## STATIC NONLINEAR ANALYSIS OF COUPLED SHEAR WALL

By

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DEPARTMENT OF CIVIL ENGINEERING Ahmedabad 382481 May 2008

# STATIC NONLINEAR ANALYSIS OF COUPLED SHEAR WALL

**Major Project** 

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering (Computer Aided Structural Analysis & Design)

By

Dhorajia Dhaval H. (06MCL003)

Guide **Dr. P.V. Patel** 



DEPARTMENT OF CIVIL ENGINEERING Ahmedabad 382481 May 2008

## CERTIFICATE

This is to certify that the Major Project entitled "Static Nonlinear Analysis of Coupled Shear Wall" submitted by Mr. Dhorajia Dhaval H. (06MCL003), towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University of Science and Technology, Ahmedabad is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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## ABSTRACT

A good amount of research and development in analysis and design of high rise structure has increased understanding about their behaviour. In case of tall structure due to height, the lateral sway of building becomes higher and so, considerations of stiffness rather than strength of structural system, control the design. The degree of stiffness depends primarily on the type of structural systems. Coupled shear wall is one of the lateral load resisting systems. It's in plane stiffness and strength are very high, so it can resist greater amount of forces. Due to its increased ductility compared to solid shear wall, it is more efficient during seismic condition.

Modeling of structural systems is very important for their realistic analysis and design. Finite element modeling and analysis of coupled shear wall building is carried out using ETABS software. Coupled shear wall is modeled considering frame and shell element. Parametric study is carried out to understand the seismic behaviour of coupled shear wall in two and three dimensional models considering different geometry. The parameters considered for the study are depth and span of coupling beam, wall length, wall height (or number of storey). Time period of two and three dimensional models is also compared with time period obtained using formula of IS: 1893 (Part-I) 2002. The results of dynamic analysis shows that time period, obtained from codal formula is lower than time computed considering two and three dimensional models for building having more than 10 storey.

Different types of support conditions are considered for calculation of time period and base shear of building. The types of support considered are fixed, hinged and flexible. The stiffness of spring support is based on coefficient of sub-grade modulus. Time period and base shear are compared for building having different types of support. Time period is also compared when wall is modeled as shall element and frame element.

In case of nonlinear analysis, coupled shear wall is modeled as frame element. Step by step procedure for static nonlinear analysis of coupled shear wall building is discussed. Comparisons of analysis results of two and three

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dimensional model are also studied. Results of nonlinear analysis shows that in flexure and shear hinge condition capacity curve (Base shear Vs. Roof displacement) are nearly same for two and three dimensional model. A parametric study is carried out by varying wall length, wall height (number of storey), span and depth of coupling beam. The earthquake loads applied on the coupled shear wall (CSW) building, are calculated as per equivalent static approach as given in IS 1893:2002. For the load calculation commercial building is considered, whose plan dimension is  $36 \times 20$  m. Capacity and demand curves are plotted for various buildings. Also failure pattern of wall and coupling beam are studied.

In dynamic as well as pushover analysis orientation of shear wall is very important. Dynamic and inelastic behaviour of centrally located coupled shear wall is good compared to other type of orientation. Compared to solid shear wall, coupled shear wall possess more ductility and it perform well in seismic conditions. The ductility comparison of different buildings, considering core and coupled core shear wall is carried out through static nonlinear (pushover) analysis for same seismic demand. Results shows that coupled core shear wall are more ductile compared core shear wall for same seismic demand.

Failure pattern of building depends on hinge property, which are used in modeling of pushover analysis. ATC 40 based hinge property represents more ductile failure compared to IS 456 based hinge property.

Finally codal provisions of IS 13920, EC 8 and ACI for analysis and design of coupled shear wall and coupling beam are discussed.

In chapter one, general aspects of nonlinearity and method of nonlinear analysis is discussed. It also includes objectives of study and scope of work. Chapter two presents the literature review on the topics related to nonlinear analysis, static and dynamic behaviour of coupled shear wall and push over analysis. In chapter three, modeling techniques and application of software ETABS (Extended Three Dimensional Analysis of Building Structures) is discussed. Chapter four describes the analysis and parametric study of coupled shear wall building for static and dynamic analysis considering different support condition. Chapter five, deals with the nonlinear behaviour of coupled shear wall building. The different orientations and their effects on dynamic and nonlinear behaviour of coupled shear wall building are discussed in chapter 6. In chapter 7, nonlinear property of hinge based on IS: 456 and limit state is discussed. The seismic design philosophy is discussed in chapter 8. Different codal provisions for design of ductile wall and coupling beam are also compared. The summary of the study, conclusions and future scope of work is given in chapter nine.

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## ABBREVIATION NOTATION AND NOMENCLATURE

2D	Two dimensional
3D	Three dimensional
В	Width of building
С	Compression
C-D	Internal failure of component
СР	Collapse provision
D-E	Residual Resistance
D. L.	Dead Load
$E_{\scriptscriptstyle D}$	Cyclic hysteretic energy dissipation at displacement of $S_{d}$
$E_{so}$	Elastic energy stored in the system at a displacement of $S_{d}$
ETABS	Extended Three dimensional Analysis of Building Structures
EQX	Earthquake Load in X direction
EQY	Earthquake Load in Y direction
F	Force
FS	Factor of safety
Ι	Moment of Inertia
IO	Immediate occupancy
Ks	The modulus of subgrade reaction
L	Length of the building
LB	Length of coupling beam
Ls	Life safety
LW	Length of wall
m1	Base moment in wall-1
m <sub>2</sub>	Base moment in wall-2
M <sub>x</sub>	Moment about X axis
My	Moment about Y axis
M <sub>xy</sub>	Moment about XY plane
M <sub>yx</sub>	Moment about YX plane
N <sub>x</sub>	Axial force in X direction
Ny	Axial force in Y direction
U	Displacement
Т	Time Period
T <sub>0</sub>	Initial Time Period

Ti	Final Time Period
Р	Axial Force
<b>q</b> <sub>a</sub>	Bearing capacity
Sa	Spectral acceleration
S <sub>d</sub>	Spectral displacement
Т	Torsional moment
$V_{Base}$	Total base shear
${eta}_{\scriptscriptstyle o}$	Structural damping
${eta}_{\scriptscriptstyle e\!f\!f}$	Effective structural damping
σ	Load intensity is applied on the soil
δ	The settlement caused by applied load
$\Delta  \delta$	Increment in settlement

#### 1.1 GENERAL

The technological revolution in the 21st century has created great demand of tall structures. From the structural engineer's point of view, tall buildings are the structures, affected by lateral forces due to wind or earthquake, because of its height. As lateral forces play important role in structural design, they must be considered from the very beginning of design. The planning of lateral load resisting system is very important in all types of tall structures design. A selection of any structural system is based on mainly justification of economy and safety criteria.

Coupled shear wall is the one of the lateral load resisting systems of high rise structure. It's in plane stiffness and strength is very high, so it can resist greater amount of seismic forces. In case of coupled shear wall, connecting coupling beams reduce the magnitude of the moments in the two walls in proportion to axial force carried by walls (Fig.1.1). Because of the relatively large lever arm (length of coupling beam) involved, a relatively small axial stress can give rise to a larger moment of resistance [1]. So coupled shear wall is more effective in resisting seismic forces compared to solid shear wall. A coupled wall structure is a combined system of frames and shear walls. The system is usually situated at the core of a medium height building and often provides spaces for elevator shafts, stairwells and storage areas [2].



Fig. 1.1 Bending moment and Shear force distribution

Generally static and dynamic analysis methods are aced for estimation of design forces in various elements of structures. In order to understand realistic behaviour of structural system only static and dynamic analysis procedure are not sufficient. Nonlinear analysis provides better understanding of structure particularly when they are subjected to seismic forces. With advances in computer hardware and software nonlinear analysis of larger structure system becomes simpler.

#### **1.2 NONLINEARITIES**

Nonlinear systems are those for which the principle of super position does not hold. Nature abounds with nonlinear systems; in fact they are the rule rather than the exception. The sources of nonlinearities can be material or constitutive, geometric, inertia, body forces or friction. The constitutive nonlinearity occurs when the stresses are nonlinear functions of the strains. The geometric nonlinearity is associated with large deformation in solids, such as beams, plates, frames and shells, resulting in nonlinear stain-displacement relations (e.g., mid-plane stretching, large curvatures of structural elements, large strains and large rotations of elements). The inertia nonlinearity may caused by the presence of concentrated or distributed masses, in a Lagrangian formulation, the kinetic energy is a function of the generalized coordinates as well as their rates and, in fluid flow, the acceleration includes a nonlinear convective term [8].

Since reinforced concrete is not an elastic material, application of the elastic analysis concepts to reinforced concrete structures is a matter of pure convenience. Even if the elastic behavior may be accepted as an idealized model for statically determinate reinforced concrete members, such a model cannot reflect the actual behavior of redundant structures.

#### **1.3 TYPES OF NONLINEARITY**

- Material Nonlinearity (plasticity, creep, viscoelasticity)
- Geometric Nonlinearity (large deformations, large strains, snap-through buckling)
- Boundary Nonlinearity (opening/closing of gaps, contact, follower force).

The example of possible nonlinear behavior, are permanent deformations and any gross changes in geometry, cracks, necking, thinning, distortions in open section beams, crippling, buckling, stress values which exceed the elastic limits of the materials, evidence of local yielding, shear bands, and temperatures above 30% of the melting temperature. In these cases, the stress is no longer proportional to the strain. Typical force-displacement curves are shown for linear and nonlinear material, as well as nonlinear stress-strain curve are shown in Fig. 1.2.



(a) Linear force displacement

Fig. 1.2 Relationship between force and displacement

#### **1.4 NONLINEAR BEHAVIOUR OF COUPLED SHEAR WALL**

When strain in structural member exists beyond their yield capacity due to material nonlinearity, plastic hinges are formed. These are the source of dissipation. By utilizing this concept, the structural response can be reduced to certain limit through controlled energy dissipation. The plastic hinges play important role in the inelastic behavior of structure. Again the inelastic behavior of earthquake resisting structural system like coupled shear wall become important in the seismic event. In case of coupled shear wall plastic hinges formed at the base and at end of every coupling beam (Fig.1.3).



Fig. 1.3 Plastic Hinge formation in coupled shear wall

It means more energy dissipation occur than solid shear wall, in which plastic hinge forms at base only. In coupled shear wall, coupling beams are the primary source of seismic energy dissipation. From the study, it is observed that as the DC (degree of coupling) increase the formulation of plastic hinge also increases in coupling beam. Therefore National Building Code of Canada has given the guideline that when DC is greater than 0.66, response reduction factor (R) is 4. And for DC less than 0.66, the response reduction factor (R) is 3.5 [5].

The coupled shear wall performs better in seismic event due to its inherent plastic behavior. The coupled shear wall can be used in newly constructed building and it can also be used for seismic rehabilitation of existing building. The important aspect is that the failure pattern of coupled shear wall should be in ductile mode. It shouldn't be in brittle mode from seismic requirement.

#### **1.5 NONLINEAR ANALYSIS**

As developments occur in highrise structure construction, new structural are developed systems to resist lateral loads which are generated due to wind or seismic event. Coupled shear wall is one of the most efficient solution to resist lateral load. Achievement of good performance under earthquake load may require study of nonlinearity either by geometry or material.

As the world moves towards the implementation of Performance Based Engineering philosophies in seismic design of structures, new seismic design provisions will require structural engineers to perform nonlinear analyses of the structures they are designing. These analyses can take the form of a full, nonlinear dynamic analysis, or of a static nonlinear Pushover Analysis. Because of the computational time required to perform a full, nonlinear dynamic analysis, the Pushover Analysis, if deemed applicable to the structure at hand, is a very attractive method for use in a current practice. For this reason, there is a need for easy to use and accurate, nonlinear Pushover Analysis tools which can easily be applied in practice. Even though recent years have seen a great amount of research in the development of such nonlinear models and techniques, there is still a great deal of knowledge missing for reinforced concrete structures [4]. Detail study is required to understanding the effect of nonlinear static as well as dynamic analysis of coupled shear wall when subjected to gravity as well as lateral loading. It is also important to understand energy dissipation through plastic hinge formation. As far as seismic forces are concerned, the inelastic behaviour of the structure plays an important role. It gives an idea about the after yield behavior, plastic hinges and through plastic hinges energy dissipations. These all parameters affected ductility of coupled shear wall. The other issues are the wall should be fail after the failure of the coupling beam in the case of coupled shear wall. Depending on geometry of the elements of coupled shear wall, flexural hinge formation and failure sequence occurs. Again the failure pattern of the coupled shear wall can be designed to achieve the structural need by considering suitable geometry of the element of coupled shear wall. Therefore to get the suitable geometry of coupled shear wall for desired failure pattern a parametric study is required.

Study of nonlinear analysis of any structure is required because of:

- Use of full strength of material.
- Overcome the limitations of assumptions of limit state theory.
- Proper economical solution.

#### **1.5.1 Static Nonlinear Analysis**

Static Nonlinear Analysis technique, also known as sequential yield analysis, or simply "push-over" analysis has gained significant popularity during the past few years. It is one of the three analysis techniques recommended by FEMA 273/274 and a main component of the Spectrum Capacity Analysis method (ATC-40). Proper application can provide valuable insights into the expected performance of structural systems and components. Misuse can lead to an erroneous understanding of the performance characteristics.

Pushover analysis is a technique by which a computer model of the building is subjected to a lateral load of a certain shape (i.e., inverted triangular or uniform). The intensity of the lateral load is slowly increased and the sequence of cracks, yielding, plastic hinge formations, and failure of various structural components is recorded. Push-over analysis can provide a significant insight into the weak links in seismic performance of a structure. A series of iterations are

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usually required during which, the structural deficiencies observed in one iteration, are rectified and followed by another. This iterative analysis and design process continues until the design satisfies a pre established performance criteria. The performance criteria for pushover analysis is generally established as the desired state of the building given a roof-top or spectral displacement amplitude.

The evaluation and retrofit design criteria is expressed as performance objectives, which define desired levels of seismic performance when the building is subjected to specified levels of seismic ground motion. Acceptable performance is measured by the level of structure and/or nonstructural damage expected from the earthquake shaking.

Three main types of Non linear Static analysis

- 1. Capacity Spectrum method (ATC 40)
- 2. Displacement Coefficient method (FEMA 273)
- 3. Secant method (CITY OF LOS ANGELES, DIVISION 95)
- In this major project "CAPACITY SPECTRUM" method is used.

Two types of Pushover analysis are:

#### 1. Load control

Used when load is known and structure is expected to support this load.

#### 2. Displacement control

Used when load is not known but displacement is known and structure is expected to lose strength and become unstable.

Essence of this procedure is a comparison between some measures of the "Demand" that earthquake place on a structure to a measure of "Capacity" of the building to resist. Three main steps involved in this method are,

- 1. Drawing Capacity Spectrum for the given building.
- 2. Finding out the demand (Demand spectrum)
- 3. Carrying out the Performance check.



Fig. 1.4 Graphical representation of CAPACITY SPECTRUM method

Base shear Vs Lateral roof displacement curve is known as capacity curve (Fig.1.5)

To generate Capacity curve:

- Apply Lateral load to the structure.
- Find out member stressed within 10 percent of its strength.
- Recording Base shear and Roof displacement
- Revising the model using zero stiffness for yielding elements.
- Applying new increment of lateral load to the revised structure such that another element yields.
- Repeating same steps up to Strength Degradation.



Fig. 1.5 Base shear Vs Lateral roof displacement curve is the Capacity curve

Develop the 2% or 5% percent damped elastic response spectrum appropriate for the specific site or as required by the IS 1893 2002 (Fig. 1.6).



Fig. 1.6 Demand spectrum

Intersection of capacity spectrum and demand spectrum is known as performance point. 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 (Fig. 1.4) [6].

### **1.6 OBJECTIVES OF STUDY**

The main objectives of present work are as follows:

- Study of static and dynamic parameters of coupled shear building in 2D and 3D models. Flexibility effect of foundation on coupled shear wall considering static and dynamic parameters.
- Static nonlinear behaviour of coupled shear wall building considering 2D and 3D modeling. Parametric study of coupling wall dimensions and their effects on failure pattern of building through pushover analysis.
- 3. Study of different orientations of coupled shear walls in same plan and their effect on static, dynamic and nonlinear behaviour of structure.
- 4. Study of nonlinear hinge property based on IS 456 and limit state design philosophy and it's effect on failure pattern of the building.
- 5. Study of different codal provisions for coupled shear wall design.

#### **1.7 SCOPE OF WORK**

To achieve above objectives the scope of major project is as:

(1) Comparative study of static and dynamic analysis between 2D and 3D model of coupled shear wall building. For study, nos. of storey has been considered 10,15,20,25 and 30. The plan and cross section elevation are as shown in Fig. 1.7. Longitudinal beam cross section is considered as 0.25×0.75m and transverse beam cross section is considered as 0.25×0.5m for all models. Slab thickness is considered as 0.125m for all model. Shear wall thickness and coupling beam width are same and they vary with nos. of stories. Column dimensions vary with no. of storey as shown in Table 1.2. Length and width of building in plan are considered 36m and 20m respectively.



Fig. 1.7 Plan and Section of coupled shear wall building

For parametric study the dimensions of coupling beam and wall width has been considered in Table 1.1 & 1.2. In all models fixed support condition is considered.

Depth	Beam span	Wall length
parameter (m)	parameter (m)	parameter (m)
0.50	1	3
0.75	1.5	4
1	2	5
1.25	2.5	6

Table 1.1 Coupling beam depth, span and wall width parameters

No. of storey	Wall width (m)	Column cross section (m)
10	0.25	0.45×0.45
15	0.30	0.5×0.5
20	0.30	0.6× 0.6
25	0.35	0.7× 0.7
30	0.35	$0.8 \times 0.8$

Table 1.2 Wall width and column cross section

- (2) Comparisons of results of static nonlinear analysis between 2D and 3D model. Parametric study on coupled shear wall dimensions for failure pattern and ductility. All dimensions and parameters are considered as shown in Table 1.1 and 1.2.
- (3) A failure patterns of differently oriented coupled shear wall building are studied, considering coupling beam depth as parameters. This study is carried out through pushover analysis. Failure pattern and ductility comparison of core shear wall and coupled core shear wall is also considered.



Fig. 1.8 Different cases of orientation of coupled shear walls in plan

Column dimensions, wall thickness and coupling beam width considered for study are shown in Table 1.2. Slab thickness is considered as 125 mm. Other beams of cross section  $0.25 \times 0.75$ m is considered respectively. Supports for all models are taken as fixed.

- (4) A failure pattern study for hinge property based on IS: 456 and a its comparison with ATC 40 based hinge property.
- (5) An illustrate an example is considered for design and detailing of 20 storey CSW building considering IS 13920 specifications. Comparison of various standards for design and detailing of coupled shear wall building using following standards,
  - a. IS: 13920
  - b. ACI
  - c. EC 8

### **1.8 ORGANIZATION OF MAJOR PROJECT**

The content of Major Project is covered in different chapters as follows

In chapter one, general aspects of behavior of coupled shear wall is discussed. Nonlinearity and method of nonlinear analysis is discussed. It also includes objectives of study and scope of work.

Chapter two presents the literature review on the topics related to nonlinear analysis, static and dynamic behaviour of coupled shear wall and push over analysis.

In chapter three, modeling techniques and application of software ETABS (Extended Three Dimensional Analysis of Building Structures) is discussed. Detailed explanations for modeling of buildings are covered in this chapter. It also gives a step by step procedure for modeling and static nonlinear analysis (Push over method). Modeling for different base condition is also discussed.

Chapter four describes the analysis and parametric study of coupled shear wall building for static and dynamic analysis of building. Time period results as obtained from dynamic analysis are compared with IS: 1893 specifications. In this chapter analysis results of 2D and 3D model are compared. Dynamic analysis considering different support conditions as well as different elements used for wall elements is also described.

Chapter five, deals with the nonlinear behaviour of coupled shear wall building. The failure pattern for different span to depth ratio of coupling beam is included in this chapter. Comparison of 2D and 3D static nonlinear analysis is also covered. The failure patterns are observed in nonlinear static behaviour of 10,15,20,25 and 30 storey CSW building by considering different parameters.

The different orientations and their effects on dynamic and nonlinear behaviour of coupled shear wall building is discussed in chapter 6. Failure pattern study is considered by coupling beam depth as parameter. Comparison of dynamic as well as nonlinear analysis of core shear wall and coupled core shear wall is also discussed.

In chapter 7, nonlinear property of hinge based on IS: 456 and limit state is discussed. Failure pattern comparison of coupled shear wall building considering hinge property based on ATC 40 and IS 456 is also presented.

The seismic design philosophy is discussed in chapter 8. Different codal provisions for designing ductile wall and coupling beam are also compared. One design example of 20 storey coupled shear wall presented with ductile detailing as per IS 13920.

The summary of the study, conclusions and future scope of work is given in chapter nine.

#### 2.1 GENERAL

In the past, analysis of coupled shear wall has been studied by many researchers and their research works were presented in different literature. In the present chapter, the review of available literature related to static, dynamic and nonlinear behaviour of coupled shear wall are presented. The study includes the review of work on topics related to static nonlinear analysis used for analysis of structure beyond elastic limit.

**Smith et al. (1991) [1]** discusses many structural system for lateral load resistance. Static and dynamic behaviour of frame, wall frame and coupled shear wall system are also explained with examples. They also illustrated an example of 20 storied CSW in which the distribution of axial forces and the moments in the walls are graphically represented.

**Paulay and Priestley (1992) [7]** explains the strategies in the location of Structural walls. The behaviour of wall and CSW are described not only for elastic stage but for the inelastic stage also. The ductility of structural wall and beam are presented.

**Taranath (1997) [2]** explains behaviour of coupled shear wall under lateral load. Theoretical background about modeling of coupled shear wall as equivalent frame element is also included. Theoretical behaviour of coupled shear wall is also compared with practical experiment.

#### 2.2 STATIC, DYNAMIC AND NONLINEAR BEHAVIOUR

**R. PARK and T. PAULAY (1974) [29]** describes about dynamic and elastoplastic behaviour of coupled shear wall. Design and ductile detailing is also discussed. From structural point of view to ensure satisfactory performance when coupled shear wall structures are exposed to severe seismic shocks, it is necessary to be able to examine at the least approximately the structures behaviour in both elastic and plastic range of loading. Desirable behaviour can be expected only if the structure is made capable of following a preferred

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sequence of yielding. From the point of view of damage control, and possible repair, it is desirable that the wall components be the last ones to suffer during the process of imposing incremental ultimate conditions.

In the study carried out by **Kuang and Chau (1998) [4]** on dynamic behaviour of stiffened coupled shear wall with flexible bases, the free vibration characteristics of stiffened coupled shear walls with flexible bases were investigated using a discrete-continuous approach. The structure was considered as both a discrete system and a continuous system at different stages of the analysis. The stiffness of the flexible foundations was represented by rotational and translational springs at the base of each shear wall. The stiffened system was reinforced by an additional stiffening beam at some level of the structure. This induced additional axial forces, and thus reduced the bending moments in the walls and the lateral deflection, and increased the natural frequencies. The effects of foundation stiffness and the stiffening beam on the free vibration characteristics of the structure were studied. The optimal location of the stiffening beam for increasing the first natural frequency of vibration was explained.

**Balkaya and Kalkan (2003) [23]** studied nonlinear seismic response evaluation of tunnel form building structures. This study presented their seismic performance evaluation based on the nonlinear pushover analyses of two case studies. The contribution of transverse walls and slab-wall interaction during the 3D action, the effects of 3D and 2D modeling on the capacity-demand relation, as well as diaphragm flexibility, torsion and damping effects were investigated. An effort was spent to develop a shell element having closing-opening and rotating crack capabilities. This study showed that the applied methodology had a considerable significance for predicting the actual capacity, failure mechanism, and evaluation of the seismic response of tunnel form buildings.

**Saatcioglu and Humar (2003) [3]** carried out work about dynamic analysis of coupled shear wall. They categorized, dynamic analysis procedure as linear (elastic) dynamic analysis considering the elastic modal response spectrum method, the numerical integration linear time history method, nonlinear (inelastic) response history analysis. They provided an overview of dynamic

analysis procedures for use in seismic design, with discussions on mathematical modeling of structures, structural elements, and hysteretic response. The determination of structural period to be used for the equivalent static force method was explained.

**Aksogan et al. (2005) [5]** carried out study on a simplified dynamic analysis of multi-bay stiffened coupled shear walls. The forced vibration analysis of a multi-bay coupled shear wall on an elastic foundation had been studied. The analysis considered shear walls with a finite number of stiffening beams, the properties of which vary from span to span and/or from section to section in the vertical direction. The continuous connection method was employed to find the structure stiffness matrix. The structure mass matrix was found with the lumped mass assumption. A time-history analysis was carried out using the Newmark numerical integration method. The two advantages of the proposed method were the simplicity of its data and the short computation time, which render it an effective method for the pre design of high-rise buildings.

**Wang and Wang (2005) [6]** presented a method to determine the first two period of natural vibration of the coupled shear wall building. He compared his Proposed method with the formula proposed by Wallace and Moehle (1992) and with the finite element analysis program SAP2000.

**S.K. DUGGAL (2007) [30]** introduce and explain all aspects of earthquakeresistance design of structures. The seismic design requirements and philosophy are briefly discussed. The principles involved in the design and conventional methods of earthquake-resistance design are introduced.

#### 2.3 MODELING

**Macleod (1990) [19]** had carry out a study on material behaviour, system behaviour and modeling. In case of material behaviour linear as well as nonlinear behaviour is explained. Element behaviour considered for linear, 2D, 3D element considering linearity as well as nonlinearity. System modeling is explained in detail for linear, non-linear, static and dynamic analysis.

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**Wand et al. (1994) [22]** had suggested modeling for asymmetric wall frame structure with varying mass and stiffness along the height. Most of the past studies on the seismic response of asymmetric buildings had concentrated on structure with uniform distribution of mass and stiffness along the building height. Such results clearly did not apply to many modern buildings with a transfer plate system at lower levels. The study proposes a simple but yet reliable formulation, capable of analyzing the seismic vibrations of irregular tall buildings with varying mass and stiffness along the building height. More specifically, the building was divided into storey. Within each storey, all wall and frame members were replaced by an equivalent flexural beam located at the center of flexure and an equivalent shear beam located at the center of shear, respectively. Different storey was connected through continuity conditions. Finally, the governing equations of coupled vibrations were established and the results of a simple example were discussed.

**Nollet and Smith (1997) [21]** carried out study on stiffened storey wall-frame structure. The shear rigidity of the frame system was increased in a story level by infilling one or more bays of the frames in that story with concrete or masonry panels, or adding bracing to the story, or increasing the sizes of the columns and girders surrounding the story. The efficiency of the concept and the evaluation of the parameters involved in the behaviour of a stiffened-story structure were demonstrated with the help of a continuum model solution. The technique was then applied to a structure as an example, which was analyzed both with the continuum model and a stiffness matrix solution.

Lu et al. (2006) [20] provided information on nonlinear FE model for RC shear wall based on multi-layer shell element and Microplane constitutive model. Nonlinear simulations for structures under disasters had been widely focused in recent years. However, precise modeling for the nonlinear behavior of reinforced concrete (RC) shear walls, which were the major lateral-force-resistant structural members in high-rise buildings, still had not been successfully solved. Based on the principles of composite material mechanics, a multi-layer shell element model was proposed to simulate the coupled in-plane/out-plane bending and the coupled in-plane bending-shear nonlinear behaviors of RC shear wall. Three

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walls under static push-over load and cyclic load were analyzed with the proposed shear wall model for demonstration. The simulation results showed that the multi-layer shell elements could correctly simulate the coupled in-plane/out-plane bending failure for tall walls and the coupled in-plane bending-shear failure for short walls. With microplane concrete constitutive law, the cyclic behavior and the damage accumulation of shear wall could be precisely modeled, which is very important for the performance-based design of structures under disaster loads.

#### 2.4 NONLINEAR ANALYSIS

**Nayar and Coull (1975) [9]** presented an elastic –plastic analysis of coupled shear walls based on wide-column frame analogy. Incremental loads were applied until a plastic hinge was formed at the heavily stressed section. This analysis helped to study the behaviour from working to ultimate load. The ductility requirement of each plastic hinge could be found out for every load increment up to collapse. And this could also calculate the ultimate load based on the practical limit of ductility.

**Applied Technology Council (ATC 40) (1997) [11]**, describes the guidelines for the linear as well as nonlinear behaviour of building elements. The inelastic hinge properties are given for the flexure as well as shear. The procedures for the inelastic analysis of building are also described.

**Bracci et al. (1997) [15]**, carried out study on seismic performance and retrofit evaluation of reinforced concrete building. A pushover analysis was essentially a step-by-step static analysis procedure that applies a distribution of lateral storey forces to a structure model developed from the moment-curvature properties of the members. The technique could be useful in evaluating structural response behaviour and identifying potential collapse mechanisms. The sequence of potential hinges could also be identified. However, during an earthquake, the sequence of hinging might be different due to the random cyclic loading in an earthquake. These potential hinge regions could then be further examined to determine their deformation capability beyond yielding.

As per **Krawinkler and Seneviratna's (1998) [14]**, opinion the push over analysis could be implemented for all structures, but it should be completed with other evaluation procedure if higher mode effects were judged to be important. No unique criterion could be established for this condition, since the importance of higher mode effects depends on the number of stories as well as on the peak(s) and plateau(s) of the design spectrum. Examples for additional evaluation procedures were, in order of preference, inelastic dynamic analysis with a representative suit of ground motions and elastic dynamic analysis using the unreduced design spectrum and suitable modal combination procedure. The latter procedure would provide estimates of elastic demand ratio that needed to be compared to acceptable values.

**Ashraf Habibullah and Stephen Pyle (1998) [28]** described about modeling for pushover analysis. Step by step procedure for analysis was presented. Interpretation of analysis results were also presented.

**Federal Emergency Management Agency (FEMA 356) (2000) [12]**, has given the guidelines for the inelastic hinge properties of all vertical elements of building. The inelastic hinge properties are given for the flexure as well as for shear of wall, coupling beam, column etc. The procedure for the inelastic static analysis has also been explained in step by step.

**Nayfeh and Frank pai (2000) [8]** provides information on nonlinearity. Different theories of beams, plates and shells are explained. Types of nonlinearity, reasons for considerations for nonlinearity, occurrence and methodology for solution are also discused. Dynamic characteristics of linear and nonlinear element are explained in detail.

**Doran (2003) [13]**, carried out study on elastic-plastic analysis of R/C coupled shear walls. Bar frame modeling was one of the good methods in coupled shear wall system in structural design. In modeling stiffness of beam was very important factor. The formula which define the ratio between plastic and elastic equivalent stiffness modification parameters were also given.

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**Sudhir K. Jain and T. Srikant (2004) [33]** discussed the concept of pushover analysis, that is becoming a popular tool in profession for (1) design the building (2) seismic evaluation of existing building and (3) developing appropriate strategy for seismic retrofitting of buildings. They shown how this analytical technique can be useful in deciding seismic retrofitting strategy and technique.

**Kunnath (2006) [10]** conducted study on assessment of nonlinear static procedure for seismic evaluation of buildings. An essential and critical component of evolving performance-based design methodologies is the accurate estimation of seismic demand parameters. Nonlinear static procedures (NSPs) are now widely used in engineering practice to predict seismic demands in building structures. While seismic demands using NSPs could be computed directly from a site-specific hazard spectrum, nonlinear time-history (NTH) analyses require an ensemble of ground motions and an associated probabilistic assessment to account for variability in earthquake recordings. Despite this advantage, simplified versions of NSP based on invariant load patterns such as those recommended in ATC-40 and FEMA-356 had well-documented limitations in terms of their inability to account for higher mode effects and the modal variations resulting from inelastic behavior. Consequently, a number of enhanced pushover procedures that overcome many of these drawbacks had also been proposed.

**Zhao et al. (2006) [16]**, had carried out study on new approach for seismic nonlinear analysis of inelastic frame structure. The nonlinear analysis referred to the evaluation of structural response considering *P*-delta effect, which was in the form of geometric nonlinearity, and inelastic behavior referred to material nonlinearity. This novel approach used finite element formulation to derive the elemental stiffness matrices, particularly to derive the geometric stiffness matrix in a general form. At the same time, this approach separates the inelastic displacement from total deflection of the structure by applying two additional constant matrices, namely, the force-recovery matrix and the moment restoring matrix in the force analogy method. The benefit behind this treatment was explicitly locating and calculating the inelastic response, together with strategically separating the coupling effect between the material nonlinearity and geometric nonlinearity, during the time history analysis. Comparison with the

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traditional incremental methods indicated that the proposed method was very time efficient as well as straightforward.

Study on nonlinear stiffness design optimization was carried out by **Chan and Hwang (2006) [17]**. Tall reinforced concrete (RC) building designs must satisfy serviceability stiffness criteria in terms of maximum lateral displacement and interstory drift. It was therefore important to assess accurately the effects of concrete cracking on lateral stiffness of such structures. This study integrated nonlinear cracking analysis methods with a powerful optimization technique and presents an effective numerical approach for the stiffness-based optimum design of tall RC buildings under service loads. A probability-based effective stiffness method was employed to identify cracked members and to modify their effective cracked stiffness. Iterative procedures were necessary for the serviceability analysis of tall RC buildings to determine their nonlinear stiffness characteristics due to concrete cracking. Design optimization based on a rigorously derived optimality criteria approach involved minimizing the cost of RC structures while satisfying the top and multiple interstory drift constraints along with member sizing requirements.

**Inel and Ozmen (2006) [18]** provided information on effect of plastic hinge properties in nonlinear analysis of reinforced concrete. Due to its simplicity, the structural engineering profession had been using the nonlinear static procedure (NSP) or pushover analysis. Modeling for such analysis required the determination of the nonlinear properties of each component in the structure, quantified by strength and deformation capacities, which depend on the modeling assumptions. Pushover analysis was carried out for either user-defined nonlinear hinge properties or default-hinge properties, available in some programs based on the FEMA-356 and ATC-40 guidelines.

### 2.5 SUMMARY

In this chapter, review of relevant literature is carried out. In the literature review, concepts of static, dynamic and nonlinear behaviour of wall frame structure, modeling, nonlinear analysis and seismic design are presented. These concepts are useful in understanding the behavior of coupled shear wall structure during occurrence of a seismic event.

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# 3. THREE DIMENSIONAL MODELING OF COUPLED SHEAR WALL

### 3.1 GENERAL

Modeling of structural systems is very important for their realistic analysis and design. Modeling of structure includes its geometry, material property of different elements, boundary conditions and loadings acting on it during its service life. Geometry of structure includes orientation of structural or non structural elements in plan as well as in elevation. Material property includes modulus of elasticity, Poisson's ratio and shear modulus etc. Boundary conditions are based on foundation conditions including soil properties. Loadings include gravitational and lateral forces developed during different conditions.

Accurate design of structure requires proper balance between safety and economy, which is governed by proper modeling. In past modeling involved mathematical parameters and simplifying assumptions, which were based on structural behavior under different loadings with different support conditions. With advancement in computing systems modeling of structural system become easier. Today modeling is carried out using software rather than manually because of complexity of geometry and loading parameters of structure.

Coupled shear wall is an efficient structural system and is widely adopted in highrise buildings. In coupled shear wall (CSW) the coupling beams are connecting the walls. Therefore the plastic hinge zones are formed at end of the coupling beams and the base of walls. Due to the introduction of coupling beam, the loads and the deformation demands are redistributed throughout the structural system rather concentrated at the base of walls. This behaviour of CSW will give more ductile failure mode compared to shear wall. The coupling beams have two main beneficial: the first one is the coupling action reduces the moments that must be resisted by individual walls and therefore CSW is more efficient lateral load resisting structural system. Secondly it provides a means by which seismic energy is dissipated over the entire height of the wall system as the coupling beam undergoes inelastic deformations, which protects the walls from excessive damage. ETABS, Extended Three dimensional Analysis of Building Structures, is a user friendly, special purpose analysis and design software developed specifically for building systems. ETABS Version 9 features an intuitive and powerful graphical interface with modeling, analytical, and design procedures. Although use of ETABS is quick and easy for simple structures, ETABS can also handle large and complicated building models, including a wide range of nonlinear behaviors.

In the present study ETABS is used mainly for the following purposes:

- 1) Modeling of coupled shear wall
- 2) Analysis of coupled shear wall and
- 3) Design of coupled shear wall structure

In this three dimensional modeling and analysis of Reinforced Concrete coupled shear wall in highrise building is discussed. Finite element modeling of coupled shear wall building for dynamic and nonlinear analysis is explored. Wall is modeled using frame element and shell element. In case of nonlinear analysis, coupled shear wall is modeled as frame element. Step by step procedure for static nonlinear analysis of coupled shear wall building is discussed. Modeling for different support conditions is also discussed in this chapter.

### **3.2 FINITE ELEMENT MODELING OF BUILDING ELEMENTS**

Finite element modeling and analysis of coupled shear wall building is carried out using ETABS software.. The shell element is used to model shell, membrane, and plate behaviour in planer and three-dimensional structures. The shell element is one type of area object. Depending on the type of section properties assigned to an area, the object can model plane stress/strain and axisymmetric solid behavior. By combining a membrane element and a plate bending element one obtains a flat shell element. This is a useful form of element for slab as well as wall, where membrane effects are not negligible. The Shell element has all six degrees of freedom at each of its connected joints. Directions of force and moment component for thin shell are shown in Fig. 3.1(a).

The Frame element is used to model beam and column behavior in planar and three-dimensional structures. The Frame element uses a general, threedimensional, beam and column formulation, which include the effects of biaxial

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bending, torsion, axial deformation, and biaxial shear deformations. The element can be prismatic or non-prismatic. The variation of the bending stiffness may be linear, parabolic, or cubic over each segment of length. The Frame element has all six degrees of freedom at both of its connected joints as shown in Fig. 3.1(b).



Fig.3.1 Elements are used in modeling of CSW building

### **3.3 MODELING IN ETABS FOR 3D DYNAMIC ANALYSIS**

ETABS gives facility for dynamic analysis for 2 D as well as 3 D model. A typical example of twenty storied coupled shear wall building is explained in this section. For the plan of the building refer Fig. 1.7. Column dimensions are considered as per Table 1.1 and 1.2. Coupling beam size considered is 0.25 x 0.5 m. Slab thickness considered as 125mm.

Modeling steps for 3D coupled shear wall building in ETABS are follows:

1. Grid lines are created for geometry of building, as per plan shown in Fig. 1.7 for 20 storey building.



Fig. 3.2 Geometry creation

2. Property is defined and assigned for beam, column and coupling beam as rectangular element. For slab and wall planer element is defined.

Properties	Click to:
Type in property to find:	Import I Wide Flange
CB	Import / wide Flange
BEAMLONG BEAMSHORT	Add I/Wide Flange 💌
CB ConoBm	Modify/Show Property
ConcCol COULMN HSS4X.237 HSS4X.250	Delete Property
HSS54X.250 HSS5X.250 HSS5X.258	OK

Sections	Click to:
DECK1 PLANK1	Add New Deck 💌
SLAB WALL	Modify/Show Section
	Delete Section
	OK
	Cancel

Fig. 3.3 Defining frame element

Fig.3.4 Defining wall and slab

3. Support condition is a very important parameter for any structure. It is the relation between foundation and soil. In this model fixed supports are provided. It has also been defined as shown in Fig. 3.5. Final 3D model with complete structural elements is shown in Fig.3.6.

Assign Restraints
Restraints in Global Directions
$\blacktriangleright$ Translation $\times$ $\blacktriangleright$ Rotation about $\times$
▼ Translation Y ▼ Rotation about Y
▼ Translation Z ▼ Rotation about Z
Fast Restraints
0K Cancel

Fig.3.5 Support condition



Fig. 3.6 3D model of 20 storey building

4. Loading parameters are defined as shown in Fig. 3.7 and 3.8. Dead load and live load are provided as gravity loads and Earth quake load is provided as per IS: 1893 -2002. All seismic parameters are shown in Fig. 3.8. Live load has been assigned 3 kN/m<sup>2</sup> except on roof.

.oads	T		Self Weight	Auto	Click To: Add New Load
Load	Туре		Multiplier	Lateral Load	
EQ	QUAKE	-	0	IS1893 2002 💌	Modify Load
DEAD	DEAD		1		
LIVE	LIVE		0		Modifu Lateral Load
EQ	QUAKE		0	IS1893 2002	modily Edicial Edda
					Delete Load
					OK

Fig.3.7 Static Load cases

Direction and Eccentricity          C × Dir       Image: C × Dir         C × Dir       Image: C × Dir         C × Dir + Eccen Y       Y Dir + Eccen X         C × Dir - Eccen Y       Y Dir - Eccen X         Ecc. Ratio (All Diaph.)       Image: C × Dir + Eccen X         Override Diaph. Eccen.       Image: C × Dir + Eccen X	Seismic Coefficients Seismic Zone Factor, Z Per Code 0.16 User Defined Soil Type III
Time Period         Ct (m)           C Approximate         Ct (m)           C Program Calc	
Story Range Top Story STORY20 - Bottom Story BASE -	OK
Factors Response Reduction Factor, R 4.	Cancel

Fig.3.8 Earthquake load application as per IS: 1893 (2002)

5. Mass source application is also provided in modeling as shown in Fig. 3.9. AS per IS:1893, 25% of live load up to  $3 \text{ kN/m}^2$ . It is applied to all floors.

Define Mass Source
Mass Definition From Self and Specified Mass From Loads From Self and Specified Mass and Loads Define Mass Multiplier for Loads Load Multiplier LIVE 0.25 LIVE 0.25 LIVE 0.25 LOBE Add Modify Delete
Include Lateral Mass Only Lump Lateral Mass at Story Levels           OK         Cancel

Fig.3.9 Mass source application

6. ETABS have facility to create rigid diaphragms action for slab. For that first select all slabs in plan view and apply as shown in Fig.3.10. It is necessary because in theory, it is assumed that during earthquake slab displace in rigid manner.







Fig. 3.11 3D model in plan

7. For modeling of wall finite element method is selected. The meshing of the model is done by using the size of element as  $250 \times 250$ mm as shown in Fig. 3.12.

Meshing Options				
Cookie Cut at Selected Line Objects (Horiz.)				
C Cookie Cut at Selected Points at Degrees (Horiz.)				
• Mesh Quads/Triangles into 144 by 144 Areas				
○ Mesh Quads/Triangles at				
Intersections with Visible Grids				
Selected Point Objects on Edges				
Intersections with Selected Line Objects				
OK Cancel				

Fig. 3.12 Meshing of wall element

8. In modeling, wall is modeled as pier, so apply pier labeling to all walls as shown in Fig. 3.13.

Pier Names	
Wall Piers         P1         P10         P10         P2         P3         P4         P5         P6         P7         P8         P9	Click to: Add New Name Change Name Delete Name OK Cancel

Fig.3.13 Pier element

# **3.4 CHECKING OF 3D MODEL FOR SEISMIC ANALYSIS**

Correct modeling is necessary for reliable analysis of structure. So each and every model has to be checked with manual calculation. In all parametric study 3D models are checked by comparing base shear of model, obtained by seismic coefficient method (manually) with ETABS base shear results. Here calculations are checked for one model shown below (Fig.3.14).

No.storey	20							
Storey height	3		_					
Lump mass c	alculatio	n				_		
Slab			Coulmn					
L	36		B (m)	0.6				
В	20		D (m)	0.6				
weight/vol.	25		H (m)	2.25				
Thickness	0.125		No.of columns	10				
			weight/vol.	25				
Ws	2250	kN	Wc	202.5	kN			
Beam			Shear wall					
X direction (1	onger sid	le)	length (m)	3				
			No. of shear wall	10				
width (m)	0.2		thickness (m)	0.3				
depth (m)	0.75		height (m)	3				
No. of beam	4		weight/vol.	25				
length (m)	36		Ws	675	kN			
weight/vol.	25					-		
wbx	540	kN	Live load	3	kN/m^2	1		
V direction (s	shorter si	de)	L.L. at intermedeate	540	ŀΝ			
width (m)	0.2	uc)	E.E. at Interinedeate	240	ni (			
denth (m)	0.5		Lump mass for roof	3569.5	kN			
No of heam	4		Lump mass for intermediate	5507.5	NI V			
length (m)	19.2		roof	4548 3	kN			
weight/vol	25		1001	1010.0				
wbv1	192	kN	Total sesmic weight	89986	kN			
wbv2	130	kN	rotar sconne weight	07700	ni (			
Coupling bea	m		7.	0.16		т	1.2075	Sec
width (m)	0.3		T	1		•	1.2075	
depth (m)	0.5		R	4				
No. of beam	5		sa/g	1.1263				
length (m)	1		Ah	0.0225				
weight/vol.	25		Base shear (kN)	2027	1			
web	18.75	kN	Base shear ETABS (kN)	1951				
			% Error	3.763				
Total wb	880.75	kN			-			
			1					

Fig. 3.14 Excel sheet for base shear calculation

### **3.5 MODELING FOR STATIC NONLINEAR ANALYSIS**

ETABS provide facility for nonlinear 3D analysis for any structure. Wall is generally modeled for analysis as shell element considering as pier and coupling beam is modeled as flexural frame element. But in the case of nonlinear analysis hinge formation in the wall element cannot be modeled. So coupled shear wall is modeled as wide column beam analogy.

### 3.5.1 Wide-column frame model

This is the most common modeling approach of coupled shear wall (Fig. 3.15). The CSW structure is replaced by an equivalent plane frame of beams and columns. The walls are modeled by column situated at the centroidal axis with rigid horizontal element transmits the rotational and vertical displacement effects at the edge of the wall to the connecting element.



Fig. 3.15 Wide-frame column model

It is a necessary to check the structural behavior of wall in both different type of modeling. For checking purpose different models are analyzed considering shell element and frame element for walls. Plan and other dimensions are considered as per Fig. 1.7 and Table 1.1 and 1.2. In all models fixed support condition is considered. The time period and base shear comparisons are shown in Fig. 3.16. and Fig. 3.17 respectively.



Fig. 3.16 Time period comparison



Fig.3.17 Base Shear comparison

From study of graphs and tables of time period and base shear it is observed that, there is not major difference in results of time period and base shear. It shows that when wall is model as frame element instead of shell element not major difference occur in structural behaviour under seismic loading.

### 3.5.2 Modeling for 3D pushover analysis

Procedure for modeling of 2D and 3D model for push over analysis is same in ETABS. In case of 2D model Seismic force applied at storey level by considering approximate analysis. In 3D model whole geometry of building is created and seismic force is calculated for model through software automatically. Modeling for 10 stored coupled shear wall building is explained in this section. The wall length, depth and span are considered as 3.0, 0.5 and 1.0m respectively. Geometry of building created is as shown in Fig. 1.7. Column cross section dimensions, wall width and coupling beam width are taken as per Table 1.1 and 1.2 for 10 storeys. The flexure and shear hinge properties of wall and coupling beam are defined in ETABS as shown in Fig. 3.18 with reference to FEMA 356-2000 and ATC 40 (Appendix-A). In the loadings gravity loads and lateral loads generated from earthquake are applied on the model. These load calculations are same as linear static case. The linear analysis and design is carried out for the corresponding responses in ETABS. Then for the nonlinear analysis two load cases are defined (Fig. 3.20 & 3.21). First one for gravity load and second for lateral load case are considered respectively. The gravity and lateral loads are applied by force and displacement action respectively.

The following general sequence of steps are followed in performing a static nonlinear analysis as follows:

- 1. Creating a model as per building plan as shown in Fig. 1.7. Note that material nonlinearity is restricted to frame and link elements, although other element types may be present in the model.
- Defining the static load cases, for dead load and earthquake, that are needed for use in the static nonlinear analysis (Define > Static Load Cases command).
- Defining hinge properties, (Define > Frame Nonlinear Hinge Properties command). Hinge property is applied as per FEMA 356 and ATC-40. For this example are defined shear and flexural hinge in wall as well as coupling beam of the coupled shear wall (Fig. 3.18).

Frame Hinge Property Data for BSH1 - M3	Frame Hinge Property Data for WSH1 - V2	
Edit	Edit	
	Point Force/SF Disp/SF	
Point Moment/SE Botation/SE	E· · ·0.4 ·10.7	
E0.2 .7	D· -0.4 -4	
C 125 4		
	B 1. 0.	
	C 1.25 4.	
	D 0.4 4.	gid Plastic
	E 0.4 [U.7]	
<u> </u>	Scaling for Force and Disp	
D 0.2 4.	s is Binid Plastic Positive Negati	ive
E 0.2 7.	Use Yield Force Force SF	
l✔ symme	ettic Vield Disp Disp SF	
Scaling for Moment and Rotation	Accession Charle Div (CD	
Positive Ne	Vegative Positive Negative	ive
Use Yield Moment Moment SF	Immediate Occupancy 2.	
Use Yield Botation Detailing CE	Liře Safetu 3	
Acceptance Criteria (Plastic Botation/SE)	Collapse Prevention 4.	
Positive Ne	Type	
Immediate Occupancy 2.	Force - Displacement	
18.0-64	C Stress - Strain	-1
Life Safety	Hinge Length	
Collapse Prevention 4.	Relative Length     Cancel	

# (a) Shear hinge in Coupling beam

(b) Shear hinge in Wall

				Edit			
Point	Moment/SF	Rotation/SF	1 II	Point	Moment/SF	Rotation/SF	1
E٠	-0.8	-6.7		E·	-0.34	-6.3	
D-	-0.8	-4		D-	-0.34	-4	
C-	-1.25	-4		C-	-1.25	-4	
B·	-1	0.		B-	-1	0.	
Α	0.	0.		A	0.	0.	
В	1.	0.		B	1.	0.	╶
С	1.25	4.		C	1.25	4.	_
D	0.8	4.	Hinge is Bigid Plastic	D	0.34	4.	Hinge is Bigid Plast
E	0.8	6.7		E	0.34	6.3	
IV Use' IV Use'	Yield Moment Mi Yield Rotation Ro	Position SF	ve Negative	V Use	e Yield Moment M e Yield Rotation R	Positiv oment SF otation SF	/e Negative
IV Use IV Use IV Use Incceptan	Yield Moment Mo Yield Rotation Ro ce Criteria (Plastic Ro ce Occupancy	Position orient SF otation SF otation/SF) Position 1.	re Negative	IV Use IV Use Accepta Immedi	e Yield Moment M e Yield Rotation R nce Criteria (Plastic R ate Occupancy	Positiv oment SF otation SF otation/SF) Positiv 1.	re Negative
Use Viceptano Immedial	Yield Moment Mi Yield Rotation Re ce Criteria (Plastic Ri ce Occupancy ty	Positi oment SF otation SF Otation/SF) 1. 3.	ve Negative	I Usa I Usa Accepta Immedi Life Sa	e Yield Moment M e Yield Rotation R nce Criteria (Plastic R ate Occupancy fety	Positiv oment SF otation SF Positiv 1. 3.	re Negative
Use Use cceptani Immedial Life Safe Collapse	Yield Moment Mi Yield Rotation Ro ce Criteria (Plastic Ri ce Occupancy ty Prevention	Positi oment SF otation /SF 0tation/SF) 1. 3. 4.	ve Negative	I Use I Use Accepta Life Sa Collaps	e Yield Moment M e Yield Rotation R nce Criteria (Plastic R ate Occupancy fety e Prevention	Positiv otation SF iotation/SF) Positiv 1. 3. 4.	re Negative

# (c) Flexural hinge in Coupling beam

(d) Flexural hinge in Wall

# Fig.3.18 Concrete Hinge property

4. Assigning hinge properties, to frame/line elements (Assign > Frame/Line > Frame Nonlinear Hinges command) (Fig.3.19).

Assign Frame Hinges (Pushover)	Assign Frame Hinges (Pushover)
Frame Hinge Data Hinge Property Relative Distance WFH1 0. Add WFH1 1. Modify Delete	Frame Hinge Data         Hinge Property         BFH1         0.375         BFH1         0.375         Add         BFH1         0.625         Modify         Delete
OK Cancel	OK Cancel

(a) Wall element

(b) Coupling beam

Fig.3.19 Assign hinge property

- 5. Running the basic linear and dynamic analyses (Analyze > Run command).
- 6. If any concrete hinge properties are based on default values to be computed by the program, concrete design about be performed so that reinforcing steel is determined.
- Define the static nonlinear load cases (Define > Static Nonlinear/Pushover Cases command), as shown in Fig. 3.20 and 3.21 2D and 3D modeled with hinges are shown in Fig. 3.22 and Fig. 3.23 respectively.

Static Nonlinear Case Name	PUSH1	
Options		
<ul> <li>Load to Level Defined by Pattern</li> </ul>	Minimum Saved Steps	1
Push to Disp. Magnitude	Maximum Null Steps	50
🔲 Use Conjugate Displ. for Control	Maximum Total Steps	200
Monitor UX - C1 STORY10 -	Maximum Iterations/Step	10
Start from Previous Case	Iteration Tolerance	1.000E-04
Save Positive Increments Only	Event Tolerance	0.01
Aember Unloading Method	Geometric Nonlinearity Effects	
Unload Entire Structure	None	
.oad Pattern	Active Structure	
Load Scale Factor	Active Gro	
DEAD - 1. Add		- Add
		Modify
Modify		Insert
Delete		Delete
	, , , , , , , , , , , , , , , , , , , ,	

Fig.3.20 Static Nonlinear load case-1

Static Nonlinear Case Na	me	PUSH2	
Options			
C Load to Level Defined by Pattern	Minimun	n Saved Steps	10
Push to Disp. Magnitude 1.2	Maximur	Maximum Null Steps	
Use Conjugate Displ. for Control	Maximur	m Total Steps	200
Monitor UX - C1 STORY1	0 💌 Maximur	m Iterations/Step	10
Start from Previous Case PUSH1	<ul> <li>Iteration</li> </ul>	Tolerance	1.000E-04
Save Positive Increments Only	Event T	olerance	0.01
Member Unloading Method	Geometric	Nonlinearity Effect	ts
Unload Entire Structure	▼ None		-
Load Pattern	Active Stru	ucture	
Load Scale Factor	Sta	Active G	duor
SEISMIC 1 Add	1	ALL	
Mad			Modify
	<u>y</u>		Insert
Dele	e		Delete
		Applu to Added F	lements Onlu

Fig.3.21 Static Nonlinear load case-2



Fig.3.22 2D model of coupled shear wall with hinge property



Fig.3.23 3D model of coupled shear wall building with hinge property

- 8. Running the static nonlinear analysis (Analyze > Run Static Nonlinear Analysis command).
- Reviewing the static nonlinear results (Display > Show Static Pushover Curve command), (Display > Show Deformed Shape command), (Display > Show Member Forces/Stress Diagram command), and (File > Print Tables > Analysis Output command).

# **3.6 INTERPRETATION OF RESULTS OF PUSHOVER ANALYSIS**

Several types of output can be obtained from the static nonlinear analysis:

- 1. Base Reaction versus Monitored Displacement can be plotted.
- Tabulated values of Base Reaction versus Monitored Displacement at each point along the pushover curve, along with tabulations of the number of hinges beyond certain control points on force-displacement curve can be viewed on the screen, printed, or saved to a file.

- 3. Base Reaction versus Monitored Displacement can be plotted in the ADRS format where the vertical axis is spectral acceleration and the horizontal axis is spectral displacement. The demand spectra can be superimposed on this plot.
- 4. Tabulated values of the capacity spectrum (ADRS capacity and demand curves), the effective period and the effective damping can be viewed on the screen, printed, or saved to a file.
- 5. The sequence of hinge formation and the color-coded state of each hinge can be viewed graphically, on a step-by-step basis, for each step of the static nonlinear case.
- 6. The member forces and stresses can be viewed graphically, on a step-by-step basis, for each step of the static nonlinear case.
- 7. Member forces and hinge results for selected members can be written to a file in spreadsheet format for subsequent processing in a spreadsheet program.
- 8. Member forces and hinge results for selected members can be written to a file in Access database format.

# **3.7 MODELING OF SUPPORTS FOR DIFFERENT TYPE OF SOIL CONDITION**

Structural behaviour depends on boundary conditions or modeling of support conditions. Usually fixed or hinged support conditions are considered. If soil below base prevents rotation or allows rotation. Structural response parameters such as displacement, time period, and base shear also depend on support condition of the structure. In the modeling of structure using software modeling of support is very important when effect of soil condition is to be considered.

The flexible support is modeled as a spring support having stiffness depending on soil bearing capacity.. The stiffness of spring is based on coefficient of sub-grade modulus. The modulus of subgrade reaction is defined as,

Where,  $\sigma$  the load intensity is applied on the soil and  $\delta$  is the settlement caused by applied load. Joseph E. Bowles [24] has suggested the following expression for approximation of K<sub>s</sub> from allowable bearing capacity, q<sub>a</sub> which is furnished by the geotechnical consultant.

$$K_s = 40 x FS x q_a$$
 (kN/m<sup>3</sup>) ... (3.2)

When this  $K_s$  value is multiplied with area of foundation stiffness of spring  $K_s$  in kN/m, is obtained which is used in modeling offspring support in ETABS.

Assign Springs		
Spring Stiffness in Global Directions         Translation ×       0.         Translation Y       0.         Translation Z       1080000         Rotation about ××       0.	Assign Restraints          Restraints in Global Directions         Image: Translation X in Rotation about X         Image: Translation Y in Rotation about Y         Image: Translation Z in Rotation about Z	
Rotation about ZZ       0.         Options       •         • Add to Existing Springs       •         • Replace Existing Springs       •         • Delete Existing Springs       •         Advanced       •         • OK       Cancel	Fast Restraints	

(a) Spring support property

(b) Restrained condition of support



In ETABS spring stiffness ( $K_s$ ) is applied in global Z direction and translation in global X, Y and rotation in Z direction are restrained as shown in Fig. 3.24.

### **3.8 SUMMARY**

Modeling is a very important part of analysis and design of any structure. Modeling in software is done such that, actual behaviour of structure is reflected in analytical results. Various studies shown that 3D analysis gives more realistic results compare to the 2D analysis. In nonlinear analysis hinge formation takes place in frame element. In present chapter 2D and 3D modeling of coupled shear wall building is discussed for static, dynamic and nonlinear analysis. Modeling of support conditions is also presented the modeling procedure discussed in this chapter is followed in subsequent chapter.

# **4** STATIC AND DYNAMIC 3D ANALYSIS OF COUPLED SHEAR WALL

### 4.1 GENERAL

This chapter deals with the static and dynamic analysis of three dimensional coupled shear wall (CSW) building. The parameters considered are coupling beam depth, coupling beam span, wall length and number of storey for study of time period and base shear distribution. The comparison of analysis results is carried out in terms of time period and base shear distribution in coupled shear wall building for Three dimensional (3D) and Two dimensional (2D) dynamic analysis.

Foundation plays an important role in behavior of structure, which is usually considered at a support condition while modeling and analysis. Usually fixed or hinged support conditions are considered depending on sub-base soil condition. Structural response parameters such as displacement, time period, and base shear depend on support condition of the structure. In the modeling of structure, using software modeling of support is very important when effect of soil condition is to be considered.

In this chapter different types of support conditions have been considered for calculation of time period and base shear of building. The results are also compared with time period based on IS: 1893 -2002 formula. In this study shear wall is modeled as shell and frame element.

# 4.2 METHODOLOGY FOR PARAMETRIC STUDY

3D and 2D analysis is done with the help of ETABS software. The step by step procedure for modeling and analysis of 2D and 3D CSW building using ETABS software is as follows:

Step-1: 3D modeling of coupled shear wall building

In 3D modeling of building the coupled shear wall is modeled as quadrilateral shell element as explained in section 3.2. The meshing of the model is done by using elements of size 250x250 mm.

Step-2: Material and Geometric Properties The material properties are:

Name of material:	Reinforced concrete
Type of material:	Isotropic
Weight per unit volume:	25 kN/m <sup>3</sup>
Modulus of Elasticity:	27386 N/mm <sup>2</sup>
Poisson's Ratio:	0.15
Concrete Compressive strength:	30 N/mm <sup>2</sup>
Reinforcement Yield stress:	415 N/mm <sup>2</sup>

The geometrical and cross sectional dimensions are considered as per Table 1.1 and Table 1.2 for different parametric study.

Step-3: Boundary condition

The fixed support conditions are assigned at the base of coupled shear wall and column.

# Step-4: Application of loading

In study of coupled shear wall the lateral loads are important. Therefore the seismic loads are considered for the analysis of coupled shear wall building. In 3D modeling of coupled shear wall building dead load, live load are applied as per IS: 875 part 1 and part 2 respectively. Earthquake load is applied as per IS: 1893- 2002. In case of 2D analysis horizontal forces are calculated as per equivalent static approach of IS: 1893-2002.

Step-5: Analysis of coupled shear wall is carried out twice. In first analysis the seismic loading is applied as per IS: 1893 time period and checked with manual calculation. After checking the model, second analysis is done considering building's time period based on dynamic analysis.

# Step-6: Study and Results

The following results are observed:

- a) Time period of 3D as well as 2D building based on dynamic analysis of all coupled shear wall building, with different parameters.
- b) Base shear distribution in wall and column for all coupled shear wall buildings.

# 4.3 STUDY OF TIME PERIOD FOR CSW- COUPLING BEAM DEPTH AS PARAMETER

The time period of coupled shear wall building depends on the stiffness of coupled shear wall, which depends on geometrical parameters. So it carries more significance to study the behavior of coupled shear wall, considering depth of coupling beam and number of storey as parameters. The behaviour of 10,15,20,25 and 30-storied CSW is studied by varying the depth of coupling beams. The depth of coupling beam is varied from 0.5 m to 1.25 m with length of wall 3 m, and span of coupling beam 1 m.

# 4.3.1 Time period comparison

The variation in time period for different depth of coupling beams is shown in Fig. 4.1. The graphs are plotted with respect to depth of coupling beam and no. of storey of coupled shear wall. From study of Fig.4.1 it is observed that, for 10 storey building there is less difference in time period for all cases. Time period of 3D analysis is nearly same to IS: 1893 time period formula. Difference between time period of 2D analysis and 3D analysis is increased with increasing number of storey. 2D model of coupled shear wall building is more flexible compared to 3D model.

# 4.3.2 Base shear distribution comparison

Total base shear of building is resisted by coupled shear wall and columns. The distribution of base shear in CSW and column for various depth of coupling beam is presented in Table 4.1.

No. of storey	Depth of coupling beam (m)	% Base shear resisted by wall	% Base shear resisted by column
10	0.5	98.43	1.57
	0.75	98.40	1.60
	1.00	98.45	1.55
	1.25	98.51	1.49
15	0.50	97.88	2.12
	0.75	97.90	2.10
	1.00	98.10	1.90
	1.25	98.34	1.66
20	0.50	96.73	3.27
	0.75	96.92	3.08
	1.00	97.02	2.98
	1.25	97.07	2.93
25	0.50	95.63	4.37
	0.75	95.86	4.14
	1.00	95.98	4.02
	1.25	96.27	3.73
30	0.50	93.75	6.25
	0.75	93.99	6.01
	1.00	94.16	5.84
	1.25	94.27	5.73

Table 4.1 Summary of base shear distribution -Coupling beam depth as parameter

From study of Table 4.1, it is observed that for same no. of storey when coupling beam depth increases, base shear in wall increases because of increase in stiffness. Over all contribution of wall for base shear decreases with increase in the number of storey.

# 4.4 STUDY OF TIME PERIOD FOR CSW- COUPLING BEAM SPAN AS PARAMETER

The study of variation in time period with different span of coupling beam is carried out. The effect on base shear distribution is also studied with different span of coupling beam. The span of coupling beam is varied from 1 m to 2.5 m with constant depth of coupling beam as 0.5 m and length of wall as 3 m.

### 4.4.1 Time period comparison

The comparison of time period obtained by IS 1893 formula and 2 D and 3D analysis with varying span of coupling beam is presented in Fig. 4.2.For 10 storey building, there is less difference in time period for all cases. Time period of 3D analysis is nearly equal to IS: 1893 time period formula. Difference between time period of 2D analysis and 3D analysis increases with increase in number of storey. 2D model of coupled shear wall of building is more flexible compared to 3D model. In 10 and 15 storey buildings time period is not affected much by span of coupling beam. But for 20, 25 and 30 storey building with increasing coupling beam span time period of building is reduced.

### 4.4.2 Base shear distribution comparison

Contribution of CSW and columns in resisting base shear is presented in Table 4.2. It is observed that for same no. of storey with increasing span of coupling beam distribution of base shear in wall decreases. It means 3D model becomes more flexible with increase in coupling beam span. With increase in number of storey contribution of wall in resisting base shear reduces.

No of storay	Span of coupling	% Base shear	% Base shear resisted
NO. OF SIDLEY	beam (m)	resisted by wall	by column
10	1.00	98.43	1.57
	1.50	97.55	2.45
	2.00	97.54	2.46
	2.50	97.20	2.80
	1.00	98.20	1.80
15	1.50	97.10	2.90
15	2.00	97.36	2.64
	2.50	96.94	3.06
	1.00	96.70	3.30
20	1.50	96.50	3.50
20	2.00	96.32	3.68
	2.50	93.04	6.96
25	1.00	95.63	4.37
	1.50	95.64	4.36
	2.00	95.44	4.56
	2.50	95.23	4.77
20	1.00	93.75	6.25
	1.50	94.41	5.59
50	2.00	93.91	6.09
	2.50	93.44	6.56

Table 4.2 Summary of base shear distribution -Coupling beam span as parameter

# 4.5 STUDY OF TIME PERIOD FOR CSW- WALL LENGTH AS PARAMETER

The study of variation in time period with different length of wall is carried out. The effect on base shear distribution is also studied with changing length of wall. The length of wall is varied from 3 m to 6 m with constant depth of coupling beam as 0.5 m and span of coupling beam as 1 m.

# 4.5.1 Time period comparison

The time period is studied with varying length of wall. The variation in time period is shown in Fig.4.3. It can be observed that, the time period obtained from ETABS is higher than the IS1893-2002 specifications for same length of wall considering different numbers of storey.

With increasing length of wall time period of 3D model is reduced drastically for all buildings. It shows that in plane stiffness of coupled shear wall of building increases with length of the wall. Reduction in time period is more noticeable for 2D model compared to 3D model for any particular building.

# 4.5.2 Base shear distribution comparison

The base shear distribution in CSW and column is summarized in Table 4.3 for wall length as parameter. Study shows that base shear distribution in wall increases with increase in length of wall for any particular no. of storey. With increase in no. of storey contribution of shear wall is reduced because due to slenderness wall become flexible.

No. of storey	Wall length (m)	% Base shear resisted by wall	% Base shear resisted by column
	3	98.43	1.57
10	4	98.75	1.25
10	5	98.78	1.22
	6	98.83	1.17
	3	97.52	2.48
15	4	98.34	1.66
15	5	98.61	1.39
	6	98.77	1.23
20	3	96.73	3.27
	4	97.77	2.23
	5	98.41	1.59
	6	98.51	1.49
	3	95.63	4.37
25	4	97.01	2.99
	5	97.72	2.28
	6	98.74	1.26
	3	93.75	6.25
30	4	96.30	3.70
30	5	97.61	2.39
	6	98.07	1.93

Table 4.3. Summary of base shear distribution -Wall length as parameter

# 4.6 DIFFERENT SUPPORT CONDITION AS PER SOIL CONDITION

Different types of support conditions have been considered for calculation of time period and base shear of building. The types of support considered are fixed, hinged and flexible. The flexible support is modeled as a spring support having stiffness depending on soil bearing capacity. Three types of soil are considered having safe bearing capacity of 150 kN/m<sup>2</sup>, 300 kN/m<sup>2</sup> and 600 kN/m<sup>2</sup> to represent soft, medium and stiff soil conditions respectively. The stiffness of spring is based on coefficient of sub-grade modulus. Same building plan considered for this study. These results have also been compared with time

period based on IS: 1893 -2002 formula. Modeling for analysis is carried out as per section 3.7.

# 4.6.1 Coupled shear wall modeling

Wall is generally modeled as shell element considering it as pier and coupling beam is modeled as flexural frame element for analysis. But in the case of static nonlinear analysis hinge formation, in the shell element cannot modeled. So at that time wall is modeled as frame element.



Fig.4.4 Wall modeled with different types of elements

It is a necessary to check the structural behavior of wall in both the type of modeling. For checking purpose different models are analyzed considering shell element and frame element for walls. Plan and other dimensions are considered as per Fig. 1.7 and Table 1.1 and 1.2.

# 4.6.2 3D Dynamic Analysis

Dynamic analysis is done for all models in which wall is modeled as frame element and shall element. Support is considered as hinged, fixed and flexible. The analysis results in terms of time period and base shear for different support condition considering wall as shell element and frame element are obtained and presented in Fig. 4.5 and Fig. 4.6 respectively. This results are also compared with that specified by IS 1893 and are shown in Fig. 4.5 and 4.6.



### Fig. 4.5 Time Period Comparison

There is not major difference in results of time period and base shear for fixed and hinged support conditions. It indicates that when wall is modeled as frame element instead of shell element, the structural behaviour of wall under seismic loading condition remains nearly same.

In case of other support condition like hinge and springs, the time period and base shear of frame element models are different from wall element models. Time period is higher when wall is modeled as frame element, which indicates that use of frame element in wall modeling under different boundary condition reduced stiffness of structure. Therefore time period increase and base shear reduces.



Fig. 4.6 Base Shear Comparison

With increase in S.B.C. time period reduces so, foundation condition shifts towards fixed condition from hinged base condition.

# 4.6.3 Results and Discussion

- Overall study of time period comparison indicates that, time period as per IS: 1893 remains constant for particular no. of storey for any dimension of coupling beam and wall. Because the formula is independent of stiffness and mass of building.
- Time period of 2D model is more than 3D model in any case. it indicates that,
   2D model is more flexible than 3D model.
- From different parametric studies, it is observed that as, when length of wall is increased for different no. of story, time period as per IS: 1893 and time period of 3D model are nearly matching with each other.

- From parametric studies it is observed that time period for 2D model, 3D model and IS: 1893-2002 formula are similar with each other for 10 storey building.
- When wall length increases percentage base shear distribution also increases in wall for any no. of story.
- With increase in number of storey, the distribution of base shear in wall is reduced in all cases, which indicates increase in flexibility.
- Time period is mainly function of stiffness and mass. Stiffness of building depends on member property of structure as well as foundation condition. Base shear of structure is inversely proportional to time period of structure.
- The time period formula as per IS: 1893-2002 is based on geometric parameter only. It does not consider actual soil stiffness and mass of structure. So it cannot give exact time period for particular soil condition. But this time period value is always low, so base shear is very high. Form safety point of view, it is good but not good form economy point.
- For economical design of the building the support conditions based on actual soil conditions should be considered.
- Base shear of structure is inversely proportional to time period of structure. Thus flexible soil condition gives higher time period and less base shear.
- For pushover analysis, it is necessary to modeled wall modeled as frame element. When wall is modeled as frame element instead of shell element, for fixed base condition, time period and base shear are similar for both the model.

# 4.7 SUMMARY

In this chapter static and dynamic analysis of coupled shear wall building is presented. Different parameters like coupling beam depth, coupling beam span, wall length and wall height (no. of storey) are considered to understand behaviour of CSW in seismic condition. Contribution of shear wall and column in resisting horizontal load due to earthquake is also studied. To understand the effect of support condition on time period and base shear of building hinge, fixed and flexible supports are considered for analysis.



Fig. 4.1 Time Period comparison coupling beam depth as parameter



Fig. 4.2 Time Period comparison coupling beam span as parameter



Fig. 4.3 Time Period comparison wall length as parameter

# 5. 3D NONLINEAR ANALYSIS OF COUPLED SHEAR WALL

### 5.1 GENERAL

The recent trend of performance-based design has brought the nonlinear static pushover analysis procedure to the forefront. Pushover analysis is a static, nonlinear procedure in which the magnitude of structural loading is incremented in accordance with a certain predefined pattern. With the increase in magnitude of the loading, weak links and failure modes of the structure are found. The loading is monotonic with the effect of cyclic behavior and load reversals being estimated by using modified monotonic force-deformation criteria and with damping approximations. Static Pushover analysis is an attempt by the structural engineering profession to evaluate the real strength of the structure and it promises to be a useful and effective tool for performance based design. FEMA-273 [12] and ATC-40 [11] documents have developed the design and analysis criteria for Pushover analysis.

The evaluation and retrofit design criteria are expressed as performance objectives, which define desired levels of seismic performance when the building is subjected to specified levels of seismic ground motion. Acceptable performance is measured by the condition of structure and nonstructural damage expected from the earthquake shaking. Damage is expressed in terms of post yield, inelastic deformation limit for various structural elements of concrete building. The analysis procedure incorporated in the methodology account for post elastic deformation of the structure by using simplified nonlinear static analysis method.

This nonlinear static procedure constitute an inelastic analysis that considers behaviour of buildings after they begin to crack and yield in response to realistic earthquake motion. This approach differs from traditional linear static procedures that reduce seismic forces to levels that allow engineers to design buildings under the assumption that they remain undamaged. Although unrealistic and potentially misleading, this simplistic approach can work well for new buildings and for regular exiting buildings. The advantage of the newer, nonlinear static procedure, when applied to exiting buildings, is that they credit the good feature of buildings at the same time that they identify deficiencies. Some buildings may be too complex to rely on the nonlinear static procedure. Those cases may require time history analysis of the nonlinear behavior of the structures during earthquake. The kind of the buildings that may require these specialized analyses are those that are highly irregular or complicated. Other examples of buildings system that may necessitate more sophisticated analysis are energy dissipation or base isolation system.

There are three main simplified static analysis methods available:

- 1. Capacity spectrum method:-(ATC-40) [11] utilizes the intersection of the capacity curve and reduced response spectrum to estimate maximum displacement.
- 2. Displacement coefficient method:-(FEMA-273) [12] utilizes the modified version of the equal displacement approximation to estimate maximum displacement.
- 3. Secant method:-(CITY OF LOS ANGELES, DIVISION 95) that uses substitute structure and secant stiffness.

Following are unique features of capacity spectrum method:

- Although there are various ways to specify the forcing function, the capacity spectrum method uses the first mode shape forcing function to push the model.
- The method uses the combined capacity spectrum- demand spectrum plot to predict performance. In comparison, the coefficient method (FEMA-273) [12] employees the target displacement based on nonlinear spectral analysis method.
- The method uses the effective hysteric damping to reduce the earthquake demand.

There are two types of pushover analysis:

# 1. Load control

The full load combination is applied. It is used when the load is known (such as gravity load) and the structure is expected to be able to support the load.

### 2. Displacement control

The magnitude of the load combination is increased or decreased as necessary until the control displacement reaches a specified value.

It is used when:

- a) Specified drifts are sought,
- b) Magnitude of the applied load is not known in advance.
- c) The structure can be expected to lose strength or become unstable, when displacement occurred in the design earthquake is known.

### 5.2 THREE DIMENSIONAL PUSHOVER ANALYSIS

In this analysis method, equivalent horizontal loading is applied on the model on an incremental basis. Three-dimensional static analysis is performed in a stepby-step manner in which the possibility of formation of plastic hinge in a member is checked in each step. If no element reaches its plastic moment capacity, then load applied is incremented and analysis is performed on the structure with new load case. Whenever any element reaches its plastic moment capacity, plastic hinge is introduced in that element. Now new analysis is performed on this structure with new earthquake load distribution, as earthquake will depend on the structural properties. Checking is done for plastic moment capacity of other elements and plastic hinge is introduced when element reaches its plastic moment capacity. At each step, the load required for each event to occur is noted down where formation of plastic hinge in any element is considered as event. This procedure is repeated until plastic mechanism is formed in the entire structure that leads to collapse of the structure. The collapse load corresponds to the load required for the final event to occur.

### **5.3 FORCE DEFORMATION CRITERIA FOR HIGNES**

The ATC-40[11] and FEMA-273 [12] documents have developed modeling procedure, acceptance criteria and analysis procedure for pushover analysis. These documents define force-deformation criteria for hinges used in pushover analysis. As shown Fig. 5.1, five points labeled A, B, C, D and E are used to define the force deflection behavior of the hinge. The response is linear to an effective yield point, B, followed by yielding (possibly with strain hardening) to

point C, followed by strength degradation to point D, followed by final collapse and loss of gravity load capacity at point E.

The three points labeled IO, LS, and CP are used to define the acceptance criteria for the hinge. IO, LS and CP stand for Immediate Occupancy, Life Safety and Collapse Prevention respectively. These are structural performances levels and can be explained as below:



Fig. 5.1 Force-Deformation for Pushover Hinge

Immediate occupancy performance level: Structural Performances Level, Immediate Occupancy, means the 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 all of their pre-earthquake strength and stiffness. The risk of life-threatening injury as a result of structural damage is very low, and the building should be safe for occupancy.

Life safety performance level: Structural performance level, life safety, means the 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. It should be possible to repair the structure; however, for economic reasons this may not be practical. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to reoccupancy. Collapse prevention performance level: Structural performance level, collapse level, means the building is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral-force-resisting system, large permanent lateral deformation of the structure. It is also characterized by degradation in the vertical-load-resisting system but it must continue to carry their gravity load demands. There may be significant risk of injury due to falling hazards from structural debris. The structure may not be technically practical to repair and is not safe for reoccupancy, as aftershock activity could induce collapse.

### **5.4 CAPACITY SPECTRUM METHOD**

The capacity spectrum is derived from an approximate nonlinear, incremental static analysis for the structure. The capacity spectrum method is a non-linear static procedure, which provide the process of performing incremental nonlinear static analysis and a capacity curve is developed for the building. This capacity curve is simply a plot of the total lateral seismic shear demand "V" on the structure, at various increment of loading, against the lateral deflection of the building at the roof, under that applied lateral force. Capacity spectrum method provides a graphical representation of the global force-displacement capacity curve (i.e. pushover curve) of the structure and it compares to the response spectra representation and retrofit design of the buildings. The graphical representation provides a clear picture of how building responds to earthquake ground motion.

Two main elements of a performance based design procedure are demand and capacity. Demand is a representation of the earthquake ground motion. Capacity is a representation of the structure's ability to resist the seismic demand. The performance is dependent on the manner that the capacity is to handle the demand. In other words, the structure must have the capacity to resist the demand of the earthquake such that the performance of the structure is compatible with the objectives of the design. The terms capacity, demand and performance point are explained below.

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## Capacity

The expected ultimate strength (in flexure, shear, or axial loading) of a structural component excluding the reduction factors are commonly used in design of concrete member. The 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.

## **Demand (Displacement)**

In nonlinear static analysis procedure, demand is represented by an estimation of the displacements or deformations that the structure is expected to undergo. This is in contrast to conventional linear elastic analysis procedure in which demand is represented by prescribed lateral forces applied to the structure. Traditional linear analysis method use lateral forces to represent a design condition. For nonlinear methods it is easier and more direct to use a set of lateral displacement 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 during the ground motion.

## Performance

Once a capacity curve and demand displacement is 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 displacement implied by the displacement demand. The performance point is derived which is the intersection of the capacity spectrum with the appropriate demand spectrum in the capacity spectrum method.

The main steps involved in these methods are,

- 1. Draw capacity curve for the given building.
- 2. Find out the demand (demand spectrum) i.e. an estimate of the maximum expected response of the building during the ground motion.
- 3. Carry out the performance check.

# 5.5 CONSTRUCTION OF BILINEAR REPRESENTATION OF CAPACITY SPECTRUM

A bilinear representation of capacity spectrum is needed to estimate the effective damping and appropriate reduction of spectral demand. Construction of the bilinear representation requires definition of the point  $a_{pi}$ ,  $d_{pi}$ . This point is the trial performance point, which is estimated by the experience to develop a reduce demand spectrum. If the reduced response spectrum is found to intersect the capacity spectrum at the estimated  $a_{pi}$ ,  $d_{pi}$  point, then that point is the performance point. The first estimate of point  $a_{pi}$ ,  $d_{pi}$  is designated as  $a_{p1}$ ,  $d_{p1}$ , the second  $a_{p2}$ ,  $d_{p2}$  and so on. Fig.5.2 shows the bilinear representation of capacity spectrum. Where, K<sub>i</sub> is initial stiffness and A<sub>1</sub> and A<sub>2</sub> are equal area.



Spectral Displacement

Fig. 5.2 Bilinear representation of capacity spectrum for capacity spectrum method

To construct the bilinear representation draw one line up from the origin at the initial stiffness of the building. Draw a second line back from the trial performance point,  $a_{pi}$ ,  $d_{pi}$ . Slope the second line such that when it intersects the first line, at point  $a_{y}$ ,  $d_{y}$ , the area designated  $A_1$  in the figure is approximately equal to the area designated  $A_2$ . The intent of setting area  $A_1$  equal to area  $A_2$  is to have equal area under the capacity spectrum and its bilinear representation, that is, to have equal energy associated with each curve.

#### **5.6 EFFECTIVE DAMPING DERIVATION**

The damping that occurs when earthquake ground motion moves a structure into the inelastic range can be viewed as a combination of damping that is inherent in the structure and hysteretic damping. Hysteretic damping is related to the area inside the loops as shown in Fig.5.3 that are formed when the earthquake force (base shear) is plotted against the structure displacement. Hysteric damping can be represented as equivalent viscous damping using equations that are available in the literature.



Fig.5.3 One cycle of motion and energy dissipation in form of hysteresis loop

The equivalent viscous damping,  $\beta_{eq}$  associated with a maximum displacement of  $d_{pi}$  can be estimated from the following equation,

 $\beta_{eq} = \beta_0 + 0.05 \text{ or } 0.02$ 

Where,

 $\beta_0$  = Hysteretic damping represented as equivalent viscous damping

0.02 or 0.05 = 2% or 5% viscous damping inherent in the steel or concrete respectively

$$\beta_0 = \frac{E_D}{4\pi E_0} \tag{5.1}$$

Where:

 $E_D$  = Energy dissipated by damping

 $E_{so}$  = Maximum strain energy

The terms ED and Eso in the above equation are illustrated in the Fig.5.3 .

 $E_D$  is the energy dissipated by the structure in a single cycle of motion,

i.e. the area enclosed by a single hysteretic loop.

 $E_{so}$  is the maximum strain energy associated with that cycle of motion,

i.e. the area of the hatched triangle.



Fig.5.4 Effective damping energy in on cycle of motion

Referring the Fig.5.4 ,  $E_D$  can be derived as  $E_D = 4x(\text{shaded area in Fig. 5.4})$   $E_D = 4(a_{pi}d_{pi} - 2A_1 - 2A_2 - 2A_3)$   $= 4[a_{pi}d_{pi} - a_yd_y - 2d_y(a_{pi} - a_y) - (d_{pi} - d_y)(a_{pi} - a_y)]$  $= 4(a_yd_{pi} - d_ya_{pi})$ 

Also referring to the Fig.5.4,  $E_{so}$  can be derived as follow

$$E_{so} = \frac{a_{pi}d_{pi}}{2} \qquad \dots (5.2)$$

$$\beta_0 = \frac{1}{4\pi} \frac{4(a_y d_{pi} - d_y a_{pi})}{\frac{a_{pi} d_{pi}}{2}} = \frac{2}{\pi} \frac{a_y d_{pi} - d_y a_{pi}}{a_{pi} d_{pi}} \qquad \dots (5.3)$$

$$\beta_0 = \frac{0.637(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} \qquad \dots (5.4)$$

And when  $\beta_0$  is written in terms of percent critical damping, the equation will become,

$$\beta_0 = \frac{63.7(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} \qquad \dots (5.5)$$

The final equation after simplification for concrete building will become,

$$\beta_{eff} = \beta_0 + 5 = \frac{63.7(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 5 \qquad \dots (5.6)$$

The existing reinforced concrete buildings are not typically ductile structures. For such buildings, calculation of the equivalent viscous damping using equation 5.6 and the idealized hysteresis loop in Fig.5.3 yields results that overestimate realistic levels of damping. In order to be consistent with these previously developed damping coefficients, as well as to consider imperfect hysteresis loops (loops reduced in area), the concept of effective viscous damping using a damping modification factor,  $\kappa$ , has been introduced. Effective viscous damping,  $\beta_{eff}$ , is defined by:

$$\beta_{eff} = \kappa \beta_0 + 5 = \frac{63.7\kappa (a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 5 \qquad \dots (5.7)$$

The  $\kappa$  factor depends on the structural behavior of the building, which in turn depends on the quality of the seismic resisting system and the duration of ground shaking. The ranges and limits for the values of  $\kappa$  assigned to the three structural behavior types are given in Table 5.1 and also classification of building based upon shaking duration is given in Table 5.2.

Structural Behavior Type	$\beta_0$ (percent)	К
Type A	≤ 16.25 > 16.25	$\frac{1.0}{1.13 - \frac{0.51(a_yd_{pi} - d_ya_{pi})}{a_{pi}d_{pi}}}$
Type B	≤25 > 25	$\frac{0.67}{0.845 - \frac{0.446(a_yd_{pi} - d_ya_{pi})}{a_{pi}d_{pi}}}$
Type C	Any value	0.33

Table 5.1 Values For Damping Modification Factor,  $\kappa$ 

Shaking	Essentially New	Average Existing	Poor Existing
duration	Building	Building	Building
Short	Type A	Type B	Type C
Long	Туре В	Type C	Type C

Table 5.2 Structural behaviour types

## **5.7 CONSTRUCTION OF DEMAND SPECTRUM**

To convert a spectrum from standard  $S_a$  Vs T format found in the building code to ADRS (Acceleration-Displacement Response Spectrum) format, it is necessary to determine the value of  $S_{di}$  for each value of  $S_{ai}$  and  $T_i$ 



Fig.5.5 Code specified demand spectrum ( $S_a$  Vs T)



Fig. 5.6 Converted demand spectrum ( $S_a Vs S_d$ )

In ADRS format, lines radiating from the origin have constant period. For point on the ADRS spectrum, the period T, can be computed using relationship given in eq.5.8. The  $S_a$  versus  $S_d$  (ADRS) representation as shown in Fig.5.6 of the

response spectra is less familiar compared to the traditional  $S_a$  versus T representation (Fig.5.5).

The effective viscous damping values obtained from Eq. 5.7 can be used to estimate spectral reduction factors. As shown in the Fig. 5.7 the spectral reduction factors are used to decrease the elastic response spectrum to a reduced response spectrum with damping greater than 5% of the critical damping.





The equations for the reduction factors  $SR_{\text{A}}$  and  $SR_{\text{v}}$  are given by:

$$SR_{A} = \frac{3.21 - 0.68 \ln(\beta_{eff})}{2.12} \qquad ...(5.9)$$
  

$$\geq Value \text{ in Table 5.3}$$

$$SR_V = \frac{2.31 - 0.41 \ln(\beta_{\rm eff})}{1.65} \qquad \dots (5.10)$$

### ≥Value in Table 5.3

Table 5.3 Minimum allowable  $SR_A$  and  $SR_v$  values

Structural Behaviour Type	$SR_A$	$SR_V$
Type A	0.33	0.50
Type B	0.44	0.56
Type C	0.56	0.67

The shape of the demand spectrum with 5% damping is controlled by the input values in the Seismic Coefficient  $C_a$ , and Seismic Coefficient  $C_v$  In ATC-40 [11], Chapter 4, an explanation of, appropriate values for,  $C_a$  and  $C_v$  are explained.

The default value for both  $C_a$  and  $C_v$  is 0.4. Demand spectra for other damping levels are created from the 5% damped spectrum using the spectral reduction factors described in Section 8.2.2.1.1 of ATC-40.

## **5.8 CONSTRUCTION OF CAPACITY SPECTRUM**

In order to construct the capacity spectrum from the pushover curve, it is necessary to do a point-by-point conversion to the first mode spectral coordinates. Any point V<sub>i</sub> and  $\delta_{roof}$  on the capacity curve as shown in Fig.5.8 is converted to the corresponding point S<sub>ai</sub> and S<sub>di</sub> on the capacity spectrum as shown in Fig.5.9 using the Eq. 5.11,

$$S_a = \frac{V_W}{\alpha_1} \qquad \qquad S_d = \frac{\Delta_{roof}}{PF_1\phi_{roof,1}} \qquad \qquad \dots (5.11)$$

Where :

 $\alpha_1$  =modal mass coefficient for the first natural mode.

$$\alpha_{1} = \frac{\left[\sum_{i=1}^{N} (w_{i}\phi_{i1})/g\right]^{2}}{\left[\sum_{i=1}^{N} w_{i}/g\right]\left[\sum_{i=1}^{N} (w_{i}\phi_{i1}^{2})/g\right]} \dots (5.12)$$

PF<sub>1</sub>=modal participation factor for the first natural mode

$$PF_{1} = \frac{\left| \sum_{i=1}^{N} (w_{i}\phi_{i1})/g \right|}{\left[ \sum_{i=1}^{N} (w_{i}\phi_{i1}^{2})/g \right]} \dots (5.13)$$

Where,

 $\phi_{roof}$  is the roof level amplitude of the mode under consideration.

 $\frac{w_i}{g}$  is mass assigned to level i.

 $\phi_{i1}$  is amplitude of model 1 at level i.

N is level which is the uppermost of the structure.

V is base shear.

W is building dead weight plus likely live loads.

 $\Delta_{roof}$  is roof displacement (V and the associated  $\Delta_{roof}$  make up points on the

. . .

capacity curve)

S<sub>a</sub> is Spectral acceleration

 $S_d$  is spectral displacement ( $S_a$  and associated  $S_d$  make up points on the capacity spectrum).



Fig.5.8 Global capacity curve of the building (V Vs  $\Delta$ )



Fig.5.9 Converted capacity spectrum (S<sub>a</sub>Vs S<sub>d</sub>)

## **5.9 PUSHOVER ANALYSIS PROCEDURE**

The step by step procedure followed in the pushover analysis is presented in this section. Steps 1 through 4 discuss about creating the computer model, steps 5 runs analysis, and steps 6 and 7 review the pushover analysis result.

- 1. Create the basic computer model (without the pushover data) in the usual manner. The graphical interface of ETABS makes this a quick and easy task. The modeling procedure is described in 3.5.2.
- 2. Define properties and acceptance criteria for the pushover hinges. The program includes the several built-in default hinge properties that are based on average values from ATC-40 [11] for concrete members and average values from FEMA-273 [12] for steel members. These built in properties can

be useful preliminary analyses, but user-defined properties are recommended for final analyses.

- Locate the pushover hinges on the model by selecting one or more frame members and assigning them one or more hinge properties and hinge locations.
- 4. Define the pushover load cases. In ETABS more than one pushover load case can be run in same analysis. Also a pushover load case can start from the final conditions of another pushover load case that was previously run in the same analysis. Typically the first pushover load case is used to apply gravity load and then subsequent lateral pushover load cases are specified to start from the final conditions of the gravity pushover. Pushover load cases can be force controlled, that is, pushed to a certain defined force level, or it can be displacement controlled, that is, pushed to a specified displacement. Typically a gravity load pushover analysis is force controlled and lateral pushover analysis are displacement controlled. ETABS allows the distribution of lateral force used in pushover to be based on a uniform acceleration in a specified direction, a specified mode shape, or a user-defined static load case.
- 5. Run the basic static analysis and, if desired, dynamic analysis. Then run the static nonlinear pushover analysis.
- 6. Display the pushover curve and the table that gives the coordinates of each step of the pushover curve and summarize the number of hinges in each state as defined in Fig. 5.1
- Review the pushover displaced shape and sequence of hinge formation on a step-by-step basis. Hinges appear when they yield and are color-coded based on their state.

## **5.10 LIMITATIONS OF PUSHOVER ANALYSIS**

Pushover analysis procedure is simple method of non-linear analysis of buildings and it has to be used with discretion and judgment. It is not adequate to address the complexity of the three-dimensional non-linear dynamic behavior.

Following are some limitations of pushover analysis.

1. Pushover analysis procedure implies that there is separation between the structural capacity and the earthquake demand. It is incorrect to assume that there exists a unique, intrinsic structural capacity of the earthquake demand.

This is primarily because on of non-linear structural behavior is load path dependent, and it is not possible to separate the loading input from the structural responses.

- 2. Pushover analysis procedure implicitly assumes that damage is a function only of the lateral deformation of the structure, neglecting duration effects, number of stress reversals and cumulative energy dissipation demand. It is generally accepted that damage of a structure is a function of both side deformation and energy [29]. The applicability of the proposed measure of damage is too simplistic, particularly for non-ductile structures whose inelastic cyclic behaviours are severally pinched and erratic.
- 3. Pushover analysis is a static analysis and neglects dynamics. During an earthquake the behavior of a non-linear yielding structure can be described by balancing the dynamic equilibrium at every time step. By focusing only on the strain of the structure during a monotonic static push, the procedure can leave a misleading impression that energy associated with the dynamic components of forces i.e. energy and viscous damping energy are significant. The energy of the structures during an earthquake is given as,

$$E_{l} = E_{k} + E_{s} + E_{v} + E_{d}$$
 ... (5.14)

Where:

 $E_{I}$  = earthquake input energy

 $E_k$  = kinetic energy

 $E_s = strain energy$ 

 $E_s$  = viscous damping energy

 $E_d$  = controlled damping energy

- 4. The procedure considers only the lateral earthquake loading and a vertical component of earthquake loading is ignored.
- 5. The procedure is based on the assumption that the response of the structure can be related to response of an equivalent single degree of freedom system. This implies that the response is controlled by a single mode, and that the shape of this mode remains constant throughout the time history response.

- 6. Pushover analysis procedure does not take into account for the progressive changes in the or modal properties that take place in a structure as it experience cyclic non-linear yielding during an earthquake. It is difficult to discuss modal properties, which are linear, of a structure that experience significantly non-linearity.
- 7. Pushover analysis fails to produce good correlation for earthquake with predominantly impulsive ground motions.
- 8. Pushover analysis gives a reasonably accurate estimate for strength of the structural frame, assuming that its elements do not fail due to secondary effects (e.g. buckling of columns) before the plastic mechanism occurs.

### 5.11 STRUCTURAL APPLICALICATIONS OF PUSHOVER ANALYSIS

Pushover analysis is very helpful to study failure pattern of the structure and capacity of the structure. There are generally main two applications, first design of new structure for desired failure patterns (performance based design) and second retrofitting of existing structure.

#### 5.11.1 Performance based design

For design of new structure, capacity of structure during preliminary stage is found out through pushover analysis. After knowing failure pattern of structure, preliminary cross section can be modified for desired failure pattern of the structure. Then further check of failure pattern is done through pushover analysis.

This processes continued upto desired failure pattern of structure is not archived. One example of 10 storey building is presented to understand this concept. Three dimensional 10 storey model of coupled shear wall building having dimensions as shown in Table 1.1 and 1.2 is considered. Plan of building is shown in Fig. 1.7. Failure pattern of this model is shown in Fig.5.10. In this model beams failed first.



Fig.5.10 Failure of 10 storey model at failure of first element

If desired failure pattern is to be obtained such that, instead of failure of beams of building, coupling beam should fail first different dimensions of beam and column are considered. The dimensions of beams and columns are modified from  $0.25 \times 0.75$  and  $0.45 \times 0.45$  to  $0.30 \times 0.75$  and  $0.55 \times 0.55$  m respectively.

Failure pattern of modified structure is shown in Fig.5.11. It is found that coupling beam failed first. Thus, through controlling stiffness of structural elements desired failure sequence of elements and numbers of hinges can be controlled.



Fig.5.11 Faiure of 10 storey model after revising stiffness

## 5.11.2 Retrofitting

The seismic evaluation and retrofit of existing concrete buildings is a great challenge for any structural engineering. The risks, measured in both lives and money, are high. Equally high is the inevitable uncertainty of future earthquake. The inherent complexity of concrete buildings and of their performance during earthquakes increases the uncertainty. Traditional design and analysis procedures developed primarily for new construction are not adequate tools for meeting this challenge.

Actual strength of designed building can be evaluated through pushover analysis. This strength is compared with real strength of that existing building. And based on that comparison, amount of retrofitting is decided. The actual seismic performance of buildings is extremely sensitive to construction quality. This is especially true for retrofitted buildings. Field conditions in existing buildings routinely vary form those shown on drawings or implied by visual inspections during the evaluation process. These changes can have significant impact on retrofit designs. Some retrofit techniques are sophisticated and require special inspections and tests. It is very important that the engineer prepare a projectspecific construction quality assurance plan. The engineer in charge should make regular inspections and receive immediate notification of any field problems [11].

#### 5.12 PUSHOVER ANALYSIS OF CSW FOR 2D AND 3D MODEL

Static nonlinear analysis is carried out using ETABS software for 2D and 3D model. Modeling is done as discussed in chapter 3 (section 3.5). Geometry for models are considered as shown in Fig.1.7 and element dimensions are considered for 10 storey building as shown in Table 1.2. Step wise loading increments are applied under PUSH 1 and PUSH 2 considering gravity and seismic load cases respectively. Push over analysis is carried out for two different 2D as well as 3D coupled shear wall building models. In one model, wall and coupling beam is assigned with shear hinge property and in another model they are assigned with flexural hinge property. Calculation of flexural hinge property is further discussed in chapter 7.

After performing nonlinear static analysis roof displacement and base shear are obtained from output file of ETABS. The capacity curve in case of flexural hinge is shown in Fig.5.10 (a) and that in case of shear hinge is shown in Fig.5.10(b).



Fig. 5.12 Comparison of capacity curve for 2D and 3D

It is observed that the displacement and base shear values of 2D model are less than 3D model. 3D model is stiffer compared to 2D model because orthogonal members connectivity at joint adds stiffness to the model. Pattern of graph for 2D as well as 3D models are same in nature.

Stages of hinge formation for flexure and shear hinges are presented for 2D model and 3D model in Table 5.4 and 5.5 respectively. From the results of nonlinear analysis it is observed that in flexure and shear hinge condition capacity curve - Base shear Vs. Roof displacement - are nearly same for 2D and 3D models.

	2-D Mo	del		3-D Model					
	Roof	Base			Roof	Base			
Steps	Displacement	shear	Stage	Steps	Displacement	shear	Stage		
	(m)	(kN)			(m)	(kN)	Ũ		
0	0	0		0	0	0			
1	0.0064	158	В	1	0.0073	168	В		
2	0.0098	209.8	IO	2	0.0101	225.4	IO		
3	0.0102	234	LS	3	0.0118	247	LS		
4	0.0105	238	CP	4	0.0113	245.6	СР		
5	0.0116	242.8	С	5	0.0121	249.2	С		
6	0.0116	119.8		6	0.0121	129.7			
7	0.0118	107.9		7	0.0124	115.8			
8	0.0119	108.6		8	0.0125	118.6			
9	0.0122	98.6		9	0.0126	114.3			
10	0.0127	93.3	р	10	0.0128	115.8	Л		
11	0.0132	98.67	D	11	0.0131	113.4	D		
12	0.013	99.4		12	0.0135	108.7			
13	0.0134	96.4		13	0.0138	101.2			
14	0.0156	98.9		14	0.0154	105.6			
15	0.0176	92.2		15	0.0184	99.2			
16	0.0163	89.9	Е	16	0.0174	97.6	Е		

Table 5.4 Stages of Hinge formation in 2D model and 3D model-Flexural hinge

Table 5.5 Stages of Hinge formation in 2D model and 3D model- Shear hinge

	2-D Mod	lel		3-D Model					
	Roof	Base			Roof	Base			
Steps	Displacement	shear	Stage Steps I		Displacement	shear	Stage		
	(m)	(kN)			(m)	(kN)	-		
0	0	0	D	0	0	0	D		
1	0.0024	69	Б	1	0.0039	88.9	D		
2	0.0066	145.9	IO	2	0.0083	169.1	IO		
3	0.0054	68	15	3	0.0083	82.9	IS		
4	0.0065	72	LS	LS 4 0.0085		85.2	LS		
5	0.006	70	CP	5	0.0085	83.9	СР		
6	0.0078	72	С	6	0.0086	84.9	С		

7	0.0081	2.9		7	0.0087	5.3	
8	0.0141	29.7		8	0.0164	42.6	
9	0.0153	37.6		9	0.0187	41.6	
10	0.016	42.7		10	0.0188	52	
11	0.0179	43.6	D	11	0.0199	51	D
12	0.0186	49.9	D	12	0.0188	56.1	D
13	0.0193	46.4		13	0.0199	54.3	
14	0.0191	58.7		14	0.0224	65.4	
15	0.0196	44.9		15	0.0224	63.7	
16	0.0239	68.4		16	0.028	88.8	
17	0.0421	102.7	Б	17	0.0453	119.4	Б
18	0.0423	0	E	18	0.0453	0	E

Formation of shear hinges at step no. 11 for 2 D model is shown in Fig. 5.13 and 3 D model is shown in Fig. 5.14.



Fig. 5.13 2D shear hinge- failure pattern of wall-step 11



Fig.5.14 3D shear hinge- failure pattern of wall-step 11

As shown in the Fig. 5.11 and 5.12, the assigned hinges start yielding at a particular value of displacement. Blue colored hinges indicate the yielding has been reached, green colour indicate the life safty level has been reached, yellow colour indicate the collapse prevention level has been reached and red colour indicate that the section has failed.

From the observation of failure pattern, it is found that the failure is not concentrated in one part of structure, but failure mechanism is distributed in most of floor beams. After the failure of all beams, columns (wall) come in to the picture to make complete failure. So failure pattern followed strong column-weak beam concept.

## 5.12.1 Derivation of Performance Point

For finding performance point, demand curve and capacity curve are required. Method for draw the demand spectrum and capecity spectrum is disscused in section 5.7 and 5.8 respectively. Pushover curve can be converted in ADRS formate through value of spectral displacement and spectral accelaresion, which can be obtained by Eq. 5.11. Table 5.6 shows various values to convert pushover (capacity curve) of 10 storey 3D building (considering shear hinge) in

to ADRS format. Three values can be calculated by using formulas given in section 5.6, 5.7 and 5.8.

Step	Teff	ßeff	Sd(C)	Sa(C)	Sd(D)	Sa(D)	ALPHA	PF*Ø
0	1.086	0.05	0	0	0.108	0.368	1	1
1	1.086	0.05	8.33E-03	0.028	0.108	0.368	0.689	1.441
2	1.086	0.05	0.017	0.057	0.108	0.368	0.689	1.441
3	1.086	0.05	0.025	0.085	0.108	0.368	0.689	1.441
4	1.086	0.05	0.031	0.107	0.108	0.368	0.689	1.441
5	1.094	0.054	0.04	0.134	0.106	0.358	0.688	1.442
6	1.108	0.062	0.048	0.158	0.104	0.341	0.687	1.443
7	1.123	0.07	0.057	0.181	0.102	0.327	0.685	1.445
8	1.141	0.077	0.065	0.201	0.101	0.313	0.683	1.446
9	1.162	0.086	0.073	0.219	0.1	0.298	0.681	1.449
10	1.183	0.094	0.083	0.238	0.099	0.285	0.678	1.452
11	1.2	0.098	0.091	0.255	0.099	0.277	0.677	1.454
12	1.216	0.101	0.099	0.271	0.1	0.271	0.675	1.456
13	1.23	0.104	0.108	0.288	0.1	0.266	0.674	1.457
14	1.244	0.106	0.118	0.306	0.101	0.262	0.673	1.459
15	1.258	0.109	0.126	0.32	0.101	0.257	0.673	1.46
16	1.258	0.109	0.126	0.32	0.101	0.256	0.673	1.46
17	1.3	0.13	0.136	0.324	0.098	0.235	0.682	1.461
18	1.339	0.148	0.146	0.329	0.097	0.219	0.689	1.461

Table 5.6 Reduced spectrum information for 3D 10 storey building

Stages of hinge formation for three dimensional 10 storey building considering shear hinge is shown in Table 5.7. Step number 0 and 20 represent the loading and unloading stage respectively. Total numbers of hinges remain constant for particular building.

Step	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	TOTAL
0	1740	0	0	0	0	0	0	0	1740
1	1740	0	0	0	0	0	0	0	1740
2	1740	0	0	0	0	0	0	0	1740
3	1738	2	0	0	0	0	0	0	1740
4	1699	41	0	0	0	0	0	0	1740
5	1664	76	0	0	0	0	0	0	1740
6	1627	113	0	0	0	0	0	0	1740

Table 5.7 Step vise hinge formulation for 3 D 10 storey building

7	1546	194	0	0	0	0	0	0	1740
8	1503	237	0	0	0	0	0	0	1740
9	1481	229	30	0	0	0	0	0	1740
10	1453	219	68	0	0	0	0	0	1740
11	1422	248	70	0	0	0	0	0	1740
12	1407	234	59	40	0	0	0	0	1740
13	1399	218	53	70	0	0	0	0	1740
14	1387	216	67	70	0	0	0	0	1740
15	1385	218	67	70	0	0	0	0	1740
16	1371	215	74	80	0	0	0	0	1740
17	1355	224	81	80	0	0	0	0	1740
18	1740	0	0	0	0	0	0	0	1740

The procedure for calculation of performance point is discussed in section 5.4. After performing static nonlinear anlysis through ETABS sofware, performance point is obtained as shown in Fig.5.15.



Fig.5.15 Capacity spectrum & Performance Point

Hence, for 3D CSW building the acceleration and displacement at performance point is obtained in ADRS form. This spectral acceleration and displacement are converted in to base shear and displacement at performance point. Thus base shear and roof displacement at the performance point are obtained. Performance point represent intersection of structure capacity and seismic demand.

#### 5.12.2 Performance check for CSW

The performance of coupled shear wall building is checked at the maximum displacement.

- The lateral load resistance of the building system, including resistance to the effect of gravity load acting through lateral displacements, should degrade by more than 20 percent of the maximum resistance of the structure. For the CSW, there is no strength degradation. The pushover curve does not degrade before the peak value. Hence, this check complies in present example.
- 2. The lateral deformation at the performance point should not exceed the allowable maximum total dirft limit. Maximum total drift is defined as the interstory drift at the performance point as shown in Table 5.8 for 10 storey building.

Storey	Max. total drift	Allowable Max. total drift
1	0.015	0.12
2	0.013	0.12
3	0.012	0.12
4	0.010	0.12
5	0.08	0.12
6	0.05	0.12
7	0.003	0.12
8	0.002	0.12
9	0.001	0.12
10	0.001	0.12

Table 5.8 Comparison of actual and permissible deformation Limit

#### **5.13 PARAMETRIC STUDY ON FAILURE PATTERNS AND DUCTILITY**

In order to have ductile failure of building and to avoid sudden collapse of building beam should fail prior to columns. Dimensions of structural member affect failure pattern. In order to understand the effect of dimension of the various structural elements on failure pattern parametric study is carried out. Failure patterns of different elements of coupled shear wall building are studied by considering different parameters as coupling beam depth, coupling beam span, wall length and numbers of storey.

#### 5.13.1 Coupling beam depth as parameters

When beam depth is considered as parameter, the results are summarized in Table 5.9. The coupling beam with depth 0.5m is failing in shear for 10 and 15-storey buildings but in other cases the beam is failing in flexure. In this study, the wall length is kept as 3.0m.

									perfo	At				
					Fail	ure of	first elen	nent	peric n	oint		Rem	ark	
Column c/s	D <sub>w</sub> (m)	H <sub>b</sub> (m)	L <sub>b</sub> (m)	T <sub>h</sub> (m)	Type of element	$\Delta$ (m)	Base shear (kN)	Storey Location	Δ (m)	Base shear (kN)	Failure of Coupling beam	Failure sequence of Coupling beam	Failure of Wall	Failure sequence of Wall
								10 storey	7					
0.45×0.45	3	0.5	1	0.25	beam	0.153	4625.23	2to6	0.15	4592.50	Shear	First	Flexure	Second
0.45×0.45	3	0.75	1	0.25	beam	0.17	4721.74	2to4	0.11	4629.36	Flexure	First	Flexure	Second
0.45×0.45	3	1	1	0.25	beam	0.2	4813.54	1&2	0.10	4759.36	Flexure	First	Flexure	Second
		1		1	1	1		15 storey	7			r	1	1
0.5×0.5	3	0.5	1	0.3	c.b.	0.17	5147.2	2to5	0.16	4924.56	Shear	First	Shear	Second
0.5×0.5	3	0.75	1	0.3	beam	0.17	5241.11	2to5	0.13	5047.36	Flexure	First	Shear	Second
0.5×0.5	3	1	1	0.3	beam	0.16	5414.10	2&3	0.11	5245.36	Flexure	First	Shear	Second
	1			1		1	1	20 storey	7	1		1		
0.6×0.6	3	0.5	1	0.3	c.b. & beam	0.21	5641.58	8to14	0.17	5426.63	Flexure	First	Shear	Second
0.6×0.6	3	0.75	1	0.3	c.b. & beam	0.17	5796.36	4to6	0.14	5625.14	Flexure	First	Shear	Second
0.6×0.6	3	1	1	0.3	c.b. & beam	0.16	5842.94	2to4	0.13	5796.52	Flexure	First	Shear	Second
								25 storey	7					
0.7×0.7	3	0.5	1	0.35	c.b. & beam	0.13	6198.25	9to16	0.12	6025.45	Flexure	First	Shear	Second
0.7×0.7	3	0.75	1	0.35	c.b. & beam	0.16	6325.25	6to13	0.14	6234.56	Flexure	First	Shear	Second
0.7×0.7	3	1	1	0.35	beam	0.17	6589.14	2to5	0.16	6425.23	Flexure	First	Shear	Second
					I.			30 storey	7					I
0.8×0.8	3	0.5	1	0.35	c.b. & beam	0.22	6742.12	14to20	0.21	6874.23	Flexure	First	Shear	Second
0.8×0.8	3	0.75	1	0.35	c.b. & beam	0.22	7258.71	12to18	0.18	7045.56	Flexure	First	Shear	Second
0.8×0.8	3	1	1	0.35	c.b. & beam	0.19	7412.65	8to11	0.15	7256.54	Flexure	First	Shear	Second

The walls are failing in flexure for 10-storey CSW and in shear in case of other storey. In case of flexural failure of walls, the hinges are formed at the base of the wall. It is observed that the walls are failing after the failure of coupling beam. Displacement at performance point decreases, when depth of coupling beam is increased, This indicates reduction in ductility with increase in depth of coupling beam.

## 5.13.2 Coupling beam span as parameter

The failure patterns of CSW are summarized in Table 5.10 when coupling beam span is considered as parameter. By increasing the span of coupling beam, the failure pattern of 0.5 m deep coupling beam is changed from shear to flexure for 10-storey CSW. By increasing span the failure pattern of wall tends to shear mode. At performance point displacement increase, with increase in span, which indicate increase in overall structural ductility.

									At pe	rformance				
					Fai	lure o	f first eler	nent		point		Rem	ark	
Column	D	Н	L	Т							Failure	Failure		
cls	(m)	<sub>b</sub> (m)	_р (m)	(m)	Type of	FΛ	Base	Storey	۸	Base	of	sequence	Failure	Failure
0/5	(111)	(111)	(111)	(111)	elemen	t (m)	shear	Location	(m)	shear (kN)	Counling	of	of Wall	sequence
					ciemen		(kN)	Location	(III)	shear (Krv)	beam	Coupling	or wan	of Wall
											ocam	beam		
								10 store	y					
0.45×0.45	3	0.5	1	0.25	beam	0.153	4625.23	2to6	0.15	4592.50	Shear	First	Flexure	Second
0.45×0.45	3	0.5	2	0.25	c.b.	0.17	5087.75	1to7	0.16	5051.75	Shear	First	Flexure	Second
0.45×0.45	3	0.5	3	0.25	c.b.	0.2	5596.52	2to10	0.19	5556.92	Flexure	First	Flexure	Second
								15 store	y					
0.5×0.5	3	0.5	1	0.3	c.b.	0.17	5147.2	2to5	0.16	4924.56	Shear	First	Shear	Second
0.5×0.5	3	0.5	2	0.3	c.b.	0.19	5661.92	3to17	0.17	5417.01	Shear	First	Shear	Second
0.5×0.5	3	0.5	3	0.3	c.b.	0.21	6228.11	4to12	0.18	5958.71	Flexure	First	Shear	Second
								20 store	y					
					c.b. &									
0.6×0.6	3	0.5	1	0.3	beam	0.21	5641.58	8to14	0.17	5426.63	Flexure	First	Shear	Second
					1 0									
					c.b. &									
0.6×0.6	3	0.5	2	0.3	beam	0.19	6205.73	6to17	0.18	5969.29	Shear	First	Shear	Second
					c.b. &									
0.6×0.6	3	0.5	3	0.3	beam	0.22	6826.31	3to11	0.19	6566.22	Shear	First	Shear	Second
								25 store	y					
					c.b. &									
0.7×0.7	3	0.5	1	0.35	beam	0.13	6198.25	9to16	0.12	6025.45	Flexure	First	Shear	Second
					a <b>b</b> 0-									
					c.o. a									
0.7×0.7	3	0.5	2	0.35	beam	0.18	6818.07	10to18	0.15	6627.99	Shear	First	Shear	Second
					c.b. &									
0.7×0.7	3	0.5	3	0.35	beam	0.21	7499.88	2to14	0.16	7290.79	Shear	First	Shear	Second
								30 store	v	I		1		
					c.b. &				2					
0.8×0.8	3	0.5	1	0.35	beam	0.22	6742.12	14to20	0.21	6874.23	Flexure	First	Shear	Second
	-													
					c.b. &									
0.8×0.8	3	0.5	2	0.35	beam	0.24	7416.33	15to21	0.22	7561.65	Flexure	First	Shear	Second
					c.b. &									
0.8×0.8	3	0.5	3	0.35	beam	0.26	8157.96	16to22	0.23	8317.81	Shear	First	Shear	Second

Table 5.10 Coupling beam span as parameters

#### 5.13.3 Wall length as parameter

From Table 5.11, the study of failure pattern for wall length parameter has been carried out. In this case the depth and span of coupling beams are kept constant as 0.5 m and 1 m respectively, wall lengths are varying from 3 m to 5 m. By increasing the wall length, the failure pattern of 0.5 m deep coupling beam is changed from shear to flexure.

Table 5.11	Wall length	as parameters
------------	-------------	---------------

								At performance						
					Fai	lure o	f first eler	nent		point		Rem	ark	
Column c/s	D <sub>w</sub> (m)	H <sub>b</sub> (m)	L <sub>b</sub> (m)	T <sub>h</sub> (m)	Type of element	Δ (m)	Base shear (kN)	Storey Location	Δ (m)	Base shear (kN)	Failure of Coupling beam	Failure sequence of Coupling beam	Failure of Wall	Failure sequence of Wall
								10 stor	ey					
$0.45 \times 0.45$	3	0.5	1	0.25	beam	0.15	4625.23	2to6	0.15	4592.50	Shear	First	Flexure	Second
$0.45 \times 0.45$	4	0.5	1	0.25	c.b.	0.16	5781.53	2to9	0.12	5740.62	Flexure	First	Shear	Second
$0.45 \times 0.45$	5	0.5	1	0.25	c.b.	0.18	7226.92	1to8	0.10	7175.78	Flexure	First	Shear	Second
								15 stor	ey					
0.5×0.5	3	0.5	1	0.3	c.b.	0.17	5147.2	2to5	0.16	4924.56	Shear	First	Shear	Second
0.5×0.5	4	0.5	1	0.3	c.b.	0.19	6434.34	8to12	0.13	6155.70	Flexure	First	Shear	Second
0.5×0.5	5	0.5	1	0.3	c.b.	0.2	8042.50	5to15	0.10	7694.62	Flexure	First	Shear	Second
20 storey														
					c.b. &									
0.6×0.6	3	0.5	1	0.3	beam	0.21	5641.58	8to14	0.17	5426.63	Flexure	First	Shear	Second
0.6×0.6	4	0.5	1	0.3	c.b.	0.22	7051.97	11to18	0.18	6783.28	Flexure	First	Shear	Second
0.6×0.6	5	0.5	1	0.3	c.b.	0.24	8814.96	12to19	0.19	8479.10	Flexure	First	Shear	Second
		r				r		25 stor	ey		1			
					c.b. &									
0.7×0.7	3	0.5	1	0.35	beam	0.13	6198.25	9to16	0.12	6025.45	Flexure	First	Shear	Second
0.7×0.7	4	0.5	1	0.35	c.b.	0.15	7747.81	7to15	0.16	7531.81	Flexure	First	Shear	Second
0.7×0.7	5	0.5	1	0.35	c.b.	0.17	9684.76	5to20	0.12	9414.76	Flexure	First	Shear	Second
								30 store	ey					
					c.b. &									
0.8×0.8	3	0.5	1	0.35	beam	0.22	6742.12	14to20	0.21	6874.23	Flexure	First	Shear	Second
0.8×0.8	4	0.5	1	0.35	c.b. & beam	0.21	8427.65	14to25	0.17	8592.78	Flexure	First	Shear	Second
0.8×0.8	5	0.5	1	0.35	c.b. & beam	0.22	10534.56	9to22	0.13	9523.54	Flexure	First	Shear	Second

The walls are failing in shear predominately with increase in wall length. Due to increase of in plane stiffness of wall displacement at performance point decreases.

## 5.14 Summary

Theoritical background for pushover analysis is discussed in this chapter. The main applications of pushover analysis in performance based design and retrofitting works is discussed. One example is considered for 2D and 3D analysis in ETABS. The results of nonlinear analysis shows that in flexure and shear hinge condition capacity curve (Base shear Vs. Roof displacement) are nearly same for 2D and 3D models. Base shear as well as roof displacemet values are more in case of 3 D model compare to 2 D model because 2D model is more flexible compare to 3D model. Base shear capacity of structure is reduced after forming hinges.

The nonlinear behaviour of CSW is greatly influenced by the wall length, span and depth of coupling beam. With increasing the depth of coupling beam, the failure pattern of coupling beam changes from shear to flexure. In 10 and 15storey CSW, with increase in span of coupling beam, failure pattern of coupling beam changes from shear to flexure. In most of cases shear wall fail in shear mode. With increasing stiffness in structure ductility of structure is reduced.

# 6. EFFECT OF ORIENTATION OF COUPLED SHEAR WALL

#### **6.1 GENERAL**

Orientation of lateral load resisting elements is very important in performance of building under seismic condition. A simple and symmetrical structure, e.g., a square, will have the greatest chance of survival under seismic condition due to:

- (1) The ability to understand the overall earthquake behaviour of a structure. It is markedly greater for simple one than that for a complex one.
- (2) The ability to understand structural details. It is considerably greater for simple structures than it is for complicated ones.

Buildings regular in plan and elevation, without re-entrant corners or discontinuities in transferring the vertical loads to the ground, display good seismic behaviour. It is important that the plan of a structure is symmetrical in both directions. In general, buildings with simple geometry in plan as shown Fig.6.1 (a) perform well during earthquakes. Buildings with re-entrant corners such as U,V,T and + shapes in plan [(Fig. 6.1(b)], may sustain significant damage during earthquakes and should be avoided. H-shaped, although symmetrical, should not be encouraged either.



Ideal for behaviour and analysis

(a) Recommended



Differential behaviour on opposite behaviour prediction problem





prediction problem



Good symmetry, analysis easy



Through symmetrical, long wings give end of long building



Re-entrant corner poor detailing

(b) Not recommended

Fig. 6.1 Geometric plans for typical building

The probable reason for the damage is the lack of proper detailing at the corners, which is complex. To reduce the bad effects of these interior corners in the plan, the building can be broken into part using a separation joint at the junction. There must be enough clearance at the separation joints so that the adjoining portions do not pound each other. Fig. 6.2 shows such cases of elongated, L-shaped and H- shaped buildings [30].



Fig. 6.2 Bracken layout concept

The different orientation of coupled shear wall in same plan of building is studied through dynamic and pushover analysis in this chapter. The advantage of coupled shear wall is also discussed.

#### **6.2 DYNAMIC ANALYSIS**

The dynamic analysis is carried out for the three dimensional structure. For parametric study, coupled shear walls are oriented in different manners. The plan and other dimensions of the structure are considered as per section 1.7.3. Modeling is done as per section 3.3. Results of time period and base shear are shown in Table 6.1 and 6.2 respectively. Time period given by dynamic analysis for different models are also compared with IS: 1893 based formula.

		Time Period(Sec.)					
PLAN	No. of	As per	As per ETAB	S 3D model			
	storey	IS:1893	Analy	/\$1\$			
<b>→X</b>			EQ x	EQ <sub>Y</sub>			
7.00 7.00 7.00 7.00	10	0.59	1.62	0.65			
7.00	15	0.89	2.01	0.95			
	20	1.18	2.94	1.46			
7,00	25	1.47	3.05	1.73			
Parallel CSW	30	1.77	3.86	2.35			
7.00 7.00 7.00 7.00	10	0.59	0.67	0.67			
3.00 L 1.00	15	0.89	1.09	1.09			
3.00 7.00 	20	1.18	1.55	1.55			
Mid side CSW	25	1.47	2.09	2.09			
	30	1.77	2.52	2.52			

Table 6.1 Time period comparison

7.00 7.00 7.0	10	0.59	0.5	0.5
3.00 1.00 3.00	15	0.89	0.84	0.84
7.00	20	1.18	1.22	1.22
	25	1.47	1.74	1.74
Central CSW	30	1.77	2.7	2.7
7.00 7.00 7.0	10	0.59	0.63	1.41
7.00	15	0.89	0.95	2.14
2.50	20	1.18	1.33	2.40
7.90	25	1.47	1.84	3.02
Outer side CSW	30	1.77	2.27	3.42

From Table 6.1, it is observed that IS 1893 specifies same time period irrespective of orientation of coupled shear wall. Time period of centrally located CSW building is nearly same compared to IS 1893 specification for lower no. of storey. Top storey displacements are summarized in Table 6.2.

		Ba	Base Shear (kN)					
PLAN	No. of storey	As per	As per ET model Ai	ABS 3D nalysis	displacement (m)			
<b>→X</b>	storey	IS:1893	EQ x	EQ <sub>Y</sub>	EQ x	EQ y		
7.00 7.00 7.00 7.00	10	1433.29	554.84	1402.81	0.06	0.01		
	15	1483.75	576.41	1462.76	0.06	0.01		
300	20	1514.22	639.74	1501.19	0.08	0.03		
	25	1585.45	682.92	1570.41	0.09	0.03		
Parallel CSW	30	1625.33	720.59	1601.21	0.11	0.05		

Table 6.2 Base shear comparison

7.00 7.00 7.00	10	1433.29	1018.19	1018.19	0.01	0.01
2.00 1.00 2.00	15	1483.75	1093.41	1093.41	0.02	0.02
7.00 	20	1514.22	1122.54	1122.54	0.03	0.03
Mid side CSW	25	1585.45	1299.94	1299.94	0.04	0.04
	30	1625.33	1356.82	1356.82	0.05	0.05
7.00 7.00 7.0	10	1455.29	1483.27	1483.27	0.01	0.01
2,00 4,200-1 (-2,00-1) 3,00 1,00 3,00 1,00	15	1513.7	1542.52	1542.52	0.01	0.01
7,00	20	1584.22	1561.44	1561.44	0.02	0.02
Central CSW	25	1625.45	1575.71	1575.71	0.03	0.03
	30	1690.76	1651.80	1651.80	0.04	0.04
7.05 7.00 7.0 7.0	10	1433.29	1398.02	520.73	0.01	0.04
7.20	15	1483.75	1416.57	538.74	0.02	0.05
7,00	20	1514.22	1489.44	658.56	0.02	0.06
Outer side CSW	25	1585.45	1501.36	753.59	0.04	0.08
	30	1625.33	1589.04	847.47	0.05	0.09

From Table 6.2 it is observed that base shear specified by IS 1893 is more compared to that obtained by dynamic analysis for different location of CSW. In case of centrally located CSW, building base shear calculated from dynamic analysis is more compared to that of in CSW and column. Compared to that of IS 1893 specifications.

From study of top storey displacement, it is observed that, top storey displacement depends on orientation of coupled shear wall. Centrally located CSW has less displacement compared to other different types of orientation. In case of asymmetrically located coupled shear wall, top storey displacement is more in out of plane compared to in plane direction of coupled shear wall.

Y DI AN	No. of	Base Shear distribution (%)					
	No. of Storey	Eq.	Х	Ec	<u>а</u> . Ү		
→ <b>X</b>		Wall	Column	Wall	Column		
	10	46.40	53.60	96.81	3.19		
7.00 7.00 7.00	15	40.98	59.02	96.79	3.21		
3.00 1.00	20	38.07	61.93	94.30	5.70		
7.00	25	35.19	64.81	94.03	5.97		
Parallel CSW	30	32.30	67.70	91.45	8.55		
7.00 7.00 7.00 7.00	10	93.07	6.93	93.07	6.93		
3.00	15	92.31	7.69	92.31	7.69		
1.00	20	91.49	8.51	91.49	8.51		
Mid side CSW	25	86.56	13.44	86.56	13.44		
	30	84.78	15.22	84.78	15.22		

Table 6.3 Base shear distribution in CSW and column

7.00 7.00 7.0	10	95.38	4.62	95.38	4.83
7.00 	15	95.23	4.77	95.23	4.77
1.00 T 3.00 	20	95.17	4.83	95.17	4.83
7.00	25	94.35	5.65	94.35	5.65
Central CSW	30	94.29	5.71	94.29	5.71
7.00 7.00 7.0 7.0 7.0	10	97.01	2.99	42.73	57.27
7.00	15	96.86	3.14	41.10	58.90
7.00	20	95.36	4.64	40.84	59.16
	25	93.14	6.86	39.47	60.53
Outer side CSW	30	92.58	7.42	39.18	60.82

Study of base shear distribution shows that, central CSW building is more effective to resist base shear compared to other arrangements of CSW. Parallel CSW and outer side CSW building are not symmetric for distribution of base shear in both the directions. Numerical comparison for all types of arrangement is shown in Table 6.3.

## **6.3 STATIC NONLINEAR ANALYSIS**

Pushover analysis is carried out for different orientation of CSW, which are used in dynamic analysis study. From pushover analysis, failure patterns are studied considering coupling beam depth as parameter. Modeling for pushover analysis is done as discussed in section 3.5.2. The results of pushover analysis are observed in terms of : (i) Failure of first element and corresponding base shear and displacement

(ii) Base shear and displacement at performance point

(iii) Nature and sequence of failure of coupling beam and shear wall.

These results are shown in Table 6.4 to 6.7 for different orientation of CSW.

Different orientation of CSW considered are similar to that discussed in section 6.2 as:

- (a) Parallel CSW
- (b) Mid side CSW
- (c) Central CSW
- (d) Out side CSW

The length of wall is 3 m and span of coupling beam is 1 m for all models. Depth of coupling beam is veried from 0.5 to 1 m to understand effect of coupling beam depth on behaviour of CSW.

					Fai	lure o	f first eler	nent	perf	At ormance point		Rem	ark	
Column c/s	D <sub>w</sub> (m)	H <sub>b</sub> (m)	L <sub>b</sub> (m)	T <sub>h</sub> (m)	Type of element	Δ (m)	Base shear (kN)	Storey Location	Δ (m)	Base shear (kN)	Failure of Coupling beam	Failure sequence of Coupling beam	Failure of Wall	Failure sequence of Wall
								10 storey	/					
0.45×0.45	3	0.5	1	0.25	c.b.	0.16	3746.44	2to8	0.14	3570.37	Shear	First	Flexure	Second
0.45×0.45	3	0.75	1	0.25	beam	0.16	3748.07	3to6	0.14	3573.43	Flexure	First	Flexure	Second
0.45×0.45	3	1	1	0.25	beam	0.16	3748.89	3 & 4	0.14	3575.83	Flexure	First	Flexure	Second
15 storey														
0.5×0.5	3	0.5	1	0.3	c.b.	0.41	4650.28	3to7	0.27	3766.40	Shear	First	Shear	Second
0.5×0.5	3	0.75	1	0.3	beam	0.41	4652.65	2to5	0.27	3770.65	Flexure	First	Shear	Second
0.5×0.5	3	1	1	0.3	beam	0.41	4653.92	4to6	0.27	3773.80	Flexure	First	Shear	Second
								20 storey	/					
0.6×0.6	3	0.5	1	0.3	c.b.	0.51	4006.17	8to17	0.42	3813.59	Shear	First	Shear	Second
0.6×0.6	3	0.75	1	0.3	beam	0.52	4008.40	12to15	0.42	3817.68	Flexure	First	Shear	Second
0.6×0.6	3	1	1	0.3	beam	0.51	4009.65	2to8	0.42	3820.97	Flexure	First	Shear	Second
			-					25 storey	/	-		-		
0.7×0.7	3	0.5	1	0.35	c.b.	0.77	5184.23	11to19	0.52	3996.56	Shear	First	Shear	Second
0.7×0.7	3	0.75	1	0.35	beam	0.78	5186.68	5to9	0.52	3998.43	Flexure	First	Shear	Second
0.7×0.7	3	1	1	0.35	beam	0.78	5187.23	2to8	0.52	4000.25	Flexure	First	Shear	Second
							-	30 storey	/	-		-		
0.8×0.8	3	0.5	1	0.35	c.b.	0.63	4784.99	15to22	0.54	4415.54	Shear	First	Shear	Second
$0.8 \times 0.8$	3	0.75	1	0.35	beam	0.58	4617.14	18to23	0.54	4421.62	Flexure	First	Shear	Second
0.8×0.8	3	1	1	0.35	beam	0.61	4744.18	5to8	0.54	4427.82	Flexure	First	Shear	Second

Table 6.4 Pushover results for parallel CSW

					Fa	ailure of	first elem	ent	perf	At ormance point		Rem	ark	
Column c/s	D <sub>w</sub> (m)	H <sub>b</sub> (m)	L <sub>b</sub> (m)	(m)	Type of element	Δ (m)	Base shear (kN)	Storey Location	Δ (m)	Base shear (kN)	Failure of Coupling beam	Failure sequence of Coupling beam	Failure of Wall	Failure sequence of Wall
							1	0 storey						
0.45×0.45	3	0.5	1	0.25	c.b.	0.50	2224.85	Base	0.2	2182.52	Shear	First	Flexure	Second
0.45×0.45	3	0.75	1	0.25	beam	0.31	2225.20	3&4	0.2	2241.12	Flexure	First	Flexure	Second
0.45×0.45	3	1	1	0.25	beam	0.31	2225.98	2	0.2	2343.25	Flexure	First	Flexure	Second
15 storey														
0.5×0.5	3	0.5	1	0.3	c.b.	0.36	2564.57	5to8	0.33	2487.84	Shear	First	Shear	Second
0.5×0.5	3	0.75	1	0.3	beam beam	0.42	2549.58	2to6 3to9	0.33	2490.42	Flexure	First First	Shear Shear	Second Second
0.5×0.5	5	1	1	0.5	ocam	0.45	2347.24	20 storey	0.55	2471.31	i iexuie	11130	Silcal	Sceona
							-	20 50010 9						
0.6×0.6	3	0.5	1	0.3	c.b.	0.59	2797.84	11to15	0.48	2677.78	Shear	First	Shear	Second
0.6×0.6	3	0.75	   1	0.3	beam	0.59	2795.14	13to18	0.48	2679.67	Flexure	First	Shear	Second
0.0X0.0	5	1	1	0.5	beam	0.02	2113.29	5&0	0.40	2082.00	Nexure	Filst	Slical	Second
				-			2	25 storey			•	r		
0.7×0.7	3	0.5	1	0.35	c.b.	0.74	2969.30	10to16	0.69	2886.14	Shear	First	Shear	Second
0.7×0.7	3	0.75	1	0.35	beam	0.74	2915.20	6to10	0.6	2853.50	Flexure	First	Shear	Second
0.7×0.7	3	1	1	0.35	beam	0.74	2918.36	8to13	0.6	2856.79	Flexure	First	Shear	Second
								30 storey			•			
0.8×0.8	3	0.5	1	0.35		0.82	3032.58	2to6	0.81	2913.46	Shear	First	Shear	Second
0.8×0.8	3	0.75	1	0.35	c.b. & beam	0.82	3034.09	12to14	0.81	2918.01	Flexure	First	Shear	Second
0.8×0.8	3	1	1	0.35	Jean	0.82	3036.84	15&16	0.81	2921.59	Flexure	First	Shear	Second

Table 6.5	Pushover	results	for	mid	side	CSW
1 abic 0.5	1 usilovel	results	101	mu	Siuc	CD H

Table 6.6 Pushover	results	for	central	CSW
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	D <sub>w</sub> (m)				Failure of first element					At ormance point	Remark			
Column l c/s (		H <sub>b</sub> (m)	L <sub>b</sub> (m)	T <sub>h</sub> (m)	Type of element	Δ (m)	Base shear (kN)	Storey Location	Δ (m	Base shear (kN)	Failure of Coupling beam	Failure sequence of Coupling beam	Failure of Wall	Failure sequence of Wall
10 storey														
0.45×0.45	3	0.5	1	0.25	c.b.	0.193	3622.22	Base	0.15	3299.90	Shear	First	Flexure	Second
0.45×0.45	3	0.75	1	0.25	c.b.	0.192	3616.27	3to7	0.15	3303.88	Flexure	First	Flexure	Second
$0.45 \times 0.45$	3	1	1	0.25	beam	0.192	3617.34	2to5	0.15	3308.29	Flexure	First	Flexure	Second
								15 storey						
0.5×0.5	3	0.5	1	0.3	c.b.	0.24	4979.26	4to10	0.16	4218.09	Shear	First	Shear	Second
0.5×0.5	3	0.75	1	0.3	beam	0.24	4993.93	2to6	0.16	4225.91	Flexure	First	Shear	Second
0.5×0.5	3	1	1	0.3	beam	0.24	5000.51	2&3	0.16	4234.74	Flexure	First	Shear	Second
20 storey														
0.6×0.6	3	0.5	1	0.3	c.b.	0.28	5948.91	8to14	0.17	5109.18	Shear	First	Shear	Second
0.6×0.6	3	0.75	1	0.3	beam	0.30	6030.62	3to9	0.17	5111.12	Flexure	First	Shear	Second
0.6×0.6	3	1	1	0.3	beam	0.30	6025.08	3&4	0.17	5119.72	Flexure	First	Shear	Second

25 storey														
0.7×0.7	3	0.5	1	0.35	c.b.	0.19	6398.77	5to17	0.17	6102.97	Shear	First	Shear	Second
0.7×0.7	3	0.75	1	0.35	beam	0.19	6421.65	3to8	0.17	6112.57	Flexure	First	Shear	Second
0.7×0.7	3	1	1	0.35	beam	0.19	6434.02	4&5	0.17	6125.33	Flexure	First	Shear	Second
30 storey														
0.8×0.8 0.8×0.8 0.8×0.8	3 3 3	0.5 0.75 1	1 1 1	0.35 0.35 0.35	c.b. Beam & c.b. Beam	0.25 0.27 0.25	7805.22 7956.12 7805.22	2to24 3to17 3to7	0.18 0.18 0.18	6620.53 6630.26 6638.53	Shear Flexure Flexure	First First First	Shear Shear Shear	Second Second Second

										At					
					Failure of first element					performance Remark			ark		
	F		Ŧ	_											
Column	$D_w$	H	L	T <sub>h</sub>					I	oont					
c/s	(m)	(m)	(m)	(m)	Type of element	Δ (m)	Base shear (kN)	Storey Location	Δ (m)	Base shear (kN)	Failure of Coupling beam	Failure sequence of Coupling beam	Failure of Wall	Failure sequence of Wall	
							1	0 storey							
0.45×0.45	3	0.5	1	0.25	c.b.	0.27	3195.90	2to6	0.14	2843.68	Shear	First	Flexure	Second	
0.45×0.45	3	0.75	1	0.25	c.b.	0.3	3344.46	3to5	0.14	2849.72	Flexure	First	Flexure	Second	
0.45×0.45	3	1	1	0.25	beam	0.31	3471.13	1&2	0.14	2857.89	Flexure	First	Flexure	Second	
15 storey															
0.5×0.5	3	0.5	1	0.3	c.b.	0.45	3057.11	6to12	0.26	2970.06	Shear	First	Shear	Second	
0.5×0.5	3	0.75	1	0.3	beam	0.44	3054.31	3to6	0.26	2973.40	Flexure	First	Shear	Second	
0.5×0.5	3	1	1	0.3	beam	0.44	3235.75	3&4	0.26	2978.13	Flexure	First	Shear	Second	
							2	0 storey							
0.6×0.6	3	0.5	1	0.3	c.b.	0.58	3237.40	5to14	0.39	3092.32	Shear	First	Shear	Second	
0.6×0.6	3	0.75	1	0.3	beam	0.54	3202.55	5to9	0.39	3096.31	Flexure	First	Shear	Second	
0.6×0.6	3	1	1	0.3	beam	0.56	3290.30	2to7	0.39	3104.34	Flexure	First	Shear	Second	
							2	5 storey	-						
0.7×0.7	3	0.5	1	0.35	c.b.	0.63	3514.37	8to17	0.53	3185.29	Shear	First	Shear	Second	
0.7×0.7	3	0.75	1	0.35	beam	0.62	3509.18	5to12	0.53	3195.37	Flexure	First	Shear	Second	
0.7×0.7	3	1	1	0.35	beam	0.66	3642.74	3to6	0.53	3199.69	Flexure	First	Shear	Second	
							3	0 storey	-						
0.8×0.8	3	0.5	1	0.35	c.b.	0.77	3496.08	12to24	0.68	3245.18	Shear	First	Shear	Second	
0.8×0.8	3	0.75	1	0.35	c.b. &beam	0.76	3471.75	13to18	0.68	3250.56	Flexure	First	Shear	Second	
$0.8 \times 0.8$	3	1	1	0.35	beam	0.74	3415.67	2&10	0.68	3255.64	Flexure	First	Shear	Second	

Table 6.7 Pushover results for out side CSW

From the results, it is observed that the base shear and displacement at performance point is more or less similar for a particular no. of storey irrespective of coupling beam depth. This indicate that coupling beam depth has not significant influence on pushover analysis.

Results of these studies indicates that failure patterns are almost same for different orientations. Base shear and displacements at performance point

varying for different orientation of CSW. It indicates that the capacity of different buildings depends on orientation of coupled shear wall for same seismic demand.

## 6.4 CORE SHEAR WALL AND COUPLED CORE SHEAR WALL

One of the most frequent uses of shear walls is in the form of box-shaped cores around stairs and elevators. As this arrangement makes structural use of vertical enclosures required around the services. Arrangement of internal cores is especially suitable for the office buildings because it gives larger open space outside of the core from the vertical elements. The walls around the core can be considered as a spatial system capable of transmitting lateral loads in both directions. Additional advantage of core structures is that being spatial structures, they have the ability to resist all types of loads: vertical loads, shear forces and bending moments in two directions, as well as torsion, especially when adequate stiffness and strength are provided between flanges of open sections. The shape of the core to a large extent is governed by the elevator and stair requirements. Variations could occure from a single rectangular core to complicated arrangements of planar shear walls [2].

The usefulness of walls in the structural planning of multistory buildings has long been recognized. When walls are situated in advantageous positions in a building, they can be very efficient in resisting lateral loads. Many shear walls contain one or more vertical rows of openings. A particularly common example of such a structure is the "shear core" of a tall building, which accommodates elevator shafts, stairwells, and service ducts. Access doors to these shafts pierce the walls. Thus, the walls on each side of openings may be interconnected by short, often deep, beams. It is customary to refer to such wall as being "coupled" by beams. Coupled shear wall is the one of the lateral load resisting systems of high rise structure. It's in plane stiffness and strength is very high, so it can resists greater amount of seismic forces. In case of coupled shear wall, connecting coupling beams reduce the magnitudes of the moments in the two walls in proportion to axial force carried by walls. Because of the relatively large lever arm (length of coupling beam) involved, a relatively small axial stress can give rise to a larger moment of resistance. So coupled shear wall is more effective in resisting seismic forces compared to solid shear wall [29].

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#### 6.5 ANALYSIS OF CORE SHEAR WALL AND COUPLED CORE SHEAR WALL

Multi storeyed core shear wall and coupled core shear wall buildings are considered for this study. The plans of the building are shown in Fig. 6.3. Column dimensions considered for study are shown in Table 1.2. Slab thickness is considered as 125 mm. Coupling beam depth is considered as 0.5m and other beams are having cross section  $0.25 \times 0.75$  m.



Figure 6.3 Structural arrangement in plan

Wall thickness and coupling beam width considered as per Table 1.2. Supports for all models are taken fixed.

## **6.5.1 DYNAMIC ANALYSIS**

For dynamic analysis, structure is modeled in ETABS. Seismic loading is applied as per IS: 1893. Time period is obtained from dynamic analysis as well as IS: 1893 based formula for both the types of arrangements. In results, all dynamic parameters are compared for both types of geometry.

Comparison of time period and base shear for core shear wall and coupled core shear wall are shown in Fig. 6.4 and 6.5 respectively.



#### Fig.6.4 Time Period comparison

From the results, it is observed that time period of core shear wall is smaller compare to coupled core shear wall. It is because of larger stiffness of core shear wall. So core shear wall attract more base shear compared to coupled core shear wall. The buildings with 10, 15, 20 storey and core shear wall is having lower time period compared to IS 1893 specification. For coupled core shear wall time period is closer to IS 1893 specifications for 10,15,20 storey building.



Figure 6.5 Base shear comparison

## **6.5.2 PUSHOVER ANALYSIS**

Modeling for pushover analysis is carried out as discussed in section 3.5.2. In case of pushover analysis, displacements and base shear at performance point are observed. Through displacement comparison, relative ductility of the structures can be estimated. The comparison of displacements is shown in Fig. 6.6.



#### Fig. 6.6 Displacement at P.P.

The results have shown that compare to coupled core shear wall core shear wall is less displaced at performance point. It indicates, that coupled core shear wall structure is more ductile compare to core shear wall structure. Failure patterns of 10 storey core shear wall as well as coupled core shear wall are shown in Fig. 6.7 and 6.8 respectively.



Fig.6.7 Failure pattern of 10 storey building model of core shear wall at staring of failure



Fig. 6.8. Failure pattern of 10 storey building model of coupled core shear wall at staring of failure

Comparison of failure pattern of both the buildings show that, in case of coupled core shear wall building, less numbers of beams are fails at staring of failure compare to core shear wall building.

The failure patterns at performance point are shown for core shear wall and coupled core shear wall in Figure 6.9 and Figure 6.10 respectively.



Fig.6.9. Failure pattern of 10 storey building model of core shear wall at performance point

Comparison of failure pattern, at performance point show that less numbers of members comes under immediate occupancy stage in case of coupled core shear wall compared to core shear wall.



Fig.6.10. Failure pattern of 10 storey building model of core coupled shear wall at performance point

## 6.6 SUMMARY

In this chapter orientation of coupled shear wall and their effect on dynamic and nonlinear behaviour is studied. For different orientations of coupled shear wall IS: 1893 specifies same time period. But in reality time period is based on overall structural in stiffness. Performance of structure having symmetric distribution of stiffness is better compare to asymmetric distribution.

In dynamic analysis, time period of core shear wall building is less compare to coupled core shear wall. Time period obtained from Dynamic analysis of all the coupled core shear wall structures is nearly match with the time period obtained from IS1893-2002. Time period as per IS: 1893 based formula is same for both core shear wall and coupled core shear wall building. Base shear of core shear wall structure is higher compared to coupled core shear wall building.

Core shear wall is stiffer red to coupled core shear wall, so core shear wall less ductile compared to core coupled shear wall building. For same seismic forces coupled core shear wall perform better compare to core shear wall building because of higher ductility.

# 7 EFFECT OF HINGE PROPERTY ON NONLINEAR ANALYSIS

#### 7.1 GENERAL

Nonlinear analysis of structures considers material non-linearity to estimate accurately the ductility (deformation behaviour) and carrying capacity of structural concrete element. Modified elastic analysis, simplified plastic analysis, lumped plasticity models and finite element methods are non-linear methods of analysis, in order of increasing complexity.

The simplest method, modified elastic analysis, uses elastic bars with elasticplastic finite regions, to model the structural behaviour. Simplified plastic analysis use either rigid-plastic or elastic-plastic relationships for the constituent materials to predict collapse load for a particular structure. The lumped plasticity model fits into this category. It uses elastic elements with plastic deformations lumped in regions of maximum moments.

The available inelastic rotation,  $\theta_a$ , of a notional plastic hinge, before the resisting moment falls below the design value, is the sum of all the associated components of inelastic curvature and rotation which are not considered in a normal elastic analysis. It is made up of components due to yielding of the steel, cracking or crushing of the concrete and total rotation of the end connection. As shown in Fig.7.1, inelastic regions are adjacent to the ends of a member or section of maximum internal moment within a member.



Fig 7.1.Collapse mechanism for fixed-end beam

#### **7.2 MOMENT CURVATURE**

The behavior of concrete sections and the influence of various factors on the behavior can be best represented by relationships between moment, curvature, and axial force. Fig. 7.2 shows a straight element of Reinforced Concrete member with equal end moments and axial forces. The radius of Curvature R is measured to the neutral axis. The radius of Curvature R, neutral axis depth  $k_d$ , concrete strain in the extreme compression fiber  $\varepsilon_c$ , and tension steel strain  $\varepsilon_s$ , will vary along the member because between the cracks the concrete will be carrying some tension [29]. Considering only a small element of length  $d_x$  of the member and using the notations of Fig7.1, the rotation between the ends of the element is given by,

$$\frac{dx}{R} = \frac{\varepsilon_c \times dx}{kd} = \frac{\varepsilon_s \times dx}{d(1-k)}$$
(7.1)

$$\frac{1}{R} = \frac{\varepsilon_c}{kd} = \frac{\varepsilon_s}{d(1-k)}$$
(7.2)



Fig.7.2 Deformation of flexural member

Now  $\frac{1}{R}$  is the curvature of the element (the rotation per unit length of member) and is given by the symbol  $\varphi$ .

$$\varphi = \frac{\varepsilon_c}{kd} = \frac{\varepsilon_s}{d(1-k)} = \frac{\varepsilon_c + \varepsilon_s}{d}$$
(7.3)

The curvature will actually vary along the length of the member because of the fluctuation of the neutral axis depth and the strains between the cracks. If the element length is small and over a crack, the curvature is given by eq. (7.3) with  $\varepsilon_c$  and  $\varepsilon_s$  as the strains at the cracked section [29].

## 7.3 ELASTO-PLASTIC BEAM BENDING

Fig. 7.3 shows elastic stress strain behaviour of material. But in case of concrete relationship between stress and strain is linear up to yield point. After yield point curve become nonlinear as shown in Fig. 7.4. Moment curvature for flexure member shows that curve remain in elastic range up to yield point.



Fig.7.3 Elastic behaviour

At yield point it pass through elastic-plastic region. Finally curve entered in plastic region (strain hardening zone), where stress is constant but strain will increase up to failure.



Fig.7.4 Nonlinear Moment-curvature relationship

#### 7.4 DUCTILE FAILURE OF R.C.C BEAM

For an under-reinforced R.C. beam the loading, deflection, section at mid-span stress-strain diagram for a section at mid-span are shown in Fig.7.5. When the load is progressively increased, strain in steel and concrete increase. It is experimentally verified that even when the concrete is heavily cracked, the strain diagram is linear. Increase in strain in steel is much higher than the strain in concrete as the strain varies linearly from neutral axis and the neutral axis is nearer to compression zone. At service load the stress and strain in steel are increased as shown in Fig.7.5-e as  $f_{st1}$  (approximately half the yield stress) and  $\epsilon_1$ . Corresponding strain in concrete is lower than  $\epsilon_1$ . At this level, deflections are small and tension cracks in concrete are invisible to the naked eye (conditions assumed in elastic design).





With the further increase in load, stresses and strains in steel and concrete increase. At a particular load, stress and strain in steel reach to yield point while the strain in concrete is below 0.0035 as the strain very linearly from neutral axis. At this load the beam does not fail as it has still capacity to develop the moment. Because of the property of the steel to yield it can resist the deflections. There are extensive cracking and deformation in tension zone before failure occurs.

With the further increase in load, the strains in the concrete and steel increase. The additional strain in concrete cause the concrete away from the neutral axis to be stressed in non-linear range and the total compression increases. Additional strain in steel causes the steel to yield but cannot increase the stress level and therefore total tension remain constant. To have an equilibrium (i.e. total compression=total tension), the neutral axis shifts towards the extreme compression fiber thereby reducing the total compression and increasing the lever arm. This increases the total moment of resistance of the beam. Concrete crushed when the strain in concrete is 0.0035.

It can be observed that before failure, the beam resists extensive cracking in tensile zone and very high deflections. This gives sufficient warning before failure to the occupants by cracks in tension zone and yielding of steel, it is termed as tensile failure. The failure in this type of beam (under-reinforced) is ductile and only this kind of design is acceptable [38].

## 7.5 M-O CURVE BASED ON IS 456 AND LIMIT STATE THEORY

M- $\theta$  curved can be plotted for designed beam section. Calculations for M- $\theta$  curve based on basic limit state conditions, are as follows:

- (1) When strain in steel as well as strain in concrete will reach at 0.002 stresses in both concrete and steel are constant.
- (2) When strain in steel reaches 0.002+0.87×fy/Es, beyond yield point, strain in concrete will reach at 0.0035.
- (3) Crushing of concrete will occur, when strain in concrete fiber reach at 0.0035.
- (4) Neutral axis remains constant for cross-section, when strain in both concrete and steel reach at maximum level.

M- $\theta$  curve is plotted for designed beam and based on that hinge property are assigned in ETABS. A worksheet is repared for calculation of M- $\theta$  curve as shown in Fig.7.6.



After calculating moment-curvature at different strain intervals they are converted in to the ETABS hinge input values by taking yield point moment as unit moment. For input of other points moment is calculated with reference to yield moment. These inputs are shown in Fig.7.7. Similarly hinge property for over reinforcement is obtained and is applied for same as frame as shown in Fig.7.8. Failure patterns considering these two different types of hinge conditions are examined by taking example of portal frame.

Point	Moment/SF	Rotation/SF				
E٠	-1.18	-38.05				
D٠	-1.12	-7				
C.	-1.05	-3				
B٠	-1	0.				
Α	0.	0.				
В	1.	0.				
С	1.05	3				
D	1.12	7				
E	1.18	38.05	Hinge is Higid Plastic			
Scaling for Moment and Hotation     Positive Negative     Use Yield Moment Moment SF						
Use Yield Rotation Rotation SF						
✓ Ose	Tield Hotation Ro	otation SF				
Acceptar	rrield Hotation Ro nce Criteria (Plastic R	otation SF				
Acceptar	nce Criteria (Plastic R	otation SF otation/SF) Positive	e Negative			
Acceptar Immedia	ried Hotation – Ho nce Criteria (Plastic R ate Occupancy	otation/SF Positive 3	e Negative			
Acceptar Immedia Life Saf	meia Hotation Ho nee Criteria (Plastic R ate Occupancy ety	otation/SF) Otation/SF) 3 4.	e Negative			



#### (a) Under reinforcement condition





## Fig.7.7 Hinge condition for different condition

Fig.7.8 Portal frame

From observation of failure patterns it is found that, failure of over-reinforced beam is sudden and hinge convert from B stage to E stage very fast. This shows that the design is uneconomical because the available strength of steel is not effectively used. Also it is unsafe as in case of overload, the failure is sudden and does not give time for occupants to take necessary action. Therefore, the design of an over- reinforced beam is not desriable.

In case of under reinforced section beam is not failed suddenly. Hinge is convert from B stage to E stage gradually. This gives sufficient warning before failure to the occupants for taking precautions against damages. The failure in this type of beam (under-reinforced) is ductile and only this kind of design is desirable.

# 7.6 PUSHOVER ANALYSIS CONSIDERING HINGE PROPERTY BASED ON IS 456 AND ATC 40

Considering different hinge properties, building is analysed in ETABS. Plan of building and necessary dimensions are taken from Fig. 1.7 and Tables 1.2 & 1.3 respectively. M- $\theta$  relationships are found for all beams and coupling beam for under-reinforced condition by using worksheet as discussed in section 7.5. Hinge property of longitudinal beam, Transverse beam and coupling beams are shown in Fig.7.9. In other model hinge property are modeled as per ATC 40 specifications as shown in Fig.7.10.



(a) Longitudinal beam

- (b) Coupling beam
- (c) Transeverse beam



		1	
Point	Moment/SF	Rotation/SF	
E-	-0.2	-7	
D٠	-0.2	-4	
C۰	-1.25	-4	
B۰	-1	0.	
Α	0.	0.	
В	1.	0.	
С	1.25	4.	
D	0.2	4.	Distance in District District
Е	0.2	7.	Hinge is Higid Plastic
Use	Yield Moment M	oment SF	
0.00	noid noiddon - Th		
Acceptar	nce Criteria (Plastic R	otation/SF)	
		Positiv	e Negative
Immedia	ate Occupancy	2.	
Life Saf	ety	3.	
Callana			
Collapsi	e Prevention	4.	

Fig.7.10 Hinge property as per ATC 40

For these two different conditions displacement and base shear at performance point are obtained and compared. Failure patterns are also compared.

## 7.7 RESULT DISCUSSION

Base shear and displacements for two different hinge properties are shown in Fig.7.11. This results are also compared in Table 7.1.



(a) Hinge property based on IS 456



(b) Hinge property ATC 40 base



	Base Shear(kN)at P.P.	Displacement (m)at P.P.
Hinge property based on IS: 456	7929.41	0.146
Hinge property based on ATC 40	7749.178	0.150

 Table 7.1 Nonlinear performance of hinge comparison

Displacement at performance point for ATC-40 based hinge property is more compared to IS: 456 based hinge property. It shows that in case of ATC 40 based criteria M- $\theta$  relationship is considered up to complete failure. While in case of IS :456 based criteria M- $\theta$  relationship is considered up to 0.0035+0.87×fy/E<sub>s</sub> strain limit.

Failure patterns for these two different types of hinge property are shown in Fig.7.12 and 7.13.



Fig.7.12 Hinge property base on IS:456 for step 4



Fig.7.13 Hinge property base on ATC 40 for step 4

The number of failure of coupling beams is more in case of IS: 456 based hinge property compared to ATC-40 based hinge property for same step no. 4 as shown in Fig.7.12 & 7.13. It shows that in ATC-40 based hinge property. Strength up to failure is considered. While in case of IS: 456 based hinge property strength is considered up to concrete crushing.

## 7.8 SUMMARY

In this chapter effect of hinge property on nonlinear analysis of structure is considered. If moment rotation M- $\theta$  relationship of under reinforced section is considered, failure of beam is more ductile compared to over reinforced section. In this chapter moment-curvature relationship for beam is obtained considering IS 456-limit state criteria. The effect of M- $\theta$  relation on results of pushover analysis is studied by taking CSW building and assigning hinge property as obtained by IS 456 specifications and ATC 40 specifications.

# 8. SEISMIC DESIGN OF COUPLED SHEAR WALL

#### **8.1 SEISMIC DESIGN REQUIREMENTS**

The two most important elements of concern to a structural engineer are calculation of seismic design forces and the means for providing sufficient ductility. In most structural engineering problems, dead loads, live loads and seismic loads can be evaluated with a fair degree of accuracy. However, the situation with regard to earthquake forces is entirely different. The loads or forces which a structure sustains during an earthquake, result directly from the distortion induced in the structure by the ground on which it rests. Base motion is characterized by displacements, velocities and accelerations, which are erratic in direction, magnitude, duration, and sequence. Earthquake loads are inertia forces related to the mass, stiffness, and energy-absorbing (e.g. damping and ductility) characteristics of the structure. The design seismic loading recommended by building codes is in the form of static lateral loading. These lateral loads depend upon the weight, the gross dimensions, and the type of structure, as well as the seismicity of the area in which it is to be built. These static design loads are used to determine the strength of the structure necessary to withstand the dynamic loads induced by earthquakes. When the proper earthquake design loads are determined by the traditional static approach, uncertainties arise from a number of factor; the most important of these are:

- (a) Not enough empirical data are available to make a reliable prediction of the character of the critical earthquake motions (i.e., amplitude, frequency characteristics, and duration) to which a proposed structure may be subjected during its lifetime.
- (b) Analysis by elastic assumptions does not take into account the change in the properties of the building materials during the progress of an earthquake. This presents difficulties in ascertaining the values of the structural parameters affecting the dynamic response (e.g., stiffness and damping), as well as the dynamic properties of the soil or supporting medium.

(c) Soil-structure interaction and geological conditions have a segnificant effect on structural performance. At present there is no clear-cut method to correctly incorporate these effects.

Despite these uncertainties, the structure should perform satisfactorily beyond the elastic-code-stipulated stress. Ductility, the foremost important property in the inelastic range, thus becomes a necessity for an earthquake-resistant design of structure. It is generally accepted that sufficient ductility will be achieved by following the standards. However, design codes are prepared for regular structures. For structures requiring high ductility, e.g. a light flexible structure attached to a large structure, careful analysis may be required.

The seismic forces specified in the code are quite small in comparison to the actual forces (4-6 times) expected at least once in the lifetime of the building. In spite of the large difference, the structures designed to the lateral loads as per codes have survived severe earthquakes. The main reason for this is the ductility of the structure, due to which energy is dissipated by post-elastic deformations; Another reason is the reduced response due to increased damping and soil-structure interaction. Fig.8.1 shows the relationship between lateral design forces for an elastic structure and for a yielding ductile structure. Much larger design forces are required for an elastic structure without ductility.



Fig.8.1 Lateral forces and ductility

During the life of a structure located in seismically active zone, it is generally expected that the structure will be subjected to many small earthquakes, a few moderate earthquakes, one or more large earthquakes, during its life span. If the earthquake motion is severe, most structures will yield in some of their elements. The energy absorption capacity of the yielding structure will limit the damage. Thus, buildings that are properly designed and detailed can survive earthquake forces, which are substantially greater than the design forces that are associated with allowable stresses in the elastic range. Hence, the structure is allowed to be damaged in case of sever shaking. The cost of securing the structure against strong shaking must be weighed against the importance of the structure and the probability of earthquakes. Seismic design concepts must consider the building's proportions, and details of ductility (capacity to yield) and reserve energy absorption capacity, to ensure that it survives the inelastic deformations that would result from a maximum expected earthquake. Special attention must be given to connections that hold the lateral force- resisting elements together [30].

#### 8.2 CODAL EARTHQUAKE RESISTANCE DESIGN PHILOSOPHY

The basic intent of design theory for earthquake-resistant structures is that buildings should be able to resist minor earthquakes without damage, resist moderated earthquakes without collapse but with some structural and nonstructural damage. This indicates that damage during earthquakes is acceptable as long as loss of life is avoided. The objective is to have structures that will behave elastically and survive without collapse under major earthquakes that might occur during the lifetime of the building. To avoid collapse during a major earthquake, members must be ductile enough to absorb and dissipate energy by post-elastic deformation. This implies that deformation beyond the yield limit is allowed without significant loss of strength [25].

Since the buildings designed by present codes may undergo relatively large inelastic deformations, it must be ensured that the structure maintains its integrity, and does not become unstable under vertical loads, while undergoing larger lateral displacements. To achieve this objective, yielding is confined to the beams, while the columns remain elastic. This is known as strong-column, weakbeam approach. The present codes recommend this, as the structures have been shown to perform better under earthquake loading with this approach.

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Structural systems that combine several lateral load-resisting elements or subsystems, generally have been observed to perform well during earthquakes. Redundancy in the structural system permits redistribution of internal forces in the event of the failure of the key elements. When the primary element or system yields or fails, the lateral force can be redistributed to a secondary system to prevent progressive failure. Without capacity for redistribution, global structural collapse can result from failure of individual members or connections. Redundancy can be provided by several means-a dual system, a system of interconnected frames that enable redistribution among frames after yield has initiated in individual frames, and continuity can alleviate the need for excesses in ductile detailing.

A building is analysed for its response to ground motion by representing the structural properties in an idealized mathematical model as an assembly of masses interconnected by springs and dampers. The tributary weight to each floor level is lumped into a single mass, and the force-deformation characteristics of the lateral force-resisting walls or frame between floor levels are transformed into equivalent storey stiffness. Because of the complexity of the calculations, the use of a computer program is necessary, even when the equivalent static force procedure is used in design [30].

## **8.3 INELASTIC BEHAVIOUR OF COUPLED SHEAR WALL**

A shear wall may contain one or more vertical rows of opening. Shear cores used in multistory buildings typically have opening for access to service shafts. The walls on either side of the openings are thus 'coupled' together by beams. Such a system is called a coupled shear wall.

When lateral loads are applied to a coupled shear wall system, moment reactions are developed at the base of each wall. In addition, a couple is also formed by axial forces in each wall that are carried across the coupling beams by vertical shear. The coupling beams usually have small span to depth ratio making the shear deformations in them significant. In a severe earthquake, the ends of coupling beam are subjected to large rotational and vertical displacements. These sections need to have adequate ductility. Properly detailed coupling beams can dissipate large amount of energy by formation of plastic hinges at their ends before any hing occurs at the wall base [29].

Coupling beam that contain inadequate shear reinforcement fail in diagonal tension. Such brittle failure results in rapid strength degradation under cyclic loading. The shear strength of the coupling beam must be greater than the shear force that can be developed when plastic hinges forms at its two ends. This requires a limit to be imposed on the tension steel content in such beams. Coupling beams that have a small span-to-depth ratio and are provided with adequate shear reinforcement in form of vertical stirrups may also fail due to sliding shear along the critical support section.

Reinforcement in the coupling beam may consist of parallel reinforcement or full length diagonal reinforcement, as shown in Fig.8.2 (a) Parallel reinforcement when adopted must be well anchored into the wall. Closely spaced stirrups must be provided over the full length of coupling beam to confined the concrete and provide adequate shear strength. Diagonal reinforcement is more effective in coupling beams as compared to parallel reinforcement. The diagonal bars must also be well anchored into the wall and restrained over their full length to prevent to buckling shown in Fig. 8.2(b) [32].

## 8.4 CODAL COMPARISON:IS:13920, ACI-318 AND EC-8

The design procedure of shear wall as specified in various codes are compared and shown in Table 8.1. The main clauses are related to thickness of wall, area of reinforcement provided and ductile detailing.

Design clauses related to coupling beam are discussed in Table 8.2. The main clauses related to coupling beams are diagonal reinforcement requirements and ductile detailing. Detailing as per different codal provisions as per IS 13920, ACI-318 and Euro code 8 are as shown in Fig.8.2, 8.3 and 8.4 respectively.

Criteria for ductile detailing of coupling beam, from diagonal reinforcement provide are similar as per IS 13920 and ACI-318.



SECTION A-A









Fig.8.4 Detailing as per EC-8

## Chapter 8. Seismic design of coupled shear wall

	IS 13920 [35]	ACI 318-05 [34]	EC 8[36]
Minimum thickness of Web	Thickness>150 mm In case of coupled shear walls	Shall not be less than 4 in.(101.6mm), or 1/30 the least distance between members that provide	> maximum of Hs/20 or 150 mm Hs = story height.
	the thickness of wall shall be at least 200 mm.	lateral support.	
Minimum reinforcement ratio	0.0025 of gross area in each direction	0.0025 of gross area in each direction	0.002 of gross area in each direction
Layer of reinforcement	If shear stress exceeds $0.25\sqrt{f_{ck}}$ or Thickness>200 mm Provide reinforcement in two layers.	Inplane shear force exceeds $2A_{cv}\sqrt{f_{c'}}$ . Provide reinforcement provide in two layers. Where, $\sqrt{f_{c'}}$ squrate root of specified compressive strength of concrete. $A_{cv}$ is gross area of concrete section bounded by	Web reinforcement should be provided in form of two grids at each face of wall.
		web thickness and length of section in the direction of shear force considered, in. <sup>2</sup>	

# Table 8.1 Ductile Wall Parameters

## Chapter 8. Seismic design of coupled shear wall

Minimum Diameter of bar	1/10 of thickness	Bar sizes 3# to 11#, 14#, 18#.	Maximum of bw/8 or 8mm. Where, bw is width of the wall	
Minimum	Least of 1w/5, 3tw, and 450 mm	<18" (457.2 mm)	Least of 20 times bar diameter or 200 mm for H-wall	
spacing of	lw is length of the wall.		Least of 25 times bar diameter or 250 mm for M-wall	
reinforcement	tw is thickness of the wall.		M means medium ductile wall.	
			H-wall H means high ductile wall.	
Boundary	Where the extreme fibre	Max comp. fiber stress $fc = P/Ag + M/Z > 0.2 fc'$	May be defined on the basis of 0.2% of	
element	compressive stress in the wall due to factored gravity loads plus factored earthquake force exceeds 0.2fck.	boundary elements are required.	compression strain. but,> 0.15 Lw or 1.5 bw.	
% of vertical reinforcement in boundary element.	> 0.8 and <6%	> 0.8 and <6%	minimum 0.8 to 5%	

Lap splices	Splicing shall be avoided in	Lap splices shall not be used for bars larger than	lap length= $\alpha$ lbnet > lsmin
	region where yielding takes	No. 11 .	lsmin= $0.3\alpha \alpha_0 (d_b/4) (f_y/f_{cd}) > 15 d_b > 200 \text{mm.}$
	place. This region is consider to	Minimum length for tension lap splices shall be as	
	extend greater of $l_w$ from base	required for class A or B splice, but > 12 in.	Where,
	of wall or $H_w/6$ whichever is	Class A splice - 1.0 ld	$d_b$ is depth of the beam.
	more. Splices in adjacent bars	Class B splice - 1.3 ld	$\alpha$ is confinement effectiveness factor, angle
	should be staggered by a	Where ld is development length in tension of	between diagonal bars and axis of a coupling
	minimum 600 mm.	deformed bar.	beam.
			$\alpha = 1$ for bar in compression
			= 1.4 for bar in compression.
			$l_{bnet} = \alpha_0 (d_b/4) (f_{yd}/f_{cd}) (Ast req/Ast pro)$
			l <sub>bnet</sub> is length of splice
			$f_{yd}$ is design value of yield strength of steel
			$f_{cd}$ is design value of concrete compressive
			strength.
			$\alpha_0$ =prevailing aspect ratio of walls of the
			structural system
			$\alpha_0=1$ for straight bar and 0.7 for curved bar.

Table 8.2 Coupling Beam Design Parameter	ers
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	IS 13920	ACI 318-05	EC 8
Diagonal reinforcement Requirement	When, Shear stress in coupling beam exceeds $\frac{0.1 \times ls \times \sqrt{f_{ck}}}{D}$ Where, ls is clear span of coupling beam and D is its overall depth.	<ul> <li>a) Coupling beams with aspect ratio, (<i>ln /h</i>) &lt; 4, shall be permitted to be reinforced with two intersecting groups of diagonally placed bars symmetrical about the mid span.</li> <li>b) Coupling beams with aspect ratio, (<i>ln /h</i>) &lt; 2, and with <i>Vu</i> exceeding 4 √<i>f<sub>c</sub></i>. <i>Acw</i> shall be reinforced with two intersecting groups of diagonally placed bars</li> <li>c) symmetrical about the mid span, unless it can be shown that loss of stiffness and strength of the coupling beams will not impair the vertical load-carrying capacity of the structure, or the egress from the structure, or the integrity of nonstructural components and their connections to the structure.</li> </ul>	The resistance to seismic actions should be provided by reinforcement arranged along both diagonals of the beam, in accordance with, $V_{Ed} \leq 2 \cdot A_{si} \cdot f_{yd} \cdot \sin \alpha$ where: $V_{Ed}$ design shear force in the coupling element ( $V_{Ed} = \cdot M_{Ed}l$ ); $A_{si}$ total area of steel bars in each diagonal direction; $\alpha$ angle between the diagonal bars and the axis of the beam.
Reinforcement & Detailing	The area of reinforcement to be provided along each diagonal in a diagonally reinforced coupling beam shell be;	Coupling beams reinforced with two intersecting groups of diagonally placed bars symmetrical about the mid span shall satisfy (a) through (f): a) Each group of diagonally placed bars shall consist	<ul> <li>a) The diagonal reinforcement is arranged in column-like elements with side at least equal to 0.5bw; its anchorage length exceeds by</li> </ul>

$A_{ad} = \frac{V_u}{1 - 1 - 1}$		of a minimum of four bars assembled in a core		50% .
$sa = 1.74 \cdot f_y \cdot \sin \alpha$		having sides measured to the outside of transverse	b)	Hoops are provided around these
Where $V_u$ is the factored shear		reinforcement no smaller than <i>bw</i> / 2, <b>p</b> erpendicular	- /	column-like elements to prevent
force, and $\alpha$ is the angle made by		to the plane of the beam and $bw / 5$ in the plane of		buckling of
the diagonal reinforcement with		the beam and perpendicular to the diagonal bars.		outiling of
the horizontal. At least 4 bars of	b)	Vn shall be determined by	c)	longitudinal bars. Hoop spacing s
8 mm diameter shall be provided	0)	vn snan be determined by		should not exceed 100 mm.
along each diagonal		$Vn = 2Avdfy\sin \alpha \leq 10 fc' Acw.$	d)	Longitudinal and transverse
along cach diagonal.		Where $\alpha$ is the angle between the diagonally placed		reinforcement is provided at both
a) The pitch of spiral or		bars and the longitudinal axis of the coupling beam.		lateral faces of the beam, meeting
spacing of ties shall	c)	Each group of diagonally placed bars shall be		the minimum requirements of EN
not be exceed 100 mm.	C)	each group of diagonality placed bars shall be		1992-1-1:200X for deep beams.
b) The diagonal or				The longitudinal reinforcement
horizontal bars of a		contining detailing clause.		should not be anchored in the
coupling beam shall be	d)	The diagonally placed bars shall be developed for		coupled walls, but only extend
anchorage length of 1.5		tension in the wall;		into them by 150 mm
times the development	e)	The diagonally placed bars shall be considered to		
length in tension.	0)	contribute to $Mn$ of the coupling beam		
		contribute to him of the coupling ocum.		
	f)	Reinforcement parallel and transverse to the		
		longitudinal axis shall be provided.		

# 8.5 DESIGN EXAMPLE OF CSW FOR 20 STOREY BUILDING AS PER IS 13920

For designing of 20 storey coupled shear wall building plan of building is considered as shown in Fig. 1.7. Other dimensions are considered as per Table 1.12 and 1.13. Modeling and analysis are done in ETABS. For the limit state design of reinforced concrete structures, the following load combinations are considered.

- (a) 1.5(DL+IL)
- (b) 1.2(DL+IL±EL)
- (c) 1.5(DL±EL)
- (d) 0.9 DL±1.5EL

When the lateral load-resisting elements are oriented along the orthogonal horizontal directions, the structure should be designed for the effects due to full design earthquake load in one horizontal direction at a time. However, when the load-resisting elements are not oriented along the orthogonal horizontal directions, then EL in the above load combinations should be replaced by  $(EL_x\pm0.3EL_y)$  or  $(EL_y\pm0.3EL_x)$ , respectively.

Design and detailing is carried out as per IS: 13920. Forces and reinforcement schedule for shear wall, coupling beam, column and other beams are shown in Table 8.3,8.4,8.5 and 8.6 respectively. Ductile detailing of one of frame-wall cross section is also shown in Fig. 8.5.

No. of storey	Axial forces (kN)	Moment (kNm)	Shear (kN)	Horizontal Reinforcement	Vertical Reinforcement	Reinforcement in B.E.
20	13850.45	1945.86	418.87	10ø @ 160 c/c	10ø @ 160 c/c	N.R.

Table 8.3 Forces and reinforcement schedule for shear wall

Table 8.4 Forces and reinforcer	nent schedule for coupling beam
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Shear (kN)	Moment (kNm)	Diagonal reinforcement	Stirrups for Diagonal Reinf. (2-legged)	l <sub>d</sub> (mm)	Horizontal Reinforcement	Vertical Reinforcement
390.23	95.87	8 nos.25ø	8ø @ 100 c/c	1250	8 nos. 10ø	8ø @ 125c/c



Shear (kN)	Moment (kNm)	Stirrups for Diagonal Reinforcement (2-legged)	l <sub>d</sub> (mm)	Top Reinforcement	Bottom Reinforcem ent
105.62	174.64	10ø @ 75 c/c in confining zone & 10ø @ 250 c/c at mid portion	1180	4nos. 16ø	5nos. 20ø

Table 8.5 Forces and reinforcement schedule for beam

## Table 8.6 Forces and reinforcement schedule for column

Axial (kN)	Moment Mx (kNm)	Moment My (kNm)	Longitudinal Reinforcement	Lateral Ties
6843.25	94.82	156.98	12 nos. 32ø	10ø @ 75 c/c in confining zone & 10ø @ 150 c/c at mid portion

# 8.6 SUMMARY

The effect of various parameters on structural behaviour, seismic response and design aspect of coupled shear wall are discussed. The seismic forces specified in the code are quite small in comparison to the actual forces (4-6 times) expected at least once in the lifetime of the building. It means structure will complete collapse at 4-6 times higher forces compare to elastic design forces.

The shear strength of the coupling beam must be greater than the shear force that can be developed when plastic hinges forms at its two ends. This requires a limit to be imposed on the tension steel content in such beams. Coupling beams that have a small span-to-depth ratio and are provided with adequate shear reinforcement in form of vertical stirrups may also fail due to sliding shear along the critical support section.

For designing ductile wall and coupling beam clauses of different codes are compared. Designing and detailing of 20 storey building is shown based on IS 13920 code.

## 9.1 SUMMARY

Coupled shear wall (CSW) system is very efficient for high rise buildings due to its in plane stiffness and ductility. As reinforced concrete is not perfectly elastic material, linear elastic analysis of coupled shear. Wall building can not reflect actual behavior of structure. The Nonlinear behavior of earthquake resistant structural system is important in the seismic event. Plastic hinges formed during deformations at the end of coupling beam and at the base of wall under lateral loading have a considerable effect on the structural response. As a result, the forces within whole structure are redistributed throughout the height. As flexural hinges are ductile in nature, are desirable in the failure pattern. The sequence of failure in CSW is similar to the strong column and weak beam concept.

Modeling is very important and major step for any structural analysis. Parametric studies are carried out considering wall as frame element as well as shell element, and seismic response of csw is compared. ETABS software is used for modeling and analysis considering different support conditions.

A parametric study is carried out to understand the dynamic behaviour of coupled shear wall in 2D and 3D models considering different geometry. The parameters of the study are depth and span of coupling beam, wall length, wall height (or number of storey). The wall length is varying from 3.0 to 6.0m; number of storey is varying from 10 to 30; depth of coupling beam is varying from 0.5 to 1.25m and span of coupling beam is varying from 1.0 to 2.5m. Time period and base shear of coupled shear wall building are compared. Lateral load distribution in wall and columns are found for 3D model. A comparative study is done to understand effect of nonlinearity in 2D and 3D analysis of coupled shear wall building.

The earthquake loads are applied on the coupled shear wall building, as per equivalent static approach given in IS 1893:2002. For the load calculation commercial building is considered, whose plan dimension is  $36 \times 20$  m. Capacity and demand curves are plotted for various buildings. In addition, failure pattern

of wall and coupling beam are studied. In this study, the span to depth ratio of coupling beam is proposed for flexural failure of CSW.

Finite element modeling of core shear wall and coupled core shear wall building for three dimensions dynamic and nonlinear analysis is explored. Comparison of time period shown that, core shear wall building is stiffer than coupled core shear wall. So core shear wall is less ductile compared to core coupled shear wall building. For same seismic forces coupled core shear wall perform better compared to core shear wall building because of higher ductility.

The hinge property is very important in pushover analysis. Comparison between hinge property based on IS:456 and ATC 40 show that, ATC 40 based hinge property is more ductile compared to that based on IS:456. In ATC 40 based hinge property, M- $\theta$  relationship is considered up to complete failure limit.

Optimization in design can be achieved by utilizing full strength of material beyond their elastic limit. For that nonlinear analysis is very important. The seismic forces specified in the code are quite small in comparison to the actual forces expected to act on building at least once in the lifetime. A design example of 20 storey coupled shear wall building is discussed.

## 9.2 CONCLUSIONS

From the above study following conclusions can be made,

## **Dynamic Behaviour**

- a. Time period of 2D model is higher compared to 3D model of coupled shear wall building. This is due to increased stiffness of 3D model compared to 2D model.
- b. Time period and base shear are more or less similar for 2D model and 3D model as compared to IS: 1893-2002 specifications, for 10 storey building. The difference is more as the number of storey increases.

## Modeling

Parametric studies show that, if wall is modeled as frame element instead of shell element, seismic behaviour of building is similar for lower height of building.

## Modeling of support condition

- a. The time period formula as per IS: 1893-2002 is based on geometrical dimensions of building only. It does not consider actual soil condition and mass of the structure.
- b. The time period given by code is generally lower in comparison to dynamic analysis. So base shear specified by IS 1893 is higher. So it cannot give realistic time period and estimate of seismic force on building for particular soil condition.
- c. For economical design of the building, the support conditions based on actual soil conditions should be considered.
- d. Time period of building depends on stiffness and mass of various components. Stiffness of building depends on member property of structure as well as foundation condition.
- Base shear of structure is inversely proportional to time period of structure. Thus flexible foundation condition gives higher time period and less base shear.
- f. For pushover analysis, it is necessary to model wall as frame element. When wall is modeled as frame element instead of shell element, fixed base condition gives similar time period and base shear compared to shell model.

## **Nonlinear Analysis**

- a. The results of nonlinear analysis shows that in flexure and shear hinge condition capacity curve (Base shear Vs. Roof displacement) are nearly same for 2D and 3D models.
- Base shear as well as roof displacemet values are more in case of 3D model compared to 2D model because 2D model is more flexible compared to 3D model.
- c. The nonlinear behaviour of CSW is greatly influenced by the wall length, span and depth of coupling beam.

- d. With increasing the depth of coupling beam, the failure pattern of coupling beam changes from shear to flexure.
- e. In 10 and 15- storey CSW, with increase in span of coupling beam, failure pattern of coupling beam changes from shear to flexure.
- f. The walls are predominately failing in shear. It is also observed that the coupling beam span to depth ratio for 2 to 2.6 is failing in flexure.
- g. With increae in stiffness of structure ductility of structure reduced for perticular height of building.

## **Dynamic Analysis of Core and Coupled core shear wall**

- a. In dynamic analysis, time period of core shear wall building is less compare to coupled core shear wall.
- b. Time period obtained from Dynamic analysis of all the coupled core shear wall structures is nearly match with the time period obtained from IS1893-2002.
- c. Time period as per IS: 1893 based formula is same for both core shear wall and coupled core shear wall building.
- d. Base shear of core shear wall structure is higher compare to coupled core shear wall building.

## Nonlinear Analysis of Core and Coupled core shear wall

- a. Core shear wall is stiffer compared to coupled core shear wall, so core shear wall is less ductile compared to coupled core shear wall building.
- b. For same seismic forces coupled core shear wall perform better compared to core shear wall building because of higher ductility.

## **Nonlinear Hinge Property**

From the comparative study of modeling of hinge properties, it is observed that ATC 40 based hinge consider strength up to failure of element, while IS: 456 based hinge consider failure up to crushing of concrete. C, D and E stages are absent in case of IS: 456 based hinge property. It indicates ATC-40 based hinge property covers strain hardening zone in contrast to IS: 456 based hinge property.

## Seismic Design

- a. The seismic forces specified in the code are quite small in comparison to the actual forces expected at least once in the lifetime of the building.
- b. Coupling beams are having a small span-to-depth ratio and are provided with adequate shear reinforcement in form of diagonal to avoid sliding shear along the critical support section.

## **9.3 FUTURE SCOPE OF WORK**

The present study can be extended as follows:

- 1. Pushover analysis tool can be developed using VC++ programming language, dedicated for CSW buildings.
- 2. Moment curvature relation ship for shear wall can be generated for different shape and same can be utilized in nonlinear pushover analysis.
- 3. Computer program for evaluating plastic hinge properties of coupled shear wall can be developed.
- 4. Higher mode effects are not considered in pushover analysis. One can consider higher mode effects on distribution of lateral force distribution.
- Nonlinear Analysis of Coupled shear wall buildings can be carried out using Site Specific Response Spectra of different sites.
- 6. Nonlinear pushover analysis can be applied to unsymmetrical coupled shear wall building.
- 7. Static nonlinear analysis can be applied to real life problems of high rise structure having coupled shear wall.

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#### **APPENDIX A**

#### Plastic Hinge Properties in FEMA 356-2000 and ATC 40

1 able 6-18	18 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedure Members Controlled by Flexure									ures—	
							Acceptable Plastic Hinge Rotation (radians)				
Conditions							Performance Level				
							Component Type				
				Rotation (radians)		Residual Strength Ratio	10	Primary		Secondary <sup>4</sup>	
			а	b	LS			СР	LS	CP	
i. Shear walls	and wa	II segments		1							
$\frac{(A_s - A'_s)f_y}{t_w l_w f'_c}$	+ <u>P</u>	$\frac{\text{Shear}}{t_w l_w \sqrt{f_c'}}$	Confined Boundary <sup>1</sup>								
≤ 0.1		≤3	Yes	0.015	0.020	0.75	0.005	0.010	0.015	0.015	0.020
≤ 0.1		≥ 6	Yes	0.010	0.015	0.40	0.004	0.008	0.010	0.010	0.015
≥ 0.25		≤ 3	Yes	0.009	0.012	0.60	0.003	0.006	0.009	0.009	0.012
≥ 0.25	1.	≥ 6	Yes	0.005	0.010	0.30	0.0015	0.003	0.005	0.005	0.010
≤ 0.1		≤ 3	No	0.008	0.015	0.60	0.002	0.004	0.008	0.008	0.015
≤ 0.1		≥ 6	No	0.006	0.010	0.30	0.002	0.004	0.006	0.006	0.010
≥ 0.25		≤ 3	No	0.003	0.005	0.25	0.001	0.002	0.003	0.003	0.005
≥ 0.25		≥ 6	No	0.002	0.004	0.20	0.001	0.001	0.002	0.002	0.004
ii. Columns su	pportir	ng discontinu	ous shear wa	lls							
Transverse reinforcement <sup>2</sup>							4				
Conforming			0.010	0.015	0.20	0.003	0.007	0.010	n.a.	n.a.	
Nonconforming			0.0	0.0	0.0	0.0	0.0	0.0	n.a.	n.a.	
iii. Shear wall	couplin	g beams									
Longitudinal reinforcement and transverse reinforcement <sup>3</sup>		$\frac{\text{Shear}}{t_w l_w \sqrt{f_c'}}$									
Conventional longitudinal <3		< 3	0.025	0.050	0.75	0.010	0.02	0.025	0.025	0.050	
reinforcement with conforming transverse reinforcement		≥6	0.02	0.040	0.50	0.005	0.010	0.020	0.020	0.040	
Conventional longitudinal ≤		≤ 3	0.020	0.035	0.50	0.006	0.012	0.020	0.020	0.035	
transverse reint	forceme	ent	≥6	0.010	0.025	0.25	0.005	0.008	0.010	0.010	0.025
Diagonal reinforcement n.a.			0.030	0.050	0.80	0.006	0.018	0.030	0.030	0.050	

Requirements for conforming transverse reinforce of hoops  $V_s \ge$  required shear strength of column. are: (a) hoops over the 3

Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the coupling beam at a spacing  $\leq d/3$ , and (b) strength of closed stirrups  $V_s \geq 3/4$  of required shear strength of the coupling beam.

For secondary coupling beams spanning <8'-0", with bottom reinforcement continuous into the supporting walls, secondary values shall be permitted to be doubled.

#### Table 6-19 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures-Members Controlled by Shear

					Acceptable Total Drift (%) or Chord Rotation (radians) <sup>1</sup> Performance Level					
		100000000000000000000000000000000000000								
		Total Drift Ratio (%), or Chord		Residual		Component Type				
			(radians) <sup>1</sup>		221	Primary		Secondary		
Conditions	d	e	c	ю	LS	CP	LS	CP		
i. Shear walls and wall segment	s						1.1	123		
All shear walls and wall segments	0.75	2.0	0.40	0.40	0.60	0.75	0.75	1.5		
ii. Shear wall coupling beams <sup>4</sup>							25.6	554		
Longitudinal reinforcement and transverse reinforcement <sup>3</sup>	$\frac{\text{Shear}}{t_w l_w \sqrt{f_c'}}$									
Conventional longitudinal reinforcement with conforming	≤ 3	0.002	0.030	0.60	0.006	0.015	0.020	0.020	0.030	
transverse reinforcement	≥8	0.016	0.024	0.30	0.005	0.012	0.016	0.016	0.024	
Conventional longitudinal reinforcement with	≤ 3	0.012	0.025	0.40	0.006	0.008	0.010	0.010	0.020	
nonconforming transverse reinforcement	≥6	0.008	0.014	0.20	0.004	0.006	0.007	0.007	0.012	

1. For shear walls and wall segments, use drift; for coupling beams, use chord rotation; refer to Figures 6-3 and 6-4.

For shear walls and wall segments where inelastic behavior is governed by shear, the axial load on the member must be ≤ 0.15 A<sub>g</sub>f<sub>c</sub><sup>2</sup>; otherwise, the member must be treated as a force-controlled component.

3. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the coupling beam at a spacing ≤ d/3, and (b) strength of closed stirrups V<sub>x</sub> ≥ 3/4 of required shear strength of the coupling beam.

4. For secondary coupling beams spanning <8'-0", with bottom reinforcement continuous into the supporting walls, secondary values shall be permitted to be doubled.

# **APPENDIX B**

#### LIST OF USEFUL WEBSITES

- <u>www.google.com</u>
- <u>http://www.pubs.asce.org/journals/jrns.html</u>
- <u>www.warez-BB.org</u>
- <u>http://www.csiberkly.com</u>
- <u>www.iitk.ac.in</u>
- <u>www.iisc.ernet.in</u>
- <u>www.nicee.org</u>
- <u>http://www.bssconline.org/NEHRP2000/comments/provisions</u>
- <u>http://icjonline.com</u>

## APPENDIX C

### LIST OF PAPERS PUBLISHED

- Dhorajia D. H. and Patel P. V., "Seismic Analysis of Coupled Shear wall Building considering different types of support condition", *National conference on Recent Trends in Geotechnical and Structural Engineering (RTGSE-2007)*, Jaipur, 22-23 December, 2007, pp. 229-232.
- Dhorajia D. H. and Patel P. V., "Three Dimension modeling of coupled Shear Wall", National conference on challenges and applications in Building Science and Technology (CAM2TBST-2008), Roorkee, 7-8 February, 2008, pp. 19-28.

## LIST OF PAPERS COMMUNICATED

- 1 Dhorajia D. H. and Patel P. V., "Static Nonlinear Analysis Coupled Shear Wall", International conference on Innovation in Building Materials, Structural Designs and Construction Practices (IBMSDCP-2008), Tamilnadu, 15-17 May, 2008. (Paper accepted)
- 2 Dhorajia D. H. and Patel P. V., "Three dimensional Dynamic and Static Nonlinear Analysis of Core and Coupled Core Shear wall building", *National Conference on Infrastructure Development (IDCE-2008)*, Hamirpur, 16–17 May 2008. (Paper accepted)
- Dhorajia D. H. and Patel P. V., "Static and Dynamic behavior of Coupled Shear Wall", 14<sup>th</sup> World Conference on Earthquake Engineering (14WCEE-2008), China, 12-17 October 2008. (Abstract accepted)