beam and column whose results are governing are designed in the design module. On clicking the Design option in the main menu bar (**Fig. 4.21**) the mainframe of Design Module will be displayed as shown in **Fig. 4.26**

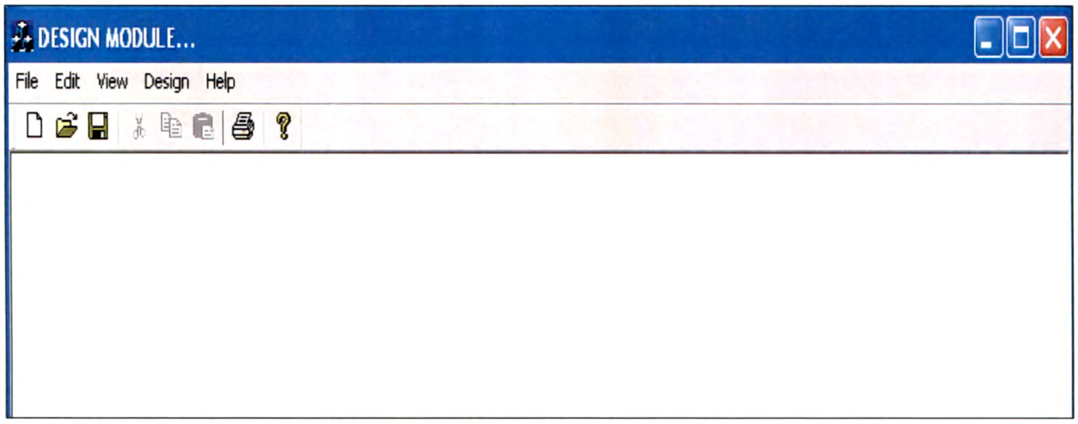


Fig. 4.26 MainFrame Screen of Design Module

"Design" menu on the mainframe has two pop-ups for beam and column design. On clicking "beam" sub-menu, the dialog box shown in **Fig. 4.27** will be displayed. Moment (Mu) and shear force (Vu) are to be entered in the dialog box besides other parameters for design.

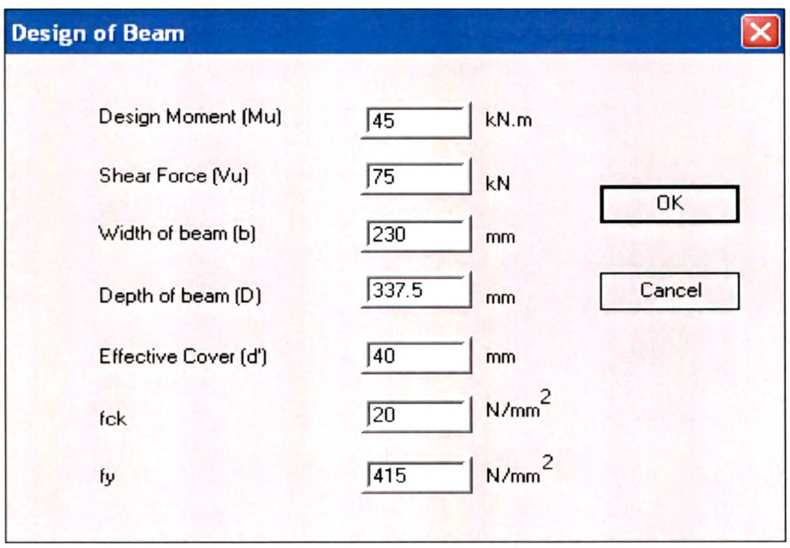


Fig. 4.27 Dialog Box for Beam Design

The design output is obtained in the form of message boxes. The first message box shows whether section is under-reinforced, balanced or over-reinforced (**Fig. 4.28**). If the message box shows that the section is "balanced or over-reinforced" dimensions of the section should be revised to get an under-reinforced section. On clicking "OK", the second message box giving effective depth, depth of neutral axis, moment of resistance and area of reinforcement will appear.

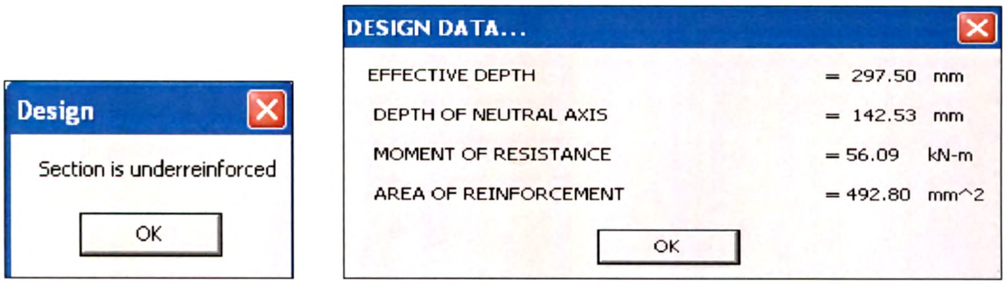


Fig. 4.28 Design Output

The next message box shows number of bars as per the chosen diameter (**Fig. 4.29**) This data will be used later in the pushover analysis, to define the hinge properties in the beam. The last message box displays the stirrups required as shown in **Fig. 4.29**.

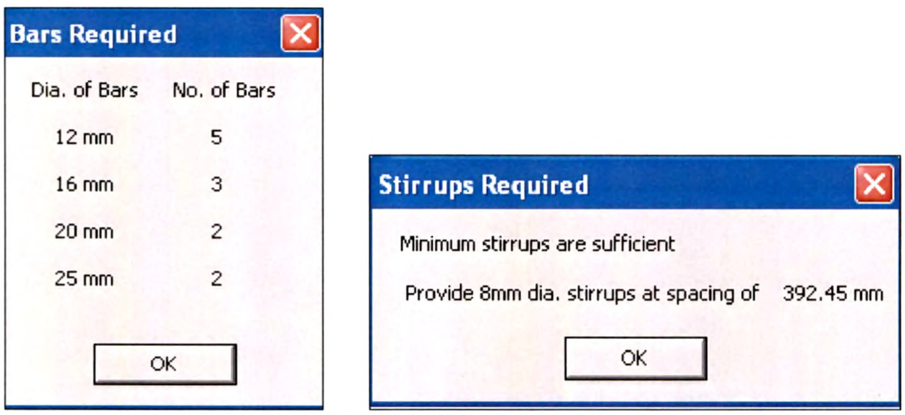


Fig. 4.29 Reinforcement Details for Beam

4.8.2.2 Column design

In the Column Design dialog box, design parameters shown in **Fig. 4.30** are to be given as input. On clicking "NEXT DATA INPUT" button, another dialog box (**Fig. 4.31**) will appear in which values of d'/D, Pu/fckbd and Mu/fckbd^2 are automatically calculated and displayed. These values are used for referring SP16 to find value of p/fck. Finally design outputs are displayed in a message box showing the area of reinforcement, diameter of bars and number of bars required (**Fig. 4.32**). This reinforcement detail is then used to get the hinge properties of the column during pushover.

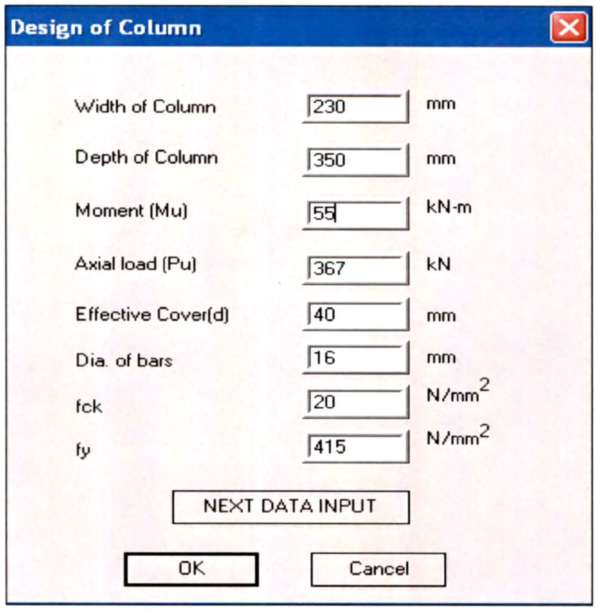


Fig. 4.30 Dialog Box for Column Design

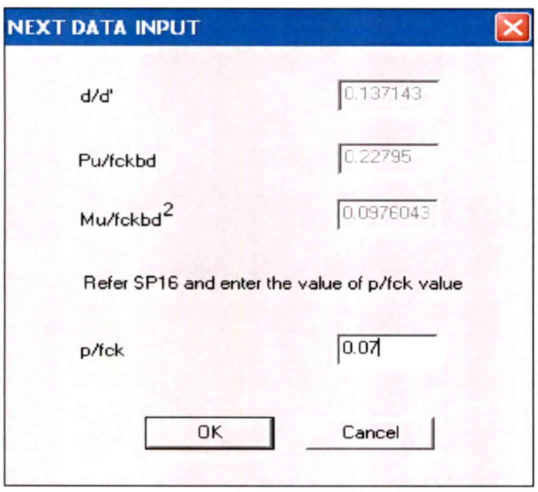


Fig. 4.31 P-M Interaction

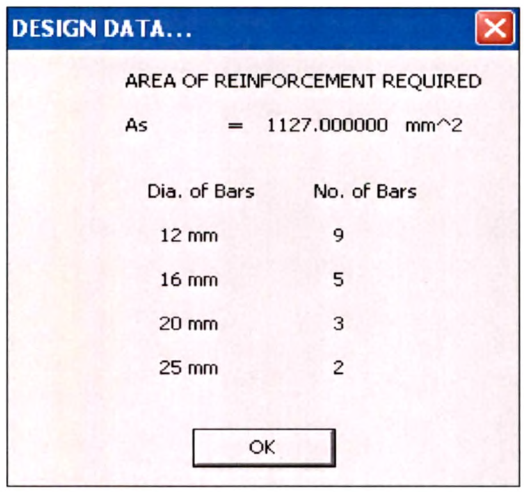


Fig. 4.32 Design of Column

4.8.3 Hinge Properties Module

In order to carry out Pushover Analysis, hinge locations and their properties are to be defined in the frame members. The hinge property will depend on the amount of reinforcement which in turn will control the moment-curvature relationship in beams and the P-M-M interaction in case of columns. A readymade software XTRACT (Cross sectional(X) Structural (TR) Analysis(A) of Components(CT)) gives the m-ф properties and P-M interaction curve for any type of cross-sectional dimension. Properties of the hinges are derived from Xtract which has been embedded within SA-DVR. On clicking the Hinge Properties menu on the Mainframe of SA-DVR (**Fig. 4.21**) Xtract will be invoked which will generate beams M-Phi curves for the given beams whilend for columns P-M-M curves are generated as described in the following paragraphs.

4.8.3.1 M-PHI relation for beams

Sections can be created by Importing from a File, as a User Defined Section, or from the template (**Fig. 4.33**). The template offers a quick way to add basic sections commonly used in design. The section template library can be accessed by selecting 'from template' option when starting a New Project. The template takes the user through a series of dialog boxes to describe the section, the reinforcing steel and the concrete material models. On clicking "Begin XTRACT" dialog box opens (**Fig. 4.34**) for section information and beam core details.

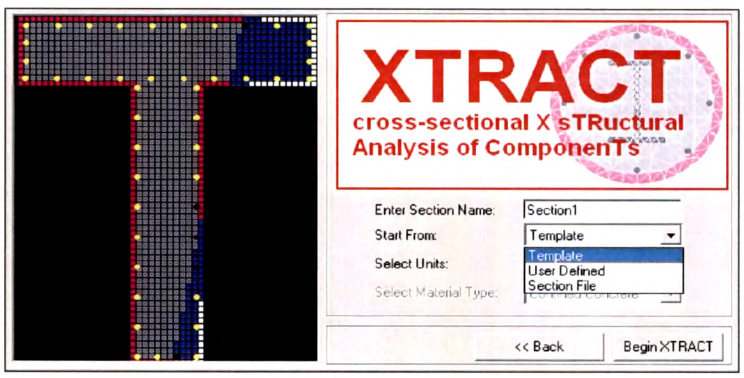


Fig. 4.33 XTRACT New Project

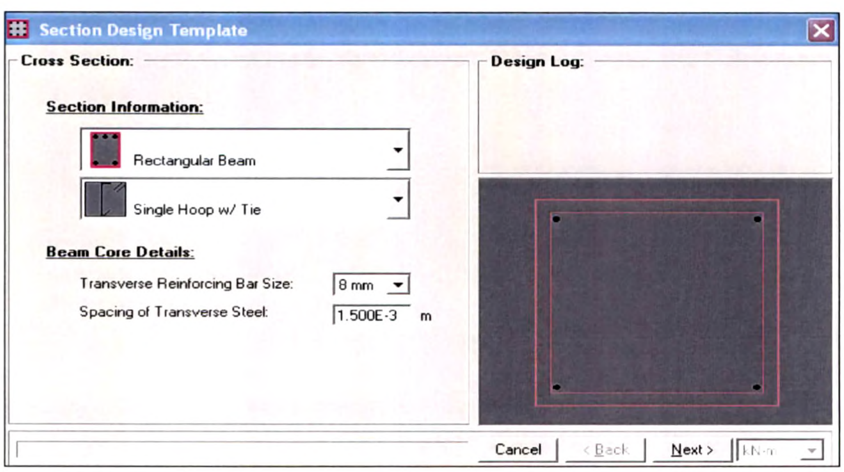


Fig. 4.34 Cross Sectional Details

On clicking Next, user has to enter geometry of the cross section (**Fig.4.35**). As the input values change, the geometry will be redrawn and the areas and reinforcing ratios will be updated. Next, the design log is updated and the Material template comes up.

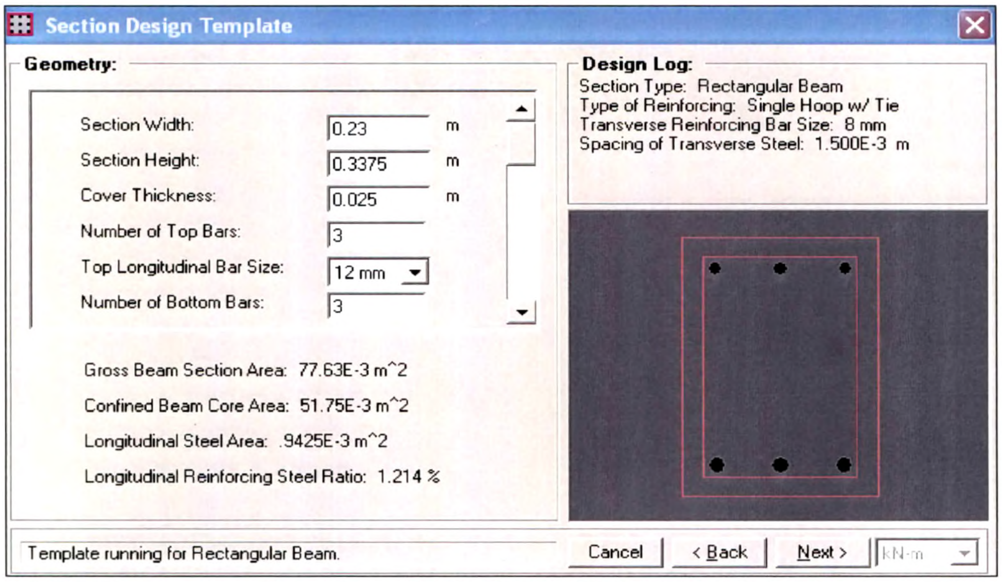


Fig. 4.35 Geometry Details

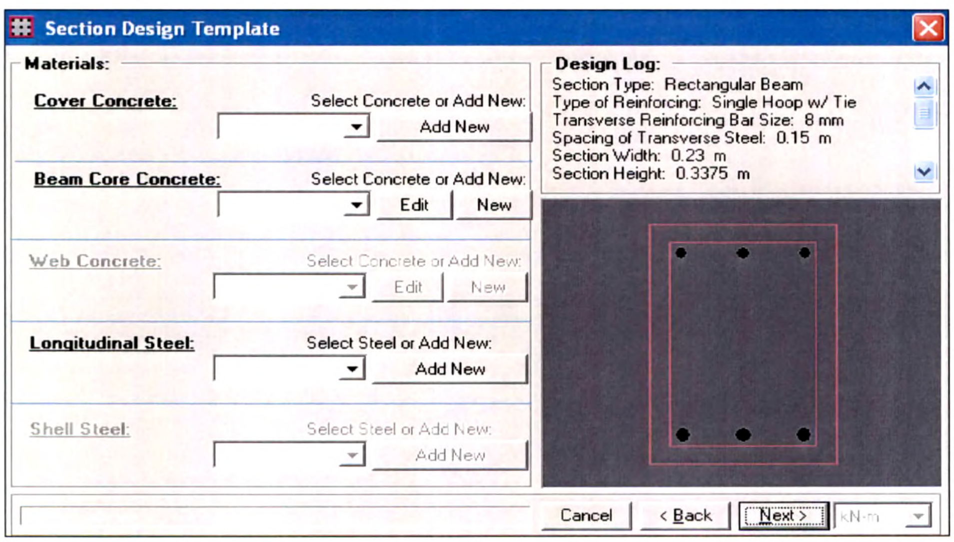


Fig. 4.36 Material Details

With the section defined, the materials need to be assigned to the various parts of the section. In this dialog box (**Fig. 4.36**) select the cover concrete (Unconfined Concrete Material Model), the core concrete (Confined Material Model), and the longitudinal steel (Bilinear with Strain Hardening Material Model). **Figs. 4.37- 4.39** show the properties for the material selected.

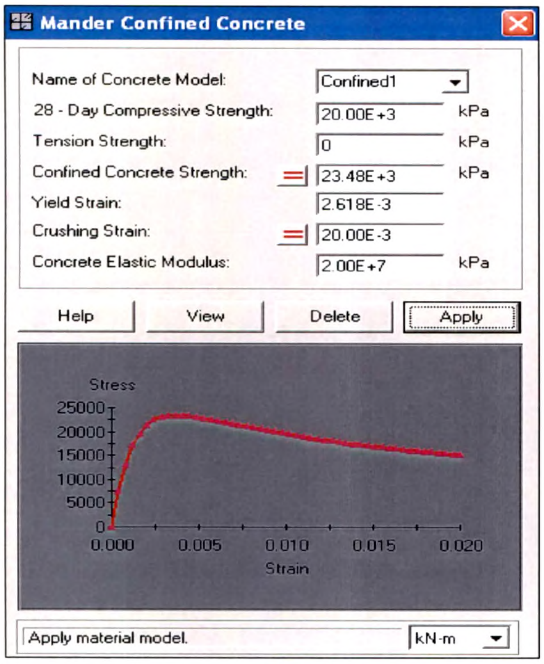


Fig. 4.37 Confined Concrete

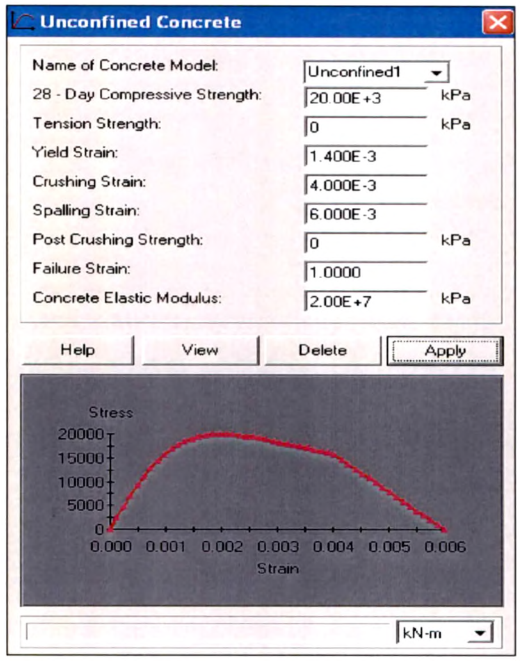


Fig. 4.38 Unconfined Concrete

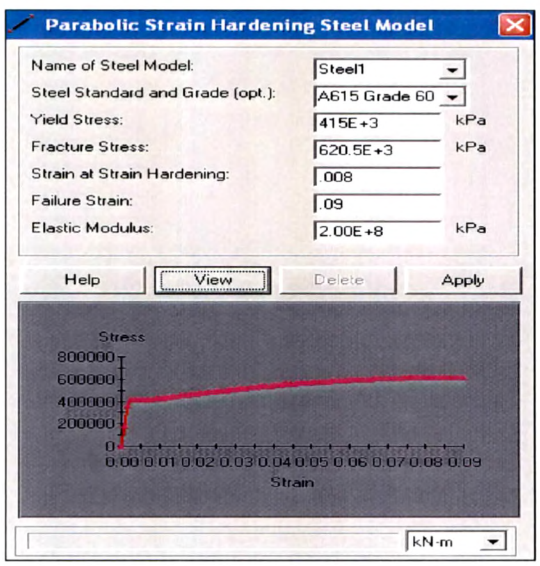


Fig. 4.39 Steel

Once the materials have been assigned to the different parts of the section (core, cover, and longitudinal bars), click the 'Next' button. The design log will be updated and then user will be taken to the final page. This dialog box shown in **Fig. 4.40** gets a user defined name for the section as also the size of mesh. The mesh size represents the average width of the triangles of the section that will be created. Tighter the mesh, more accurate the solution, but higher the computational cost. To create the section and close the template, click the 'Create Section' button.

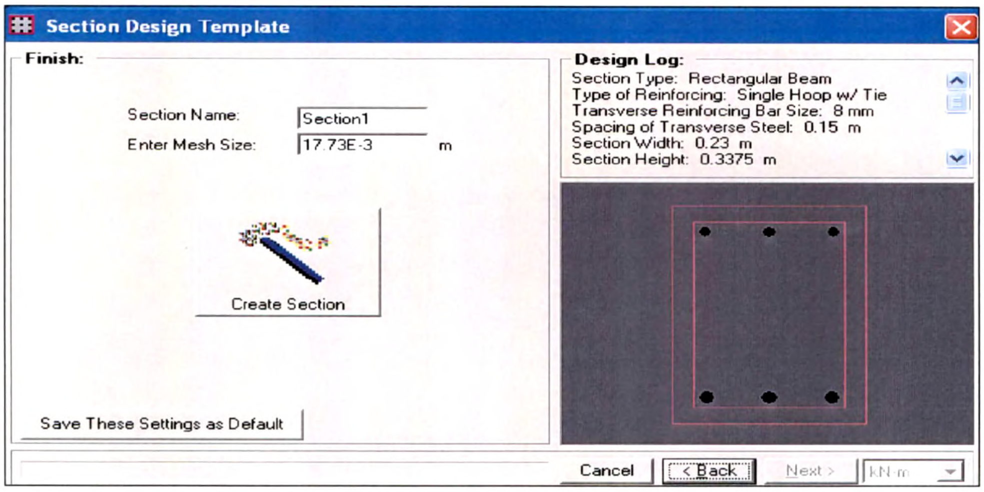


Fig. 4.40 Creating a Section

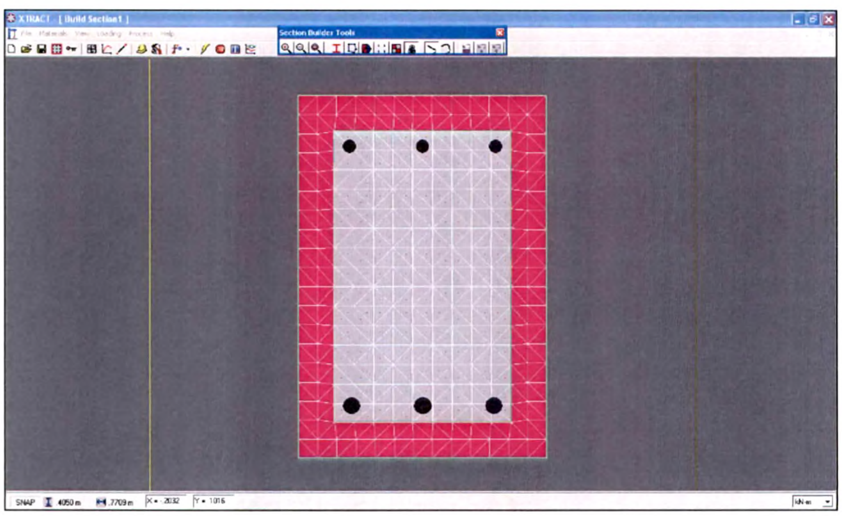


Fig. 4.41 Final Section with Discretization

Once a section has been created, various loadings can be added. The loading options are Moment Curvature, PM Interaction, and Capacity Orbit (Moment-

Moment Interaction). All loadings and sections can be verified in the Project Manager as the project grows (**Fig. 4.41**).

To add a Moment Curvature Analysis on a cross section that has been created and after the Material Models have been defined, click Moment Curvature in the Loading Menu (**Fig. 4.42**) from the main title bar, right click the folder in the Project Manager that bears the section name and select Moment Curvature, or click the 'Add Moment Curvature' icon from the Main Toolbar.

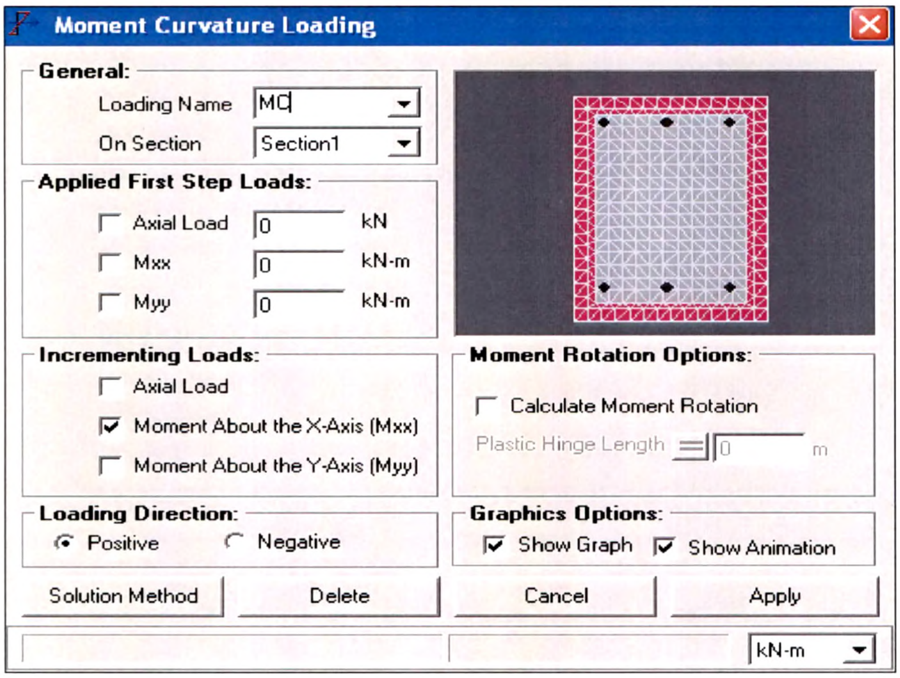


Fig. 4.42 Moment Curvature

After assigning the load click "Run Analysis" on the Process menu. Click on "View" menu (**Fig. 4.43**) to open the Project Manager and then open the analysis result (**Fig. 4.44**). The analysis report (**Fig.4.45**) shows the value of moment at first yield and ultimate moment which is used in the pushover analysis of 2D plane frame.

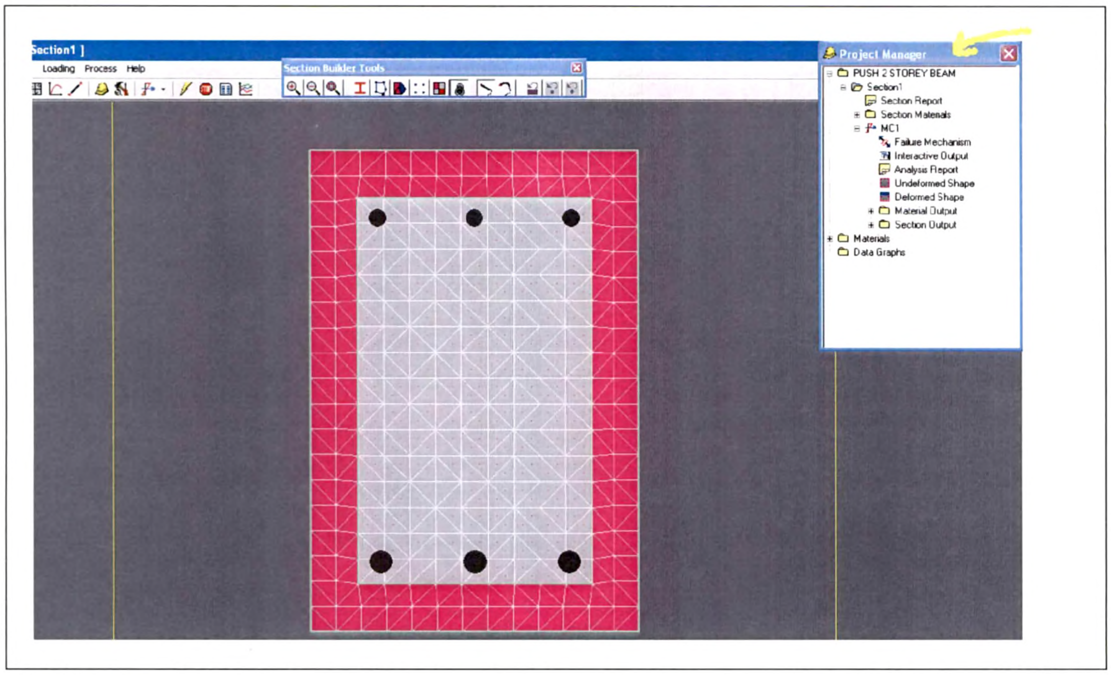
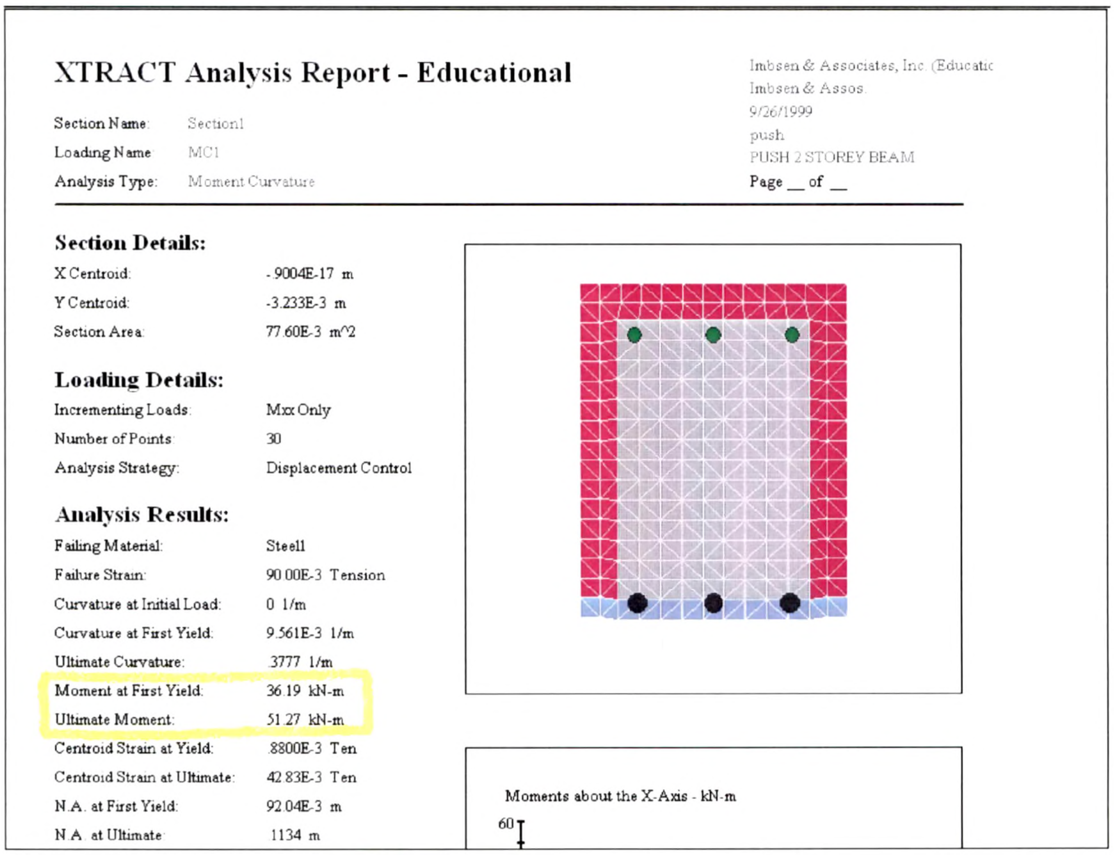


Fig. 4.43 Project Manager



XTRACT Analysis Report - Educational

Imbsen & Associates, Inc. (Educatic
Imbsen & Assos.
9/26/1999
push
PUSH 2 STOREY BEAM
Page __ of __

Section Name:	Section1
Loading Name:	MC1
Analysis Type:	Moment Curvature

Section Details:

X Centroid:	-.9004E-17 m
Y Centroid:	-3.233E-3 m
Section Area:	77.60E-3 m^2

Loading Details:

Incrementing Loads:	Mxx Only
Number of Points:	30
Analysis Strategy:	Displacement Control

Analysis Results:

Failing Material:	Steel1
Failure Strain:	90.00E-3 Tension
Curvature at Initial Load:	0 1/m
Curvature at First Yield:	9.561E-3 1/m
Ultimate Curvature:	.3777 1/m
Moment at First Yield:	36.19 kN-m
Ultimate Moment:	51.27 kN-m
Centroid Strain at Yield:	.8800E-3 Ten
Centroid Strain at Ultimate:	42.83E-3 Ten
N.A. at First Yield:	92.04E-3 m
N.A. at Ultimate:	.1134 m

Moments about the X-Axis - kN-m
60

Fig. 4.44 Yield and Ultimate Moments

4.8.3.2 P-M-M curve for columns

The section is created in a manner similar to the one described for the M-Phi curve and then loading option PM Interaction is applied (**Fig. 4.45**).

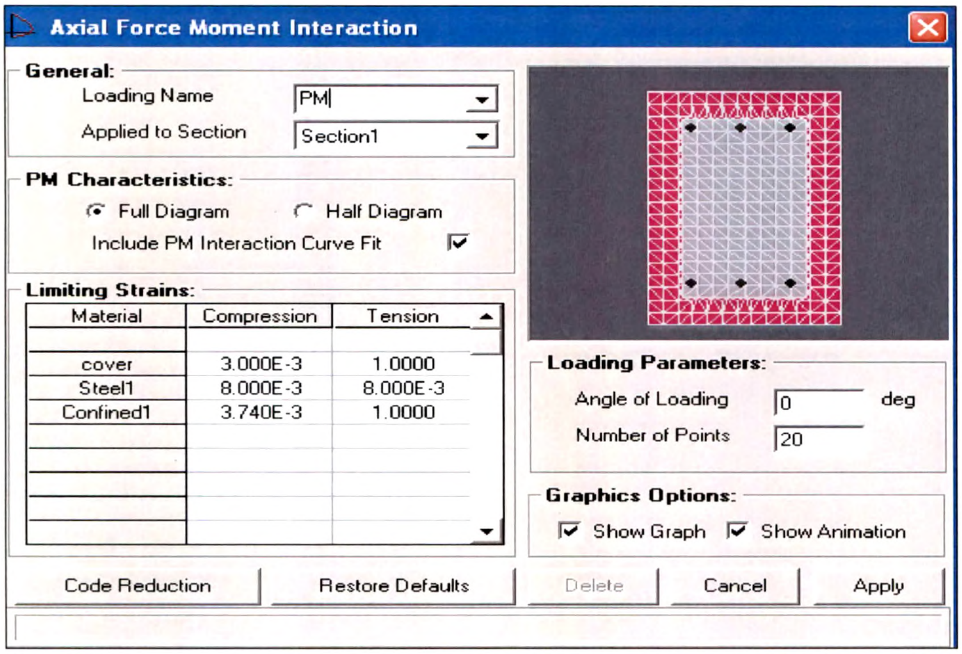


Fig. 4.45 P-M Interaction

Different interaction factors are specified for different locations within the Axial Force Moment Interaction Curve. These factors, when applied, produce the code reduced PM Interaction diagram (**Fig. 4.46**). Because XTRACT is code independent, the user can apply any system of factors representative of any code.

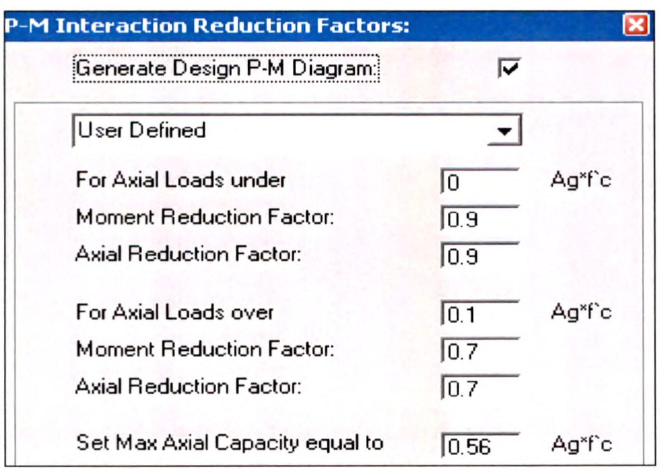


Fig. 4.46 P-M Reduction Factors

Factor groups can be added permanently to the program and reused repeatedly. These factor groups are saved in a file titled ReductionFactors.dat located in the XTRACT directory. Factor groups can be added in the Options dialog or directly in the ReductionFactors.dat file using a text editor. The part of analysis report is depicted in **Fig. 4.47.**

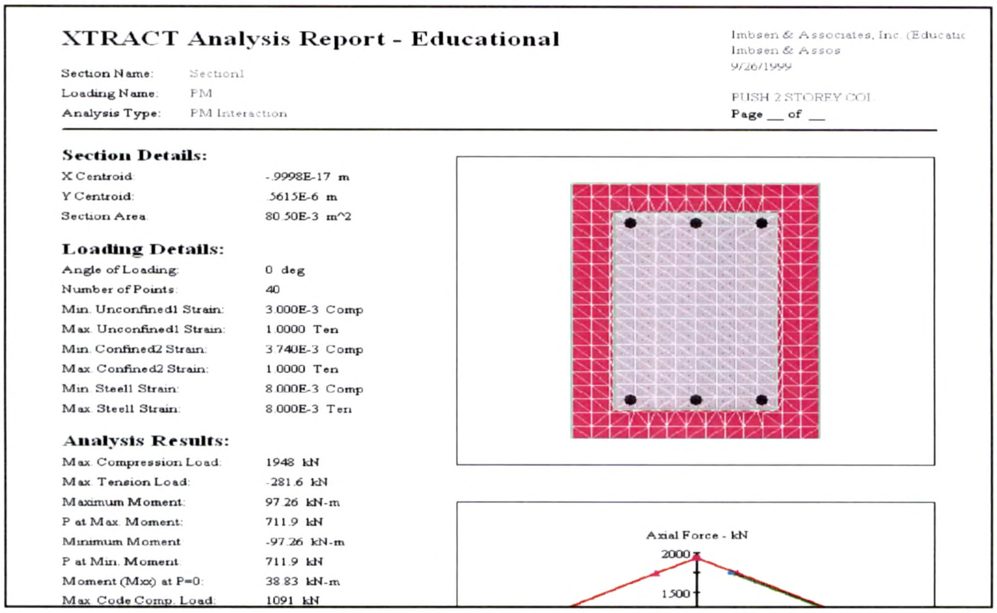


Fig. 4.47 Analysis Results for PM values

4.8.4 Pushover Analysis Module

On clicking the Pushover option in the main menu bar (**Fig. 4.21**) another mainframe of Pushover Module will appear on the screen which is as shown in **Fig. 4.48**. From menu bar on selecting "Input" menu, document class will be invoked and dialog box shown in **Fig. 4.49** will appear on the main screen.

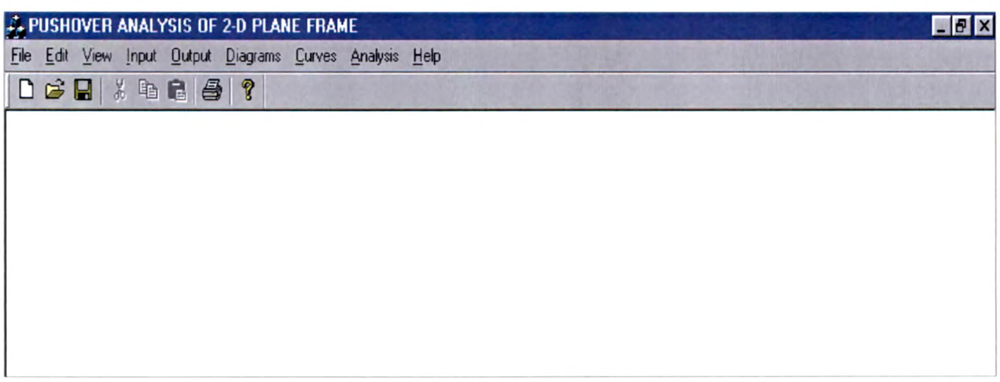


Fig. 4.48 Main Menu Bar

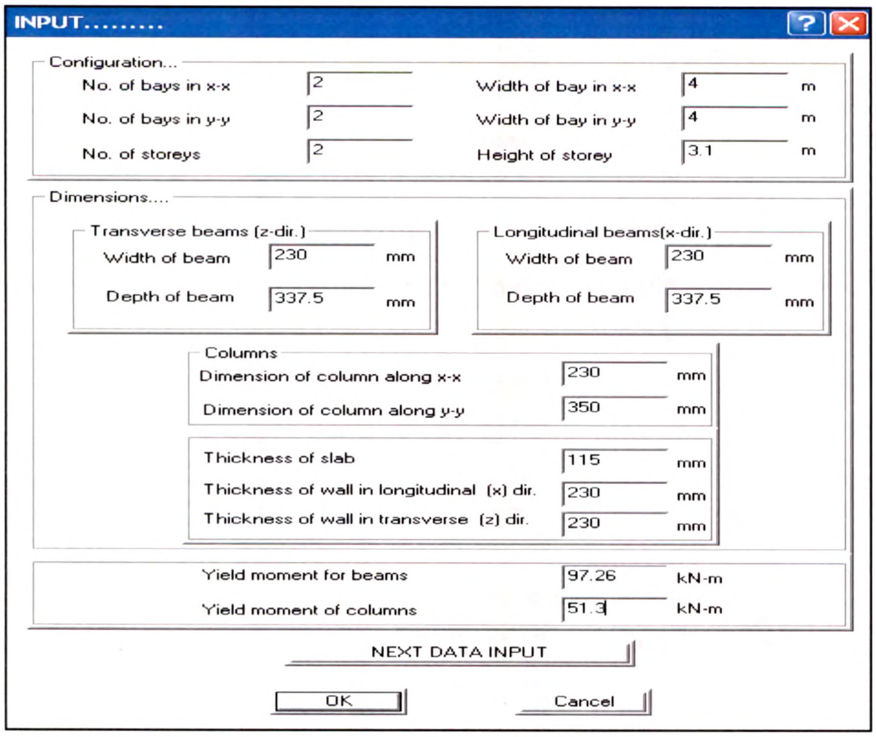


Fig. 4.49 Input Dialog Box

After giving data in the dialog box (**Fig.4.49**), click on “Next Data Input” for giving seismic parameters, support conditions, modulus of elasticity and live load in their relevant edit boxes (**Fig. 4.50**).

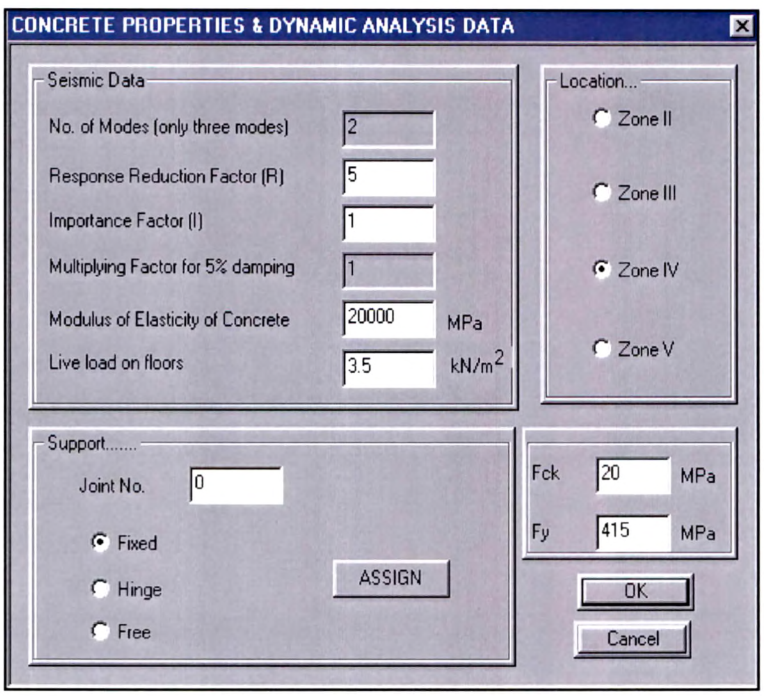


Fig. 4.50 Dialog Box

On pressing "OK" button of the " NEXT DATA INPUT" dialog box, the main dialog box as shown in **Fig. 4.21** will reappear. On pressing "OK" button of the main dialog box the function OninputOK will be executed. The member function of CInput class, will generate the input files "input.txt" and "input1.txt".

By clicking "Analysis" button from the main menu the plane frame will be analyzed for the assigned data using the input data file as described earlier.

Results of the package can be extracted in report form as well as graphical form. Text output can be extracted by clicking "Output" from main menu bar, as text file of notepad, as shown below:

```
No. of bays along x-x are :2          Spacing of bays along x-x : 4 m
No. of bays along y-y are :2          Spacing of bays along y-y : 4 m
No. of storeys : 2                    Storey height :3.1 m
--------------------------------------------------------------------
Thickness of slab: 115 mm
Thickness of wall in longitudinal direction (x-direction): 230 mm
Thickness of wall in transverse direction (z-direction): 230 mm
Dimension of columns along x-x: 230 mm
Dimension of columns along y-y: 350 mm
Width of beams in longitudinal direction: 230 mm
Depth of beams in longitudinal direction: 337.5 mm
Width of beams in transverse direction: 230 mm
Depth of beams in transverse direction: 337.5 mm
Live load on floors: 3.5 kN/m2
No. of modes to be considered: 2
Modulus of elasticity of concrete is: 20000 MPa
--------------------------------------------------------------------
Results at floor level
----------------------
Weight of beams :93.15 kN
Weight of slab :296.0013 kN
Weight of columns :56.1487 kN
Weight of walls:684.48 kN
Total weight :1129.78  kN
```

Results at roof level

Weight of beams:93.15 kN

Weight of slab : 184 kN

Weight of columns : 28.0744 kN

Weight of walls: 342.24 kN

Total weight : 647.464 kN

Moment of inertia of column @ x-x: 0.000354871 m4

Stiffness of column @ x-x: 2858.88 kN/m

No. of columns: 3

Actual storey stiffness: 8576.65 kN/m

Response Reduction Factor: 5

Importance Factor: 1 Zone Factor (Z) = 0.24

---------------STRUCTURAL DETAILS------------------

Number of members = 10

Number of joints = 9

Modulus of elasticity = 2E+007

----------------JOINT CO-ORDINATES--------------------

Joint No.	X-coordinate	Y-coordinate
0	0	0.0
1	0	3.1
2	0	6.2
3	4	0.0
4	4	3.1
5	4	6.2
6	8	0.0
7	8	3.1
8	8	6.2

--------------------MEMBER CONNECTIVITY------------------

Member No.	j-end	k-end
1	0	1
2	1	2
3	3	4
4	4	5
5	6	7
6	7	8
7	1	4
8	2	5
9	4	7
10	5	8

MEM No.	E (kPa)	I (m^3)	A (m^2)	Mp (kN-m)
1	2e+007	0.000821771	0.0805	97.26
2	2e+007	0.000821771	0.0805	97.26
3	2e+007	0.000821771	0.0805	97.26
4	2e+007	0.000821771	0.0805	97.26
5	2e+007	0.000821771	0.0805	97.26
6	2e+007	0.000821771	0.0805	97.26
7	2e+007	0.000736831	0.077625	51.3
8	2e+007	0.000736831	0.077625	51.3
9	2e+007	0.000736831	0.077625	51.3
10	2e+007	0.000736831	0.077625	51.3

Number of Non-plastic displacements = 27

Number of total displacements = 47

------------MEMBER DISPLACEMENT NUMBERS-----------------

0	1	2	3	4	5	27	28
3	4	5	6	7	8	29	30
9	10	11	12	13	14	31	32
12	13	14	15	16	17	33	34
18	19	20	21	22	23	35	36
21	22	23	24	25	26	37	38
3	4	5	12	13	14	39	40
6	7	8	15	16	17	41	42
12	13	14	21	22	23	43	44
15	16	17	24	25	26	45	46

Member	Length	cx	cy
1	3.1	0	1
2	3.1	0	1
3	3.1	0	1
4	3.1	0	1
5	3.1	0	1
6	3.1	0	1
7	4	1	0
8	4	1	0
9	4	1	0
10	4	1	0

--------------------SUPPORT STATUS---------------------------

0 0 0	1 1 1	1 1 1
0 0 0	1 1 1	1 1 1
0 0 0	1 1 1	1 1 1

---------------------------JOINT LOADS------------------------

Number of joint loads = 2

4	51.33	0	0
5	46.74	0	0

Stage 1

CUMULATIVE LOAD FACTOR IS = 1.10699

CUMULATIVE FIXED END MOMENTS AND AXIAL FORCE

MEM NO.	MOMENT I-END	MOMENT J-END	TENSION	SHEAR FORCE
1	-65.49	-37.87	36.44	33.34
2	-13.43	-26.37	12.26	12.84
3	-74.49	-55.34	-5.588e-007	41.88
4	-35.5	-45.31	1.96e-006	26.07
5	-65.49	-37.87	-36.44	33.34
6	-13.43	-26.37	-12.26	12.84
7	51.3	45.42	20.5	-24.18
8	26.37	22.66	12.84	-12.26
9	45.42	51.3	-20.5	-24.18
10	22.66	26.37	-12.84	-12.26

CUMULATIVE DEFORMATION VALUES ARE

X-MOVEMENT	Y-MOVEMENT	Z-ROTATION
0	0	0
0.00907	7.02e-005	0.0026
0.0172	9.38e-005	0.00138
0	0	0
0.00913	-1.08e-012	0.00181
0.0172	2.7e-012	0.00088
0	0	0
0.00907	-7.02e-005	0.0026
0.0172	-9.38e-005	0.00138

AT THE END OF STAGE 1

PLASTIC HINGE LOCATIONS

MEM NO.	I-END	J-END
1	0	0
2	0	0
3	0	0
4	0	0
5	0	0
6	0	0
7	1	0
8	0	0
9	0	0
10	0	0

Stage 2

CUMULATIVE LOAD FACTOR IS = 1.11

CUMULATIVE FIXED END MOMENTS AND AXIAL FORCE

MEM NO.	MOMENT I-END	MOMENT J-END	TENSION	SHEAR FORCE
1	-65.49	-37.87	36.44	33.34
2	-13.43	-26.37	12.26	12.84
3	-74.49	-55.34	2.568e-006	41.88
4	-35.50	-45.31	2.239e-006	26.07
5	-65.49	-37.87	-36.44	33.34
6	-13.43	-26.37	-12.26	12.84
7	51.30	45.42	20.5	-24.18
8	26.37	22.66	12.84	-12.26
9	45.42	51.3	-20.5	-24.18
10	22.66	26.37	-12.84	-12.26

CUMULATIVE DEFORMATION VALUES ARE

X-MOVEMENT	Y-MOVEMENT	Z-ROTATION
0	0	0
0.00907	7.02e-005	0.0026
0.0172	9.38e-005	0.00138
0	0	0
0.00913	4.94e-012	0.00181
0.0172	9.26e-012	0.00088
0	0	0
0.00907	-7.02e-005	0.0026
0.0172	-9.38e-005	0.00138

AT THE END OF STAGE 2
PLASTIC HINGE LOCATION ARE

MEM NO.	I-END	J-END
1	0	0
2	0	0
3	0	0
4	0	0
5	0	0
6	0	0
7	1	0
8	0	0
9	0	2
10	0	0

Stage 10

CUMULATIVE LOAD FACTOR IS = 1.55

CUMULATIVE FIXED END MOMENTS AND AXIAL FORCE

MEM NO.	MOMENT I-END	MOMENT J-END	TENSION	SHEAR FORCE
1	-97.26	-55.21	50.63	49.18
2	3.91	-51.3	24.98	15.29
3	-97.26	-69.72	4.578e-006	53.86
4	-32.88	-97.26	4.73e-006	41.98
5	-97.26	-55.21	-50.63	49.18
6	3.91	-51.3	-24.98	15.29
7	51.3	51.3	33.9	-25.65
8	51.3	48.63	15.29	-24.98
9	51.3	51.3	-33.9	-25.65
10	48.63	51.3	-15.29	-24.98

CUMULATIVE DEFORMATION VALUES

X-MOVEMENT	Y-MOVEMENT	Z-ROTATION
0	0	0
0.0294	9.75e-005	0.00907
0.0518	0.000146	0.00386
0	0	0
0.0295	8.81e-012	0.00819
0.0518	1.79e-011	0.00212
0	0	0
0.0294	-9.75e-005	0.00907
0.0518	-0.000146	0.00386

AT THE END OF STAGE 10

PLASTIC HINGE LOCATIONS

MEM NO.	I-END	J-END
1	6	0
2	0	0
3	5	0
4	0	10
5	7	0
6	0	0
7	1	3
8	8	0
9	4	2
10	0	9

The above results show the moment at i and j end, axial force and shear forces in each member at every stage of increment of loads until the collapse mechanism is formed. The results also give the cumulative deformation values at each joint of the frame. For two-storey two bay frame it is seen that the collapse mechanism will form at the end of stage 10.

While performing pushover analysis, pushover curve (capacity curve) is developed for the building. Capacity Curve is a plot of Base shear v/s Roof displacement. On completion of each stage the base shear and roof displacements are obtained which are plotted on (Y-axis) and (X-axis) respectively. The capacity curve is displayed by clicking on the "Capacity curve" on the "Curve" menu of the main menu bar.

Capacity spectrum is obtained by transforming Base shear (V) to spectral acceleration (Sa) and Roof displacement (d) to spectral displacement (Sd) as explained earlier. The capacity spectrum is displayed by clicking on the "Capacity Spectrum" on the "Curve" menu of the main menu bar (**Fig.4.51**).

Demand Spectra is obtained by transforming the coordinates of Response Spectra "Spectral Acceleration (Sa/g) and Time period (T)" to "Spectral Acceleration and Spectral Displacement" respectively. The values of conversion of capacity curve to capacity spectrum is tabulated in **Table 4.5**.

After completing pushover analysis, in order to know the expected seismic performance of the structure, the Capacity Spectrum Method is used. The Capacity spectrum is superimposed over the Demand Spectrum to get the intersection point which is the Performance Point. The Performance Point gives the estimated displacement demand on the structure for the specified level of seismic hazard.

Table 4. 5 Conversion of Capacity Curve to Capacity Spectrum

Shear Force (V),kN	Spectral Acceleration (Sa)kN	Roof Displacement (d) m	Spectral Displacement (Sd) (m)
0	0	0	0
88.32	0.33099934	0.0074	0.03685
97.17	0.364167386	0.02	0.00996
108.12	0.405205082	0.0275	0.13697
114.00	0.427241762	0.0325	0.16187
118.5	0.444106569	0.0486	0.24206

Figure 4.51 shows the capacity curve, which is developed using ordinates of base shear V and roof displacement whereas **Fig. 4.52** shows the capacity spectrum overlapped on demand spectra. Capacity spectrum is developed using the ordinates of Spectral acceleration Sa and Spectral Displacement Sd as given in **Table 4.6**.

As shown in **Fig. 4.52** the capacity spectra do not intersect with the demand spectra in this particular case, which indicates that the structure does not behave satisfactorily during the earthquake and that it needs to be retrofitted.

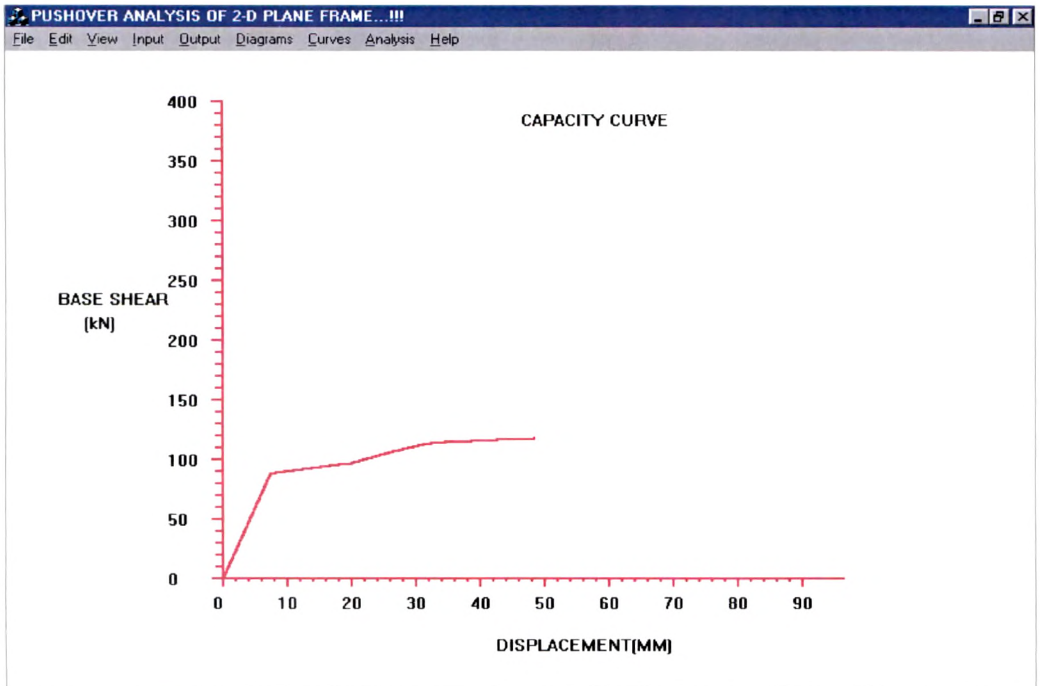


Fig. 4.51 Capacity Curve

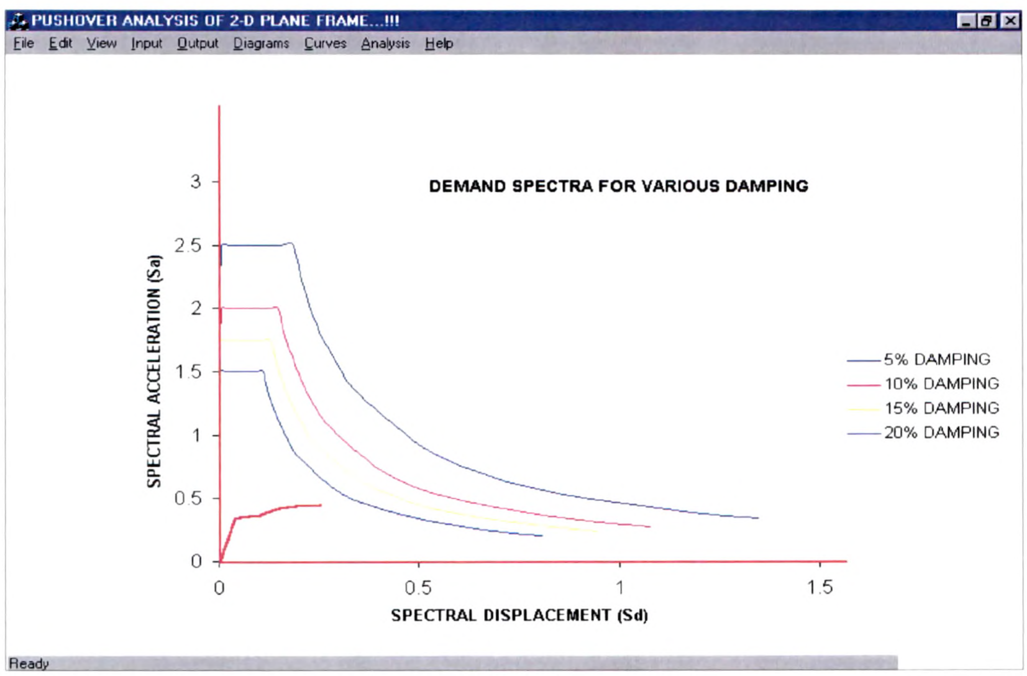


Fig. 4.52 Capacity Spectrum Overlapped on Demand Spectra

4.9 VERIFICATION OF RESULTS

Table 4.6 shows the comparison of moment (M3) obtained from SAP2000 and that obtained from the Pushover Module of SA-DVR. First column of the table gives the member numbers, second column gives the distances between i and j ends of the member. It can be seen that the results of the last stage of SA-DVR match substantially with those of SAP. Similarly **Table 4.7** compares the Base Shear vs. Roof Displacements obtained from both the software and are found to be comparable.

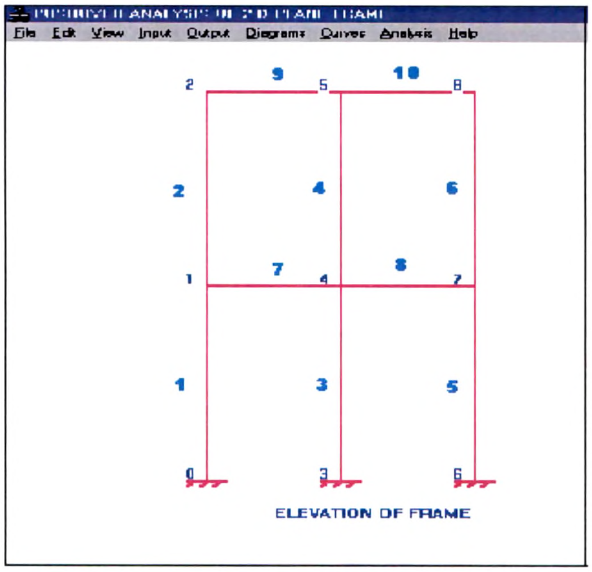


Fig. 4.53 G+1 Plane Frame

Table 4.6 Comparison of Results of Pushover in SAP and SA-DVR

Member No	Length m	SAP2000 M3 in kNm		SA-DVR M3 in kNm	
		i end	j end	i end	j end
1	3.1	-94.02	-62.82	-97.26	-55.21
2	3.1	4.36	-51.41	3.91	-51.31
3	3.1	-87.68	-77.07	-97.26	-69.72
4	3.1	-30.64	-93.14	-32.88	-97.26
5	3.1	-90.53	-62.01	-97.26	-55.21
6	3.1	17.23	-51.33	3.91	-51.32
7	4.0	51.39	51.40	51.32	51.32
8	4.0	51.39	42.46	51.32	48.63
9	4.0	51.39	51.41	51.32	51.32
10	4.0	50.77	51.33	48.63	51.32

Table 4.7 Coordinates of Pushover Curve

BASE SHEAR		DISPLACEMENT	
SAP	SA-DVR	SAP	SA-DVR
0	0	0	0
129.129	122.1	0.028189	0.03105
139.252	132.92	0.031185	0.03705
140.991	141.46	0.032045	0.04275
170.079	152.22	0.078469	0.0777

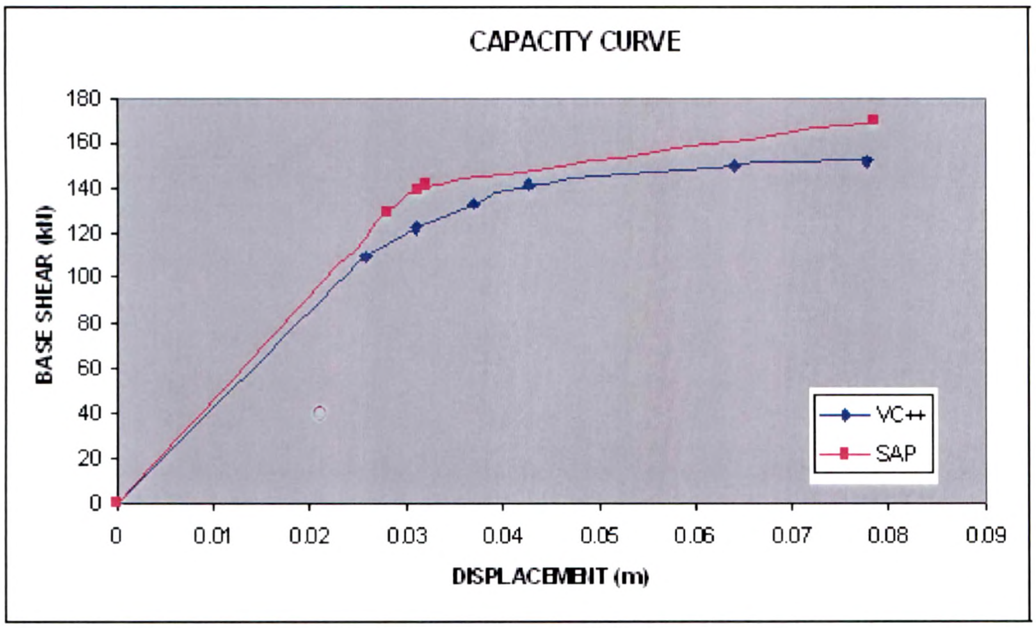


Fig. 4.54 Pushover Curve from SAP and SA-DVR

Fig.4.54 shows the comparison of the Capacity Curve developed in SA-DVR and SAP. A similar example for a 3 storey building was tested in the pushover module. **Table 4.8** shows the comparison of moment (M3) obtained from SAP and those obtained from SA-DVR

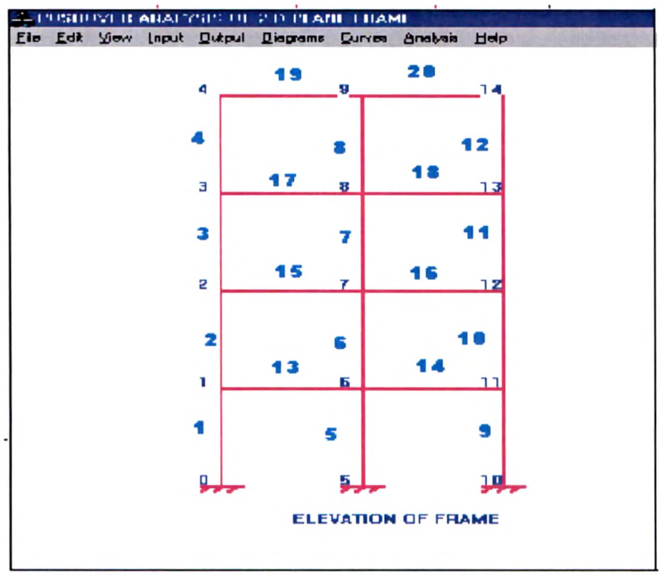


Fig. 4.55 G+3 Story Frame

Table 4.8 Comparison of Results of Pushover in SAP and SA-DVR

Member No	Length m	SAP M3 in kNm		SA-DVR M3 in kNm	
		j and	k end	j and	k end
1	3.1	-113.14	-25.87	-120.6	-20.08
2	3.1	-29.75	-63.06	-31.19	-73.88
3	3.1	4.68	-96.88	22.61	-95.62
4	3.1	14.69	-51.40	44.32	-51.27
5	3.1	-99.09	-39.46	-120.64	-38.52
6	3.1	-60.48	-90.26	-64.02	-103.21
7	3.1	22.58	-110.34	0.46	-114.13
8	3.1	7.88	-93.30	11.60	-102.54
9	3.1	-106.49	-29.02	-120.6	-20.08
10	3.1	-43.92	-70.53	-31.19	-73.88
11	3.1	15.44	-93.36	22.61	-95.59
12	3.1	46.92	-51.40	44.32	-51.27
13	3.1	51.38	51.40	51.27	51.27
14	3.1	51.39	51.40	51.27	51.27
15	3.1	51.39	51.40	51.27	51.27
16	3.1	51.39	47.79	51.27	51.27
17	3.1	51.39	51.39	51.27	51.27
18	3.1	51.28	51.58	51.27	51.27
19	3.1	50.45	50.45	51.27	51.27
20	3.1	45.50	51.40	51.27	51.27

Table 4.7 shows the coordinates of roof displacement and base shear obtained from SAP and SA-DVR for the G+3 story frame shown in **Fig. 4.55**. **Figure 4.56** shows the comparison of the Capacity Curve developed in SA-DVR and SAP.

Table 4.9 Coordinates of Pushover Curve

BASE SHEAR (kN)		DISPLACEMENT (m)	
SAP	SA-DVR	SAP	SA-DVR
0.00	0.00	0.000000	0.000000
9.62	8.16	0.002965	0.029800
90.00	89.90	0.030000	0.034100
100.00	91.23	0.039600	0.035000
115.00	116.87	0.059479	0.068700
127.16	122.77	0.088009	0.079600
137.43	124.86	0.122879	0.084700
137.36	129.18	0.122884	0.096700
140.71	139.93	0.137419	0.221000

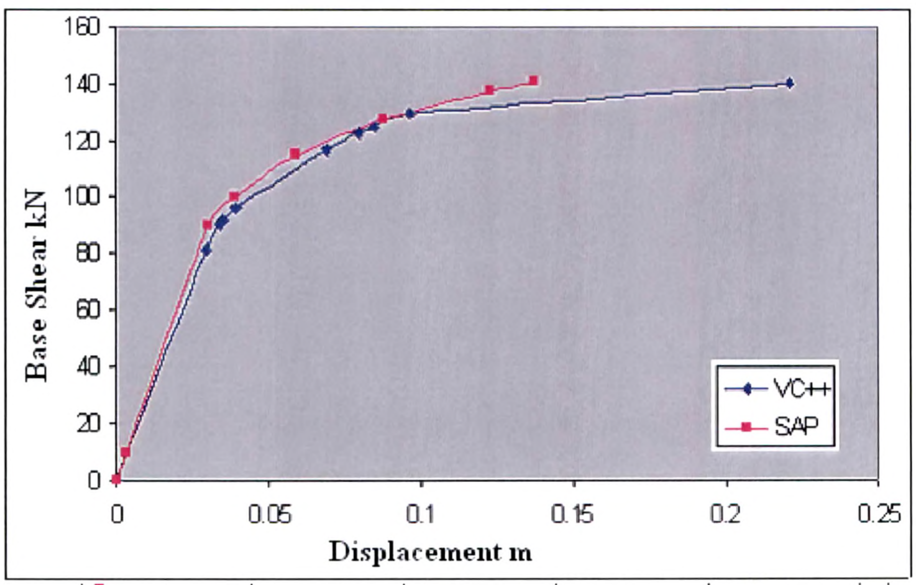


Fig. 4. 56 Comparison of Pushover Curve of SAP and SA-DVR

4.10 LIMITATIONS OF PUSHOVER ANALYSIS

The Pushover analysis treats non-linearity in a more explicit manner but the proposed procedure suffers from following fundamental deficiencies:

- The pushover analysis procedure implies that there is a separation between the structural capacity and earthquake demand.
- It is incorrect to assume that there exists a unique, intrinsic structural capacity irrespective of the earthquake demand. Nonlinear structural behavior is load path dependent, and it is not possible to separate the loading input from the structural responses.
- The pushover analysis procedure implicitly assumes that damage is a function only of the lateral deformation of the structure, neglecting duration effects and cumulative energy dissipation demand. It is generally accepted that damage of a structure is a function of both deformation and energy. The applicability of the proposed measure of damage is too simplistic, particularly for non-ductile structures whose inelastic cyclical behaviors are severely pinched and erratic.
- The pushover analysis is a static analysis, and neglects dynamics. During an earthquake, the behavior of a nonlinear yielding structure can be described by balancing the dynamic equilibrium at every time step. By focusing only on the strain energy of a structure during a monotonic static push, the procedure can leave a misleading impression that energy associated with the dynamic components of forces, i.e. kinetic energy and viscous damping energy, are insignificant.
- The pushover analysis procedure is overly simplifying. The procedure assumes that it is possible to "substructure" a nonlinear 3-D structure, and to characterize its behavior by two parameters, base shear and roof drift.
- Pushover analysis procedure does not take into account the progressive changes in the modal properties that take place in a structure as it experiences cyclic nonlinear yielding during an earthquake. It is difficult to discuss modal properties, which are linear, of a structure that experience significant non-linearity.
- Pushover analysis fails to produce good co-relation for earthquakes with predominantly impulsive ground motions.
- Retrofitting based on pushover analysis may be costly at the rate of 10-50% of cost of new construction, but nevertheless, the performance of the structure is to be given priority [81]
- The principal sources of uncertainties in Pushover results include:

- Definition of the ground motion including, intensity, duration, phasing, and frequency content
- Analysis of the distribution of deformations and stresses produced in the structure in response to the ground motion
- Knowledge of the actual configuration, strengths, deformations, and energy absorption and dissipation capacities of the structure in its as-constructed and maintained condition
- Determination of specific damage to structural and nonstructural components, in response to defined ground motions

4.11 CLOSING REMARKS

- For existing structures the reinforcement details should be carefully taken in order to get the hinge properties. Hinge properties and hinge locations play an important role in the final results of pushover analysis.
- As pushover analysis is performance based analysis, the results cannot be generalized and used on other structures. Hence they are structure specific.
- The particular case study undertaken shows that the structure is not able to meet the performance standards as the members are not designed to meet the specific earthquake demand. Hence major retrofitting may be undertaken and the building should be analysed post retrofitting to ensure improved performance under earthquake loads.
- The test case results match with SAP pushover analysis substantially. The small variation that is encountered may be due to the response spectrum generated as per IS 456 in SA-DVR while SAP follows FEMA guidelines.
- Some variation may also be attributed to the Hinge properties generated by SAP which are automatically calculated while SA-DVR gets these from Xtract software, embedded within the main module.
- Pushover analysis gives more accurate prediction of global displacement and inter-story drift as compared to linear methods of analysis. However the inherent limitations of the method make its employability limited to structures with few modes.