



Pushover Analysis of RCC Framed Structure

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ABSTRACT

From the effects of significant earthquakes (since the early 1980s) it is concluded that the seismic risks in urban areas are increasing and are far from socio-economically acceptable levels. There is an urgent need to reverse this situation and it is believed that one of the most effective ways of doing this is through: (1) the development of more reliable seismic standards and code provisions than those currently available and (2) their stringent implementation for the complete engineering of new engineering facilities. A performance-based design is aimed at controlling the structural damage based on precise estimations of proper response parameters. This is possible if more accurate analyses are carried out, including all potential important factors involved in the structural behavior.

With an emphasis on providing stakeholders, owner the information needed to make rational business or safety-related decisions, design practice has moved toward predictive methods for assessing potential seismic performance and has led to the development of performance based engineering methods for seismic design.

1. INTRODUCTION

1.1 General

Earthquakes are one of the greatest natural hazards which can make a uncountable damage to our earth. Earthquake is unpredictable hazard and we need a tool which can help us to come across with this hazard. Engineering can make a tool for the constructions which can sustain in these Earthquakes. Performance based design is gaining a new dimensions in the seismic design philosophy Where in the near field ground motion (usually acceleration) is to be considered. Earthquake loads are to be carefully modeled so as to be assess the real behavior of the structure with the clear understanding that damage is expected but it should be regulated. In this Project , pushover analysis which is iterative procedure shall be looked upon as an alternate for the orthodox analysis procedures. This study focused on pushover analysis of multistoried R.C building subjected them to monotonically increasing lateral forces with an invariant height wise distribution until the preset performance level is reached. The promise of performance-based seismic engineering (PBSE) is to produce structures with predictable seismic performance. Comprehensive effort by professionals from several disciplines is required to turn this promise into a reality. Earthquakes can create serious damage to structures & the damage to structures causes deaths, injuries, economic loss, and loss of functions. The structures already built are unprotected to future earthquakes. Earthquake risk is associated with seismic hazard, vulnerability of buildings, exposure. Seismic hazard quantifies the probable ground motion that can occur at site. Unpredictability of building is important is causing risk to life. India has witnessed Bhuj earthquake of Jan 26, 2001

(fig 1.1). Sumatra earthquake of December 26, 2004, Kashmir earthquake of October 8, 2005, Nepal earthquakes in beside country & disasters due to earthquakes in the current decades. Most of the casualties were due to collapse of the buildings and or due to lack of rescue and relief to the victims under the collapsed buildings .Earthquakes do not kill people but poorly by unprotected designs of constructions. These earthquakes have clearly shown us that we need to have a comprehensive strategy for disaster managements which should include planning, design and construction of earthquake resistance buildings through strict compliance of Codal provisions for earthquake countermeasures. As of today, it is unfortunate that despite of having all the scientific know-how in the field of earthquake engineering and the Codal provisions for construction in seismic areas, developed and brought out by the Bureau of Indian Standards, on account of ignorance or fear of added cost we continue to find utter neglect of our engineers and builders. Common people are not savvy to many aspects of earthquakes disaster and better building practices in seismic zones of the country.

2. LITERATURE REVIEW

Earthquake engineering is most sensible engineering field in the world. Performance based design approach is new concept and lot of research is going on the validating this approach. Number of scientist gone through this approach and explained the varieties of approaches it properties and limitations. Some of them are discussed below.

Peter Fajfar et al (2000) ^[1] presented a relatively simple nonlinear method for the seismic analysis of structures (the N2 method). It combines the pushover analysis of a multi-

degree-of-freedom (MDOF) model with the response spectrum analysis of an equivalent single-degree-of-freedom (SDOF) system. The method is formulated in the acceleration- displacement format, which enables the visual interpretation of the procedure and of the relations between the basic quantities controlling the seismic response. Inelastic spectra, rather than elastic spectra with equivalent damping and period, were applied. This feature represents the major difference with respect to the capacity spectrum method. Moreover, demand quantities can be obtained without iteration. Generally, the results of the N2 method are reasonably accurate, provided that the structure oscillates predominantly in the first mode. In the work, the method is described and discussed, and its basic derivatives are given. The similarities and differences between the proposed method and the FEMA 273 and ATC 40 nonlinear static analysis procedures are discussed. Application of the method is illustrated by means of an example.

J. B. Mander (2001)[10] reviewed from an historical perspective past and current developments in earthquake engineered structures. Based on the present state-of-the-practice in New Zealand, and a world-view of the state-of-the-art, he argued that in order to make progress towards the building of seismic resilient communities, research and development activities should focus on performance-based design which gives the engineer the ability to inform clients/owners of the expected degree of damage to enable a better management of seismic risk. To achieve expected performance outcomes it will be necessary to supplement, current force-based design standards with displacement-based design methodologies.

Improved design methodologies alone will not lead to a significantly superior level of seismic resilient communities, but rather lead to a superior standard of performance-based engineered structures where the post-earthquake outcome will be known with a certain degree of confidence. This paper gives two philosophical approaches that are referred to as Control and Repairability of Damage (CARD), and Damage Avoidance Design (DAD)

Qiang Xue, et al (2003)[14] presented a performance-based seismic design procedure, which is directly associated with pre-quantified performance criteria, by employing a displacement-based approach. A lower bound of yielding displacement of the structure to satisfy these performance criteria was proposed. This approach is general and applicable for any type of reduced response spectrum that taking into account of the inelastic behavior provided the spectrum reduction factor regarding each spectral region is given. The procedure can be extended to fulfill multiple performance objectives and to consider special effects such as the near-fault and accumulative damage. In the presented design procedure of the building, the k factor plays an important role in controlling the design strength. The simplicity and applicability of the proposed procedure is demonstrated through numerical examples. The proposed design procedure, which starts from the pre-quantified performance objectives, is transparent and straightforward to present the underlying concept of —performance based design. Non-linear time history analysis verified that this approach is applicable to control the target displacement to the performance acceptable limit. Its flexibility in considering special effect such as near-fault or strong

motion duration and simplicity in a proposed multiple performance objectives design are demonstrated.

Andreas J. Kappos et al (2004)^[1] proposed a performance-based design procedure for realistic 3D reinforced concrete (R/C) buildings, which involves the use of advanced analytical tools. The proposed method was then applied to a regular multistory reinforced concrete 3D frame building and was found to lead to better seismic performance than the standard code (Euro code 8) procedure, and in addition led to a more economic design of transverse reinforcement in the members that develop very little inelastic behavior even for very strong earthquakes.

The building was first designed to a standard code procedure, and then redesigned to the proposed method. Due to its high regularity, the building was designed using both versions of the method (based on either inelastic dynamic or inelastic static analysis). In addition, several alternative designs to the new method were carried out. All designs were subsequently assessed for a number of performance objectives, using both local and global criteria. A six-storey R/C, doubly symmetric structure (three 3 m spans in y -direction, three spans of 6,4,6 m in x -direction) was selected as a test of the proposed procedure. The building was first designed to the provisions of the current Greek Seismic Code, which is very similar to Euro code 8 (CEN, 1995) [9] ductility class Ml (medium), for a design ground acceleration of 0.25g, assuming class A soil conditions (stiff deposits). Earthquake loading was combined with gravity loading $G + 0.3 LL$. The materials used in the structure are C20/25 (characteristic cylinder strength of 20 MPa) concrete, and S400 steel (characteristic yield strength of 400 MPa). Square column crosssections (from 300 to 450 mm) were used, with reinforcement ratios not exceeding about 2% (the minimum reinforcement ratio for columns was 1%). Beam sections varied from 200×400 to 300×650 (mm²).

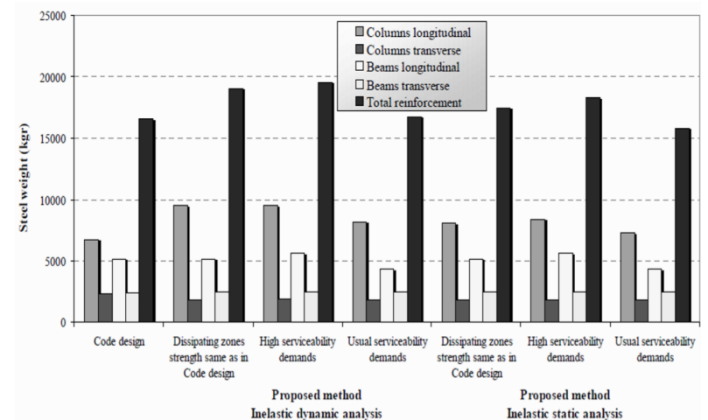


Figure 2.1 - Required amount of steel in beams and columns for all designs

Both elastic and inelastic (dynamic and static) analyses of the structure were carried out using SAP 2000 Nonlinear (Computers and Structures, 2000), adopting a member-by-member modeling approach. Inelastic beam (and column) members were modeled as elastic elements with inelastic springs (plastic hinges) at their ends; the effective rigidity of T-beams was taken equal to 40% the gross section rigidity (EI_g), while for columns 80% of EI_g was assumed. The moment curvature characteristics of the plastic hinges were estimated from section analysis using appropriate non-linear

constitutive laws for concrete and steel (Penelis and Kappos, 1997) ^[27]; member strength and ductility were estimated on the basis of the nonlinear section analysis results.

Vipul Prakash (2004) ^[9] gives the prospects for Performance Based Engineering (PBE) in India. He lists the pre-requisites that made the emergence of PBE possible in California, compares the situation in India and discusses the tasks and difficulties for implementing PBE in India.

In India, the criteria for earthquake resistant design of structures are given in IS 1893, published by the Bureau of Indian Standards (BIS). IS 1893-2002 reduced the number of seismic zones to four by merging zone I with zone II and adopted a modified CIS64 scale for seismic zoning and dropped references to the MMI scale. The mapping of zones to intensities in IS 1893-2002 is given in Table 2.1.

Table 2.1 - Mapping Seismic Zones to Intensities in IS 1893-2002

In IS 189320	02
Seismic Zone	Mapped to a Modified CIS-
II	VI and below
III	VII
IV	VIII
V	IX and above

In US, building performance levels are divided into structural performance levels (SP-1 to SP6) and nonstructural performance levels (NP-A to NP-E), and then a combination of structural and nonstructural performance levels is set as the performance objective to be met at a given level of earthquake. These combinations can be approximately mapped to the damage grades specified in EMS-98 as follows:

IS 1893- 2002 specifies two levels of earthquakes – Maximum Considered Earthquake (MCE) and Design Basis Earthquake (DBE). In Clause 6.1.3, it states the performance objective as follows: —The design approach adopted in this standard is to ensure that structures possess at least a minimum strength to withstand minor earthquakes (< DBE), which occur frequently, without damage; resist moderate earthquake (DBE) without significant structural damage though some nonstructural damage may occur; and aims that structures withstand a major earthquake (MCE) without collapse.

Table 2.2 Comparison of Damage Grades as per EMS-98 and Building Performance Levels

Damage Grade as per EMS-98	Approximate Building Performance Combination in PBE
Grade 1 (no structural damage, slight nonstructural damage)	SP-1 (immediate occupancy) + NP-A (operational) = 1-A (operational)
Grade 2 (slight structural damage, moderate nonstructural damage)	SP-1 (immediate occupancy) + NP-B (immediate occupancy) = 1-B (immediate occupancy)
Grade 3 (moderate structural damage, heavy on structural damage)	SP-3 (life safety) + NP-C (life safety) = 3-C (life safety)
Grade 4 (heavy structural damage, very heavy nonstructural damage)	SP-5 (structural stability) + NP-E (not considered) = 5-E (structural stability)
Grade 5 (very heavy structural damage)	SP-6 (not considered) + NP-E (not considered) = 6-E (not considered)

In PBE, merely stating a performance objective is not sufficient; it has to be followed up by analyses or a methodology for ensuring that the stated performance objectives will indeed be met by the evaluated structures. PBE thus requires much tighter language and cross-referencing to be used in the specifications.

X.-K. Zou et al (2005)[19] present an effective computer-based technique that incorporates pushover analysis together with numerical optimization procedures to automate the pushover drift performance design of reinforced concrete (RC) buildings. Performance-based design using nonlinear pushover analysis, is a highly iterative process needed to meet designer-specified and code

requirements. This paper presents an effective computer-based technique that incorporates pushover analysis together with numerical optimization procedures to automate the pushover drift performance design. Steel reinforcement, as compared with concrete materials, appears to be the more cost-effective material that can be effectively used to control drift beyond the occurrence of first yielding and to provide the required ductility of RC building frameworks.

In this study, steel reinforcement ratios are taken as design variables during the design optimization process. Using the principle of virtual work, the nonlinear inelastic seismic drift responses generated by the pushover analysis can be

explicitly expressed in terms of element design variables. An optimality criteria technique is presented in this paper for solving the explicit performance-based seismic design optimization problem for RC buildings. Two building frame examples are presented to illustrate the effectiveness and practicality of the proposed optimal design method.

The design optimization procedure for limiting performance-based seismic drifts of an RC building structure is listed as follows:

1. Establish an initial design with optimal member dimensions, which can be obtained from the elastic seismic design optimization by minimizing the concrete cost of an RC structure subjected to a minor earthquake loading using the elastic response spectrum analysis method.
2. Determine the design spectra, corresponding to different earthquake demand levels, which will be used in the nonlinear pushover analysis.
3. Conduct a static virtual load analysis to obtain the member internal forces that will be used in formulating inelastic drift responses by employing the principle of virtual work.
4. On the basis of the optimal member size, determine the minimum and maximum size bounds of the steel reinforcement ratios, ρ and ρ' , in accordance with the strength-based code requirements.

R. K. Goel and A. K. Chopra (2006)^[5] presented an improved Direct Displacement Based Design Procedure for Performance-Based seismic design of structures. Direct displacement-based design requires a simplified procedure to estimate the seismic deformation of an inelastic SDF system, representing the first (elastic) mode of vibration of the structure. This step is usually accomplished by analysis of an equivalent linear system using elastic design spectra. In their work, an equally simple procedure is developed that is based on the well-known concepts of inelastic design spectra. This procedure provides: (1) accurate values of displacement and ductility demands, and (2) a structural design that satisfies the design criteria for allowable plastic rotation. In contrast, the existing procedure using elastic design spectra for equivalent linear systems is shown to underestimate significantly the displacement and ductility demands.

In this work, it is demonstrated that the deformation and ductility factor that are estimated in designing the structure by this procedure are much smaller than the deformation and ductility demands determined by nonlinear analysis of the system using inelastic design spectra. Furthermore, it has been shown that the plastic rotation demand on structures designed by this procedure may exceed the acceptable value of the plastic rotation.

Qiang Xue, Chia-Wei Wu et al (2007)^[15] summarized the development of the seismic design draft code for buildings in Taiwan using performance-based seismic design methodology and case studied following the guidelines in the paper. They presented the design of a reinforced concrete building by using the draft code.

In their study first, the current seismic design code provisions are examined according to the theoretical basis of PBSD to identify which methodologies of PBSD need to be incorporated into the current seismic design code. Then, a PBSD flowchart is presented. Finally, a draft of the

proposed code is described. Transparent seismic design objectives for buildings of different use groups have been established qualitatively and interpreted quantitatively as performance criteria including drift limits. Site feasibility requirements, conceptual design scopes and basic rules have been proposed. Performance objective-oriented procedures for preliminary design and seismic performance evaluation have been presented. Suggestions on seismic performance criteria and the evaluation of existing buildings have been made. In order to provide clear, easy to follow guidelines, comparisons and case studies have also been conducted.

The performance-based seismic design code introduces a transparent platform in which the owners and designers can exchange their views on the expected seismic performance of the buildings under different levels of earthquakes. For buildings of different seismic use groups, specific performance goals are established without employing an importance factor. Performance levels are quantified through parameters associated with structural strength, stiffness and ductility. Conceptual design rules with focuses on redundancy and uniform continuity of strength, stiffness and ductility are specified. A performance objective-oriented preliminary design procedure is presented with consideration of flexibility. Preliminary checks on the interstory drift limit may help in finding the stiffness deficiencies earlier in the preliminary design stage and save some computational effort, particularly for steel structures.

The differences between seismic performance assessments of new buildings and those of existing buildings are pointed out. In engineering practice, member size standards and construction convenience are usually considered. A structure designed in this way usually has a lower ductility capacity than that specified in the code because structural ductility is not uniformly distributed. However, the structural strength and stiffness are usually higher than the demand. Therefore, the prescribed performance objective is usually satisfied.

According to the case study, if the same column size has been adopted for the first several floors, a higher reinforcement ratio assigned to the first 2 stories is helpful for uniform distribution of system ductility. Adopting the performance criteria in the draft code, direct displacement-based design procedures have been applied successfully for moment resisting frames without iteration. The performance criteria associated with stiffness or displacement as suggested in the draft code should not be used either as optimized design criteria or in a direct displacement-based design procedure for structural systems other than moment resisting frames.

In this draft code, the design of nonstructural components is done to accommodate either acceleration or displacement. No specific criterion regarding economic loss is provided. The nonstructural damage is limited by the structural drift limit.

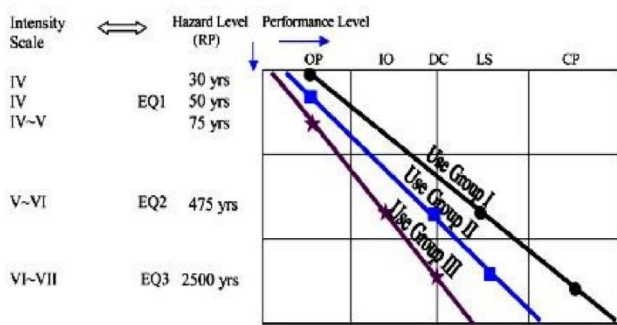


Figure 2.2 - Performance Objectives

As shown in Fig. 2.2, three seismic hazard levels were considered and can be distinguished by return period, probability of exceedance, or corresponding site intensity scale. Performance of a building has been classified into 5 levels, Operational (OP), Immediate Occupancy (IO), Damage Control (DC), Life Safety (LS) and Collapse Prevention (CP).

Whittaker, Y. N. Huang et al (2007) [18] summarize the next (second) generation tools and procedures for performance-based earthquake engineering in the United States. The methodology, which is described in detail in the draft Guidelines for the Seismic Performance Assessment of Buildings, builds on the first generation deterministic procedures, which were developed in the ATC-33 project in the mid 1990s and in ASCE Standard: ASCE/SEI 41-06 Seismic

Nilesh M. Kashid (2011) [12] Says that, During the last few decades, earthquake engineering has undergone significant development. Initially, structures were designed without considering seismic loading. Later, it was observed that the structures designed for some lateral loads like wind etc. performed significantly well than those designed for gravity loading only. Hence, the importance of considering earthquake forces in the design process is realized and seismic resistant design became a practice. Further with the advent of time history analysis and better understanding of seismic response. The importance of ductility of structure is realized in resisting even higher seismic loads than the designed loads. The design base shear force is reduced by introducing a response reduction factor to consider an inelastic displacement capacity of ductile structure in dissipating the energy. Further, with more understanding of structural behaviour at micro-level or element level, the concept of "capacity design" was introduced and this forced to decide the required performance of the structure right at the design stage itself. Today, the seismic design codes of various countries are being revised to decide performance criteria of the buildings that will suit the existing design and construction practices in the respective countries. In this paper, an attempt has been made to develop a possible method of seismic analysis that can be incorporated in the existing Indian Seismic Code I.S.: 1893- 2002. For the purpose, some of the key features the existing methodology followed by Federal Emergency Management Agency (FEMA) of United States of America is made use of.

Sudhanshu Sood (2011) [16] mentioned his paper that, A performance-based design is aimed at controlling the structural damage based on precise estimations of proper response parameters. Performance-based seismic design

explicitly evaluates how a building is likely to perform; given the potential hazard it is likely to experience, considering uncertainties inherent in the quantification of potential hazard and uncertainties in assessment of the actual building response. It is an iterative process that begins with the selection of performance objectives, followed by the development of a preliminary design, an assessment as to whether or not the design meets the performance objectives, and finally redesign and reassessment, if required, until the desired performance level is achieved. In this present study two R.C buildings, one symmetrical and one unsymmetrical in plan (designed according to IS 456:2000) are analysed using Pushover Analysis and redesigning by changing the main reinforcement of various frame elements and again analyzing. The pushover analysis has been carried out using SAP2000, a product of Computers and Structures International. A total of 24 cases for a particular four storey building located in ZoneIV have been analyzed, changing reinforcement of different structural elements,

3. METHODOLOGY

3.1 Introduction

Force Based design is a traditional approach to Seismic Design of a Building. Using the Response Spectrum the design lateral forces on the Building are determined & the members are designed to withstand these forces. In this approach, there is no measure of the deformation capability of a member or of the building. At best, an elastic drift is computed under the design forces and checked against an elastic drift limit. Alternatively, an inelastic drift is estimated from the calculated elastic drift by multiplying the later by a factor and checking the inelastic drift against an inelastic drift limit.

In performance based analysis the deformations of the members and the building as a whole are quantified under the lateral forces of an earthquake of a certain level of seismic hazard. The deformations or strains are better quantities to assess damage than stresses or forces. A performance based analysis requires a nonlinear lateral load versus deformation curve as the deformations are expected to go beyond the elastic curve. The performance based analysis gives the analyst more choices of "PERFORMANCE" of the building as compared to the limit states of collapse and serviceability in a design based on limit state method.

3.2 Performance-Based Seismic Design Process

Performance-based design is an iterative process that begins with the selection of performance objectives, followed by the development of a preliminary design, an assessment as to whether or not the design meets the performance objectives, and finally redesign and reassessment, if required, until the desired performance level is achieved.

3.3 Select Performance Objectives

The process begins with the selection of design criteria stated in the form of one or more performance objectives. Performance objectives are statements of the acceptable risk of incurring different levels of damage and the consequential losses that occur as a result of this damage, at a specified level of seismic hazard. Since losses can be associated with structural damage, nonstructural damage, or

both, performance objectives must be expressed considering the potential performance of

3.3.1 Performance Levels

Building performance is a combination of the performance of the structure, the nonstructural elements and systems and the contents. Although structural and non – structural performance may be affected by similar seismic response parameters, it may be convenient to consider the two separately. This approach allows for selection of different performance levels for structural and non-structural systems. A common practice in seismic rehabilitation might be to select life safety performance level for the structural system and a hazard mitigated approach involving anchoring heavy items for the non structural components [FEMA 273:1997]. Although different performance levels may be selected for structural and non-structural systems, in general the approach to evaluating acceptability is nonetheless the same.

The decision to include or exclude performance of the building contents may be influenced by the significance of the hazard passed by the contents or whether the structural engineer has control over how contents are introduced and used in a building. Except for specialized facilities or contents, the structural engineer's responsibility normally does not include contents.

A wide range of performance can be targeted in building design, ranging from damage onset to collapse. Performance terminology usually is selected so that owners, risk managers, or users without a structural engineering background can readily understand it. As a result, the terminology itself may be somewhat ambiguous in engineering terms. Unambiguous definition of the performance levels or states must accompany the terminology.

A series of performance definition has been selected for this document. The series ranges from what is being defined as damage onset through collapse. These performance levels are presently roughly in order of increasing damage and decreasing function, as illustrated in figure 3.2. The performance levels are shown in the ranges likely for most buildings. Precisely where the performance level falls for a specific building will depend on the building configuration, material and details. Precise definition, rather than ranges, will be needed in the code implementation.

3.3.2 Building Performance Levels

3.3.2.1 Operational Level

This Building Performance Level is a combination of the Structural Immediate Occupancy Level and the Nonstructural Operational Level. Buildings meeting this performance level are expected to sustain minimal or no damage to their structural and nonstructural components. The building is suitable for its normal occupancy and use, although possibly in a slightly impaired mode, with power, water, and other

Figure 3.2: Capacity Curve

required utilities provided from emergency sources, and possibly with some nonessential systems not functioning. Buildings meeting this performance level pose an extremely low risk to life safety. Under very low levels of earthquake ground motion, most buildings should be able to meet or exceed this performance level. Typically, however, it will

not be economically practical to design for this performance under severe levels of ground shaking, except for buildings that house essential services.

3.3.2.2 Immediate Occupancy Level

This Building Performance Level is a combination of the Structural and Nonstructural Immediate Occupancy levels. Buildings meeting this performance level are expected to sustain minimal or no damage to their structural elements and only minor damage to their nonstructural components. While it would be safe to reoccupy a building meeting this performance level immediately following a major earthquake, nonstructural systems may not function due to either a lack of electrical power or internal damage to equipment. Therefore, although immediate reoccupancy of the building is possible, it may be necessary to perform some cleanup and repair, and await the restoration of utility service, before the building could function in a normal mode. The risk to life safety at this performance level is very low. Many building owners may wish to achieve this level of performance when the building is subjected to moderate levels of earthquake ground motion. In addition, some owners may desire such performance for very important buildings, under severe levels of earthquake ground shaking. This level provides most of the protection obtained under the Operational Level, without the cost of providing standby utilities and performing rigorous seismic qualification of equipment performance.

3.3.2.3 Life Safety Level

This Building Performance Level is a combination of the Structural and Nonstructural Life Safety levels. Buildings meeting this level may experience extensive damage to structural and nonstructural components. Repairs may be required before reoccupancy of the building occurs, and repair may be deemed economically impractical. The risk to life in buildings meeting this performance level is low. Many building owners will desire to meet this performance level for a severe level of ground shaking.

Table 3.1 - Details of Performance level

Performance Level	Structural Performance	Non Structural Performance
Operational (O)	<ol style="list-style-type: none"> 1. Very light damage 2. No permanent drift 3. Substantially original strength and stiffness 	<ol style="list-style-type: none"> 1. Negligible damage. 2. Power & other utilities are available.
Immediate Occupancy (IO)	<ol style="list-style-type: none"> 1. Light damage 2. No permanent drift 3. Substantially original strength & stiffness Minor cracking 4. Elevators can be restarted 5. Fire protection operable 	<ol style="list-style-type: none"> 1. Equipments & content secure but may not operate due to mechanical/utility failure.
Life Safety (LS)	<ol style="list-style-type: none"> 1. Moderate damage 2. Some permanent drift Residual strength & stiffness in all stories 3. Gravity elements function 4. Building may be beyond economical repair 	<ol style="list-style-type: none"> 1. Falling hazard mitigated but extensive systems damage.
Collapse Prevention (CP)	<ol style="list-style-type: none"> 1. Severe damage 2. Large permanent drifts 3. Little residual strength & stiffness 4. Gravity elements function Some exits blocked 5. Building near collapse 	<ol style="list-style-type: none"> 1. Extensive damage.

3.3.2.4 Collapse Prevention Level

This Building Performance Level consists of the Structural Collapse Prevention Level with no consideration of nonstructural vulnerabilities. Buildings meeting this performance level may pose a significant hazard to life safety resulting from failure of nonstructural components. However, because the building itself does not collapse, gross loss of life should be avoided. Many buildings meeting this level will be complete economic losses.

3.4 Seismic Hazard

For any type of solution details estimation of problem is assessed, so before design the seismic hazard is assessed. The most important seismic hazard to the majority of the building is earthquake ground shaking. The hazard posed by shaking includes both the motions imparted to the isolated structure as well as the hazard from adjacent building due to pounding, shared elements, or falling debris. Other hazard that may be considered in special cases include ground failure (liquefaction and lateral spreading, land sliding, differential settlement, or surface fault rupture); loading, flooding, or other water damage due to tsunamis; or collateral hazard such as fire or hazardous chemical release. These mentioned hazards are not usually of direct interest in displacement based design, although in special cases the effects of ground movement will be considered.

3.4.1 Seismic hazard levels:

Seismic hazard level should be selected considering the building function, its targeted performance, and its expected service life. Seismic hazard level can be stated in terms of probability that those levels will be exceeded over an established time. A common reference time is 50 years, which relates conveniently to both the service of a

conventional building structure as well as the adult life of a typical occupant or investor, and hence relates both to property risk and life risk.

Some performance objectives aimed at providing enhanced performance such as continued occupancy will consider a seismic hazard defined, for example by 10% reliability of design for life safety has considered a 10% probability of exceedance level in 50 years, or a return period of approximately 475 years.

Four levels of earthquake hazard are defined for the performance objectives. EQ-I represents a frequent event and is defined as the earthquake that has an 87% probability of being exceeded in a 50 year period (annual probability of exceedance of 4% or mean recurrence interval of approximately 25 years). EQ-II has a mean recurrence interval of 72 years, EQ-III has a recurrence interval between 250 and 800 years, and EQ-IV has a recurrence interval between 800 and 2500 years.

3.4.2 Combined Performance level and Seismic Hazard Level

Three standard performance objectives are defined as shown in Figure 3.3. Basic Safety Objective (BSO) is recommended for PBSE of standard occupancy structures and requires level 1 performance in EQ-I, level 2 performances in EQ-II, level 3 performance in EQ-III and level 4 performance in EQ-IV. Enhanced Objective 1 (EQI) is a higher objective than BSO and requires level 1 performance in EQ-II, level 2 performance in EQ-III and level 3 performance in EQ-IV. EQ-IV is considered for safety critical facilities and includes level 1 performance in the EQ-I, EQ-II and EQ-III and level 2 performance for EQ-IV.

As seen from Figure (1.1) a performance objective is a combination of a hazard level and the expected performance under that hazard. The hazard level is determined by a probabilistic hazard analysis (PSHA) and expressed in terms of the annual frequency of exceedance or return period. The SEAOC Blue Book and SEAOC Vision 2000 report (1995) define the performance levels in both qualitative and quantitative terms. The quantitative levels are defined through limiting values of measurable response parameters, such as story drifts and ductility demands. The DBD method presented here uses hazard estimates represented by Uniform Hazard Spectra given by codes, for example NBCC 2005, and quantitative displacement, drift and ductility measures to satisfy the desired performance objectives.

3.5 Push Over Analysis

Pushover Analysis is a nonlinear, static procedure in which the lateral loads' magnitude is incrementally increased, maintaining a predefined distribution pattern along the height of the building. Weak links and failure modes of the buildings are found with the help of the increase in the magnitude of the loads.

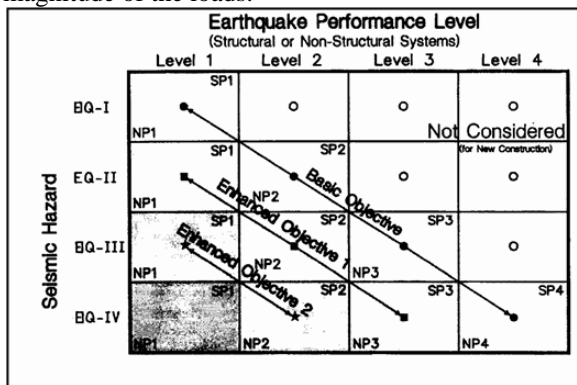


Figure 3.3 - Standard Performance Objectives (SEAOC Blue Book)

In Pushover analysis, a static horizontal force profile, usually proportional to the design force profiles specified in the codes, is applied to the structure. The force profile is then incremented in small steps and the structure is analyzed at each step. As the loads are increased, the building undergoes yielding at a few locations. Every time such yielding takes place, the structural properties are modified approximately to reflect the yielding. The analysis is continued till the structure collapses, or the building reaches certain level of lateral displacement.

3.6 Need For Pushover Analysis

Conventionally, seismic assessment and design has relied on linear or equivalent linear (with reduced stiffness) analysis of structural systems. In this approach, simple models are used for various components of the structure, which is subjected to seismic forces evaluated from elastic or design spectra, and reduced by force reduction (or behavior) factors. The ensuing displacements are amplified to account for the reduction of applied forces. This procedure, though simple and easy to apply in the design office environment, suffers from the following shortcomings:

1. The force reduction factors recommended in codes of practice are approximate and do not necessarily represent the specific structure under consideration.

2. When critical zones of a structure enter into the inelastic range, the force and deformation distribution change significantly. This change is not represented by a global reduction of forces.
3. The mechanism that will most likely perpetuate collapse is unlikely to be that represented by the elastic action and deformation distribution.
4. The global and particularly the local distribution of deformations in the inelastic range may bear no resemblance to those in the elastic range. The same applies to the values of deformations, not just the distribution. As a consequence of the above, the reduced forces - amplified deformations linear elastic approach fails to fit within the principle of failure mode control, which is part of performance based assessment and design. This in turn has led to an increase in the use of inelastic analysis as a more realistic means of assessing the deformational state in structures subjected to strong ground motion.

The pushover analysis is a significant step forward by giving consideration to those inelastic response characteristics that will distinguish between good and bad performance in severe earthquakes. The non-linear static pushover analysis is a partial and relatively simple intermediate solution to the complex problem of predicting force and deformation demands imposed on a structure and its elements due to ground motion.

Here, the important terms are static and analysis. Static implies that a static method is being employed to represent a dynamic phenomenon; a representation that is adequate in many cases but doomed to failure in some cases. Analysis implies that a system solution has been created already and the pushover is employed to evaluate the solution and modify it as needed.

The pushover is a part of an evaluation process and provides estimates of demands imposed on structures and elements. Hence, there is always a need of a method which is more rational and accurate and at the same time able to identify seismic deficiencies correctly and that too in correct order of vulnerability. Pushover analysis is able to satisfy these criteria satisfactorily and in a convenient way.

3.7 Description Of Pushover Analysis

The non-linear static pushover procedure was originally formulated and suggested by two agencies namely, federal emergency management agency (FEMA) and applied technical council (ATC), under their seismic rehabilitation programs and guidelines. This is included in the documents FEMA-273, FEMA-356 and ATC40.3.4.2.1 Introduction to FEMA-273. The primary purpose of FEMA-273 document is to provide technically sound and nationally acceptable guidelines for the seismic rehabilitation of buildings. The Guidelines for the Seismic Rehabilitation of Buildings are intended to serve as a ready tool for design professionals for carrying out the design and analysis of buildings, a reference document for building regulatory officials, and a foundation for the future development and implementation of building code provisions and standards.

3.8 Capacity Spectrum Method

One of the methods used to determine the performance point is the Capacity Spectrum Method, also known as the

Acceleration-Displacement Response Spectra method (ADRS). The Capacity Spectrum method requires that both the capacity curve and the demand curve be represented in response spectral ordinates.

3.8.1 Capacity Spectrum, Demand Spectrum, Performance Point

As graphically presented in Figure 3.6, the nonlinear static analysis procedure requires determination of three primary elements: capacity, demand and performance. The capacity spectrum can be obtained through the pushover analysis, which is generally produced based on the first mode response of the structure assuming that the fundamental mode of vibration is the predominant response of the structure. This pushover capacity curve approximates how a structure behaves beyond the elastic limit under seismic loadings.

The demand spectrum curve is normally estimated by reducing the standard elastic 5% damped design spectrum by the spectral reduction method. The intersection of the pushover capacity and demand spectrum curves defines the “performance point” as shown in Figure 3.6. At the performance point, the resulting responses of the building should then be checked using certain acceptability criteria. The responses can be checked against acceptability limits on both global system levels (such as the lateral load stability and the inter-story drift) and local element levels (such as the element strength and the sectional plastic rotation). When the responses of a structure do not meet the targeted performance level, the structure needs to be resized and the design process repeated until a solution for the desired performance level is reached. In general, the determination of the satisfactory performance response that fulfill both the system level response and element level response requires a highly iterative trial-and-error design procedure even with the aid of today’s engineering computer software.

3.8.2 Conversion of Pushover curve to Capacity Spectrum Curve

To convert a spectrum from the standard S_a (Spectra Acceleration) vs T (Period) format found in the building codes to ADRS format, it is necessary to determine the value of S_{d_i} (Spectral Displacement) for each point on the curve, $S_{a_i} T_i$ This can be done with the equation:

$$S_{d_i} = \frac{T_i^2}{4\pi^2} S_{a_i g}$$

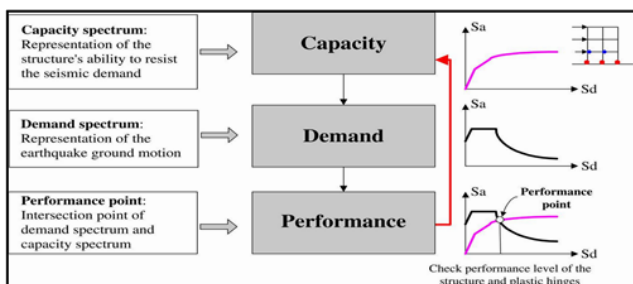


Fig 3.6 - Nonlinear Static analysis procedure

Standard demand response spectra contain a range of constant spectral acceleration and a second range of constant spectral velocity; S_v . Spectral acceleration, S_a and displacement at period T_i are given by:

$$S_{a_i g} = \frac{2\pi T_i S_v}{T_i} = \frac{2\pi S_v}{T_i}$$

The capacity spectrum can be developed from the pushover curve by a point by point conversion to the first mode spectral coordinates. Any point V_i (Base Shear), δ_i (Roof Displacement) on the capacity (pushover) curve is converted to the corresponding point S_{a_i} , S_{d_i} on the capacity spectrum using the equations:

$$S_{d_i} = \frac{V_i/W}{\alpha_1} \delta_i = \frac{PF_1 * \Phi_{1,ROOF}}{\alpha_1} \delta_i$$

Where α_1 and PF_1 , are the modal mass coefficients and participation factors for the first natural mode of the structure respectively. $\Phi_{1,roof}$ is the roof level amplitude of the first mode.

The modal participation factors and modal coefficient are calculated as:

$$PF_1 = \frac{\sum_{i=1}^n (W_i \phi_{i1}) / g}{\sum_{i=1}^n (W_i \phi_{i1}^2) / g}$$

$$\alpha_1 = \frac{[\sum_{i=1}^n (W_1 \phi_{i1}) / g]^2}{[\sum_{i=1}^n (\frac{W_i}{g})][\sum_{i=1}^n \frac{W_1 \phi_{i1}^2}{g}]}$$

Where w_i is the weight at any level i .

As displacement increase, the period of the structure lengthens. This is reflected directly in the capacity spectrum. Inelastic displacements increase damping and reduce demand. The Capacity Spectrum Method reduces the demand to find an intersection with the capacity spectrum, where the displacement is consistent with the implied damping. Figure 3.7 shows the conversion of Pushover curve to capacity spectrum curve. The damping that occurs when the structure is pushed into the inelastic range can be viewed as a combination of viscous and hysteretic damping. Hysteretic damping can be represented as equivalent viscous damping. Thus, the total effective damping can be estimated as:

$$\beta_{eff} = \lambda \beta_0 + 0.05$$

Where β_0 is the hysteretic damping and 0.05 is the assumed 5% viscous damping inherent in the structure. The λ -factor

(called κ -factor in ATC-40) is a modification factor to account for the extent to which the actual building hysteresis is well represented by the bilinear representation of the capacity spectrum (See Table 3.7 & 3.8 and Figure 3.10).

The term β_0 can be calculated using:

$$\beta_0 = \frac{1}{4\pi} \frac{E_D}{E_{SO}}$$

Where E_D is the energy dissipated by damping and E_{SO} is the maximum strain energy. The physical significance is explained in Fig. 3.8.

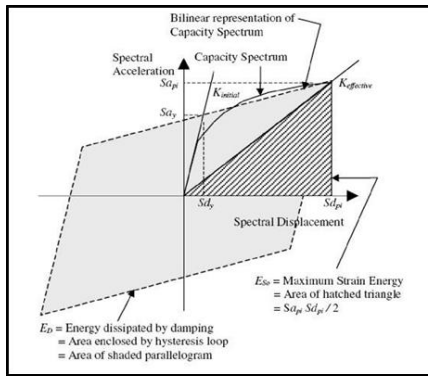


Figure 3.8 - Derivation of Energy dissipated by Damping

Table 3.2 - Structural Behaviour Types

Shaking Duration	Essentially new Building	Average Existing Building	Poor Existing Building
Short	Type A	Type B	Type C
Long	Type B	Type C	Type C

Table 3.3 - Values for damping modification values, λ

Structural Type	Behaviour	β_0	Λ
Type A		≤ 16.25	1
		≥ 16.25	1.13 - 0.51
Type B		≤ 25	0.67
		≥ 25	0.845 - 0.446
Type C		Any Value	0.33

To account for the damping, the response spectrum is reduced by reduction factors SR_A and SR_V which are given by:

$$SR_A = \frac{1}{B_\delta} = \frac{3.21 - 0.68 \ln(\beta_{eff})}{2.12}$$

$$SR_V = \frac{1}{B_l} = \frac{2.31 - 0.41 \ln(\beta_{eff})}{1.65}$$

Both SR_A and SR_V must be greater than or equal to allowable values. The elastic response spectrum (5% damped) is thus reduced to a response spectrum with damping values greater than 5% critically damped (See Figure 3.9).

3.8.2.1 Determination of Performance Point

There are three procedures described in ATC-40 to find the performance point. The most transparent and most convenient method for programming is

Procedure A, which uses a set of equations described in ATC-40.

Procedure B is also an iterative method to find the performance point, which uses the assumption that the yield point and the post yield slope of the bilinear representation, remains constant. This is adequate for most cases; however, in some cases this assumption may not be valid.

Procedure C is graphical method that is convenient for hand as well as software analysis. SAP2000 uses this method for the determination of performance point. To find the performance point using Procedure C the following steps are used:

First of all, the single demand spectrum (variable damping) curve is constructed by doing the following for each point on the Pushover Curve:

1. Draw a radial line through a point on the Pushover curve. This is a line of constant period.
2. Calculate the damping associated with the point on the curve, based on the area under the curve upto that point.
3. Construct the demand spectrum, plotting it for the same damping level as associated with the point on the pushover curve.
4. The intersection point for the radial line and associated demand spectrum represents a point on the Single Demand Spectrum (Variable Damping Curve).
5. A number of arbitrary points are taken on the Pushover curve and such points are obtained.
6. A curve is then drawn by joining through these points. The intersection of this curve with the original pushover curve gives the Performance Point of the Structure as shown in fig. 3.10.

3.9 Important Terms

1. Displacement Ductility

The ability of a structure or member to undergo inelastic deformations beyond the initial yield deformation with no decrease in the load resistance.

$$\mu = \frac{\Delta m}{\Delta y}$$

Figure 3.10 Capacity Spectrum Procedure C to Determine Performance Point 2. Inelastic displacement demand

Expressed in terms of Inelastic Displacement demand Ratio (IDDR). IDDR represents the ratio of inelastic displacement demand over the ultimate inelastic displacement capacity.

$$IDDR = \frac{\text{Inelastic Displacement Demand}}{\text{Ultimate Inelastic Displacement Capacity}}$$

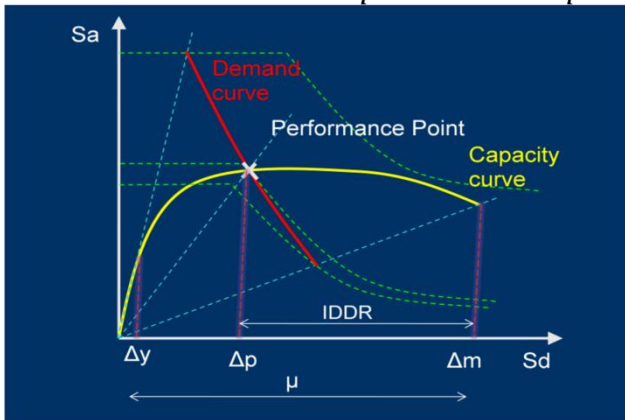


Figure 3.11 - Ductility and IDDR

Where,

- Δy = yield deformation
- Δp = displacement at PP
- Δm = maximum displacement.

$$\mu = \frac{\Delta m}{\Delta y}$$

$$IDDR = \frac{\Delta p}{\Delta m}$$

3. Plastic Rotations

The sequence of plastic hinge formation and state of hinge at various levels of building performance can be obtained from SAP output. This gives the information about the weakest member

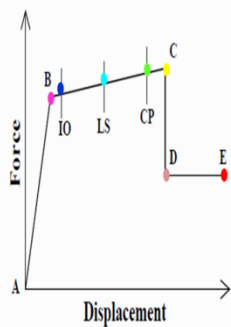
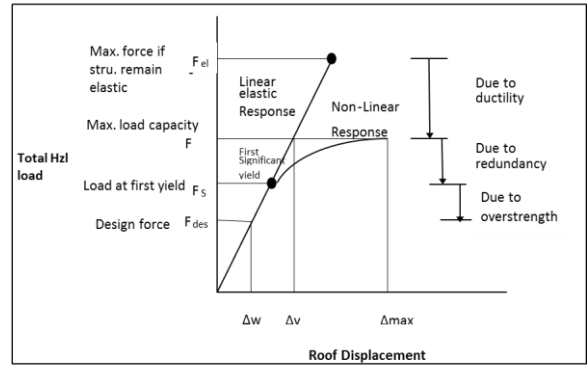


Figure 3.12 - Determination of Performance Point (FEMA)

4. Response Reduction Factor:



Allowable Limits:

FEMA – 356 has specified allowable limits for IDR, Plastic rotations, and IDDR values.

Table 3.4 - Inter-storey drift ratio (IDR)

Structural System	OP	IO	DC	LS	CP
Masonry Shear wall system	0.005	0.007	0.007	0.007	0.009
Others	0.005	0.010	0.015	0.020	0.025

Table 3.5 - Plastic rotations (radian)

Structural System	IO	LS	CP
Beams	0.005	0.02	0.025
Columns	0.005	0.01	0.02

Table 3.6 - Inelastic displacement demand (IDDR)

Performance Level	OP	IO	DC	LS	CP
IDDR	0.0	0.2	0.4	0.6	0.8

This limit helps as a guide lines for the validation of model results. The results within the limits given above should be accepted.

4 -SAP2000 MODELLING AND ANALYSIS

4.1 General

The main objective of performance based seismic design of buildings is to avoid total catastrophic damage and to restrict the structural damages caused to the performance limit of the building. For this purpose Static pushover analysis is used to evaluate the real strength of the structure and it promises to be a useful and effective tool for performance based design.

4.2 Description of structure

The building considered for analysis is a typical G + 11 storey R.C. building. The plan area of building is 15 x 15 m with 3 m as height of each typical storey. It consists of 3 bays of 5m each in X-direction and 3 bays of 5m each in Y-direction.

Table 4.1 - Basic Parameters Considered For Analysis

Sr. No.	Basic Parameters Considered	
1	Grade of Concrete	M20
2	Grade of Steel	Fe415
3	Floor to Floor Height	3m
4	Plinth Height above GL	0.55m
5	Depth of Foundation Below GL	0.65m
6	Parapet Height	1.5m
7	External Wall Thickness	0.23m
8	Internal Wall Thickness	0.15m
9	Live Load on Floor	3 KN/m ²
10	Live Load on Roof	1.5 KN/m ²
11	Floor Finishes	1 KN/m ²
12	Roof Treatment	1.5 KN/m ²
13	Density Of Concrete	25 KN/m ³
14	Density Of Masonry Wall	20 KN/m ³

The sectional properties of various elements obtained based on gravity analysis and used as initial sizes for further analysis are presented as below -

Table 4.2 - Initial Member Sizes Considered For Analysis

Sr. No.	Member Dimensions	Sizes in mm
A	Columns at	500 X 500
B	Beams at each floor level	300 X 300
C	Thickness of Slab at each floor level	150

The considered earthquake parameters used for analysis are as follows -

Table 4.3 - Earthquake Parameters Considered For Analysis

Sr. No.	Earthquake Parameters	
1	Soil Type	Hard
2	Importance Factor	1
3	Time Period	Program Calculated
4	Seismic Zone	III
5	Building Frame Type	SMRF
6	Seismic Zone Factor	0.16

The Required codes are -

Table 4.4 - Codes Used For Analysis

Sr. No.	Codes Used	
1	R.C.C. design	IS 456: 2000
2	Earthquake resistant design of structures	IS1893:2002

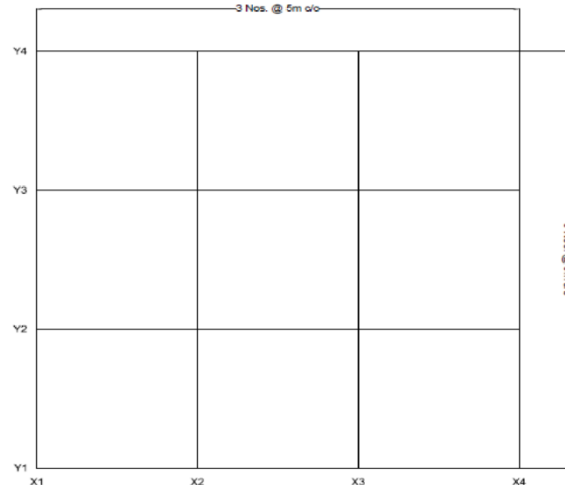


Figure 4.1 – Plan



Figure 4.2 – Elevation

4.3 Step By Step Procedure

- A. Click the File Menu > New Model command or the New Model button. The form shown in Fig will display. Verify that the default units are set to KN, m, C.
- B. The New Model form allows for the quick generation of numerous model types using parametric generation techniques. However, in this project the model will started using only the grid generation. When laying out the grid, it is

important that geometry defined accurately represents the major geometrical aspects of the model so, it is advisable to spend time carefully planning the number and spacing of the grid lines. Select the Grid Only button and then form shown in Fig will display.

C. The Quick Grid Lines form (Fig.4.4) is used to specify the grids and spacing in the X, Y and Z directions. Set the number of grids lines to 4 in the X and Y direction, and to 13 in the Z directions. Type 5, 5, 3 into X, Y, Z directions spacing edit boxes respectively. The values specified in the First Grid Line Location area locate the origin of the grids lines; make sure that these values are all set to zero for this tutorial. Click the OK button to continue.

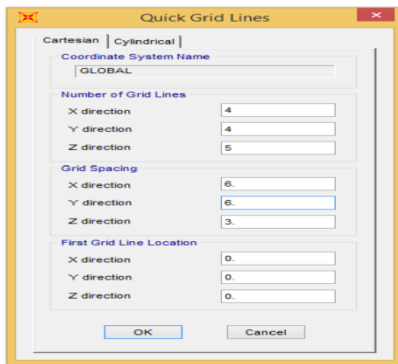


Figure 4.4 - Quick Grid Line Form

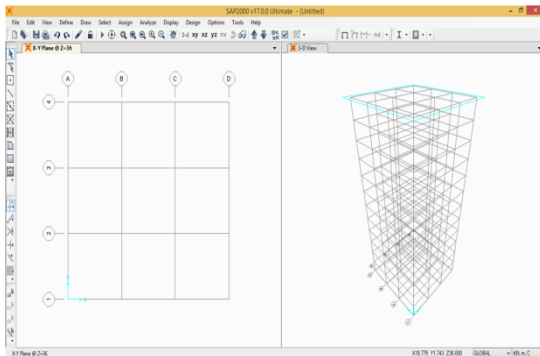


Figure 4.5 - SAP2000 Windows

Step 2 - Begin a New Model

Use the Define menu > Materials command to add, modify, or delete a material property definition. The material property definitions are then used in defining the structural objects (frame sections, cable sections, tendon sections, area sections, and solid properties.)

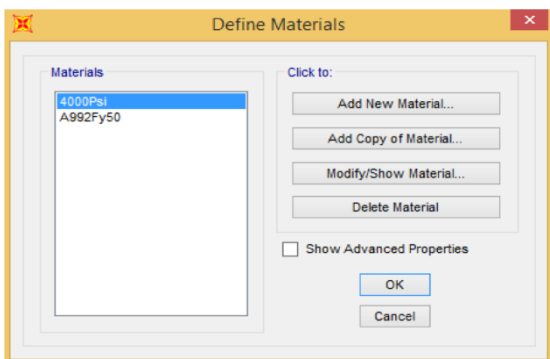


Figure 4.6 - Define Material Form

A. Click the Define menu > Materials command, the Define Material form (Fig.4.6) will display. Highlight a 4000Psi in the Materials display list Then Click the Modify/Show Material button, the form shown in Fig. 8 will display.

B. In Material Name and display color edit box, type M20 and in Material Type select concrete from drop down list.

C. Set Weight per unit Volume as 25. Set Modulus of Elasticity to 22360679.774

$$(5000 \sqrt{f_{ck}}$$

as per IS 456). Set Poisson's Ratio to 0.2. Set Specified Concrete Compressive Strength to 20000 and then click OK button.

D. Highlight an A992Fy50 in the Materials display list Then Click on Modify/Show Material button, the form shown in Fig.4.6 will display.

E. In Material Name and display color edit box, type Fe415 and in Material Type select rebar from drop down list.

F. Set Minimum yield stress (F_y), Minimum tensile stress (F_u), Expected yield stress, (F_{ye}) and Expected tensile stress (F_{ue}) to 415000, 498000, 518750 and 622500. Click OK buttons on Material Property Data form and Add materials form to exit all forms.

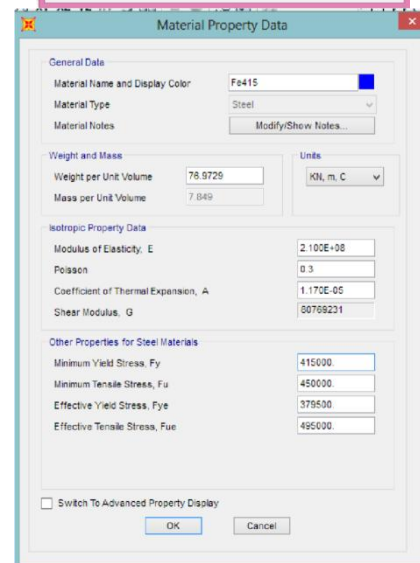
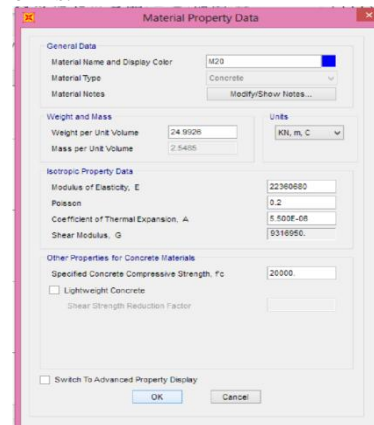


Figure 4.7 - Material Property Data Concrete

Figure 4.8 - Material Property Data Rebar

Step 2 - Define Frame Section

Defining a frame section makes the section available for assignment to selected objects. The Define menu > Frame Sections command can be used to (a) Import sections from predefined databases, (b) Define frame section properties on the basis of their dimensions, (c) review and modify section properties and (d) Delete section properties.

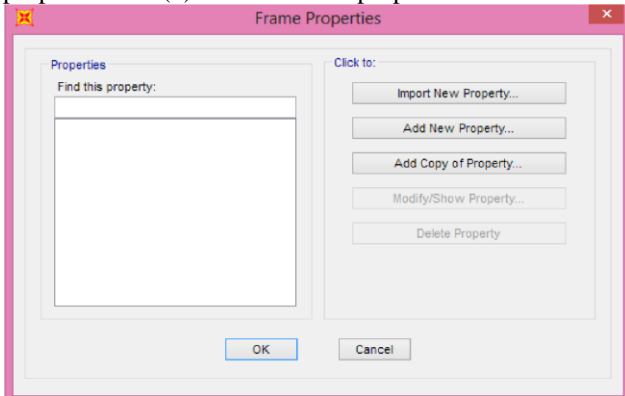


Figure 4.9 - Frame Properties Form

A. Click the Define Menu > Section Properties > Frame Sections command, which will display the frame properties form (Fig 4.9)

B. Click the Add New Property button, which will display form shown in Fig.4.10.

C. in Frame Section Property Type select Concrete from drop down list and click the rectangular button, which will display form shown in Fig 4.11 In Section Name Area, Type C-500x500.

- In Depth and Width edit box, Type 0.5 and 0.5 respectively.

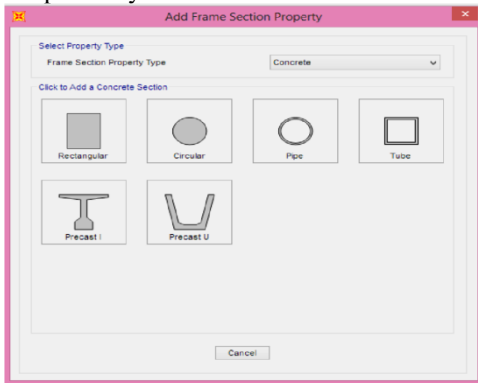


Figure 4.10 - Add Frame Section Property

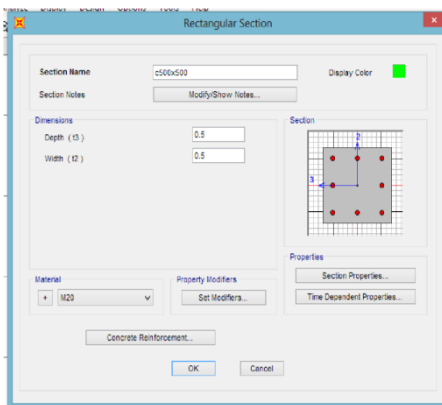


Figure 4.11 - Rectangular Section

- D. Click the Concrete Reinforcement button, Reinforcement Data form (Fig 4.12) will appear.
- In Rebar Material Area, Select Fe 415 from list as a Rebar Material of longitudinal Bars and confinement bars (Ties).
- In Design Type Area, Select Column (P-M2-M3 Design) Option.
- In configuration of reinforcement area, select Rectangular.
- In Longitudinal Bars - Rectangular Configuration Area, Type 0.05 in Cover to for confinement bars edit box.
- In Confinement Bars Area, Select 10d from drop down list as confinement Bar Size; Type 0.2 in Longitudinal Spacing of Confinement Bars edit box and Type 2 in Number of Confinement Bars in 3-dir and 2-dir.
- In set Check / Design Area, select Reinforcement to be designed.
- Click OK buttons on Reinforcement Data form and rectangular section form. E. Click the Add New Property button, which will display from shown in Fig 4.10

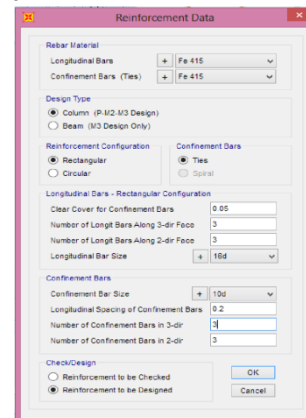


Figure 4.12 - Reinforcement Data Form for Column

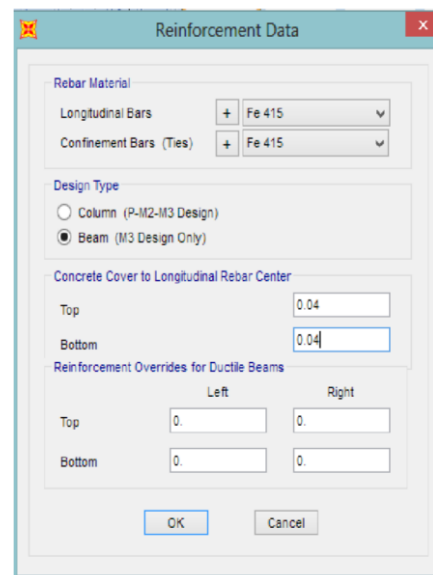


Figure 4.13 - Reinforcement Data Form for Beam

- F. Make sure that in Frame section Property Type, Concrete is selected and click the Rectangular button, which will display from shown in Fig 4.11.
- In Section Name Area, Type B-300x300.

- In Depth and width edit box Type 0.3 and 0.3 respectively.
- G. Click the Concrete Reinforcement button, Reinforcement data form (Fig 4.12) will appear.
- In Rebar Material Area, Select Fe415 from list as a rebar material of Longitudinal Bars and Confinement Bars.
- In Design Type Area, Select Beam (M3 Design only) option, which will display reinforcement data form (Fig 4.13) for beam.
- In Concrete Cover to Rebar Centre Area, Type 0.04 top and bottom edit box.
- Click the OK buttons on reinforcement data form, rectangular section form and frame properties form.

Step 4 - Add Frame Objects

In this step, Frame Objects with the associated column and beam sections list are drawn using the grids and snap- options and generated using Edit menu Command

A. Draw Frame Objects XZ. Made sure that the X-Z plane @ Y=0 view is active. Then select the DRAW FRAME or Cable option from left hand side tool bar which will give the properties of the objects which have to be draw (Fig 4.14).

Confirm the properties given in earlier stage are updated .Here we will select first C- 500 X 500 property first.

Properties of Object	
Line Object Type	Straight Frame
Section	c500x500
Moment Releases	Continuous
XY Plane Offset Normal	0.
Drawing Control Type	None <space bar>

Figure 4.14 - Properties of Object Table of Column

- B. Select the Column Properties which was given in earlier step and draw the nodes
- C. Replicate the assigned columns in Y direction as well as Z directions using Replicate command of Edit Menu as shown in Fig. 4.16

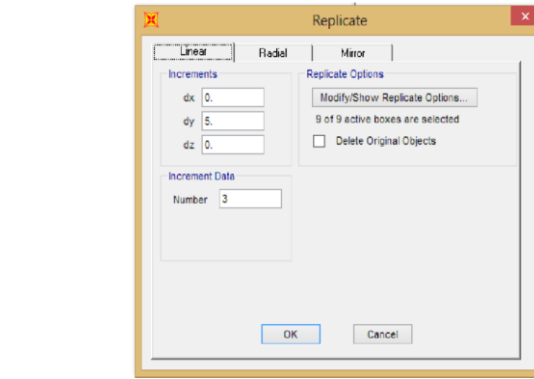
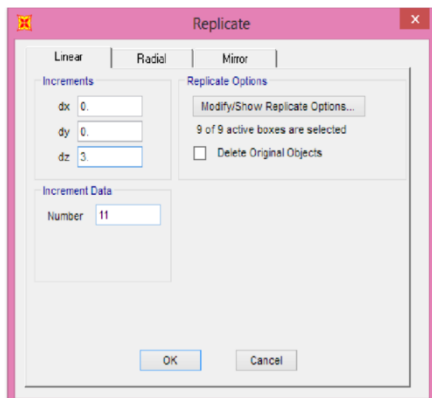


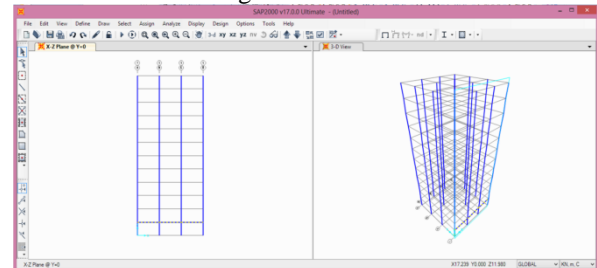
Figure 4.16 - Replication of assigned columns to Y and Z Directions

D. Now again Select the Then select the DRAW FRAME or Cable option from left hand side tool bar which will give the properties of the objects which have to be draw (Fig 4.17). Now we will select property of beam by changing section to B300x300.

Properties of Object	
Line Object Type	Straight Frame
Section	b 300x300
Moment Releases	Continuous
XY Plane Offset Normal	0.
Drawing Control Type	None <space bar>

Figure 4.17 - Properties of Object Table of Beam

E. Select the Column Properties which was given in earlier step and draw the nodes of beams of FF as Shown in Fig 4.18.



F. Replicate the assigned Beams in Y direction as well as Z directions using Replicate command of Edit Menu as shown in Figure 4.19. Similar procedure adopted for assigning beam in XY direction and then replicate it along Z direction and it will give the all nodes as per plan selected.

G. After completion of assignment of frame objects, check the all objects are assigned appropriately. The Model will have now beams and columns allotted as per Fig.

H. After assignment of objects column bases should be restrained. In this Step, supports for the frame are defined. Make sure X-Y plane @ Z=0 view is active, And that the program is in select mode.

- Select the Support Nodes i.e. nodes at Z=0

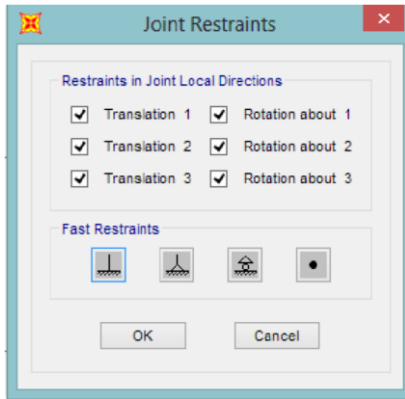


Figure 4.21 - Joint Restraints Form

- Click the Assign Menu > Joint > Restraints command to bring up the Joint Restraints form (Fig 4.21).
- Click the Fixed Support button to assign restraints in the Translation and rotation in 1, 2 and 3 directions. Click Ok to accept changes.

A. Now we have to assign the slab as an AREA object occurring in the model in the form of shell. Click the Define> Section properties > Area Sections. Area section form will arrive. Select the shell section type and click on add new section. The shell section form will arrive (Fig 4.22). Fill the appropriate information and assumptions and click on OK.

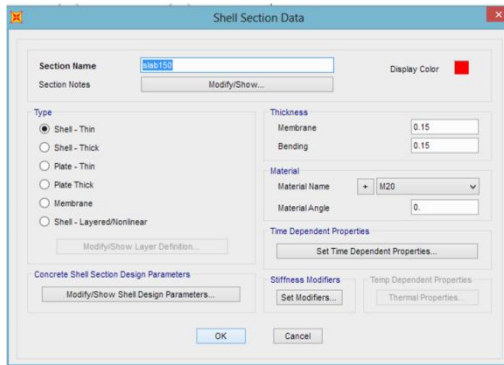


Figure 4.22 - Shell Section Data Form

- J. Now made sure that X-Y plane @ Y=3 view is active. Select Draw Rectangular Option from left hand side tool bar and assigned the slab area to the model. After assigning the all slabs @ Y=3, select them and replicate along Z direction. Then the model will have all its object (Fig. 4.23).

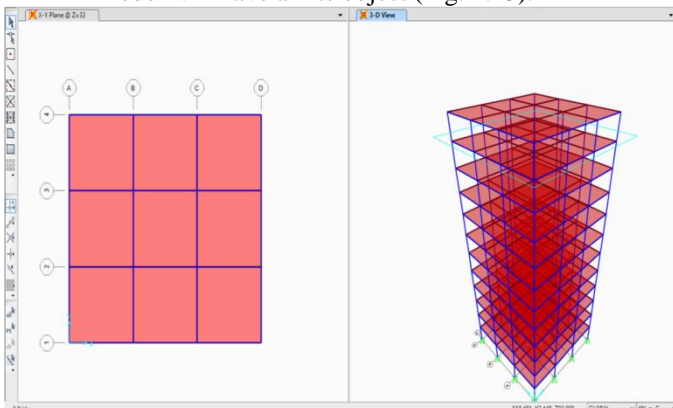


Figure 4.23 - Object Filled Model

Step 5 - Define Load Pattern

The loads used in this problem consist of Dead, Dead Wall, Dead Slab, Dead FF (Floor finish), Dead RT (Roof treatment), Live and Live Roof loads acting in the gravity direction.

A. Click the Define menu > Load Patterns command to bring up the Define Load Patterns form (Fig. 4.24). Note there is a single default load case defined, which is a Dead Load case with self-weight (DEAD). Note that the self-weight multiplier is set to 1 for the default case. This indicates that this load pattern will Load Patterns and Load Cases exist, and they may be different.

However, the program automatically creates a corresponding Load case when a load pattern is defined, and the load cases are available for review at the time the analysis is run.

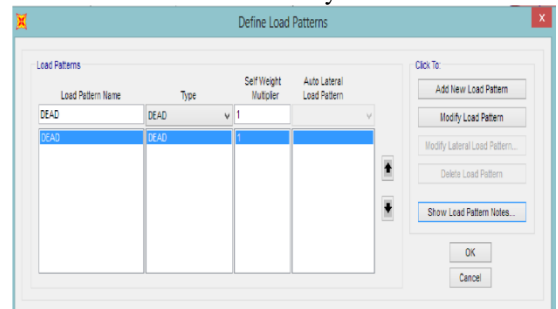


Figure 4.24 - Define Load Pattern Form

- B. Click in the edit Load Pattern Name column. Type the name of the new load pattern, DEAD Wall. Select a type of Load from the drop down list; in this case, select Super Dead. Make sure that the self-weight multiplier is set to zero. Click the Add New Load Pattern button to add the Dead Wall load to the load list.
- C. Repeat item B, to add Dead Slab, Dead FF, Dead RT load cases.
- D. Type the name of the new load pattern, LIVE. Select LIVE, a Type of load from the drop-down list. Make sure that the Self Weight Multiplier is set to zero. Click the Add New Load Pattern button to add the Live load to the load list.
- E. Type the name of the new load pattern, LIVE ROOF. Select ROOF LIVE, a Type of load from the drop-down list. Make sure that the Self Weight Multiplier is set to zero. Click the Add New Load Pattern button to add the Live load to the load list.
- F. The Define Loads form should now appear as shown in Fig 4.25. Click the Ok button in that form to accept the newly defined static load cases.

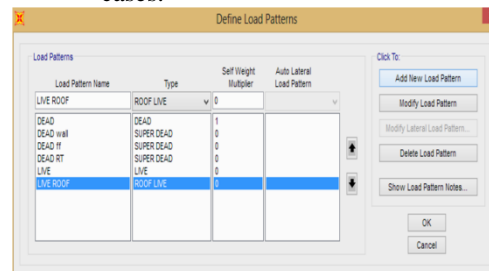


Figure 4.25 - Define Load Pattern

Step 5 - Assign Loads

A. In this step, the Dead Load and Live Load will be applied to the model. The Dead floor finish, Live load first act on slab and then transferred to the beam by Trapezoidal-Triangle rule. So these Loads Assigned to Slabs. The Dead Wall Load Directly transferred to the beams so this load should be applied to beams only.

B. First we will apply gravity loads to the slab i.e. Dead load, Dead FF and then will apply the Live loads. For application of dead load first select all the slabs by clicking on select menu > Properties > Area Section. Select the area section of slab. All Slabs automatically get selected.

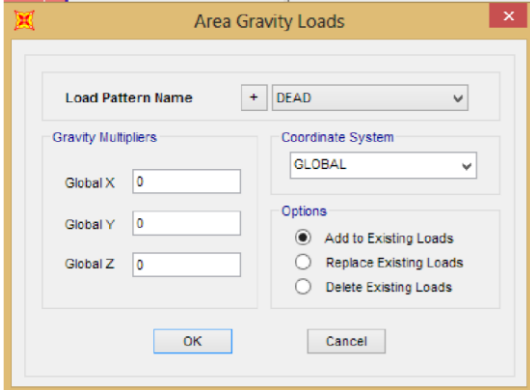


Figure 4.26 - Area Gravity Loads

A. For assigning the dead load click on Assign menu > Area Loads > Gravity load. The software automatically calculate the dead load. Select Add to Existing load tab as shown in Fig.4.26 And click on OK.

B. For assigning external gravity load like dead load select all slabs and click on Assign menu > Area Loads > Uniform shell to frame and assign the loads as per Fig.4.27 and select Add to existing load tab and click on Ok. Similar procedure adopted for live load.

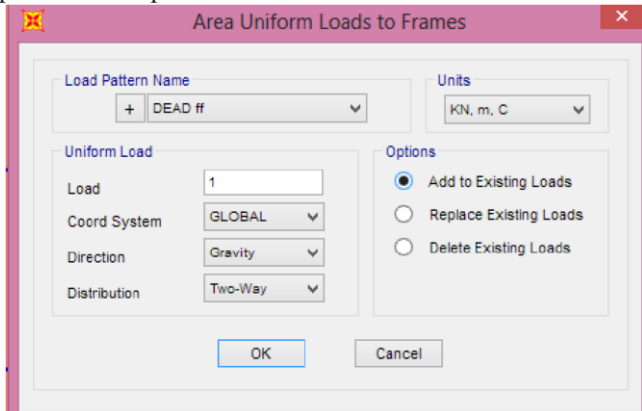


Figure 4.27 - Area Uniform Loads to Frames Form

C. Dead Load of wall directly acts upon the beams so this dead load should assigned to beams only. So, calculate the wall load the select all the beams and select Assign menu> Frame Loads > Distributed. Fill the frame distributed form as per Fig.4.28. And select add to existing load tab. and click on OK.

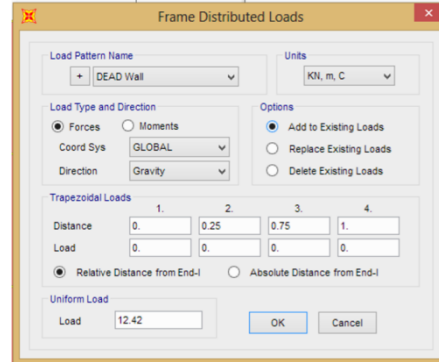


Figure 4.28 Frame Distributed Loads

Step 6 - Assign Rigid Diaphragm

Make sure that X-Y Plane @ Z= 3 view is active and that the program is in the select mode.

A. Select all nodes at Z=3, 6, 9 up to 36 level.

B. Click the Assign Menu > Joint > Constraints command. This brings up the Assign / Define Constraint form (Fig 4.29).

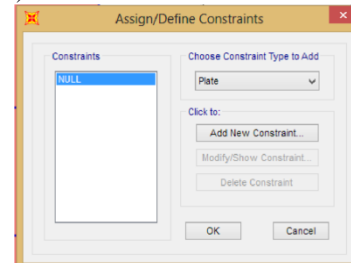


Figure 4.29 - Assign / Define Constraints Form

C. In the Choose Constraint Type to Add Area, click the drop down list and select Diaphragm.

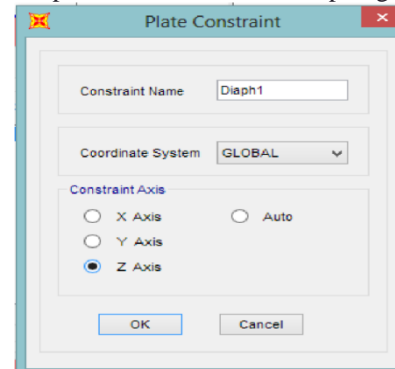


Figure 4.30 - Assign / Define Plate Constraints Form

D. Click the Add New Constraints button, which will display Diaphragm Constraint form (Fig 4.30).

E. Type Floor Diaphragm in the Constraint name edit box.

F. Verify that Z axis option is selected in the Constraint Axis area.

G. Check assign a different diaphragm constraints to each different selected Z level box.

H. Click the OK buttons on the Diaphragm Constraint and the Assign / Define Constraints forms to exist all forms.

Step 7 Assign Earthquake Load Case

Click the Define Menu > Load Cases command, which will display the Define load cases form.

A. Click the Add New Load case Button, which will display Define Load case data form.(Fig 4.41)

- B. In Load Case Name Area, type EQ-X
- C. In Load Case Type Area, Select Earthquake from drop down list.
- D. In Modal Combination Area, Select CQC option.

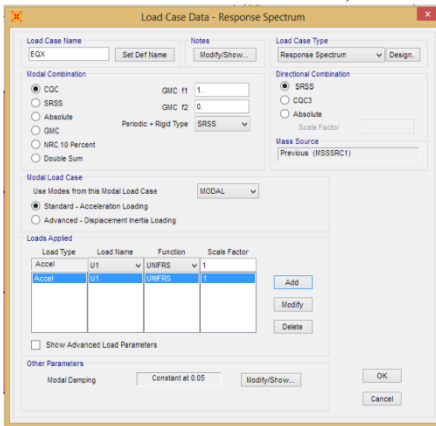


Figure 4.31 - Load Case Data- Response Spectrum Form

- E. In Load applied area,
 - In Load type area, select Accel from drop down list.
 - In Load Name Area, select U1 from drop down list.
 - In Function area select IS1893 II from drop down list.
 - In scale factor edit box type 0.3.
 - Click the Add button.
- C. Click OK button on the Load Case Data- Quake form to accept the EQ-X analysis case.
- D. Click the Add new load case button on the Define Load Case form.
- E. In Load Case Name Area, Type EQ-Y.
- F. Repeat Item D-E.
- G. Click OK button.

Step 8 Define Mass Source

- A. Click the Define menu > Mass source command, which will display the Define mass source form (Fig.4.32)

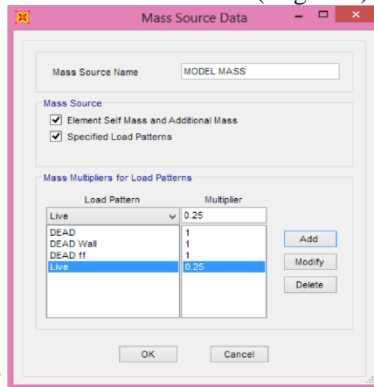


Figure 4.32 - Mass Source Data

- Form B. In Mass Definition Area, Select from loads option.
- C. In Define Mass Multiplier for loads area.
 - In Load area, select dead from drop down list.
 - Type 1 in Multiplier edit box.
 - Click the Add button to add mass source.
 - Repeat Item 1-3 for DEAD WALL, DEAD FF and DEAD RT load.
 - In Load area, Select LIVE from drop down list.
 - Type 0.25 (as live load $\leq 3 \text{ KN/m}^2$) in Multiplier edit box.

- D. Click OK button on the Define Mass Source form to accept mass sources.
- E. Click the File menu > SAVE command or save button to save the model.

Step 9 Run Analysis

In this step, the Analysis will be run.

- A. Click the Analyze menu > Run Analysis command or the Run Analysis button, to bring up the set Load Cases to Run form (Fig.4.33).

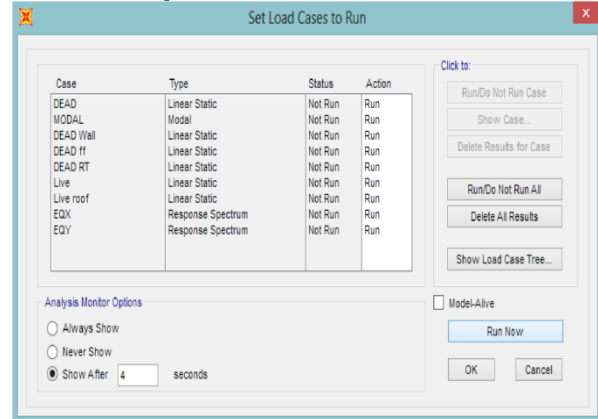


Figure 4.33 - Set Analysis Cases to Run Form

- B. Click the Run now button on the set load cases to run form, which will display SAP Analysis Monitors. The program will create the analysis model from your object based SAP 2000 model. The information may be accessed at a later time by going to File Menu > Show Input/ Output Text Files command and selecting the file with .LOG extension.
- C. When the analysis is finished, the message "ANALYSIS COMPLETE" will display. Close the analysis window. The program automatically displays a deformed shape view of the model, and the model is locked. The model is locked when LOCK/ UNLOCK model button depressed. Locking the model prevents any changes to the model that would invalidate the analysis results.

Step 10 Define Load Combinations

- A. Click the Define Menu > Combination command, which will display the Define Response Combinations form.
- B. Click the Add New Combo button, which will display the Response

Combination Data form (Fig 4.34)

- Type 1.5 (DL+LL) in the Response Combination Name edit box.
- Select Linear Add from the Combination type drop down list if it is not already selected.

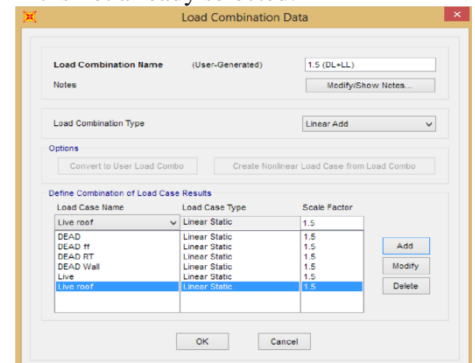


Figure 4.34 - Response Combination Data form

- Select dead in the Case Name from drop down list (if it is not already selected) and type 1.5 in the Scale Factor edit Box (If it is not already there) and click the Add button.
- Select DEAD wall in the case Name from drop down list and click Add button.
- Select DEAD FF in the Case Name from drop down list and click the Add button.
- Select DEAD RT in the Case Name from drop down list and click the Add button.
- Select Live in the Case Name from drop down list and click the Add button.
- Click Ok button on Response Combination Data form to accept Combination.

A. Similar procedure adopted to add all load combinations given below:-

- $1.2(DL+LL+EQX)$
- $1.2(DL+LL-EQX)$
- $1.2(DL+LL+/-EQY)$
- $1.5(DL +/-EQX)$
- $1.5(DL +/-EQY)$
- $0.9DL +/-1.5EQX$
- $0.9DL +/-1.5EQY$

Step 11 - Push over Analysis

In this step we carry out the Static nonlinear push over analysis.

A. For carrying out we have to assign hinges to column and beams. Select all the beams with nodes first. Select Assign > Frame > Hinges will give the Frame Hinge Assignment Data Form. First select hinge property as Auto and keep relative distance 0 first and will get auto Hinge assignment data form. (fig 4.35) Select TABLE FORM FEMA 356 for auto hinge type and select Table 67 (Concrete beams flexural) it will automatically select M3 degree of freedom and click on OK.

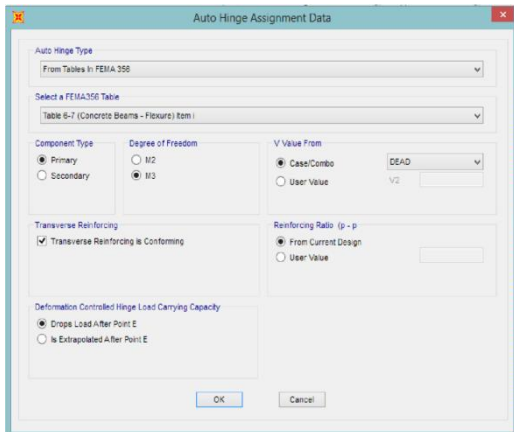


Figure 4.35 - Auto Hinge Assignment Data form for Beams

- B. After assigning one hinge to the beams assign other hinge property same as auto but change the relative distance to 1 and similar procedure as above should be adopted. Then we will get the Frame hinge assignment form as fig 4.35. Click on Ok. The hinges will get assigned to the beams.
- C. Now for assigning the hinges to the column select all columns with nodes and similar procedure as per "hinges assigned to beams" is adopted. Here while selecting auto hinge type select FEMA 356

and select table 6-8 as shown in fig 4.36. And click on OK.

- D. The all frame members are assigned with hinges should be checked. It will appear on the screen with hinge number and property. It will appear as Fig. 4.37.

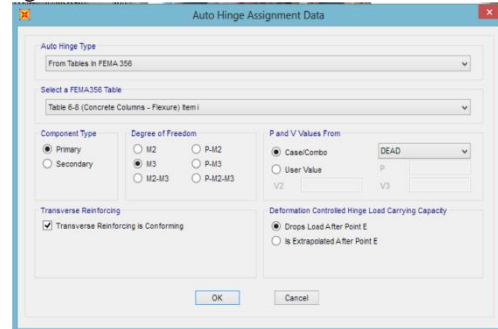


Figure 4.36 - Auto Hinge Assignment Data form for Columns

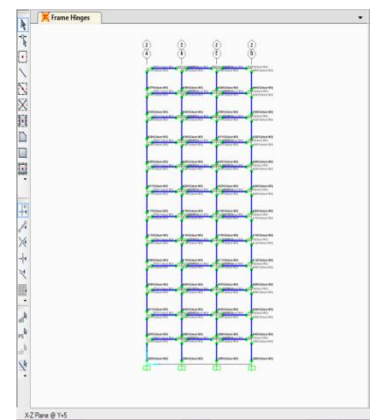


Figure 4.37 - Frame with Hinges

- E. Now we have to set the model for push over analysis case. For setting out push over case first we have modify the DEAD load as a nonlinear. Select Define > Load pattern > Dead Load > Modify case. Load case data form as per Fig 4.38 will appear change the analysis type from linear to nonlinear and click on OK.

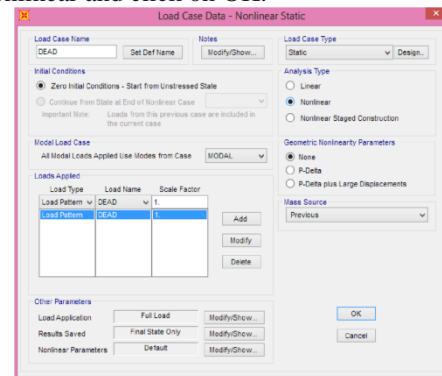


Figure 4.38 - Load Case Data- Nonlinear Static Form

- F. For adding push over case select Define menu > Load case pattern > Add new case. Write "PUSH" as Load Case Name and modify the form as per fig 4.39.

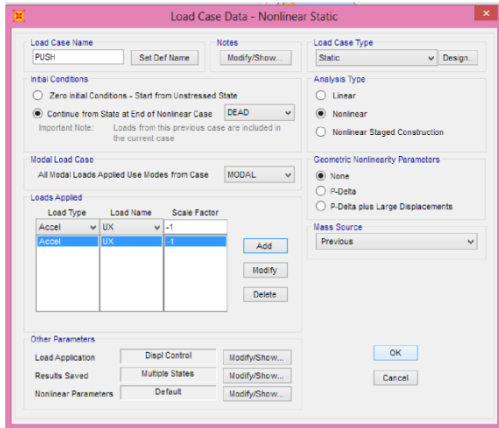


Figure 4.39 - Load Case Data for Push over Case

G. While setting other parameters from fig 4.39, for load application select modify tab and change the load application control for static nonlinear analysis as Fig 4.40.

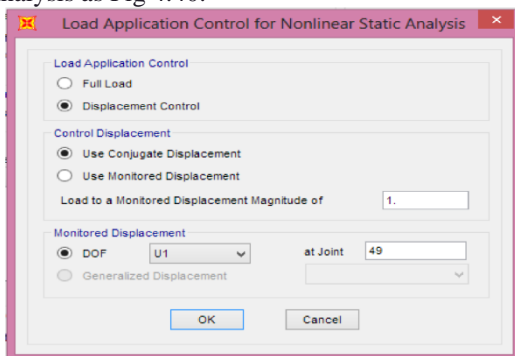


Figure 4.40 - Load Application Control for Nonlinear Static Analysis

H. For setting of Results saved of other parameters click on Modify / Show tab and change the parameters as per fig 4.41. And click on OK button.

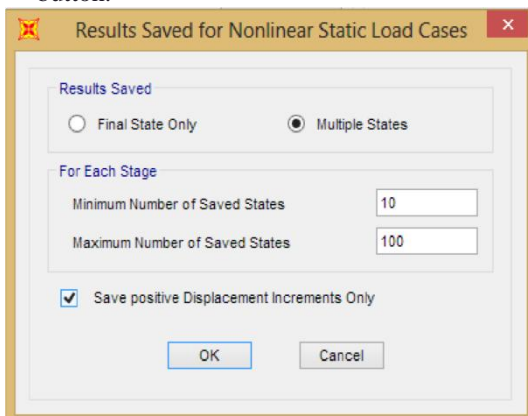


Figure 4.41 - Results Saved for Nonlinear Static Load Cases

J. After addition of Push Load case the model should be analyzed for the static push over analysis. Select Analyze > Run Analysis, "Set load cases to run analysis" window will obtain. Run only Dead load case and push over load case as per Fig 4.42. And click on the Run Now.

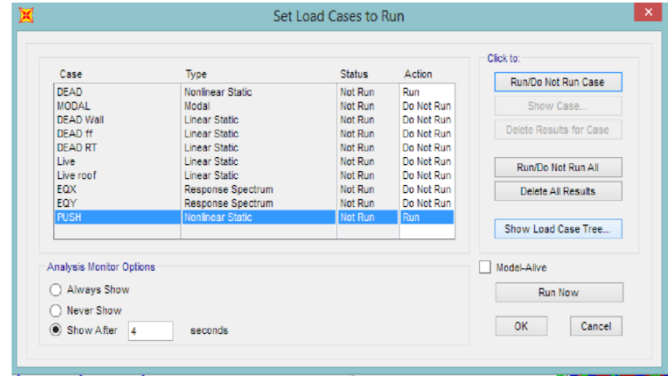


Figure 4.42 - Set Load Cases to Run Form

L. After Completion of analysis select Display > Show Static Pushover Curve and check the various parameters. To change the user parameters click on modify tab and change the user coefficients Ca and Cv as per fig 4.43. And click on Ok. The modified ATC 40 capacity curve is referred to get the results.

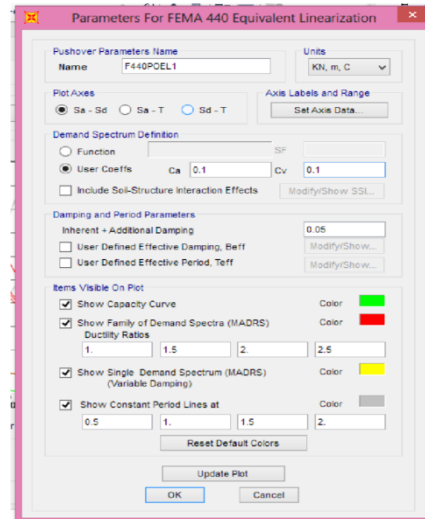


Figure 4.43 - Parameters for FEMA 440 Equivalent Linearization

4.4 Results

We get results from the Static push over curve in terms of Ductility ratio, and performance point which helps to evaluate the Response reduction factor for both immediate occupancy level and life safety level.

A. For Immediate Occupancy Level:-

Formulation of Response reduction factor:-

- Ductility factor:-

Using equation for ductility factor, derived by Miranda and Bertero,

$$R_{\mu} = \left(\frac{\mu - 1}{\phi} + 1 \right)$$

Where, Φ for medium soil is given by:-

$$\phi = 1 + \left(\frac{1}{12T - \mu T} \right) - \left(\frac{2}{5T} \times e^{-2(\ln(T) - 0.2)} \right)$$

For Immediate Occupancy Level, we have

= 2.210 sec. and $\mu = 1.148$ (from Fig 4.44)
 Thus, $\Phi = 1.042$
 $R\mu = \{[(1.148-1) / 1.042] + 1\}$
 = 1.270

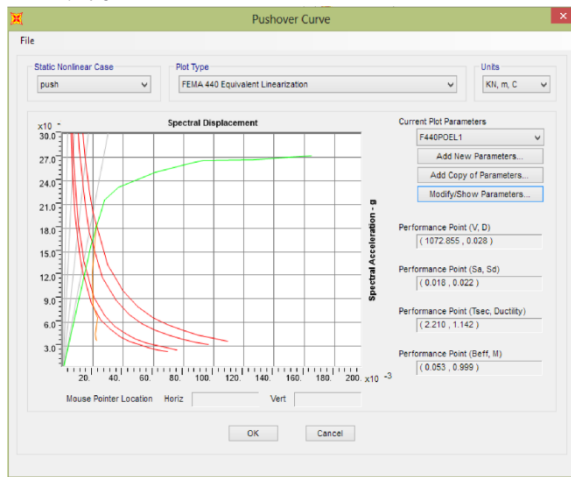


Figure 4.44 – Determination of Performance Point For IO

- Over strength factor:-

Over strength factor (R_s) = V_0 / V_D

Where,

V_0 = Maximum Base Shear corresponding to performance point
 V_D = Design Base Shear Therefore,

$R_s = (1072.855 / 520.596) = 2.060$

Therefore, the Over strength factor (R_s) of structure for IO level is 2.060.

- Redundancy factor:-

The redundancy factor depends upon the number of lines of vertical seismic framing.

Values to be considered are defined in ATC-19 as shown below:

Table No.4.5 – Redundancy Factor Table from ATC-19 for IO

Sr. No.	Lines of Vertical Seismic Framing	Drift redundancy Factor
1	2	0.71
2	3	0.86
3	≥ 4	1.00

Thus, the Response reduction factor is given by:

$R = R_s \times R\mu \times R_R$

= 2.060 X 1.270 X 1.0
 = 2.61

B. For Life Safety:-

Formulation of Response reduction factor:-

- Ductility factor:-

Using equation for ductility factor, derived by Miranda and Bertero,

Where, Φ for medium soil is given by: -

For Life Safety Level, we have

$T = 2.592$ sec. and $\mu = 1.644$

Thus, $\Phi = 0.660$

$R\mu = \{[(1.644-1)/ 0.660] + 1\}$
 = 1.970

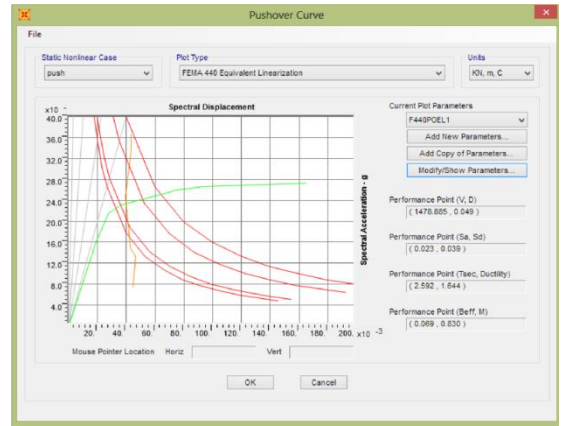


Figure 4.45 – Determination of Performance Point for LS Over strength factor:-

Over strength factor (R_s) = V_0 / V_D

Where,

V_0 = Maximum Base Shear corresponding to performance point
 V_D = Design Base Shear Therefore,

$R_s = (1478.885 / 520.596) = 2.840$

Therefore, the Over strength factor (R_s) of structure for LS level is 2.84.

Redundancy factor:-

The redundancy factor depends upon the number of lines of vertical seismic framing.

Values to be considered are defined in ATC-19 as shown below:

Table No.4.6 - Redundancy Factor Table from ATC-19 for LS

Sr. No.	Lines of Vertical Seismic Framing	Drift redundancy Factor
1	2	0.71
2	3	0.86
3	≥ 4	1.00

Thus, the Response reduction factor is given by:

$R = R_s \times R\mu \times R_R$

= 2.840 X 1.970 X 1.0
 = 5.596

C. Hinge Results

Hinges are the points in the structure or frame which help us to analyze the structural behavior. Their rotation amount helps us verify the results. Figure 4.45 and 4.46 helps to get hinges behavior and their rotation. The color combination shown below the Fig 4.46 indicates the performance of hinges in given push case.

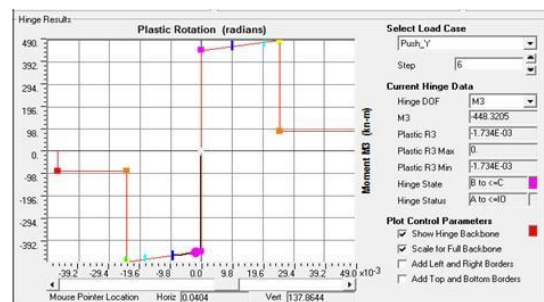


Figure 4.4.6 – Hinge result

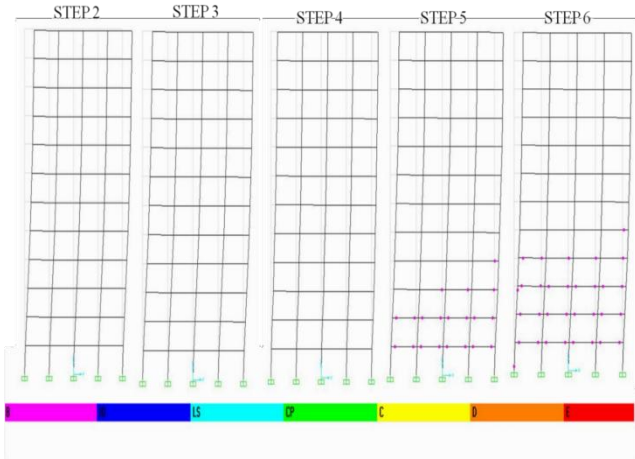


Figure 4.4.7 – Building Behavior in Push Over Case

All above results are mentioned in below table – Table 4.7 – Results for IO and LS

Parameter	Immediate Occupancy		Life Safety	
	X Direction	Y Direction	X Direction	Y Direction
Base Shear	1072.85 5 kN	1072.85 5 kN	1478.88 5 kN	1478.88 5 kN
Time Period	2.210 sec	2.210 sec.	2.592 sec.	2.592 sec.
IDR	0.69	0.69	0.243	0.243
IDR allowable	1	1	0.25	0.25
Ductility Ratio	1.148	1.148	1.644	1.644
IDDR	0.1	0.2	0.55	0.55
IDDR allowable	0.2	0.2	0.6	0.6
Plastic Rotation (°)	0.0041 rad.	0.0041 rad	0.00561 rad.	0.00561 rad.
allowable	0.005 rad.	0.005 rad.	0.02 rad.	0.02 rad.
Response Reduction Factor (R)	2.61		5.596	

The Above results are checked with guidelines given in FEMA 356 mentioned in this report in Table 3.4, Table 3.5, and Table 3.6, which shows that results are within limits & appropriate.

5. CONCLUSION AND FUTURE SCOPE

5.1 General

In the present study, the non-linear response of RCC frame using SAP2000 under the loading has been carried out with the intention to study the relative importance of several factors in the non-linear analysis of RC frames.

5.2 Conclusions

Following Conclusions are made from this study:-

1. The frame behaved linearly elastic up to a base shear value of around 520 KN. At the value of

base-shear 1072 KN, it depicted non-linearity in its behavior. Increase in deflection has been observed to be more with load increments at base-shear of 1072 KN showing the elasto-plastic behavior.

2. The joints of the structure have displayed rapid degradation and the inter storey deflections have increased rapidly in non-linear zone. Severe damages have occurred at joints at lower floors whereas moderate damages have been observed in the first and second floors. Minor damage has been observed at roof level.
3. The frame has shown variety of failures like beam-column joint failure, flexural failures and shear failures. Prominent failures are joint failures. Flexural failures have been seen in beams due to X-directional loading.
4. It has been observed that the top storey experienced major damages in this case opposite to the case of frame.
5. Micro cracks have been observed to appear even when the frame is in its elastic zone. The cracks have been found increasing with the increase in deflections.

5.3 Recommendations

The literature review and analysis procedure utilized in this thesis has provided useful insight for future application of SAP2000 for analysis. It helps in comparing the results with experimental results data and study the difference between them. This study helped in understanding the basic approach of design and performance based seismic approach. Modeling the RCC frame in SAP2000 software gives good and efficient results which can be included in future research.

4. Future Scope

In the present study, frame used is symmetrical and square. The frame can be unsymmetrical and of irregular shapes. It has been studied under monotonic loads. The frame can be studied under cyclic-loading to monitor the variation in loaddeflection curves at given time history. The relationship between torsion moment and deflection of building in seismic event can be derived.

As this approach of seismic design is still in research, there are various parameters like allowable deflection of top floor, soil strata interaction are in current research and this approach can be modified according to it.

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