STATIC AND DYNAMIC ANALYSIS OF COUPLED SHEAR WALL IN HIGHRISE STRUCTURE

By

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DEPARTMENT OF CIVIL ENGINEERING Ahmedabad 382481 May 2006

STATIC AND DYNAMIC ANALYSIS OF COUPLED SHEAR WALL IN HIGHRISE STRUCTURE

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

Kirti Sundar Dash (04MCL002)

Guide Prof. P.V. Patel



DEPARTMENT OF CIVIL ENGINEERING Ahmedabad 382481 May 2006

CERTIFICATE

This is to certify that the Major Project entitled "Static and Dynamic Analysis of Coupled Shear Wall in Highrise Structure" submitted by Mr. Kirti Sundar Dash (04MCL002), 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

Taipei 101, Petronas twin Towers, Sears Tower and World Trade Centre are just a few examples testifying to the human aspiration to build increasingly highrise structures. The basic structural behavior of highrise structure is similar to the cantilever coming out of the ground. The lateral forces generated from wind and earthquake is critical in highrise structures. Therefore to resists these lateral forces, lateral load resisting structural systems are required. The examples of lateral load resisting structural systems are rigid frame, braced frame, shear wall and coupled shear wall.

From the past seismic events like, it observed that structural wall subjected to brittle kind of failure due to insufficient ductility to control its behavior. Then the researchers came up with new idea by connecting the walls through the framing beams, which is known as coupled shear wall. In CSW, the forces are redistributed through out the entire structure instead of concentrating at base of walls, which is happened in case of structural wall. The performance index of the coupling action is the degree of coupling. The degree of coupling (DC) depends on the geometry of CSW. The geometry of CSW includes wall length, wall height (or number of storey) depth and span of coupling beam. Therefore a parametric study is carried out to study the behavior of CSW. The behavior of CSW includes elastic static, elastic dynamic and inelastic static behavior.

From the elastic static behavior of CSW, it is observed that as the depth of coupling beam increases the degree of coupling increases at the lower storey and DC increases for the lower depth of coupling beam at higher storey. The time period of CSW is studied and compared with current codal provision of IS 1893 : 2002, Wallace and Moehle's formula and O. Chaallal's formula. From the analysis results of ETABS, it is observed that the time period is closer to the Wallace and Moehle's formula and there is considerable difference between codal provision and analysis results. In the present scenario, the codal provision needs to incorporate the dimensions of CSW elements in the time period formula for CSW building. As far as inelastic behavior is concern, the failure pattern of CSW is studied considering different parameters. The guideline for well proportioned

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CSW is proposed in terms of span to depth ratio of coupling beam. When the degree of coupling beam increases the formation of plastic hinges also increases. More the coupling action more will be the dissipation of energy. Therefore response reduction factor in case of CSW building can be increased in compared to structural wall building.

The seismic design of CSW and its foundation for different parameters is carried out. The effect of soil condition on the responses of CSW and its foundation are also studied. As the allowable bearing capacity of soil decreases the axial force decreases and moment increases in the wall rapidly at lower storey. For the implementation of CSW concept, the response of CSW in a 51-storey building is studied. At present, the construction of 51-storey building is going on in Mumbai.

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NOTATION

n	Number of stories
t	Wall thickness
W	Unit floor weight including tributary wall weight
L	Distance between center of gravity of wall 1 and 2
М	Modal mass participation
Р	Axial load in wall
Т	Time period
W	Average seismic weight
h _n	Building height
h _s	Mean storey height
A_{st}	Area of Steel reinforcement
C _u	Compressive force along diagonal
D_w	Wall length
E _c	Concrete modulus of elasticity
Fo	peak value of earthquake induced resisting force in corresponding
	to elastic system
Fy	Yield strength of inelastic system
H _b	Coupling beam Depth
L _b	Coupling beam Span
M_0	Total overturning moment
M_1	Moment resisted by wall 1
M ₂	Moment resisted by wall 2
T _u	Tensile force along diagonal
V_{u}	Factored Shear force
p	Ratio of wall area to floor plan area for the walls aligned in the
	direction the period is calculated
Δ_{o}	Maximum deformation in corresponding to F_y for elastic system
Δ_{y}	Yield deformation in corresponding to inelastic system
Δ_{m}	Maximum deformation corresponds to inelastic system
μ	Ductility factor
# , ф	TOR Steel
CSW	Coupled shear wall
DC	Degree of coupling

1.1 HISTORY

In the design of multistoried building the lateral forces like wind, earthquake and thermal load are critical due to height of the structure. To resist these lateral forces, efficient lateral load resisting systems are required. The developments of structural systems are depending on the available materials, the level of construction technology, services systems and use of the building. In the ancient age, timber and bricks were the two materials available for construction. Timber has lacked strength for buildings of more than about five stories, and very susceptible to fire hazard. Brick has high compressive strength and fire resistance. In 1891, the sixteen storied Monadnock Building in Chicago was constructed with the 2m thick load bearing masonry walls [1]. The spaces in the lower floors were largely occupied by the walls. The demand for space was increased due to industrialization, which was led for the innovations of the higher strength and structurally more efficient materials wrought iron and subsequently steel and introduction of elevator. In 1931, the 102 storied Empire State Building of structural system braced steel frame was constructed up to height of 381m. At that time the tallest concrete building was Exchange building in Seattle of only 23 stories. In 1973, 110 storied framed tube structure World Trade Centre in New York, and in 1974, 442m high bundled tube structure Sears Tower in Chicago were constructed [1].

Reinforced concrete tall buildings were introduced approximately two decades after the first steel tall building. In 1960, the multistoried buildings in R.C.C. with shear wall concept were started. That was a major step toward the reinforced concrete high rise structure with the introduction of shear walls for resisting horizontal loading. The shear wall structural system has given a significant development in structural forms of concrete high rise buildings, freeing them from the previous 20 to 25 storey height limitations of the rigid frame systems. At that time M20 grade concrete could be achievable. In the last decade, with the development of the high strength concrete, up to M130 grade concrete can [2] be achieved and similarly high strength steel 400 MPa and 500 MPa of yield strength are available. The Petronas twin tower in Malaysia, which is tube in tube high strength concrete structure, was constructed in 1992-1998. The structural system of 88 storied twin tower of height 452 m is the combination of shear wall and framed structure and up to M80 grade concrete was used in this project [3]. In 1970s, a number of important structures using steel plate shear wall were designed and constructed in USA and Japan. The 35 story office building in Kobe, Japan was designed with steel shear wall structural system [4].

Earlier designs of concrete structural walls exhibited classical brittle shear failure modes in some earthquakes, particularly in the 1964 Alska event. Then the researchers came up with new concept for the remedial measures of the structures. That concept was Coupled Shear wall [5].

1.2 SHEAR WALL

Shear wall or Structural wall is the structural element, which principally resists the lateral loads generated from the earthquake or wind load, with minimum deflection or storey drift due to it's large in plane stiffness. From the name shear wall (SW), it seems that the shear might control the behavior but in real sense the flexure controls the behavior. Therefore some of the authors prefer to mention structural wall rather shear wall [6]. The structural walls provide the satisfactory results to meet the basic criteria (i.e. stiffness, strength and ductility) of the designer. The great advantage of structural wall is, due to it's large in plane stiffness, it protects the nonstructural damage through limiting inter storey drift. A special detailing of it provides a dependable ductile behavior. Otherwise the structural walls are inherently brittle in poor detailing. Therefore code specified a lower ductility factors for the structural wall than frames.

When the aspect ratio (height / length) of structural wall is equal to or exceed 4, the wall is considered as "Slender". If the aspect ratio of wall is equal to or less than 2, it is considered as "Squat". The slender structural wall is governed by flexure and forms a plastic flexural hinge near the base of the wall under lateral load. The ductility of the wall depends on (a) percentage of longitudinal reinforcement concentrated near the boundaries of wall, (b) axial load, (c) lateral shear, (d) thickness of web and (e) reinforcement in web. The higher axial load stress and higher shear stress reduce the flexural ductility and energy

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absorbing capability of wall. Squat wall normally governed by shear strength. The shear strength of Squat wall is limited by diagonal tension, sliding shear, or diagonal compression. In inelastic behavior of Squat wall, the shear hinge is forming at the base of wall under lateral load [6-8].

1.2.1 Type of Shear Wall

As per ATC-40, the concrete wall elements are classified as solid walls, punched walls, perforated walls or coupled walls (FIGURE 1.1). Punched walls have openings those aren't aligned vertically. Perforated walls are made of vertical and horizontal wall segments those are arranged in a regular pattern. Coupled shear walls are special case of perforated walls, where two or more walls are connected by horizontal framing components. In coupled walls and perforated walls, the vertical components will often be referred to as wall piers. The horizontal framing components are the coupling beam, whereas those of perforated walls will be referred to as spandrels [8].



FIGURE 1.1 WALL ELEMENTS OF BUILDING

Shear wall can be designed in different structural geometries with different materials or combination of materials. The shear walls also can be categorized as per the construction material and the geometrical shape, which is discussed in the next sub-sections.

1.2.1.1 Based on Materials

The structural wall can be constructed by using materials like Timber, Masonry, Steel, Concrete or Composite (concrete and steel). The reinforced concrete is used widely for the construction of structural wall not only in lowrise building but also highrise buildings. Based on the materials type of shear walls are shown in FIGURE 1.2.

Chapter 1 Introduction



FIGURE 1.2 SHEAR WALL BASED ON MATERAILS

1.2.1.2 Based on Geometry

Mainly the structural walls based on different geometry can be categorized as Planar wall and Core wall. In the category of planar walls, the special type of walls are Rectangular shear wall, Coupled shear wall, Rectangular shear wall with boundary element and Coupled shear wall with boundary element. Similarly the core walls are having special types are Box wall and Coupled Box wall. The FIGURE 1.3 shows the different types of shear wall based on geometry.

1.2.2 Planning Aspects of Shear wall

The structural designer decides the most desirable location for the structural wall in floor plan by consultation with Architect. The major structural aspects to be considered for the planning of walls are [6]:

- a) Lateral Load Intensities
- b) Architectural Context
- c) Regularity and preferably symmetry in the positioning of walls within the building will reduce adverse torsion effects.
- d) Type of Shear wall used



f) Coupled box wall

FIGURE 1.3 SHEAR WALL IN DIFFERENT GEOMETRY

e) Torsional stability

For the best torsional stability as many of the walls as possible should be located at periphery of the building. The walls on each side may be individual cantilevers or they may be coupled to each other.

f) Available overturning capacity of he foundations

The another aspect is the more gravity load should be carried by the structural wall, the less will be the demand for flexural reinforcement in the wall and the foundation become more stable to absorb the overturning moments generated in the wall.

g) Ductility aspect

The configuration of walls in elevation should be checked to ensure that feasible shear resistance and flexural strength with adequate ductility capacity can readily be achieved.

h) Openings in floors

Efficiency of force transfer from diaphragms to walls should be proper, where large openings exist in the floors.

1.2.3 Behavior of Shear Wall

When seismic forces tend to push the shear wall, it causes an overturning moment. The overturning moment causes tension and compression boundary forces in the shear wall. Therefore the concentration of internal stresses are more in the vertical extreme edges, which is shown in FIGURE 1.4. When height to width ratio great than 2, the shear wall behave like a vertical slender cantilever beam. The primary mode of deformation is bending; shear deformation are small. Squat shear walls usually have height to width ratio less than half. For the squat shear wall amount of shear deformation is significant.



FIGURE 1.4 SHEAR WALL BEHAVIOR

1.3 COUPLED SHEAR WALL

In the cantilever shear walls, inelastic behavior of the entire wall is dependent on the plastic hinge zone at the wall base, where large rotations and yielding of reinforcement takes place. As a result the stiffness, strength, ductility and means of dissipation of energy of the entire structural system are focused on the base portion of shear wall FIGURE 1.5(a). Brittle failure mechanism and deformation modes with limited ductility aren't permitted to control behavior during the design seismic event. Example of failures are like concrete crushing in boundary element, diagonal tension or compression caused by shear, sliding shear along construction joints, shear or bond failure along lapped or anchorage, instability and bar buckling of principal compression steel. Subsequently research has given the solutions to overcome this problem. These walls should be designed through appropriate reinforcing of ordinary cantilever walls or through the use of concrete coupled walls.

Coupled shear wall (CSW) is an efficient structural system to resist the lateral loads due to wind and earthquake in high multistoried building. It can be a series of walls coupled by beams and slabs or slabs or as central core structure with openings to accommodate doors, elevator wells, windows, doors and corridor.

1.3.1 Behavior of Coupled Shear wall

When the lateral loads are acting on the coupled shear walls, both of walls bend in flexure and coupling beam is displaced vertically in opposite direction to each other at the ends. The vertical displacement causes the double curvature bending of the beam. The double curvature bending induces internal shear in the beam. The induced shear causes axial tension and compression at the c. g. line of the wall in windward and leeward side respectively. The couple generated by the lever arm in between axial tension and compression force resists the external moment, which is shown in FIGURE 1.5(b).

In a cantilever wall the total overturning moment, M_0 is resisted at the base in the form of flexural stresses. While in coupled shear walls the axial forces as well as moments are being resisted. At any section the following equilibrium condition should be satisfied [6, 9]:





FIGURE 1.5 FORCES IN (a) STRUCTURAL WALL, (b) COUPLED SHEAR WALL

Where M_1 and M_2 are the moments resisted by the wall 1 and 2 respectively. The coupling beams have two main beneficial [9]:

- 1. It reduces the moment that must be resisted by the individual walls.
- 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 protect the walls from excessive damage.

1.3.2 Degree of coupling

The degree of coupling is the performance index of the coupled shear wall. From numerous analytical as well as experimental studies, it is proved that the structural behavior of the coupled shear wall is greatly influenced by the behavior of their coupling system. The coupling system depends on the geometry of the coupling beam relative to the walls. The coupled shear wall will respond to the lateral loadings on the basis of its degree of coupling [6, 9].



FIGURE 1.6 (a) DEFORMED SHAPE OF CSW, (b) STRAIN DISTRIBUTION OF CSW

Degree of Coupling =
$$\frac{P \times L}{M_0}$$
 (1.2)

Depending on degree of coupling, as shown in FIGURE 1.6 the coupled shear wall categories as [9]:

- a) Low degree of coupling
- b) Intermediate degree of coupling
- c) High degree of coupling

When the degree of coupling is very less (approximately less than 0.33), there is virtually no composite action. The both of the walls deflect independently in a flexural mode like a braced frame. If the stiffness of coupling beam will vary with respect to stiffness of walls, at certain degree of coupling the coupled wall system will behave like a rigid frame. Then the coupled shear wall will deflect in shear mode. In between the above two extreme condition the coupled shear wall behave in flexural-shear mode for the intermediate degree of coupling like wall-frame structure (FIGURE 1.7). For this reason coupled shear wall may be considered as the generic high-rise structural system and coupled wall theory as

a generalized theory for flexural-shear cantilevers. The degree of coupling becomes major issue in different standards, when it relates with the response reduction factor (R).



FIGURE 1.7 MODE OF DEFLECTION

The Canadian concrete standard (CSA 1994) directly linked the DC to response reduction factor as follows [9]:

$$DC \ge 0.66 \Rightarrow R = 4.0$$
 (Ductile fully coupled) (1.3)

$$DC \le 0.66 \Rightarrow R = 3.5$$
 (Ductile partially coupled) (1.4)

The above equations indicate that the less coupling effect lead to lesser dissipation of energy and vice versa. Therefore the more coupling action is the source of dissipation of energy.

1.4 OBJECTIVE OF STUDY

Highrise structures are the symbol of prestige from the beginning of civilization. The rapid growth of population and limited space again fuel the issue to many folds. The structural aspect of the highrise structure is critical to the lateral loadings generated from earthquake and wind. To resist these lateral loads, lateral load resisting structural system like coupled shear wall is required. The elastic behavior of CSW leads to the reduction of moments in wall due to the coupling phenomenon. The coupling action greatly influenced by the degree of coupling. The properties of the elements of a structure always controls it's behavior. Similarly the sizes of wall and coupling beam and height of CSW also control its behavior. To meet the structural needs, stability, strength, safety and economy, the suitable element sizes are required. Therefore a parametric study on CSW is conducted considering the parameters as depth and span of coupling beam, wall length and number of storey. The effects of these parameters on the responses and DC of the CSW will justify the choice of the suitable geometry. For the same, degree of coupling and the responses (axial force, moment and shear) in wall and coupling beam is studied for different height of CSW.

The dynamic excitation, earthquake and wind, are to be responded by the structure as per the dynamic characteristic like natural time period. The natural time period depends on mass and stiffness of structure. The geometry of the structure governs the mass and stiffness of the structure. Therefore the dynamic behavior of CSW is also studied with the parameters as defined above. The time period given in codal provisions, various authors and software results are compared.

As per as earthquake is concern, the inelastic behavior of the structure plays a vital role. It gives an idea about the after yield behavior. The after yield behavior depends on ductility of the structure. By incorporating the flexural hinges in the structure, the ductility can be improved. The other issue is the wall should fail after the failure of the coupling beam in case of CSW. The above two issue, flexural hinge formation and the failure sequence, depends on the geometry of the elements of CSW. Again the failure pattern of the CSW can be designed to meet the structural need by considering suitable geometry of the elements of CSW. Therefore to get the suitable geometry of CSW for desired failure pattern a parametric study is conducted.

The behavior of the structure can be simulated in reality through the structural design. The structural design and detailing of CSW also carried out and compared with shear wall design. The forces generated with in the structure transfer to the soil strata through the foundation. Here the effects of response generated with in the CSW on it's foundation are also studied. To study the implementation of the CSW concept in live project, the analysis of 51-storied highrise residential building in Mumbai is carried out and the responses of the CSW in building are studied.

1.5 SCOPE OF PRESENT WORK

The present work deals with the behavior of coupled shear wall in highrise structure. The behavior of coupled shear wall includes,

- a) Linear static behavior
- b) Linear dynamic behavior
- c) Nonlinear static behavior

To study all these behavior of CSW, the parameters are defined as depth and span of coupling beam, wall length and number of storey. The depth of coupling beam is varying from 0.5 to 1.5m, span of coupling beam is 1.0 to 3.0m, wall length is 2.0 to 6.0m and the number of storey is 5 to 30. The details of parameters are given in TABLE 1.1-1.3 (SHEET No.1). In the parametric study, CSW is the lateral load resisting system of commercial building, for which typical floor plan and elevation of CSW are shown in FIGURE 1.8 and 1.9(SHEET No. 1).

The responses and properties observed in the parametric study of linear static behavior of CSW are,

- a) Axial force, shear and moments (Structural Wall)
- b) Shear and moment (Coupling beam)
- c) Degree of coupling

In the linear dynamic behavior of CSW, the natural time period of CSW is studied for all the parameters. The natural time period obtained from codal provision, various formulae and ETABS [10] results are compared. The failure patterns are observed in nonlinear static behavior of 10, 15 and 20 storey CSW. The criterion for desire failure pattern is proposed in terms of span to depth ratio of coupling beam. Seismic design of CSW is carried out for 15-storey considering all parameters. The design results of CSW and uncoupled wall are compared. The effects of responses of CSW on the foundation are also studied. The CSW concept is implemented in 51-storey residential building at Mumbai, which is a live project.

1.6 ORGANIZATION OF REPORT

The Report comprises nine chapters. The chapter 1 is the introductory part of the report, in the beginning history of lateral load resisting structural system is discussed. Then general discussion of Shear wall, types of SW and its behavior **Chapter 1 Introduction**

SHEET No. 1

and different aspects to be considered for the planning of SW are also highlighted. It explains about the need of coupled shear wall and its behavior. The importance of degree of coupling with respect to response reduction factor is also explained. The chapter also emphasizes the objective of study and scope of the present work. The chapter 2 deals with the literature review. In literature review the essence of works related to Coupled shear wall was done by different authors from the year of 1967 to 2005 is discussed in three categories. The categorization is done based on linear static, linear dynamic and nonlinear static behavior of CSW.

In the chapter 3, the approaches for the modeling of CSW are discussed. The responses like axial forces, shear and moment in wall and coupling beam considering continuum, wide-column frame and finite element model are compared. In chapter 4, the methodology applied for the parametric study of coupled shear wall is illustrated with a typical example. The responses of CSW from the elastic static analysis for the depth and span of coupling beam, wall length and number of storey as parameters are discussed. The degree of coupling and stress in horizontal direction of CSW are also studied.

In chapter 5, time period of CSW and CSW building are compared with analysis results from ETABS, formula given by O. Chaallal, Wallace & Moehle and IS 1893 (Part 1) : 2002. The mode shapes of the CSW for the different modes are also studied in this chapter. The chapter 6 deals with the inelastic behavior of CSW. The failure pattern of CSW is discussed by considering different parameters. For the desired failure pattern the relationship in terms of span to depth ratio of coupling beam is proposed in this chapter.

The design of wall, coupling beam and the foundation of 15-storied CSW are discussed in chapter 7. And the design results and responses of uncoupled wall and CSW are compared considering with the different parameters. The effects of responses on the foundation of CSW for different parameters are also studied. The behavior of coupled shear wall is discussed in chapter 8 considering lateral load resisting system of 51-storied building. The study is summarized and concluded in the chapter 9. And the CSW related future scope of work is given in this chapter.

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2.1 GENERAL

The analysis, design and experimental procedure and their conclusions for coupled shear wall are presented by many authors. They tried with different analytical and experimental model to simulate the actual structural static and dynamic behavior of coupled shear wall. There are many assumptions involved with the procedures. The researchers came up with new realistic concepts from time to time to achieve the more realistic behavior. In this chapter the study carried out by different authors based on the behavior of coupled shear wall is discussed in three categories. The categories are linear static, linear dynamic and nonlinear static behavior.

2.2 REVIEW OF WORK ON CSW

The researchers had carried out experimental as well as analytical study for the elastic and inelastic behavior of CSW, which are discussed in different category.

2.2.1 Linear Static Behavior

The works carried by the researchers on linear static behavior of CSW are discussed below.

Coull and Choudhury (1967) [11] had given curves for the rapid evaluation of the stresses and maximum deflections in CSW subjected to either a triangularly distributed lateral load or a point load at the top. The curves were derived from the continuous connection theory, in which the discrete system of connecting beams was replaced by an equivalent continuous medium.

Smith et al. (1991) [1] discussed the different theory for the analysis of coupled shear wall. They also illustrated an example of 20 storied CSW in which the distribution of axial forces and the moments in the walls were graphically represented.

Paulay and Priestley (1992) [6] explained the strategies in the location of Structural walls. The behavior of structural wall and CSW were described not only for the elastic stage but for the inelastic stage also. The ductility aspect of structural wall and coupling beam were studied.

Medhekar and Jain (1993) [12] proposed the design and detailing approach for the RC shear walls with an example. They also discussed the behavior of RC shear wall under seismic loading and factors influencing structural responses of walls. The influencing factors are height to depth ratio, type of loading, flexural, shear, diagonal and special transverse reinforcement, concrete strength and section shape. The behavior of CSW was also explained.

Chaallal et al. (1996) [9] dealt with the classification of the coupled shear wall and evaluation of degree of coupling between the walls and the coupling beams. For the classification of the coupled shear wall, a simple method which was based on the dynamic properties of coupled shear wall was also proposed. One numerical example was presented for the applicability of the proposed methods. The evaluation of the degree of coupling requires the structural analysis of coupled shear wall. But it is difficult in the preliminary proportioning process. A simple method was proposed in this paper for the evaluation of degree of coupling without the structural analysis. <u>The author also proposed formula for the fundamental periods of CSW and explain the importance of response</u> <u>reduction factor with respect to degree of coupling in different codal provisions</u>.

Arslan et al. (2000) [13] elucidated the required number of stiffening beams in the coupled shear walls to efficiently control the large drift at top and bending moment at bottom in tall multistoried building. They had solved examples with computer program and verified by SAP90 structural analysis program.

Doran (2003) [14] had presented an equivalent bar frame modeling for the analysis of coupled shear wall structure. In the bar model, the stiffness of the coupling beam was modified by a factor which depend on the geometrical and material aspects.

Harries et al. (2004) [15] had done parametric study of over 2000 coupled core wall geometries to investigate appropriate parameters to identify efficient coupled shear wall geometry. They had varied geometrical parameters like number of stories, building height, length of wall, breadth of wall, length of coupling beam, and depth of coupling beam. The responses were calculated by elastic analysis of coupled shear walls.

Hindi and Hassan (2004) [16] had prepared a theoretical model to study the behavior of diagonally reinforced coupling beams. The assumption in model was that all loads were resisted by diagonal tension and compression. <u>The diagonal compression was carried by the diagonal reinforcement and the concrete core surrounded by the diagonal bars in that direction. Where as the diagonal tension was carried only by the diagonal reinforcement. Either rupture of diagonal reinforcement or concrete core defines failure of the coupling beam.</u>

Lam et al. (2005) [17] had experimentally proposed that the steel plate embedded composite coupling beam with shear studs was more effective than the normal R.C.C beams under worst earthquake or wind load condition.

Jain and Murty [18] proposed draft provisions and commentary on code IS:13920. In the context of coupled shear wall, the proposed changes are as follows,

- a) "In case of coupled shear walls, the thickness of the wall shall be at least 200mm (Clause No. 9.1.2)". And in the present code minimum thickness is 150mm.
- b) If the earthquake induced shear stress in the coupling beam exceeds $(0.1 \times l_s \times \sqrt{f_{ck}})/D$ or when l_s / D is less than or equal to 3, where l_s is the clear span of the coupling beam and D is overall depth, the entire earthquake induced shear and flexure shall, preferably be resisted by diagonal reinforcement (Clause No. 9.5.1)". The authors explained experiments to show the reduction in efficacy of longitudinal reinforcing steel in coupling beams in walls when l_s / D is more than 3.

2.2.2 Linear Dynamic Behavior

The studies carried out by the researchers in dynamic behavior of CSW are discussed below.

Skattum (1971) [19] had studied the free vibration of planar coupled shear wall, where the walls were coupled together by a system of discrete spandrel beams. He assumed the replacement of spandrel beams by a continuous system of laminae and determined the natural frequencies and mode shapes. He had presented the results in graphical form, from which natural frequencies of coupled shear wall can be extracted.

Mukherjee and Coull (1973) [20] extended the continuous connection method of analysis for study of the free vibration of CSW structures. The theoretical result of natural frequency was compared with the results from tests on model structures. The results were used for the evaluation of dynamic response of the structure applying the lateral displacement.

Tso and Rutenberg (1977) [21] presented a procedure on the response spectrum technique to estimate the design response for CSW under seismic loading. They had proposed some design charts from which the responses like base shear and natural frequency can be calculated.

Cheung et al. (1977) [22] used the finite strip procedure to predict the free vibration response of both planar and non-planar coupled shear wall. He compared the frequency from two different approaches. Both the approaches considered that the walls were vertical cantilever strip. And comparison was made between modeling the spandrel beams as discrete beams and as an equivalent continuum. He observed the same frequency from both the approaches.

Basu (1983) [23] proposed the seismic design charts for the coupled shear walls. According to him the design charts prepared by Wai K. Tso and Avigdor Rutenberg [21] may not be sufficiently accurate for the entire range of practical dimensions of wall, because the author in his approximate analysis assumed the CSW as slender cantilever. Basu approximated the coupled shear wall as

continuum of uniform properties. And in his results, it was shown that the <u>mode</u> <u>shapes of CSW cannot be approximated by the corresponding mode shapes of</u> <u>the slender cantilever</u>.

Lu and Jiang (2003) [24] had conducted shaking table test of the rectangular as well as core coupled shear walls for cyclic loading condition. In this test the failure process and mechanism were studied. The phenomenon of shear lag of the core wall was observed distinctly.

Wang and Wang (2005) [25] presented a method to determine the first two periods 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.

2.2.3 Nonlinear Static Behavior

Many researchers had worked on the inelastic behavior of CSW, which are discussed in this section.

Nayar and Coull (1975) [26] presented an elastic-plastic analysis of coupled shear walls based on the wide-column frame analogy. Incremental loads were applied until a plastic hinge was formed at the most heavily stressed section. This analysis helped to study the behavior from working to ultimate load. The ductility requirement of each plastic hinge could be found out for every load increment up to collapse. And his method also can calculate the ultimate load based on the practical limit of ductility.

Takayanagi and Schnobrich (1979) [27] investigated the non-linear responses of the coupled shear wall under dynamic and static loads. They compared the results like yielding moment, axial forces and shear of the wall as well as beam and their corresponding roof displacement, between analytical and test structural models.

Keshavarzian and Schnobrich (1985) [28] presented a member-by-member modeling technique for non-linear analysis of R/C coupled shear wall under static and dynamic loads. In modeling part they had considered the line elements

Chapter 2 Literature Review

representing the beams and walls connected by rigid links. And coupling beams were idealized as elastic line elements with inelastic rotational springs located at the member ends.

Chaallal and Ghlamallah (1996) [29] had studied the nonlinearity of the foundation and supporting ground with the walls and coupling beam. From the study it was found that for the flexible foundation the fundamental periods lengthen by a maximum of 33% and it amplified deflections by maximum of 81%. That reduced the maximum shear and flexural stress in the walls and the coupling beams in lower stories.

Applied Technology Council (ATC 40) (1997) [8], describes the guidelines for the linear as well as nonlinear behavior of building elements. The inelastic hinge properties are given for the flexure as well as shear in different tables. The procedures for the inelastic analysis of building are also described.

Chaallal and Gauthier (2000) [30] proposed on the nonlinear dynamic analysis of ductile CSW to evaluate the dynamic amplification factor (which take into account the effects of higher modes of vibration associated with the hinge formation in the wall) for Canadian region. <u>He also mentioned that the time period as per code is more conservative than the actual in coupled shear wall building.</u> The time period as per code is suitable for the wall without opening but for walls with opening should be as per O. Chaallal, D. Gauthier, & P. Malenfant (1996) paper.

Federal Emergency Management Agency (FEMA 356) (2000) [7], has given the guidelines for the inelastic hinge properties of all building vertical element. 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 also explained in step by step.

Chaallal and Gauthier (2001) [31] had analytical studied on nonlinear deformation and ductility response of reinforced concrete ductile coupled shear walls under seismic loadings. Results indicated that the maximum interstorey drift from dynamic analyses was less than that obtained from static analyses

with NBCC specified lateral forces. It was also found to be substantially lower for tall CSW compared to short or medium-height walls, and decreased only slightly as the degree of coupling increased within the range considered in this study. The ductility demand factor for walls as well as coupling beams was decreased with increase in number of storey irrespective of degree of coupling (DC). <u>As the DC increased the number of plastic hinge formations increased in the coupling beams and drastically decreased in the walls.</u>

Sherif et al. (2002) [32] had discussed the proposed design procedure and modeling of the hybrid coupled shear wall. The finite element model was created to study the static nonlinear analysis (or pushover analysis) and the important aspects as material inelasticity, concrete cracking, opening and closure of the gap between the embedded steel beams and the walls, and geometric nonlinearity. They found that this method gave appropriate results for the low & medium rise building but not for the high rise building. Because <u>in static nonlinear analysis the first mode was considered as the critical, but for the high rise building higher modes are critical.</u> The analytical results were also compared with the test results.

Sherif and Christopher (2002) [33] had explained the procedure for the nonlinear static analysis of hybrid coupled shear walls. They studied the variation of parameters like target displacement, base shear, wall reactions, storey drifts over the height, wall plastic rotation, coupling beam rotations and wall beam connection response with respect to the coupling ratio. The authors had mentioned that the pushover analysis, which was implemented in this isn't representing the true behavior of system under strong earthquake.

Zhao and Kwan (2003) [34] conducted test to study the nonlinear behavior of reinforced concrete coupling beams. They observed that a coupling beam failing in shear would have a relatively low ductility, a coupling beam failing in flexure could have a ductility ratio of 10 or higher. Therefore the coupling beam should be designed to fail in flexure and have shear strength at least 20% higher than the flexural strength.
Hassan and Sherif (2004) [35] had investigated the dynamic nonlinear seismic behavior of hybrid coupled shear wall (i.e. concrete walls with steel coupling beam). They developed finite element models for the analysis and verified from the test data. The parameters studied were interstory drift, dynamic base shear magnification, wall rotation, coupling beam plastic rotation.

Lu and Chen (2005) [36] had proposed a nonlinear macro model to simulate the coupled shear wall. The model for walls having some key characters, such as shifting of neutral axis within the wall section and interaction among flexure, shear and axial forces. The nonlinear model for coupling beams represented the characters like elastic, plastic, shear and interface bond slip action. They had conducted a pseudo static loading test on three coupled shear wall specimens to validate the analytical results.

3.1 INTRODUCTION

A reasonably accurate assessment of the CSW behavior is necessary to form a proper representative model for analysis. The accuracy of analytical modeling of complex CSW systems has always been of concern to the Structural Engineer. The modeling of structure for analysis is dependent to some extent on the approach to analysis. For the approximate analysis of CSW the continuum model is used. The accurate methods for the analysis of CSW are wide-column frame method and finite element method. In wide-column frame method, the CSW is modeled by equivalent wide columns that consist of a column at centroidal axis of the wall, with rigid arms at the beam levels to represent the effects of the wall's width. In finite element method the shell elements are used for the modeling of CSW. This chapter deals with the different modeling techniques for the approximate and accurate analysis of CSW. The ETABS [10], which is finite based software, is used for the modeling and analysis of CSW for the wide-column frame method and finite element method.

3.2 MODELING APPROACHES

The integrated design process of structural system is shown in FIGURE 3.1.



FIGURE 3.1 DESIGN PROCESS OF STRUCTURAL SYSTEM

After the completion of the architectural model of the building, the structural engineer takes the responsibility to make the strong, stable and durable structure. For the same a mathematical model of building, representing its characteristics, is prepared. Finally a "Model" of the structure is analyzed to estimate the response of structure under worst combination of loads. For the modeling and analysis of structures software can be used. In which the excitation (Loads, vibrations, settlements, thermal changes) can be given to the

Chapter 3 Modeling of Coupled Shear Wall

structural model and the corresponding responses (Displacements, strains, stresses) obtained to be used in the design process. The proper representative structural model will simulate the real structural system accurately. If the modeling of the structure is improper, the rest of the design process will be affected and design output may result into unsafe or uneconomical building. The objectives of modeling a CSW for analysis or design of a structure are (a) to accurately estimate the storey deflection shape of the wall and (b) to determine the distribution of forces to that wall and coupling beams under load.

As far as CSW is concern, the modeling approaches of it always a matter of challenge to the structural engineer. There are different approaches to model the coupled shear wall as follows:

- (i) Continuum model / Lamina model (FIGURE 3.2)
- (ii) Wide-column frame model / Linear element model (FIGURE 3.3)
- (iii) Finite element model (FIGURE 3.4)
- (iv) Plastic model (FIGURE 3.5)

3.2.1 Continuum model

The most important approximate method is the continuous medium technique (FIGURE 3.2). In this method, all horizontal connecting elements (coupling beam) are assumed as an equivalent continuous connecting medium between the vertical elements (walls). This can be achieved with reasonable accuracy only for a uniform system of connecting beams and walls. The two dimensional plane structure is transformed into essentially one-dimensional one. This makes possible to study the behavior of structure in the form of an ordinary linear differential equation, which can give a closed form solution.

3.2.2 Wide-column frame model

This is the most common coupled shear wall model (FIGURE 3.3). The CSW structure is replaced by an equivalent plane frame of beams and columns. The wide walls are modeled by a line column situated at the centrodial axis with rigid horizontal elements connecting the centrodial axis to the outer fibers at each floor levels. The rigid horizontal element transmits the rotational and vertical displacement effects at the edge of the wall to the connecting beams.



FIGURE 3.2 CONTINUUM MODEL



FIGURE 3.4 FINITE ELEMENT MODEL



FIGURE 3.3 WIDE-COLUMN FRAME MODEL



FIGURE 3.5 PLASTIC MODEL

3.2.3 Finite element model

Finite element model is generalized approach to study the behavior of structure. If the wall contains irregular openings, then it may be difficult to represent the structure by plane frame model. In that case it is better to use the finite element model (FIGURE 3.4). In this technique, the surface concerned is divided into a series of elements, generally rectangular, triangular or quadrilateral connected at a discrete set of nodes on their boundaries. By setting up the structural stiffness matrix of the elements, all the nodal displacements and associated forces can be solved. The technique has the advantage that a refined mesh may be made in regions of high stress gradient or particularly complex geometry and a much coarser mesh in regions of low or uniform stress. At the time of meshing for the finite element analysis, the rectangular elements should be as square as possible, triangular elements should be equilateral, and quadrilateral elements should be parallelograms with equal sides, for more accurate results. The finite element method is now well established and documented and used for structural

analysis through general purpose programs like ANSYS 8.0, SAP 2000, ETABS [10].

3.2.4 Plastic model

The plastic collapse mechanism of a coupled shear wall can be modeled as shown in FIGURE 3.5 using the plastic moment capacity formulations for walls and coupling beams developed [37]. This model can be used to determine the ultimate shear capacity of the wall and the maximum shears and axial forces to the walls and coupling beams.

3.3 SELECTION OF SOFTWARE

There are various aspects are involved in the selection of software for the parametric study. The software studied are ANSYS 8.0 and ETABS 8.0. The main criteria are the limitations of the resources and the expected responses needed from software. Firstly the ANSYS was studied for the parametric study purposes. The element used for this study of coupled shear wall is 8 nodded quadrilateral element (PLANE82). The element size in the meshing is 100 x 100 mm. The responses studied from static analysis. The modeling of couple shear wall in ANSYS is shown in FIGURE 3.6. The coupled shear wall is modeled for 5-storey building. As it was mentioned in chapter 1, the coupled shear wall is studied for 10, 15, 20, 25, and 30 storey. For more stories the number of nodes as well as the number of elements will increase for a suitable meshing. Due to the limitations of resources it wasn't possible to continue with this software. Therefore the second option ETABS is tried to overcome the above mentioned problems.



FIGURE 3.6 USING ANSYS (a) MODELING (b) MESHING AND LOADING OF CSW

3.4 COMPARATIVE STUDY OF DIFFERENT MODELING TECHNIQUE

For the study of different modeling technique of CSW, worked example of coupled shear wall structure given in Coull and Smith [1], is considered. The real structure of coupled shear wall is shown in FIGURE 3.7.



FIGURE 3.8 DISTRIBUTIONS OF AXIAL FORCES AND BENDING MOMENTS IN WALLS (CONTINUUM MODEL)

The distribution of axial forces and the bending moments in both of the wall is shown in FIGURE 3.8 for the continuum model.

The wide-column frame model of CSW is carried out by using ETABS, which is shown in FIGURE 3.9. The distribution of axial forces and moments in the walls are shown in FIGURE 3.10.



FIGURE 3.9 WIDE-COLUMN FRAME MODEL OF CSW USING ETABS



FIGURE 3.10 DISTRIBUTIONS OF AXIAL FORCES & BENDING MOMENTS IN WALL (WIDE-COLUMN FRAME MODEL)

The finite model of CSW is shown in FIGURE 3.11. The FE modeling is carried out by using ETABS. The distribution of axial force and bending moment in walls are shown in FIGURE 3.12.



FIGURE 3.11 FINITE ELEMENT MODEL OF CSW USING ETABS



FIGURE 3.12 DISTRIBUTIONS OF AXIAL FORCES AND BENDING MOMENTS IN WALLS (FINITE ELEMENT MODEL)

					Axial	force	Coup	ling		
	Momen	t (kNm)	Shear	: (kN)	(k	N)	bear	m	Degree	
							Momen	Shear	of	Deflection
Model	Wall 1	Wall 2	Wall 1	Wall 2	Wall 1	Wall 2	t (kNm)	(kN)	coupling	(m)
Continuum										
model	2902	7964	247	677	1876	1876	102	102	0.58	0.016
Wide-column										
frame model	3805	8797	370	554	1659	1659	112	112	0.51	0.022
Finite element										
model	3903	9199	372	550	1596	1596	124	468	0.49	0.023

TABLE 3.1 COMPARISON OF MAXIMUM RESPONSES IN COUPLED SHEAR WALL

The maximum responses axial force, moment and shear force in wall and coupling beam obtained from continuum model, wide-column model and finite element model approaches are given in Table 3.1.

3.5 CSW MODEL IN WOODEN LOADING FRAME

To study the behavior of CSW, one model is prepared, which is made up of resinous material. The model of CSW is fixed in the wooden loading frame. Then the static lateral loads are applied along the major axis at floor levels. The FIGURE 3.13(a) is the undeformed shape of 6-storey CSW model on the wooden loading frame. After application of lateral load, the FIGURE 3.13(b) shows the deformed shape of CSW. The behavior of CSW is simulated through this model. One of the significant features to be noticed here the double curvature bending of coupling beam, which is mainly responsible for the coupling action in the CSW. The deformation at different storey level of coupling beam is visualized after application of lateral load. The behavior of CSW for different depth of coupling beam on the same model can also be visualized.

3.6 CLOSING REMARKS

From the comparative study for different modeling approaches, it is observed that the shear in the coupling beam in finite element gives higher results than other approaches. In the continuum model, axial force and degree of coupling is more than other approaches. Also the moment in the walls, moment and shear in coupling beam are less in compared to wide-column frame and finite element model. Wide-column frame model is commonly used in practice. This approach is giving an intermediate result of responses between continuum and finite element model. But the shear in the coupling beam is obtained considerably less.



(b) DEFORMED SHAPE OF CSW FIGURE 3.13 CSW MODEL IN WOODEN LOADING FRAME

4.1 INTRODUCTION

This chapter deals with the static behavior of coupled shear wall. The methodology for the parametric study is illustrated with an example. The responses like axial force, bending moment and shear in wall as well as the shear and bending moment in coupling beam are studied for the individual parameters. The parameters are beam depth, wall length, beam span and number of storey. Then comparison is made on the behavior of coupled shear wall as per the different parameters. In section 4.5, the behavior of coupled shear wall is explained by varying the beam depth for each storey. Similarly in section 4.6 and 4.7, the behavior of coupled shear wall is discussed by varying the depth of the wall and span of the beam for each storey.

4.2 METHODOLOGY FOR PARAMETRIC STUDY

Methodology adopted for the modeling and application of loading are explained in this section. Also the procedure of parametric study in totality is given. The procedure is explained in the following steps:

Step-1: Modeling and Meshing of coupled shear walls

The coupled shear wall is modeled in 2-dimension and quadrilateral shell element is considered, which is explained in section 3.2.3. The meshing of the model is done by using the sizes of elements as 250×250 mm.

Step-2: Material and Geometrical Properties

The material properties are:

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

The geometrical properties are to be considered as per the TABLE 1.1 (SHEET No. 1).

Step-3: Boundary condition

The boundary conditions applied at the base of coupled shear wall is fixed.

Step-4: Application of loadings

In the study of coupled shear wall the lateral loads are critical. Therefore the seismic loads are considered for the analysis of the CSW. Exact determination of the earthquake forces is almost impossible. The reason is, the nature of earthquake motions can't be predicted and the response of the structural elements to these ground motion, isn't quite known. By making the best approximation, which reduces the problem to a one dimensional body force system and most of the codes use this simplified version for their specifications regarding the earthquake forces.

There are two approaches for incorporating the effect of earthquake forces as:

- a) Quasi-Static Approach
- b) Dynamic Approach

The Quasi-Static approach is most common, in which the earthquake forces are treated as static horizontal forces and the resulting stress are calculated and checked against specified safe values. In this parametric study the Quasi-Static approach is adopted for the calculation of horizontal forces, which are generated from earthquake effects.

The loadings, which are applied on the coupled shear wall, are calculated from the building plan as shown in FIGURE 1.8 (SHEET No. 1). The horizontal forces are calculated as per the IS: 1893-2002 specifications for equivalent static approach. Then these horizontal forces are applied on the CSW model.

Step-5: Analysis of coupled shear wall

Step-6: Study of Results

The results are to be noted:

- a) Axial forces, bending moments and shear force in walls
- b) Shear force and bending moment in coupling beams
- c) Degree of coupling

The checking of the results:

Equilibrium conditions to be checked for the assurance of correct results, which are obtained from the analysis. The general equilibrium conditions are,

$$\Sigma F_x = 0, \ \Sigma F_y = 0, \ \text{and} \ \Sigma M_z = 0$$
 (4.1)

For the justification of the $F_x = 0$, storey shear and for the $F_y = 0$, the axial forces in the wall at any section should be checked. As per as the moment is concern, at any section the following equilibrium statement should be satisfied:

$$M_0 = M_1 + M_2 + P \times L$$
 (Ref. 1.1)

Here the total external moments (M_0) are resisted by two walls ($M_1 \& M_2$) and due to the coupling action of the beams (P × L). The above steps are repeated for the all the problems, which are formulated in TABLE 1.1-1.3 (SHEET No. 1).

4.3 EXAMPLE OF TEN STORIED COUPLED SHEAR WALL

A typical example of ten storied coupled shear wall building is explained in this section. For the plan of the building refer FIGURE 1.8 (SHEET No. 1).

Data:

Number of storey:	10
Storey Height:	3 m
Building Height:	30 m
Building Length:	36 m
Building Width:	20 m
Length of wall:	3 m
Thickness of wall:	0.25 m
Span of coupling beam:	1.0 m
Depth of coupling beam:	0.75 m
Width of coupling beam:	0.25 m
Slab thickness:	0.125 m
Floor beam depth along shorter direction:	0.5 m
Floor beam depth along longer direction:	0.75 m
Column size:	0.45 x 0.45 m
Zone Factor (Zone III):	0.16
Soil condition:	Medium
Importance Factor:	1.0
Response reduction factor:	4.0
Live load:	3.0 kN/m ²

Considering above data, the modeling, meshing, material and geometrical properties and boundary conditions are applied in ETABS as per the step-wise discussions in section 4.2. The next step is the lateral load calculation.

Lateral load calculation:

Time period as per IS: 1893-2002 (= $0.09 \times h/\sqrt{d}$): 0.6037 secondSpectral accélération coefficient (Sa/g):2.253Horizontal seismic coefficient (Ah):0.045

For the calculation of lateral load generated from earthquake is shown in TABLE 4.1.

	Seismic weight, W _i	H _i			Lateral load on CSW, Q _{ic}	Moment at base
Storey	(kN)	(m)	$W_i \times H_i^2$	$Q_{i}(kN)$	(kN)	(kNm)
10	3813	30	3.43E+06	457	91	2740
9	4429	27	3.23E+06	430	86	2320
8	4429	24	2.55E+06	339	68	1629
7	4429	21	1.95E+06	260	52	1091
6	4429	18	1.44E+06	191	38	687
5	4429	15	9.97E+05	133	26	398
4	4429	12	6.38E+05	85	17	204
3	4429	9	3.59E+05	48	10	86
2	4429	6	1.59E+05	21	4	25
1	4429	3	3.99E+04	5	1	3
Total	43678		1.48E+07			9183

 TABLE 4.1 LATERAL LOAD CALCULATION FOR CSW

N.B. :- $Q_{ic} = Q_i / 5$

Base shear ($V_b = A_h \times W$) = 1968 kN

External moment at base $(M_0) = 9183$ kNm

After application of calculated lateral loads on CSW model, the analysis is performed in ETABS.

Study of Results:

The results obtained from the analysis are shown in TABLE 4.2. The degree of coupling can be calculated as,

Degree of coupling = $(P \times L) / M_0$

= 1704 × 4 / 9183 = 0.742

Checking of the Results:

Equilibrium Condition - 1 $\Sigma F_x = 0$

From the TABLE 4.3, it is found that external shear (column 5) and total shear carried by the CSW (column 6) are same at any floor level. So it is satisfying $\sum F_x = 0$ condition.

	Axial	force	Bending	Moment	Sh	ear	Shear Force	Bending Moment
	(k	N)	(kN	Im)	(k	N)	(kN)	(kNm)
Storey	Wall 1	Wall 2	Wall 1	Wall 2	Wall 1	Wall 2	Beam	Beam
10	49	-49	41	37	39	52	65	25
9	153	-153	95	99	82	95	114	36
8	291	-291	185	191	118	127	151	47
7	462	-462	290	294	146	151	186	58
6	659	-659	399	401	167	170	215	66
5	876	-876	509	510	180	182	235	72
4	1104	-1104	622	622	189	190	247	76
3	1333	-1333	745	745	194	195	248	77
2	1547	-1547	907	906	196	196	229	70
1	1704	-1704	1184	1184	197	197	164	46

TABLE 4.2 LINEAR STATIC ANALYSIS RESULTS OF CSW

N.B.:- -ve sign for compression and +ve sign for tension

TABLE 4.3 CHECKING FOR HORIZONTAL FORCE EQUILIBRIUM, $\sum F_x = 0$

	From	From TA	BLE 4.2,	External	
	TABLE	Sh	ear	storey	Total shear
	4.1, Qic	(k	N)	shear	in wall
Storey	(kN)	Wall 1	Wall 2	(kN)	(kN)
(1)	(2)	(3)	(4)	(5)	(6)=(3)+(4)
10	91	39	52	91	91
9	86	82	95	177	177
8	68	118	127	245	245
7	52	146	151	297	297
6	38	167	170	335	335
5	26	180	182	361	361
4	17	189	190	379	379
3	10	194	195	388	388
2	4	196	196	393	393
1	1	197	197	394	394

Equilibrium Condition – 2 $\Sigma F_y = 0$

From the TABLE 4.2, it is observed that the axial force in wall-1 is tension and wall-2 compression having same value in each level. So it is satisfying the equilibrium condition $\Sigma F_v = 0$.

Equilibrium Condition – 3 $\Sigma M_z = 0$

Let's consider the moments at base level of the CSW, from TABLE 4.1

External	moment $(M_0) =$	9183 kNm
External	moment $(M_0) =$	9183 kNm

Moment of	carried l	by	wall-1	(M_1)	=	1184	kNm

Moment carried by wall-2 (M_2) = 1184 kNm

Axial forces in wall (P) = 1704 kN

Lever arm (L) =

Applying the equilibrium equation,

4.0 m

$$M_0 = M_1 + M_2 + P \times L$$

$$M_1 + M_2 + P \times L = 1184 + 1184 + 1704 \times 4.0$$

$$= 9183 \text{ kNm} = M_0 \text{ (Proved)}$$

From the above calculations it is justified the moment equilibrium condition. In the same way it can be proved at each level, which is shown in TABLE 4.4.

	Lateral load on CSW On	External Moment at storey base Mo	Moment (kNm)		Axial force in wall	$M_1 + M_2 + P \\ \times L =$
Storey	(kN)	(kNm)	Wall 1	Wall 2	(kN)	(kN)
10	91	274	41	37	49	274
9	86	806	95	99	153	806
8	68	1541	185	191	291	1541
7	52	2432	290	294	462	2432
6	38	3438	399	401	659	3438
5	26	4524	509	510	876	4524
4	17	5660	622	622	1104	5660
3	10	6825	745	745	1333	6825
2	4	8003	907	906	1547	8002
1	1	9183	1184	1184	1704	9183

TABLE 4.4 CHECKING FOR MOMENT EQUILIBRIUM

4.4 STRESSES IN COUPLED SHEAR WALL

The stress in coupled shear wall is studied as horizontal direct stress (S11), axial stress or vertical direct stress (S22) and shear stress (S12). For the study of stresses 10-storied CSW is considered and the wall length, depth and span of beam are 3.0, 0.75 and 1.0m respectively (FIGURE 4.1).



FIGURE 4.1 STRESSES OF 10-STORIED CSW

The size of shell element is 0.25×0.25 m. Stresses are studied considering the second layer from top of 0.75m and 1.0m depth of coupling beam at storey no. 3 (FIGURE 4.2(a)). The variation of stresses in the horizontal direction of CSW at X-X level is shown in FIGURE 4.2.



The diagonal tensile and compressive stresses are observed in contour of S11 shown in FIGURE 4.1 (a) and similar study can be made from stress diagram as shown in FIGURE 4.2 (b). The axial stress in CSW is observed that one wall is subjected to tension and other one is in compression (FIGURE 4.1 (b) & 4.2 (c)). The shear stress in CSW is critical in coupling beam (FIGURE 4.1 (c) & 4.2 (d)).

4.5 STUDY OF CSW - BEAM DEPTH AS PARAMETER

The degree of coupling depends on the depth of beam. So it carries more significant to study the behavior of coupled shear wall, considering depth of coupling beam and number of storey as parameter. The maximum responses for beam depth as parameters are given in tabulated form at the end of this section.

4.5.1 Coupled Shear Wall (5 – Storey) – Beam depth as parameter

The behavior of 5-Storied CSW is studied by varying the depth of coupling beams. The responses are plotted with respect to the height (H) of CSW.



FIGURE 4.3 FORCES AND DC IN 5-STORIED CSW (BEAM DEPTH AS PARAMETER)

The axial forces in wall increases and bending moment in the wall decreases with increase in depth of coupling beam respectively (FIGURE4.3 (a) & (b)). The shear force and bending moment in coupling beam decreases and DC increases as the depth of coupling beam increases (FIGURE4.3 (c), (d) & (e)).

4.5.2 Coupled Shear Wall (10 – Storey) – Beam depth as parameter

The responses in 10-storied CSW are studied with beam depth parameter.



FIGURE 4.4 FORCES AND DC IN 10-STORIED CSW (BEAM DEPTH AS PARAMETER)

The behavior of 10-storied CSW is observed similar to the 5-storied (FIGURE 4.4). Only the difference is the 0.5 and 0.75m depth of coupling beam at the 10^{th} storey having more DC, where as 1.5 and 1.25m depth of coupling at base is more DC (FIGURE 4.4 (f)).

4.5.3 Coupled Shear Wall (15 – Storey) – Beam depth as parameter

The forces in wall and coupling beam are studied with varying beam depth.



FIGURE 4.5 FORCES AND DC IN 15-STORIED CSW (BEAM DEPTH AS PARAMETER)

The axial force in wall increases and bending moment in wall reduces with increase in depth of coupling beam for 15-storied CSW (FIGURE 4.5 (a) & (b)). Shear force and bending moment in coupling beam decrease with increase in depth (FIGURE 4.5 (d) & (e)). The DC at base increases for 1.5m depth of coupling beam compare to others (FIGURE 4.5 (f)).

4.5.4 Coupled Shear Wall (20 – Storey) – Beam depth as parameter

The forces in wall and coupling beam are studied with varying beam depth for 20- storied CSW.



FIGURE 4.6 FORCES AND DC IN 20-STORIED CSW (BEAM DEPTH AS PARAMETER)

The responses observed in 20-storied CSW are shown in FIGURE 4.6. The axial force and bending moment in wall increases and decreases with increase in depth of coupling beam respectively. The shear force is critical in compare to bending moment in coupling beam. The shear force and B.M. reduces as the depth of coupling increases. The DC is varying similar to the 15-storied CSW.

4.5.5 Coupled Shear Wall (25 – Storey) – Beam depth as parameter

The axial force, shear force and B.M. in wall and coupling beam are studied with varying beam depth for 25- storied CSW and DC also studied for the same.



FIGURE 4.7 FORCES AND DC IN 25-STORIED CSW (BEAM DEPTH AS PARAMETER)

The axial forces, shear and bending moment in wall of 25-storied CSW is shown in FIGURE 4.7 (a), (b) & (c). The shear force and bending moment in coupling beam are shown in FIGURE 4.7 (d) & (e). The axial force increases and B.M. decreases in wall, when the depth of coupling beam increases. And the shear force as well as B.M. in coupling beam decreases with increase in beam depth.

4.5.6 Coupled Shear Wall (30 – Storey) – Beam depth as parameter

In 30-storied CSW, the axial force, B.M. and shear in wall, shear force and B.M. in coupling beam and DC are studied with beam depth as parameter.



FIGURE 4.8 FORCES AND DC IN 30-STORIED CSW (BEAM DEPTH AS PARAMETER)

The axial force increases and bending moment decreases in wall of 30-storey CSW and shear force and bending moment in coupling beam decreases with increases in depth of coupling beam (FIGURE 4.8).

The maximum forces in wall and coupling beams are summarized in TABLE 4.5 and 4.6.

					Max.	Max.	Max.		Max. Moment
					Axial	Moment in	Moment in	Max. Shear in	in coupling
Prob.	Dw	Hb	Lb		forces in	Wall 1	Wall 2	coupling Beam	Beam (kNm),
No	(m)	(m)	(m)	D.C.	Wall (kN)	(kNm)	(kNm)	(kN), (Location)	(Location)
5	-Store	y CSV	V						
								152	35
1	3.0	0.5	1.0	0.59	349	492	488	(STOREY-3)	(STOREY-3)
								107	31
2	3.0	0.75	1.0	0.65	388	414	411	(STOREY-2)	(STOREY-3)
								80	27
3	3.0	1.0	1.0	0.69	409	371	369	(STOREY-2)	(STOREY-2)
								64	24
4	3.0	1.25	1.0	0.71	424	342	339	(STOREY-2)	(STOREY-2)
								53	21
5	3.0	1.5	1.0	0.73	436	319	317	(STOREY-2)	(STOREY-2)
1	0-Stor	rey CS	W						
								382	88
6	3.0	0.5	1.0	0.7	1617	1356	1355	(STOREY-4)	(STOREY-4)
								249	77
7	3.0	0.75	1.0	0.74	1704	1185	1184	(STOREY-3)	(STOREY-3)
								180	65
8	3.0	1.0	1.0	0.76	1751	1092	1092	(STOREY-3)	(STOREY-3)
								140	56
9	3.0	1.25	1.0	0.78	1782	1031	1030	(STOREY-3)	(STOREY-3)
								114	49
10	3.0	1.50	1.0	0.79	1806	984	984	(STOREY-3)	(STOREY-3)
15	5-Store	ey CSV	N						
								426	101
11	3.0	0.50	1.0	0.75	2647	1776	1776	(STOREY-5)	(STOREY-5)
								270	88
12	3.0	0.75	1.0	0.77	2740	1593	1593	(STOREY-4)	(STOREY-4)
								193	75
13	3.0	1.00	1.0	0.79	2790	1496	1496	(STOREY-4)	(STOREY-3)
								150	65
14	3.0	1.25	1.0	0.8	2823	1431	1431	(STOREY-3)	(STOREY-3)
								123	57
15	3.0	1.50	1.0	0.8	2836	1377	1376	(STOREY-3)	(STOREY-3)

TABLE 4.5 SUMMARY OF MAXIMUM FORCES IN 5, 10, & 15-STOREY CSW(BEAM DEPTH AS PARAMETER)

TABLE 4.6 SUMMARY OF MAXIMUM FORCES IN 20, 25, & 30-STOREY CSW

(BEAM DEPTH AS PARAMETER)

						Max.		Max. Shear in	Max. Moment
					Max. Axial	Moment in	Max.	coupling Beam	in coupling
Prob.	$\mathbf{D}\mathbf{w}$	Hb	Lb		forces in	Wall 1	Moment in	(kN),	Beam (kNm),
No	(m)	(m)	(m)	D.C.	Wall (kN)	(kNm)	Wall 2 (kNm)	(Location)	(Location)
20-	Store	y CSV	W						
								449	110
16	3.0	0.50	1.0	0.77	3715	2196	2196	(STOREY-6)	(STOREY-5)
								282	96
17	3.0	0.75	1.0	0.79	3811	2007	2007	(STOREY-5)	(STOREY-4)
								201	83
18	3.0	1.00	1.0	0.8	3862	1907	1907	(STOREY-4)	(STOREY-3)
								156	73
19	3.0	1.25	1.0	0.81	3898	1840	1840	(STOREY-4)	(STOREY-3)
								129	65
20	3.0	1.50	1.0	0.81	3925	1790	1789	(STOREY-3)	(STOREY-3)
25-	Store	y CSV	N						
								467	119
21	3.0	0.50	1.0	0.79	4826	2631	2631	(STOREY-6)	(STOREY-5)
								291	105
22	3.0	0.75	1.0	0.8	4925	2436	2436	(STOREY-5)	(STOREY-4)
								208	91
23	3.0	1.00	1.0	0.81	4979	2333	2333	(STOREY-4)	(STOREY-3)
								162	81
24	3.0	1.25	1.0	0.82	5015	2264	2264	(STOREY-4)	(STOREY-3)
								134	73
25	3.0	1.50	1.0	0.82	5043	2212	2212	(STOREY-3)	(STOREY-3)
30-	Store	y CSV	N						
								483	127
26	3.0	0.50	1.0	0.8	5993	3088	3088	(STOREY-7)	(STOREY-5)
								300	113
27	3.0	0.75	1.0	0.81	6095	2887	2887	(STOREY-5)	(STOREY-4)
								215	100
28	3.0	1.00	1.0	0.82	6151	2781	2781	(STOREY-5)	(STOREY-3)
								168	89
29	3.0	1.25	1.0	0.82	6189	2709	2709	(STOREY-4)	(STOREY-3)
				_				140	82
30	3.0	1.50	1.0	0.82	6218	2656	2656	(STOREY-3)	(STOREY-3)

4.6 STUDY OF CSW - BEAM SPAN AS PARAMETER

The behavior of CSW also depends on the span of coupling beam. So it is important to study the static behavior of coupled shear wall with considering beam span of coupling beam and number of storey as parameter. The maximum responses for beam span as parameter are summarized at the end of the this section.

4.6.1 Coupled Shear Wall (5 – Storey) – Beam span as parameter

The behavior of 5-Storied Coupled Shear Wall is studied by varying the span of coupling beams.



FIGURE 4.9 FORCES AND DC IN 5-STORIED CSW (BEAM SPAN AS PARAMETER)

The axial forces in the wall decreases and bending moment in the wall increases with increase in span of coupling beam respectively (FIGURE4.9 (a) & (b)). The shear force and bending moment in coupling beam increases and DC decreases as the span of coupling beam increases (FIGURE4.9 (c), (d) & (e)).

4.6.2 Coupled Shear Wall (10 – Storey) – Beam span as parameter

The responses and degree of coupling in 10-storied CSW are studied with beam span as parameter.



FIGURE 4.10 FORCES AND DC IN 10-STORIED CSW (BEAM SPAN AS PARAMETER)

The behavior of 10-storied CSW is observed similar to the 5-storied (FIGURE 4.10). Only the difference is the 3.0 and 2.5m span of coupling beam at the 10^{th} storey having more DC, where as 1.0 and 1.5m span of coupling at base is more DC (FIGURE 4.10 (f)).

4.6.3 Coupled Shear Wall (15 – Storey) – Beam span as parameter

The forces in wall and coupling beam of 15-storied CSW are studied with varying beam span from 1.0 to 3.0 m.



FIGURE 4.11 FORCES AND DC IN 15-STORIED CSW (BEAM SPAN AS PARAMETER)

The axial force in wall decreases and bending moment in wall at base increases with increase in span of coupling beam for 15-storied CSW (FIGURE 4.11 (a) & (b)). Shear force and bending moment in coupling beam increase with increase

in span (FIGURE 4.11 (d) & (e)). The DC at base decreases for higher span of coupling beam like 3.0 m at base compare to others (FIGURE 4.11 (f)).

4.6.4 Coupled Shear Wall (20 – Storey) – Beam span as parameter

The forces in wall and coupling beam are studied with varying beam span for 20storied CSW.



FIGURE 4.12 FORCES AND DC IN 20-STORIED CSW (BEAM SPAN AS PARAMETER)

The responses observed in 20-storied CSW are shown in FIGURE 4.12. The axial force and bending moment in wall decreases and increases with increase in span of coupling beam respectively. The shear force is critical in compare to bending moment in coupling beam. The shear force and bending moment increase as the span of coupling increases. The DC is varying similar to the 15-storied CSW.

4.6.5 Coupled Shear Wall (25 – Storey) – Beam span as parameter

The axial force, shear and B.M. in wall and coupling beam are studied with varying beam span for 25- storied CSW and DC also studied for the same.



FIGURE 4.13 FORCES AND DC IN 25-STORIED CSW (BEAM SPAN AS PARAMETER)

The axial forces, shear and bending moment in wall of 25-storied CSW is shown in FIGURE 4.13 (a), (b) & (c). The shear force and bending moment in coupling beam are shown in FIGURE 4.13 (d) & (e). The axial force decreases and B.M. increases in wall, when the span of coupling beam increases. And the shear force as well as B.M. in coupling beam increases with increase in span of beam.

4.6.6 Coupled Shear Wall (30 - Storey) - Beam span as parameter

In 30-storied CSW, the axial force, B.M. and shear in wall, shear and B.M. in coupling beam and DC are studied with beam span as parameter.



FIGURE 4.14 FORCES AND DC IN 30-STORIED CSW (BEAM SPAN AS PARAMETER)

The responses and DC of 30-storied CSW observed from is shown in FIGURE 4.14. The DC is increasing at 30-storey level with increase in span of beam.

The maximum forces in wall and coupling beams are summarized in TABLE 4.7 and 4.8.

					Max.	Max.	Max.	Max. Shear in	Max. Moment in
					Axial	Moment in	Moment in	coupling	coupling Beam
Prob.	Dw	Hb	Lb		forces in	Wall 1	Wall 2	Beam (kN),	(kNm),
No	(m)	(m)	(m)	D.C.	Wall (kN)	(kNm)	(kNm)	(Location)	(Location)
5-S	torey	y CS	W						
31	3.0	0.5	1.0	0.59	349	492	488	152 (STOREY-3)	34 (STOREY-3)
32	3.0	0.5	1.5	0.53	277	566	562	181 (STOREY-3)	51 (STOREY-5)
33	3.0	0.5	2.0	0.46	219	642	637	189 (STOREY-3)	73 (STOREY-5)
34	3.0	0.5	2.5	0.4	174	712	705	190 (STOREY-4)	91 (STOREY-5)
35	3.0	0.5	3.0	0.35	140	772	765	184 (STOREY-4)	105 (STOREY-5)
10-5	Store	y CS	W						
36	3.0	0.5	1.0	0.7	1617	1356	1355	382 (STOREY-4)	88 (STOREY-4)
37	3.0	0.5	1.5	0.68	1384	1478	1476	480 (STOREY-4)	128 (STOREY-5)
38	3.0	0.5	2.0	0.65	1185	1629	1628	534 (STOREY-5)	163 (STOREY-5)
39	3.0	0.5	2.5	0.61	1018	1791	1789	564 (STOREY-5)	203 (STOREY-10)
40	3.0	0.5	3.0	0.58	880	1952	1949	575 (STOREY-5)	268 (STOREY-10)
15-	Store	ey CS	SW						
41	3.0	0.5	1.0	0.75	2647	1776	1776	426 (STOREY-5)	101 (STOREY-5)
42	3.0	0.5	1.5	0.74	2322	1847	1846	558 (STOREY-5)	153 (STOREY-6)
43	3.0	0.5	2.0	0.72	2039	1974	1973	646 (STOREY-6)	202 (STOREY-6)
44	3.0	0.5	2.5	0.7	1797	2128	2128	697 (STOREY-6)	246 (STOREY-7)
45	3.0	0.5	3.0	0.68	1592	2293	2293	730 (STOBEY-7)	287 (STOBEY-8)

TABLE 4.7 SUMMARY OF MAXIMUM FORCES IN 5, 10, & 15-STOREY CSW(BEAM SPAN AS PARAMETER)

TABLE 4.8 SUMMARY OF MAXIMUM FORCES IN 20, 25, & 30-STOREY CSW

r			1						
					Max.	Max.	Max.	Max. Shear	Max. Moment in
					Axial	Moment in	Moment in	in coupling	coupling Beam
Proh	Dw	Hb	Lb		forces in	Wall 1	Wall 2	Beam (kN),	(kNm),
No	(m)	(m)	(m)	D.C.	Wall (kN)	(kNm)	(kNm)	(Location)	(Location)
20-Storey CSW								· · · · ·	
								110	110
46	3.0	0.5	1.0	0.77	3715	2196	2196	(STOREY-6)	(STOREY-5)
								603	172
47	3.0	0.5	1.5	0.77	3299	2204	2204	(STOREY-6)	(STOREY-6)
								713	230
48	3.0	0.5	2.0	0.76	2934	2290	2290	(STOREY-7)	(STOREY-7)
								787	284
49	3.0	0.5	2.5	0.75	2620	2421	2421	(STOREY-8)	(STOREY-8)
								838	335
50	3.0	0.5	3.0	0.73	2350	2575	2575	(STOREY-8)	(STOREY-9)
25-Storey CSW									
								467	119
51	3.0	0.5	1.0	0.79	4826	2631	2631	(STOREY-6)	(STOREY-5)
								634	187
52	3.0	0.5	1.5	0.79	4317	2570	2570	(STOREY-7)	(STOREY-7)
								759	254
53	3.0	0.5	2.0	0.79	3870	2609	2609	(STOREY-8)	(STOREY-8)
								851	317
54	3.0	0.5	2.5	0.78	3482	2707	2707	(STOREY-9)	(STOREY-9)
								916	376
55	3.0	0.5	3.0	0.77	3147	2841	2841	(STOREY-9)	(STOREY-10)
30-Storey CSW									
								483	127
56	3.0	0.5	1.0	0.8	5993	3088	3088	(STOREY-7)	(STOREY-5)
								660	203
57	3.0	0.5	1.5	0.8	5387	2954	2954	(STOREY-8)	(STOREY-7)
								797	277
58	3.0	0.5	2.0	0.8	4853	2941	2941	(STOREY-9)	(STOREY-8)
								901	348
59	3.0	0.5	2.5	0.8	4390	3003	3003	(STOREY-10)	(STOREY-9)
								979	415
60	3.0	0.5	3.0	0.79	3988	3112	3112	(STOREY-10)	(STOREY-10)

(BEAM SPAN AS PARAMETER)

4.7 STUDY OF CSW - WALL LENGTH AS PARAMETER

The walls are the vertical elements of CSW. With change in geometry of wall the behavior of CSW changes. To study those effects, here the wall length considered as parameter with number of storey. The maximum responses are given in TABLE 4.7 for wall length as parameter.

4.7.1 Coupled Shear Wall (5 – Storey) – Wall Length as parameter

The behavior of 5-Storied Coupled Shear Wall is studied by varying the wall length.



FIGURE 4.15 FORCES AND DC IN 5-STORIED CSW (WALL LENGTH AS PARAMETER)

The axial force decreases and bending moment in the wall increases with increase in wall length respectively (FIGURE4.15 (a) & (b)). The shear force and bending moment in coupling beam decreases and DC decreases as the wall length of CSW increases (FIGURE4.15 (c), (d) & (e)).

4.7.2 Coupled Shear Wall (10 – Storey) – Wall Length as parameter

The responses and degree of coupling in 10-storied CSW are studied with wall length as parameter and the wall length is varying from 2.0 to 6.0 m. The axial force in wall reduces and B.M. increases as the length of wall increases.



FIGURE 4.16 FORCES AND DC IN 10-STORIED CSW (WALL LENGTH AS PARAMETER)

The behavior of 10-storied CSW is observed similar to the 5-storied (FIGURE 4.16). Only the difference is the 6.0 and 5.0 m length of wall at the 10^{th} storey having more DC, where as 2.0 and 3.5m span of coupling at base is more DC (FIGURE 4.16 (f)).

4.7.3 Coupled Shear Wall (15 – Storey) – Wall Length as parameter

The forces in wall and coupling beam are studied with varying wall length from 2.0 to 6.0 m. Here the 15-storey CSW is considered for the study. The thickness of wall is 0.3 m.



FIGURE 4.17 FORCES AND DC IN 15-STORIED CSW (WALL LENGTH AS PARAMETER)

The axial force in wall decreases and bending moment in wall increases with increase in length of wall for 15-storied CSW (FIGURE 4.17 (a) & (b)). Shear force and bending moment in coupling beam decrease with increase in length of
wall (FIGURE 4.17 (d) & (e)). The DC at base increases for lower length of wall in compare to others (FIGURE 4.17 (f)).

4.7.4 Coupled Shear Wall (20 – Storey) – Wall Length as parameter

The forces in wall and coupling beam are studied with varying length of wall for 20- storied CSW.



FIGURE 4.18 FORCES AND DC IN 20-STORIED CSW (WALL LENGTH AS PARAMETER)

The responses observed in 20-storied CSW are shown in FIGURE 4.18. The axial force and bending moment in wall decreases and increases with increase in

length of wall respectively. The shear force is critical in compare to bending moment in coupling beam. The shear force and bending moment reduces as the length of wall increases. The DC is varying similar to the 15-storied CSW.

4.7.5 Coupled Shear Wall (25 – Storey) – Wall Length as parameter

The axial force, shear and B.M. in wall and coupling beam are studied with varying wall length for 25- storied CSW and DC also studied for the same.



FIGURE 4.19 FORCES AND DC IN 25-STORIED CSW (WALL LENGTH AS PARAMETER)

The axial forces, shear and bending moment in wall of 25-storied CSW is shown in FIGURE 4.19 (a), (b) & (c). The shear force and bending moment in coupling beam are shown in FIGURE 4.19 (d) & (e). The axial force decreases and B.M. increases in wall, when the length of wall increases. And the shear force as well as bending moment in coupling beam decreases with increase in length of wall.

4.7.6 Coupled Shear Wall (30 – Storey) – Wall Length as parameter

In 30-storied CSW, the axial force, B.M. and shear in wall, shear force and B.M. in coupling beam and DC are studied with wall length as parameter.



FIGURE 4.20 FORCES AND DC IN 30-STORIED CSW (WALL LENGTH AS PARAMETER)

The responses and DC of 30-storied CSW observed from is shown in FIGURE 4.20. With the increases in wall length, the axial force decreases and shear and bending moment in wall increases. Whereas in the coupling beam shear force and bending moment decreases with increases in length of wall. The DC at the top storey increases as wall length increases.

The maximum forces in wall and coupling beams are summarized in TABLE 4.9 and 4.10.

					Max.	Max.	Max.		Max. Moment in
					Axial	Moment in	Moment in	Max. Shear in	coupling Beam
Prob.	Dw	Hb	Lb		forces in	Wall 1	Wall 2	coupling Beam	(kNm),
No	(m)	(m)	(m)	D.C.	Wall (kN)	(kNm)	(kNm)	(kN), (Location)	(Location)
5-S	Storey	CS	N						
61	2.0	0.5	1.0	0.67	511	386	385	228 (STOREY-2)	52 (STOREY-3)
62	3.0	0.5	1.0	0.59	349	492	488	152 (STOREY-3)	34 (STOREY-3)
63	4.0	0.5	1.0	0.53	260	579	570	114 (STOREY-3)	25 (STOREY-5)
64	5.0	0.5	1.0	0.49	205	658	636	90 (STOREY-3)	20 (STOREY-4)
65	6.0	0.5	1.0	0.45	167	735	690	74 (STOREY-3)	16 (STOREY-4)
10-9	Store	y CS	W						
66	2.0	0.5	1.0	0.76	2252	1054	1054	538 (STOREY-3)	131 (STOREY-3)
67	3.0	0.5	1.0	0.7	1617	1356	1355	382 (STOREY-4)	88 (STOREY-4)
68	4.0	0.5	1.0	0.66	1260	1600	1597	293 (STOREY-4)	66 (STOREY-4)
69	5.0	0.5	1.0	0.63	1032	1812	1805	237 (STOREY-4)	53 (STOREY-5)
70	6.0	0.5	1.0	0.61	876	2004	1992	198 (STOREY-5)	44 (STOREY-5)
15-9	Store	y CS	W						
71	2.0	0.5	1.0	0.8	3610	1373	1373	582 (STOREY-4)	151 (STOREY-4)
72	3.0	0.5	1.0	0.75	2647	1776	1776	426 (STOREY-5)	101 (STOREY-5)
73	4.0	0.5	1.0	0.71	2101	2101	2100	337 (STOREY-5)	77 (STOREY-5)
74	5.0	0.5	1.0	0.69	1751	2385	2383	278 (STOREY-5)	62 (STOREY-6)
75	6.0	0.5	1.0	0.67	1510	2646	2642	238 (STOREY-6)	53 (STOREY-6)

TABLE 4.9 SUMMARY OF MAXIMUM FORCES IN 5, 10, & 15-STOREY CSW(WALL LENGTH AS PARAMETER)

TABLE 4.10 SUMMARY OF MAXIMUM FORCES IN 20, 25, & 30-STOREY CSW

					Max.	Max.	Max.	Max. Shear in	Max. Moment in
					Axial	Moment in	Moment in	coupling	coupling Beam
Prob	Dw	Hb	Lb		forces in	Wall 1	Wall 2	Beam (kN),	(kNm),
No	(m)	(m)	(m)	D.C.	Wall (kN)	(kNm)	(kNm)	(Location)	(Location)
20	-Store	y CS	W						
								607	170
76	2.0	0.5	1.0	0.82	5030	1702	1702	(STOREY-5)	(STOREY-4)
								449	110
77	3.0	0.5	1.0	0.77	3715	2196	2196	(STOREY-6)	(STOREY-5)
								359	84
78	4.0	0.5	1.0	0.74	2966	2590	2590	(STOREY-6)	(STOREY-6)
70	5.0	0.5		0.70	0.405	0000		300	68
79	5.0	0.5	1.0	0.72	2485	2929	2929	(STOREY-6)	(STOREY-7)
		0.5		0.00	0405	0011		257	
80	6.0	0.5	1.0	0.69	2135	3211	3209	(STOREY-7)	(STOREY-7)
20		y Co	vv					000	100
81	20	05	10	0.83	6510	2043	2043	628 (STOREV-5)	188 (STOREV-4)
01	2.0	0.5	1.0	0.00	0010	2043	2040	(01011213)	110
82	3.0	0.5	10	0 79	4826	2631	2631	(STOBEY-6)	(STOREY-5)
02	0.0	0.0		0.70	1020	2001	2001	375	89
83	4.0	0.5	1.0	0.76	3866	3094	3093	(STOREY-7)	(STOREY-6)
								315	73
84	5.0	0.5	1.0	0.74	3248	3489	3488	(STOREY-7)	(STOREY-7)
								273	62
85	6.0	0.5	1.0	0.72	2818	3844	3843	(STOREY-8)	(STOREY-8)
30	-Store	y CS	W						
								648	206
86	2.0	0.5	1.0	0.83	8068	2403	2403	(STOREY-6)	(STOREY-4)
								483	127
87	3.0	0.5	1.0	0.8	5993	3088	3088	(STOREY-7)	(STOREY-5)
								389	95
88	4.0	0.5	1.0	0.77	4809	3622	3621	(STOREY-7)	(STOREY-7)
		o -		o ==	40.17	4070	4070	328	77
89	5.0	0.5	1.0	0.75	4047	4073	4073	(STOREY-8)	(STOREY-8)
		0.5	10	0.70	0540	4 4 7 7	4477	285	
90	6.0	0.5	1.0	0.73	3516	44//	44//	(STOREY-8)	(STOREY-8)

(WALL LENGTH AS PARAMETER)

4.8 COMPARATIVE STUDY ON DEGREE OF COUPLING

The degree of coupling is the performance index of coupled shear wall. As the number of storey increases from 5 to 30 storey, the DC is studied at the base and top storey with different parameters. When beam depth as parameter, the DC at base increases with increase in depth of coupling beam (FIGURE 4.21(a)). For 5-storied CSW, the change in depth of coupling beam makes considerable change in DC, whereas in 30-storied CSW the variations are very close. At the

top storey the lower depth of coupling beam is having more DC as shown in FIGURE 4.21(b).



FIGURE 4.21 DEGREE OF COUPLING (COUPLING BEAM DEPTH AS PARAMETER)





FIGURE 4.23 DEGREE OF COUPLING (WALL LENGTH AS PARAMETER)

The DC decreases with increases in span of coupling beam at the base of CSW (FIGURE 4.22(a)). At the top storey DC increases with increases in span of coupling beam (FIGURE 4.23(b)). For lowrise CSW (5-storey) at base and top

storey the DC decreases with increases in wall length. But for highrise CSW structure, the DC decreases at base and increases at top storey with increases in wall length. The difference in DC for lowrise CSW in compared to highrise CSW structure is more with change in parameters.

4.9 CLOSING REMARKS

The parameters considered for the linear static behavior of CSW are depth and span of coupling beam, wall length and number of storey. When the depth of coupling beam as parameter, in wall the axial force increases and bending moment decrease with increase in depth of coupling beam. And the shear force and bending moment in coupling beam decreases. For higher depth of coupling beam at the base of wall gives more DC and lower depth of coupling beam at top storey gives more DC.

When span of coupling beam as parameter, in the wall axial force decreases and bending moment increases with increases in span of coupling beam. And the shear force and bending moment in coupling beam increases. For lower span of coupling beam at base of wall gives more DC and greater span of coupling beam at top storey gives more DC. For wall length as parameter, the axial force and bending moment in wall decreases and increases respectively with increase in wall length. And shear force and bending moment decreases in coupling beam. The lesser wall length gives more DC at base and greater wall length at top storey gives more DC.

5.1 INTRODUCTION

The study of responses due to the seismic excitation in CSW is important as far as earthquake is concern. The seismic behavior of structure also depends on natural time period of structure. The accuracy in the estimation of natural time period reflect in the estimation of forces with in structure. When the estimated natural time period is shorter than the actual, then the structure will be over designed. It is shown that the codal formulae provide periods that are generally shorter than measured periods and the current code formulae for estimating the natural time period of concrete shear wall buildings are grossly inadequate [38]. Therefore a parametric study has been carried out to investigate the fundamental time period of CSW and compare with the formula given by O. Chaallal [9]. And also the fundamental period of CSW building is estimated as per IS1893-2002 and compared with the Wallace and Moehle's formula (1992) [9, 25, 39].

5.2 TIME PERIOD OF COUPLED SHEAR WALL

For the estimation of time period of coupled shear wall building and coupled shear wall was given by different codes and the researchers, which is discussed below.

The formula given by NBCC (1990, 1995) for estimating the fundamental period of multistory wall systems, including coupled shear wall are,

$$T_{\text{code}} = 0.09 h_n / \sqrt{2D_w + L_b} \qquad \text{for stiff coupling beam}$$
(5.1)

$$T_{\text{code}} = 0.09 h_n / \sqrt{D_w}$$
 for flexible coupling beam (5.2)

Where h_n = building height; D_w = wall length; and L_b = span of coupling beam. In Eq. (5.1) & (5.2), the seismic weight effect isn't taken into account.

The formula proposed by Wallace and Moehle (1992) for the fundamental periods of structural wall buildings are,

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$$T_{wallace} = 6.2 \frac{h_w}{(2D_w + L_b)} n \sqrt{\frac{wh_s}{gE_c p}} \quad \text{for stiff coupling beam}$$
(5.3)

$$T_{wallace} = 6.2 \frac{h_w}{D_w} n_v \sqrt{\frac{wh_s}{gE_cp}}$$
 for flexible coupling beam (5.4)

Where n = number of stories; w = unit floor weight including tributary wall weight; h_s = mean storey height; p = ratio of wall area to floor plan area for the walls aligned in the direction the period is calculated; g = acceleration due to gravity; and E_c = concrete modulus of elasticity. This formula doesn't include the coupling parameters, which is important for CSW. O. Chaallal (1996) proposed formula for estimating the fundamental time period of CSW as follows:

$$T = 3n \left(\frac{L_b}{H_b}\right)^{0.19} \left(\frac{h_w}{D_w}\right)^{0.76} \sqrt{\frac{W}{gE_c t}}$$
(5.5)

Where $h_w = CSW$ height; $H_b =$ depth of coupling beam; t = wall thickness; and W = average seismic weight.

5.3 RESULTS AND DISCUSSION OF PARAMETRIC STUDY

The time period of CSW and CSW building are studied considering the parameters are depth and span of coupling beam, wall length and number of storey. The parameters are defined in TABLE 1.1-1.3 (SHEET No. 1).

5.3.1 Time period of Coupled Shear Wall

When coupling beam depth as parameter the results of the coupled shear wall from ETABS and O. Chaallal's formula as shown in FIGURE 5.1. When depth of coupling beam as parameter, the fundamental time period of coupled shear wall decrease with the increase in depth of coupling beam. As per the ETABS results (FIGURE 5.1 and TABLE 5.1), for the higher storey CSW the nature of curve tends to straight line, when the depth of coupling beam is increased. But it was observed from FIGURE 5.1, that the time period calculated from O. Chaallal's formula has a similar pattern of variation for all the stories. For 15 to 30-storied CSW, the time period obtained from O. Chaallal's formula and ETABS results is matching between 0.6m to 0.8m depth of coupling beam. The values written within bracket in TABLE 5.1 are calculated from O. Chaallal's formula.





Beam depth, m	0.5	0.75	1.0	1.25	1.5
	0.262	0.238	0.227	0.22	0.215
5 storey	(0.235)	(0.217)	(0.206)	(0.197)	(0.191)
	0.715	0.68	0.665	0.657	0.652
10 storey	(0.749)	(0.694)	(0.657)	(0.63)	(0.609)
	1.356	1.319	1.304	1.297	1.292
15 storey	(1.422)	(1.317)	(1.247)	(1.196)	(1.155)
	2.311	2.272	2.257	2.25	2.247
20 storey	(2.384)	(2.208)	(2.091)	(2.004)	(1.936)
	3.316	3.279	3.267	3.262	3.261
25 storey	(3.572)	(3.308)	(3.133)	(3.003)	(2.901)
	4.71	4.673	4.663	4.66	4.66
30 storey	(4.989)	(4.62)	(4.375)	(4.194)	(4.052)

TABLE 5.1 TIME PERIOD OF CSW (COUPLING BEAM DEPTH AS PARAMTER)

When coupling beam span as parameter the results of the coupled shear wall from ETABS and O. Chaallal's formula as shown in FIGURE 5.2. When beam span as parameter from the ETABS results it is observed that up to a particular span of coupling beam the fundamental time period decrease then it increase for the higher storey. For example in 15, 20, and 25 storey up to span 1.5, 2.0 and 2.5m the time period is decreasing respectively, and then the time period increasing (FIGURE 5.2 TABLE 5.2). The time period from O. Chaallal's formula is increasing with increase in span of CSW and the nature of curve is same for all the stories (FIGURE 5.2). The values written within bracket in TABLE 5.2 are calculated from O. Chaallal's formula.



FIGURE 5.2 TIME PERIOD OF CSW (COUPLING BEAM SPAN AS PARAMETER)

Beam span, m	1	1.5	2	2.5	3
	0.262	0.286	0.313	0.337	0.359
5 storey	(0.235)	(0.254)	(0.268)	(0.28)	(0.289)
	0.715	0.732	0.772	0.821	0.877
10 storey	(0.749)	(0.809)	(0.855)	(0.892)	(0.923)
	1.356	1.33	1.35	1.396	1.465
15 storey	(1.422)	(1.536)	(1.623)	(1.693)	(1.753)
	2.311	2.21	2.183	2.203	2.265
20 storey	(2.384)	(2.575)	(2.72)	(2.837)	(2.937)
	3.316	3.125	3.029	2.998	3.031
25 storey	(3.572)	(3.858)	(4.075)	(4.252)	(4.402)
	4.71	4.398	4.209	4.106	4.091
30 storey	(4.989)	(5.389)	(5.691)	(5.938)	(6.147)

 TABLE 5.2 TIME PERIOD OF CSW (COUPLING BEAM SPAN AS PARAMTER)

When Wall Length as parameter the results of the coupled shear wall from ETABS and O. Chaallal's formula as shown in FIGURE 5.3 & 5.4 (TABLE 5.3). When the length of wall as the parameter, the fundamental time period of coupled shear wall decrease with the increase in wall length. The time period of CSW from ETABS results for 10 to 30 storey (3.0m to 6.0m) gives shorter value than the O. Chaallal's formula. For 10 to 30 storey CSW, the time period obtained from O. Chaallal's formula for the length of wall 2.5 to 3.0m is matching with the ETABS results. The values written within bracket in TABLE 5.3 are calculated from O. Chaallal's formula.





FIGURE 5.4 TIME PERIOD OF CSW (WALL LENGTH AS PARAMETER) TABLE 5.3 TIME PERIOD OF CSW (WALL LENGTH AS PARAMETER)

Wall length, m	2	3	4	5	6
	0.386	0.262	0.2	0.163	0.139
5 storey	(0.314)	(0.235)	(0.192)	(0.165)	(0.146)
	1.085	0.715	0.533	0.426	0.357
10 storey	(1.0)	(0.749)	(0.613)	(0.527)	(0.467)
	2.082	1.356	1.001	0.792	0.656
15 storey	(1.891)	(1.422)	(1.169)	(1.008)	(0.896)
	3.573	2.311	1.693	1.331	1.095
20 storey	(3.17)	(2.384)	(1.959)	(1.688)	(1.5)
	5.13	3.316	2.426	1.904	1.563
25 storey	(4.753)	(3.572)	(2.933)	(2.528)	(2.245)
	7.302	4.71	3.437	2.691	2.204
30 storey	(6.642)	(4.989)	(4.095)	(3.526)	(3.13)

5.3.2 Time period of Coupled Shear Wall Building

The time period of Coupled Shear Wall building is calculated from Wallace and Moehle's formula, ETABS results and from IS1893-2002 (clause 7.6.2) as shown in FIGURE 5.5 to 5.6. The same parameters are considered for the study of time period of CSW building.

At first, the time period is studied for the coupling beam depth as parameter. The time period of CSW from ETABS results and time period of CSW building obtained from Wallace and Moehle's formula are closer to each others. There is a considerable difference between time obtained from IS1893-2002 (clause 7.6.2) given in equation (5.6) and the Wallace and Moehle's formula and ETABS results (FIGURE 5.5). For example, from FIGURE 5.5, the time period for 30-storey CSW building is calculated as per Wallace & Moehle's formula is 5.544 seconds. And ETABS result is 4.66 seconds but from IS1893-2002 the value obtained as 1.811 seconds (TABLE 5.4).

$$T_a = \frac{0.09h}{\sqrt{d}} \tag{5.6}$$



FIGURE 5.5 TIME PERIOD OF CSW BUILDING AND CSW (COUPLING BEAM DEPTH AS PARAMETER)

When span of coupling beam and beam depth as parameter, the time period obtained from IS 1893 : 2002 formula are closer up to 10-storey. But the

difference considerably increases for higher storey (FIGURE 5.6). The point to be noted that the time period obtained from ETABS results and Wallace & Moehle's formula are closer for 5 to 30 storey (TABLE 5.5).

Storey	Beam depth, m	0.5	0.75	1.0	1.25	1.5
	Wallace formula	0.17	0.17	0.17	0.17	0.17
	IS 1893-2002	0.302	0.302	0.302	0.302	0.302
5 storey	ETABS	0.262	0.238	0.227	0.22	0.215
	Wallace formula	0.64	0.64	0.64	0.64	0.64
	IS 1893-2002	0.604	0.604	0.604	0.604	0.604
10 storey	ETABS	0.715	0.68	0.665	0.657	0.652
	Wallace formula	1.388	1.388	1.388	1.388	1.388
	IS 1893-2002	0.906	0.906	0.906	0.906	0.906
15 storey	ETABS	1.356	1.319	1.304	1.297	1.292
	Wallace formula	2.403	2.403	2.403	2.403	2.403
	IS 1893-2002	1.207	1.207	1.207	1.207	1.207
20 storey	ETABS	2.311	2.272	2.257	2.25	2.247
	Wallace formula	3.8	3.8	3.8	3.8	3.8
	IS 1893-2002	1.509	1.509	1.509	1.509	1.509
25 storey	ETABS	3.316	3.279	3.267	3.262	3.261
	Wallace formula	5.544	5.544	5.544	5.544	5.544
	IS 1893-2002	1.811	1.811	1.811	1.811	1.811
30 storey	ETABS	4.71	4.673	4.663	4.66	4.66

TABLE 5.4 TIME PERIOD OF CSW BUILDING AND CSW(BEAM DEPTH AS PARAMETER)

TABLE 5.5 TIME PERIOD OF CSW BUILDING AND CSW(BEAM SPAN AS PARAMETER)

Storey	Beam span, m	1	1.5	2	2.5	3
	Wallace formula	0.17	0.158	0.148	0.14	0.132
	IS 1893-2002	0.302	0.302	0.302	0.302	0.302
5 storey	ETABS	0.262	0.286	0.313	0.337	0.359
	Wallace formula	0.639	0.597	0.559	0.526	0.497
	IS 1893-2002	0.604	0.604	0.604	0.604	0.604
10 storey	ETABS	0.715	0.732	0.772	0.821	0.877
	Wallace formula	1.338	1.249	1.171	1.102	1.041
	IS 1893-2002	0.906	0.906	0.906	0.906	0.906
15 storey	ETABS	1.356	1.33	1.35	1.396	1.465
	Wallace formula	2.403	2.242	2.102	1.979	1.869
	IS 1893-2002	1.207	1.207	1.207	1.207	1.207
20 storey	ETABS	2.311	2.21	2.183	2.203	2.265
	Wallace formula	3.798	3.545	3.323	3.128	2.954
	IS 1893-2002	1.509	1.509	1.509	1.509	1.509
25 storey	ETABS	3.316	3.125	3.029	2.998	3.031
	Wallace formula	5.542	5.172	4.849	4.564	4.31
	IS 1893-2002	1.811	1.811	1.811	1.811	1.811
30 storey	ETABS	4.71	4.398	4.209	4.106	4.091

5 -0 + 0



Similarly for wall length as parameter, from FIGURE 5.7, it can be observed that, the time period obtained from Wallace & Moehle's formula is quite higher than the IS1893-2002. The time period results are matching at 10, 15, 20, 25-storey

2

(a) FOR COUPLING BEAM SPAN = 3.0m

1

IS 1893 : 2002 CSW Time period-ETABS

3

Time period (Second)

FIGURE 5.6 TIME PERIOD OF CSW BUILDING AND CSW (COUPLING BEAM SPAN AS PARAMETER)

Δ

5

for 3.0, 4.0, 5.0 and 6.0m wall length respectively (FIGURE 5.7 and TABLE 5.6). By referring IS1893-2002, the base shear obtained is much higher value due to shorter time period. Therefore the design of CSW building is quite conservative.



(WALL LENGTH AS PARAMETER)

Storey	Wall length, m	2	3.0	4	5.0	6
	Wallace formula	0.286	0.17	0.116	0.086	0.068
	IS 1893-2002	0.302	0.302	0.302	0.302	0.302
5 storey	ETABS	0.386	0.262	0.2	0.163	0.139
	Wallace formula	1.075	0.639	0.439	0.327	0.257
	IS 1893-2002	0.604	0.604	0.604	0.604	0.604
10 storey	ETABS	1.085	0.715	0.533	0.426	0.357
	Wallace formula	2.24	1.338	0.922	0.689	0.543
	IS 1893-2002	0.906	0.906	0.906	0.906	0.906
15 storey	ETABS	2.082	1.356	1.001	0.792	0.656
	Wallace formula	4.026	2.403	1.654	1.237	0.975
	IS 1893-2002	1.207	1.207	1.207	1.207	1.207
20 storey	ETABS	3.573	2.311	1.693	1.331	1.095
	Wallace formula	6.367	3.798	2.614	1.953	1.539
	IS 1893-2002	1.509	1.509	1.509	1.509	1.509
25 storey	ETABS	5.13	3.316	2.426	1.904	1.563
	Wallace formula	9.296	5.542	3.812	2.847	2.242
	IS 1893-2002	1.811	1.811	1.811	1.811	1.811
30 storey	ETABS	7.302	4.71	3.437	2.691	2.204

TABLE 5.6 TIME PERIOD OF CSW BUILDING AND CSW (WALL LENGTH AS PARAMETER)

5.4 MODE SHAPE OF COUPLED SHEAR WALL

The different mode shapes of 5-storied CSW are shown in FIGURE 5.8. Here T and M in the FIGURE 5.8 indicate the time period and modal mass participation of the given mode. In the first mode of 5-storey CSW, the modal mass participation is 73.1% and time period is 0.262 seconds, which are the maximum among others.

The FIGURE 5.9 and TABLE 5.7 show the first five modes of 5, 10, 15, 20, 25 and 30-storey CSW. The parameters, wall length, depth and span of coupling beam are considered 3.0, 0.5 and 1.0m respectively. As the number of storey increase from 5 to 30 the modal participation in the first mode reduces from 73.1 to 63.1. At the same time, time period increases from 0.262 to 4.709 seconds. In higher modes, the modal masses are contributing more for the higher storey. The modal mass participation in second mode for 5-storey CSW is 18.3 and it increases to 20.21% for 20-storey CSW. Then it reduces to 20.05% in 30-storey. In case of other modes, the modal mass participation is in increasing rate. For example in mode 3 for 5-storey CSW, modal mass participation is 5.8% and then it increases to 6.53% for 30-storey CSW.

Chapter 5 Linear Dynamic Behavior of Coupled Shear Wall



	Period	М	ΣM		Period	М	ΣM
Mode	(second)	(%)	$\overline{(\%)}$	Mode	(Second)	(%)	$\overline{(\%)}$
For 5 storey				For 20 storey			
1	0.262	73.10	73.10	1	2.311	64.40	64.40
2	0.065	18.30	91.39	2	0.444	20.21	84.61
3	0.030	5.87	97.26	3	0.187	6.10	90.71
4	0.025	0.00	97.26	4	0.111	2.95	93.66
5	0.021	0.00	97.26	5	0.076	1.75	95.41
For 10 storey				For 25 storey			
1	0.715	68.10	68.10	1	3.316	63.63	63.63
2	0.162	19.11	87.21	2	0.607	20.18	83.81
3	0.072	5.73	92.94	3	0.250	6.35	90.16
4	0.044	3.03	95.98	4	0.146	3.03	93.19
5	0.030	1.78	97.75	5	0.099	1.76	94.95
For 15 storey				For 30 storey			
1	1.356	65.70	65.70	1	4.709	63.14	63.14
2	0.279	19.96	85.66	2	0.835	20.05	83.19
3	0.121	5.81	91.47	3	0.336	6.53	89.72
4	0.073	2.94	94.41	4	0.194	3.13	92.84
5	0.050	1.78	96.20	5	0.131	1.79	94.63

TABLE 5.7 MODAL MASS PARTICIPATION OF CSW

5.5 CLOSING REMARKS

The fundamental time period obtained from O. Chaallal's formula is having same pattern of variation for all storey (i.e. 5 to 30 storey). But from ETABS results, variation pattern of the time period is different for different storey as the parameters changes. The differences in values of time period between ETABS and O. Chaallal increase with increase in the dimension of parameters and the number of storey. When depth of coupling beam is the parameter, for 30 storied CSW and coupling beam depth of 1.5m the maximum 13% difference in two results were observed. Similarly for span of coupling and wall length as parameter the maximum difference in results attain much higher value.

As the number of storey increase, the fundamental time period of coupled shear wall is very close to the fundamental time period of coupled shear wall building obtained from Wallace & Moehle's formula. But there is a considerable difference between the time period obtained from Moehle's formula and IS1893-2002 (clause 7.6.2).

6.1 INTRODUCTION

Nonlinear structural behavior may be caused either by geometric or material nonlinearities. Geometric nonlinearities typically associated with large deformations. Material nonlinearities are associated with inelastic behavior exhibited by members strained beyond their yield capacity. The plastic hinges formed in this process have a considerable effect on the structural response. These are the source of dissipation of energy. By utilizing this concept, the structural response can be reduced to certain limit through controlled energy dissipation. The plastic hinges play an 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.

The design principle given in IS 1893: 2002 ensure that structures resist moderate earthquakes (DBE) without significant structural damage and aims that structures withstand a major earthquake without collapse [39]. As the major earthquake is concern; Asian continent experienced M9.0 magnitude of earthquake generated tsunami. So there is more possibility of failure of the structure in such events. Because the actual forces on the structure at the time of seismic event is much greater than the design forces, though there is material ductility and over strength criteria. Therefore there is a need to study the desired failure pattern of earthquake resistant structural system like CSW. The every structure has certain capacity. When demand exceeds the capacity of structure failure occurs. That failure pattern may be brittle or ductile. For the safety of life in seismic event ductile failure pattern is desired. To get the desired ductile failure pattern of CSW, this parametric study is carried out.

6.1.1 Ductility and Response Reduction Factor

The ductility and response reduction factor are two major issues in the inelastic behavior. Therefore to study the inelastic behavior of CSW, the ductility and response reduction factor concept becomes more important. Ductility of a structure, or its members, is the capacity to undergo large inelastic deformations without significant loss of strength or stiffness [39].



FIGURE 6.1 FORCE ~ DEFORMATION CURVE

The deformation and force relationship is shown in FIGURE 6.1 for elastic and inelastic system [40]. The structure in both the cases is same, but in case-1 structure is purely elastic. And in case-2 the structure is inelastic having certain ductile property. The forces and deformations are shown in FIGURE 6.1 are as follows:

 F_{o} = peak value of earthquake induced resisting force in corresponding to elastic system

 F_y = Yield strength of inelastic system

 Δ_o = Maximum deformation in corresponding to F_y for elastic system

 Δ_y = Yield deformation in corresponding to inelastic system

 Δ_m = Maximum deformation corresponds to inelastic system

The ductility factor can be defined as,

Ductility factor,
$$\mu = \frac{\Delta_m}{\Delta_y}$$
 (6.1)

For the elastic deformation, $\mu = 1.0$ and inelastic deformation $\mu > 1.0$. Response reduction factor can be defined as,

Response reduction factor,
$$R = \frac{F_o}{F_v}$$
 (6.2)

The relation between $\boldsymbol{\mu}$ and R is,

$$R = \frac{F_o}{F_y} = \frac{\Delta_o}{\Delta_y} \quad \text{and} \quad \mu = \frac{\Delta_m}{\Delta_y} = \frac{\Delta_m}{\Delta_o} \times \frac{\Delta_o}{\Delta_y} = \frac{\Delta_m}{\Delta_o} \times R$$

The ductility factor and response reduction factor aren't same. For the incorporation of μ and R in seismic analysis of building, the empirical relationship is [6],

For low time period building,
$$\mu = \frac{R^2 + 1}{2}$$

For high time period building (T > 1.0 second), $\mu = R$







FIGURE 6.3 DUCTILITY AND RESPONSE RELATIONSHIP

The FIGURE 6.2 and 6.3 show that with increase in ductility of the structure the response and strength decreases. The reduction in forces is due to yielding and damping. Therefore the more ductile structure has the more response reduction factor.

6.2 INELASTIC BEHAVIOR OF COUPLED SHEAR WALL

The inelastic behavior of cantilever shear walls is dependent on the plastic hinge zone at the wall base, where large rotations and yielding of reinforcement takes place. As a result, the stiffness, strength, ductility and means of dissipating energy of the entire structural system are wholly focused on the response of base region.



FIGURE 6.4 HINGE FORMATION IN SHEAR WALL AND COUPLED SHEAR WALL

The coupling beams connect two or more structural walls in CSW. In the inelastic behavior, the formation of plastic hinges occurred at the end of coupling beams and base of walls. As a result the forces with in the whole structure redistributed throughout the height instead of concentrating at particular region. (In CSW, the maximum redistribution of forces in wall and coupling beam is 30 and 20% allowed respectively) [6]. The desired full energy dissipating mechanism in coupled walls will be similar to that in multistory frames with strong columns and weak beams [6]. The more plastic hinge will form, the more energy will dissipate. In coupled shear wall, coupling beams are the primary source of energy dissipation. Also degree of coupling has great influence over the energy dissipation. From the study, it is observed that as the DC increases the formation of plastic hinge also increases in coupling beam. Therefore NBCC has given the guideline that when DC is greater than 0.66, response reduction factor (R) is 3.5.

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

6.3 NONLINEAR STATIC ANALYSIS

In Nonlinear static analysis, a model directly incorporating inelastic material response is displaced to a target displacement and resulting internal deformations and forces are determined. The mathematical model of the building is subjected to monotonically increasing lateral forces or displacements until either a target displacement is exceeded or the building collapses. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. The target displacement may be calculated by procedure given in FEMA 273 (FIGURE 6.5) [41]. In approximation, the target displacement may be considered as 0.04 times the height of the building [41].

A Nonlinear static procedure provides a graphical representation of the global force-displacement capacity curve of the structure (i.e. pushover curve) and compares it to the response spectra representations of the earthquake demands.

The Nonlinear static method requires three primary elements: capacity, demand and performance. In this method both demand response spectra and structural capacity curves be plotted in the spectral acceleration vs. spectral displacement domain. Spectra plotted in this format are known as Acceleration-Displacement Response Spectra (ADRS). To convert a spectrum from the standard S_a vs T format given in IS 1893-2002 to ADRS format, it is necessary to determine the value of S_d for each point on the curve. This can be done with the equation:

$$S_d = \frac{T^2}{4\pi^2} S_d \tag{6.3}$$

Similarly the procedure is given in ATC-40 to develop the capacity spectrum from the capacity curve or pushover curve [8]. The ADRS format allows the demand spectrum to be overlaid on the capacity spectrum of building. The intersection of the demand and capacity spectra gives the performance point.



FIGURE 6.5 NONLINEAR STATIC APPROACH

6.3.1 Analysis Procedure

The Nonlinear analysis procedure followed for the inelastic behavior of coupled shear wall is discussed below.

Step-1: Modeling

Here the coupled shear wall is modeled as wide-column frame. The wall is modeled as column element and coupling beam as beam element (chapter 3). The rigidity is applied on the beam, which is in the wall region for transferring vertical deformation and rotation.

Step-2: Loading

The dead load and the earthquake load calculated as per equivalent static approach as given in IS1893-2002 is applied on the mathematical model of CSW.

Step-3: Hinge Properties

The flexural and shear hinge properties is defined as per the FEMA 356 (Table 6-18 & 6-19) and ATC 40. After the hinge properties are defined, those are assigned at the end of coupling beam and at the base of wall.

Step-4: Linear Analysis and Design of CSW

The linear analysis of CSW is carried with the help of ETABS. The wall and the coupling beam are designed for the responses obtained from the linear analysis.

Step-5: Static Nonlinear Load case

For the Static Nonlinear analysis, the static load cases are to be defined.

Step-6: Static Nonlinear analysis

With the help of ETABS software, the Static Nonlinear analysis is performed.

Step-7: Review of Static Nonlinear results

The results to be reviewed from the Static Nonlinear analysis of CSW are capacity curve and the pattern of failure in deformed shape.

Plastic hinge properties in FEMA 356-2000 and ATC 40 (Section 6.3.1, Step 3):

Table 6-18 N N	lodeling Para lembers Con	meters and Nu trolled by Flexu	merical Jre	Accept	ance Crite	ria for Nc	nlinear	Proced	ures—		
						Acce	Acceptable Plastic Hinge Rotation (radians)				
							Perfor	mance L	evel		
			Plactic	Hinga	Besidual			Compon	ent Type	,	
			Rota (rad	ation lans)	Strength Ratio		Prin	nary	Secor	Secondary ⁴	
Conditions			а	b	с	10	LS	CP	LS	CP	
i. Shear walls and	t wall segment	S	•	· · · · ·							
$\frac{(\mathcal{A}_{s} - \mathcal{A}_{s}')f_{y} + F}{t_{w}t_{w}f_{c}'}$	$\frac{\text{Shear}}{t_w l_w \sqrt{f_s}}$	Confined ⊟ Boundary ¹									
≤ 0.1	≤3	Yes	0.015	0.020	0.75	0.005	0.010	0.015	0.015	0.020	
≤ 0,1	≥ 6	Yeş	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	≥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	
iii. Shear wall cou	Ipling beams			-							
Longitudinal reinforcement and transverse reinforcement ³ $\frac{St}{t_w l}$		$\frac{\text{Shear}}{t_w l_w \sqrt{f_c}}$									
Conventional long	itudinal	≤ 3	0.025	0.050	0.75	0.010	0.02	0.025	0.025	0.050	
reinforcement with conforming transverse reinforcement		26	0.02	0.040	0.50	0.005	0.010	0.020	0.020	0.040	
Conventional long	fuctinal	≤ 3	6.020	0.035	0.60	0.006	0.012	0.020	0.020	0.035	
reinforcement with nonconforming transverse reinforcement		26	8,010	0,025	0,25	0,005	0,008	0,010	0,010	0,025	
Diagonal reinforce	rivent	n.a.	0.030	0.050	0.80	0.006	0.018	0.030	0.030	0.050	

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

					Acce	ptable T Rotat	otal Drift tion (rad	t (%) or 0 lans) ¹	Chord
						Perfc	vmance	Level	
		Total Dr (%), or	ift Ratio Chord	Residual		Component Type			
		Rotation (radians) ¹		Strength Ratio		Primary		Secondary	
Conditions		d	ø	c	10	LS	CP	LS	CP
i. Shear walls and wall segments	i. Shear walls and wall segments								
All shear walls and wall segments	All shear walls and wall segments ²			0,40	0,40	Q.60	0.75	0,75	1,5
ii. Shear wall coupling beams ⁴									
Longitudinal reinforcement and transverse reinforcement ³	$\frac{\text{Shear}}{t_w l_{w*} \sqrt{f_c'}}$								
Conventional longitudinal reinforcement with conforming	≤3	0.002	0.030	0.60	0.006	0.015	0.020	0.020	0.038
transverse remiorcement	≥6	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	0.005	0.008	Ó.010	0.010	0.028
reinforcentent	≥6	0.008	0.014	0.20	0.004	0.006	0.007	0.007	0.012

6.3.2 Hinge Properties

Action in the Static Nonlinear analysis is classified as deformation controlled and forced controlled. A deformation controlled action is one that has an associated deformation that is allowed to exceed the yield value and the maximum associated deformation is limited by the ductility capacity of the component. A forced controlled action is one that has an associated deformation that isn't allowed to exceed the yield value.



In FIGURE 6.6, point B corresponds to significant yielding, point C corresponds to the point where significant lateral resistance is assumed to be lost, and point E corresponds to the point where gravity load resistance is assumed to be lost. The load-deformation relationship in FIGURE 6.6(a) referred to shear walls having inelastic behavior under lateral loading that is governed by flexure. The x-axis of FIGURE 6.6(a) should be taken as the rotation over plastic hinge region at the end of the member. The hinge rotation at point B corresponds to the yield point, θ_y and it is given by the following expression:

$$\theta_{y} = \left(\frac{M_{y}}{E_{c}I}\right) L_{p} \tag{7.2}$$

Where M_y = Yield moment capacity of wall, E_c = Concrete modulus, I = Member moment of inertia and L_p = Plastic hinge length. For the analytical models of walls the value of L_p shall be equal to 0.5 times the flexural depth of the element but less than one storey height [41].

When the Shear controls the inelastic behavior of wall, it is more appropriate to use drift as the deformation value in FIGURE 6.6(b). The member drift also shown in FIGURE 6.7(b).



(c) CHORD ROTATION FOR COUPLING BEAM FIGURE 6.7 INELASTIC HINGE BEHAVIOR OF CSW

For the coupling beams, the deformation measure to be used in FIGURE 6.6(b) is the chord rotation for the member as shown in FIGURE 6.7(c). Chord rotation is the most representative measure of the deformed state of a coupling beam, whether its inelastic response is governed by flexure or by shear [41].

6.4 PARAMETRIC STUDY ON NONLINEAR BEHAVIOR OF CSW

The nonlinear behavior of CSW is studied for 10, 15 and 20-storey. The other parameters, wall length, depth and span of coupling beam are considered for the study as per given in TABLE 1.1-1.3.

6.4.1 Nonlinear Static Analysis of 10-storey CSW

The wall length, depth and span of coupling beam are considered as 3.0, 0.5, and 1.0m respectively. The model for nonlinear analysis of CSW in this study is wide-column frame (FIGURE 6.8). The shell element isn't considered due to the limitations in the ETABS. ETABS has facility for defining hinges for the line

element or frame element [10]. The flexural and shear hinge properties of wall and coupling beam are defined in ETABS as shown FIGURE 6.8 with reference to tables given in FEMA 356-2000 and ATC 40. In the loadings gravity loads and lateral loads generated from earthquake are applied on the model. These load calculations are same as in linear static case. The linear analysis and design is carried out for the corresponding responses in ETABS. Then for nonlinear analysis two load cases are defined (FIGURE 6.9). First one for gravity load and second for lateral load case is defined. The gravity load and lateral load are to be applied by force and displacement action respectively.





(a) CSW MODEL (b)FLEXURAL HINGE IN WALL

(c) SHEAR HINGE IN WALL



FIGURE 6.8 MODEL AND INELASTIC HINGE PROPERTIES OF CSW



After all these inputs the nonlinear static analysis of CSW is performed using ETABS software. The failure patterns of CSW for flexural hinge are observed from the pushover or capacity curve obtained from analysis (FIGURE 6.10 and TABLE 6.1). There is sudden drop in base shear observed in between step 5 and 6. And at this step corresponding displacement is 0.0121m, where the beams are failing in flexure (FIGURE 6.12(a)).



TABLE 6.1 CAPACITY CURVE (FLEXURAL HINGE)

Step	Displacement (meter)	Base Force (kN)
0	1.77E-05	0
1	0.0073	168.0
2	0.0101	225.4
3	0.0118	247.0
4	0.0118	245.6
5	0.0121	249.2
6	0.0121	129.7



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When shear hinges are defined in the model, in between step 2 and 3 there is a drop in base force for 0.0083m displacement (FIGURE 6.11 and TABLE 6.2). In this region of capacity curve the shear hinges are formed in the coupling beam as shown in FIGURE 6.12(b). The point is to be noted that for 0.0121m displacement the coupling beam is failing in flexure, where as for displacement 0.0083m displacement the coupling beams are failing in shear. Therefore the beam is critically failing in shear failure.

The wall is failing in the shear hinge model, in between step 11 and 12 for displacement of 0.5087m at 2nd and 3rd-storey level (FIGURE 6.13(a) and TABLE 6.2). When flexure and shear hinges are both incorporated in the model, the significant feature in this model is the wall is failing in flexure for the displacement of 0.0453 m (FIGURE 13 & 14, TABLE 6.3).





TABLE 6.3 CAPACITY CURVE(FLEXURE AND SHEAR HINGE)

	Displacement	Base force				
Step	(meter)	(kN)				
0	1.77E-05	0				
1	0.0039	88.9				
2	0.0083	169.1				
3	0.0083	82.9				
4	0.0085	85.2				
5	0.0085	83.9				
6	0.0086	84.9				
7	0.0087	5.3				
8	0.0164	42.6				
9	0.0164	41.6				
10	0.0187	52.0				
11	0.0188	51.0				
12	0.0199	56.1				
13	0.0199	54.3				
14	0.0224	65.4				
15	0.0224	63.7				
16	0.028	88.8				
17	0.0453	119.4				
18	0.0453	-34.9				

6.4.2 Results and discussion

The nonlinear behavior of CSW is studied for 10, 15 and 20-storey considering all the parameters. The study on failure pattern of CSW is carried out for other cases similar to the procedure explained in section 6.4.1. In each case three models are studied incorporating flexural hinge, shear hinge and flexure and shear hinge both. The failure pattern is studied from their capacity curve and the hinge formation.

When beam depth as parameter the results are summarized in TABLE 6.4. The coupling beam is failing in shear for beam depth 0.5m at 10 and 15-storey but other cases the beam is failing in flexure. In this case, the wall length is kept 3.0m. The walls are failing in flexure for 10-storey CSW and walls are failing in shear in case of other storeys. In case of flexural failure of walls, the hinges are formed at the base of wall. The sequence of failure is concern; the walls are failing after the failure of coupling beams.

The failure patterns of CSW are summarized in TABLE 6.5 for beam span as parameter. By increasing the span, the failure pattern of 0.5m coupling beam depth is changing from shear to flexure for 10-storey CSW.

				Failu	re of	Coupling							
				beam			Fai	lure c	of Wall	Remark			
											Failure		Failure
Problem	\mathbf{D}_{w}	H_{b}	L_{b}	Δ	F	Storey	Δ	F	Storey	Failure	sequence	Failure	sequence
No.	(m)	(m)	(m)	(mm)	(kN)	Location	(mm)	(kN)	Location	of beam	of beam	of wall	of wall
10 STOREY													
6	3	0.5	1	108	8	2 to 6	119	45	Base	Shear	First	Flexure	Second
7	3	0.75	1	252	11	2 to 10	154	43	Base	Flexure	First	Flexure	Second
8	3	1	1	266	11	1 to 10	154	42	Base	Flexure	First	Flexure	Second
9	3	1.25	1	263	10	1 to 10	154	41	Base	Flexure	First	Flexure	Second
10	3	1.5	1	268	10	1 to 10	154	40	Base	Flexure	First	Flexure	Second
15 STOREY													
11	3	0.5	1	189	21	2 to 8	2419	1558	3&4	Shear	First	Shear	Second
12	3	0.75	1	270	29	4 to 15	2420	1558	3 & 4	Flexure	First	Shear	Second
13	3	1	1	273	28	2 to 15	2423	1559	3 & 4	Flexure	First	Shear	Second
14	3	1.25	1	272	28	3 to 15	2423	1559	3 & 4	Flexure	First	Shear	Second
15	3	1.5	1	277	28	2 to 15	2423	1559	3&4	Flexure	First	Shear	Second
20 STOREY													
16	3	0.5	1	367	195	7 to 12		NA		Flexure	First	NA	
17	3	0.75	1	313	105	5 to 12	1622	552	3&4	Flexure	First	Shear	Second
18	3	1	1	290	82	4 to 19	1624	462	2	Flexure	First	Shear	Second
19	3	1.25	1	288	75	3 to 18	1633	428	1	Flexure	First	Shear	Second
20	3	1.5	1	295	74	2 to 19	1631	410	Base	Flexure	First	Shear	Second

TABLE 6.4 SUMMARY OF FAILURE PATTERN (BEAM DEPTH AS PARAMETER)

N.B.:- The Δ and F are the Roof displacement and Base force respectively.

			Failure of Coupling										
				beam			Failure of Wall			Remark			
										Failure	Failure		Failure
Problem	D_{w}	H_{b}	L _b	Δ	F	Storey	Δ	F	Storey	of	sequence	Failure	sequence
No.	(m)	(m)	(m)	(mm)	(kN)	Location	(mm)	(kN)	Location	beam	of beam	of wall	of wall
10 STOREY													
36	3	0.5	1	108	8	2 to 6	119	45	Base	Shear	First	Flexure	Second
37	3	0.5	1.5	176	8.5	4		NA		Shear	First	NA	
38	3	0.5	2	421	32	7 to 10	1350	71	2	Flexure	First	Shear	Second
39	3	0.5	2.5	510	41	9 & 10	1335	77	2	Flexure	First	Shear	Second
40	3	0.5	3	437	73	6 to 10	428	83	Base	Flexure	First	Flexure	Second
15 STOREY													
41	3	0.5	1	189	21	2 to 8	2419	1558	3 & 4	Shear	First	Shear	Second
42	3	0.5	1.5	209	22	4 to 7	1630	1088	1	Shear	First	Shear	Second
43	3	0.5	2	412	128	1 to 4	412	128	Base	Flexure	Second	Flexure	First
44	3	0.5	2.5	488	78	15	1578	180	3 & 4	Flexure	First	Shear	Second
45	3	0.5	3	556	113	14 & 15	1561	185	4	Flexure	First	Shear	Second
20 STOREY													
46	3	0.5	1	367	195	7 to 12		NA		Flexure	First	NA	
47	3	0.5	1.5	119	28	8		NA		Shear	First	NA	
48	3	0.5	2	171	35	7		NA		Shear	First	NA	
49	3	0.5	2.5	477	124	18 & 19	1559	331	5	Flexure	First	Shear	Second
50	3	0.5	3	531	171	20	1543	328	5	Flexure	First	Shear	Second
By increasing span the failure pattern of wall in more cases are shear. In TABLE 6.6, the failure pattern is studied for wall length as parameter. In this case the depth and span of coupling beams are kept constant, only wall lengths are varying. By increasing the wall length, the failure pattern of 0.5m coupling beam depth is changing from shear to flexure. And the walls are failing in shear predominately with increase in Wall length.

				Failure of Coupling									
					bea	eam Failure of Wall			of Wall	Remark			
										Failure	Failure		Failure
Problem	$D_{\rm w}$	H_b	L _b	Δ	F	Storey	Δ	F	Storey	of	sequence	Failure	sequence
No.	(m)	(m)	(m)	(mm)	(kN)	Location	(mm)	(kN)	Location	beam	of beam	of wall	of wall
10) STO	REY	-										
66	2	0.5	1	88	11	1 to 7	170	208	Base	Shear	First	Flexure	Second
67	3	0.5	1	108	8	2 to 6	119	45	Base	Shear	First	Flexure	Second
68	4	0.5	1	264	7	6 to 10	1813	183	2	Flexure	First	Shear	Second
69	5	0.5	1	290	4	6 to 10	2261	119	2	Flexure	First	Shear	Second
70	6	0.5	1	300	3	7 to 10	2719	85	2	Flexure	First	Shear	Second
1.	5 STO	REY											
71	2	0.5	1	107	3	1 to 10	100	376	Base	Shear	First	Flexure	Second
72	3	0.5	1	189	21	2 to 8	2419	1558	3 & 4	Shear	First	Shear	Second
73	4	0.5	1	266	15	8 to 15	2160	592	2	Flexure	First	Shear	Second
74	5	0.5	1	290	9	8 to 15	2707	384	1	Flexure	First	Shear	Second
75	6	0.5	1	298	6	8 to 15	3257	271	2	Flexure	First	Shear	Second
20) STO	REY											
76	2	0.5	1	133	53	2 to 13		NA		Shear	First	NA	
77	3	0.5	1	367	195	7 to 12		NA		Flexure	First	NA	
78	4	0.5	1	270	33	13 to 20		NA		Flexure	First	NA	
79	5	0.5	1	294	20	14 to 20	2701	888	2	Flexure	First	Shear	Second
80	6	0.5	1	358	397	1 to 3	407	71	Base	Flexure	Second	Flexure	First

 TABLE 6.6 SUMMARY OF FAILURE PATTERN (WALL LENGTH AS PARAMETER)

6.4.2.1 Plastic hinge and degree of coupling

There is a significant effect of degree of coupling on plastic hinge formation. It can be studied from FIGURE 6.15, as the depth of coupling beam increase the more number of plastic hinges are formed. This is same for all the stories, which are studied. The degree of coupling increases with increasing in depth of coupling at the lower storey. The points to be noted here is as the degree of coupling increases the formation of number of hinges are also increases. That means with increasing the coupling action more amount of energy can be dissipated. Since the plastic hinges are the source of dissipation of energy. The guidelines given in NBCC can be highlighted here is that when DC is greater than 0.66, R value is to be considered as 4.0.



FIGURE 6.15 RELATIONSHIP BETWEEN PLASIC HINGE AND DEGREE OF COUPLING

6.5 CLOSING REMARKS

The nonlinear behavior 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 15-storey CSW, with increase in span of coupling beam, failure pattern of coupling beam changes from shear to flexure. More cases in wall for span of coupling beam as parameter, the shear failure is critical. Similarly for wall length as parameter, the failure pattern of 0.5m depth coupling beam changes from shear to flexure. And 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.

Here the number of hinge formation with respect to increases in coupling beam depth is studied and the effect of number of hinge formation on degree of coupling is also observed. The important aspect to be noted as the depth of coupling beam increases the number of hinge formation increases. In other way, the degree of coupling at base increases with increase in depth of coupling beam. From the above two study, it can be concluded that as the degree of coupling increase the number of hinge formation also increase. <u>More coupling action dissipates more energy</u>.

7. SEISMIC DESIGN OF COUPLED SHEAR WALL AND STUDY OF ITS FOUNDATION

7.1 INTRODUCTION

The earthquake in the present Indian scenario is a major issue, which is learned from the past experience like Kashmir, Sumatra and Bhuj earthquake. The earthquake resistant structural systems are required to resist the lateral load generated from seismic effects. The coupled shear wall is one of the earthquake resistant structural systems. The mechanism in the CSW gives more ductility to the structure. In ultimate stage the plastic hinges are formed in the coupling beams and at the base of the walls. These plastic hinges are the means of dissipation of energy. For the implementation of CSW in the construction needs the proper seismic design and ductile detailing. The seismic design depends on the behavior of CSW. Therefore a parametric study is performed to understand the behavior and then seismic design is carried out for the same. The 15-storied coupled shear wall building is studied with the parameters as depth and span of coupling beams and the length of wall. The design forces and reinforcements are obtained in the analysis and design of coupled wall and the uncoupled wall are compared in this chapter. The coupled shear wall and Shear Wall is modeled and analyzed in ETABS software. The behavior of CSW along with its foundation is also studied, considering soil as elastic support.

7.2 DESIGN CONSIDERATIONS

The 15-storey coupled shear wall is considered for the seismic design study. The other parameters considered are wall length, depth and span of coupling beam.

7.2.1 Coupled Shear Wall

The typical floor plan of CSW building is shown in FIGURE 1.8 (SHEET No. 1). The parameters considered in study are, span and depth of coupling beam and wall length, which are shown in TABLE 1.1 to 1.3. The CSW is modeled and analyzed using ETABS software.

Earthquake loads are calculated as per IS1893:2002 by using equivalent static approach for zone = III, response reduction factor = 4, and importance factor =

1. Wallace formula for time period of CSW building is used for seismic load calculation due to inadequacy of the time period formula, which is given in IS 1893-2002 []. The Wallace formula (1992) is,

$$T_{wallace} = 6.2 \frac{h_w}{(2D_w + L_b)} n \sqrt{\frac{wh_s}{gE_cp}}$$
(Ref 5.3)

Where n = number of storey, D_w = wall length, w = unit floor weight, h_w = height of coupled shear wall, p = ratio of wall area to floor plan area, h_s = mean storey height, g = acceleration due to gravity; E_c = concrete modulus of elasticity and L_b = span of coupling beam.

7.2.2 Uncoupled Shear Wall

The problem formulation for the uncoupled shear wall is given in TABLE 7.1. The building plan and number of storey is to be considered same as coupled shear wall. The concrete grade of M30 and Fe415 grade steel are considered for the material properties in this study.

TABLE 7.1 PROBLEM FORMULATIONS FOR 15-STOREY UNCOUPLED SHEAR WALL

Problem. No.	а	b	с	d	e
Length of Wall (meter)	2.0	3.0	4.0	5.0	6.0

7.3 DESIGN APPROACH FOR COUPLED SHEAR WALL

The design of CSW involves the design of wall and coupling beam. The designing approach for the wall and coupling beam are discussed in the subsequent sections.

7.3.1 Design of Couple Wall

It will be necessary to design the walls to withstand the bending moments and forces acting on wall elements of the coupled wall system. The bending moment pattern is similar to that of simple cantilever walls. In addition, because of the coupling system there will be considerable axial forces which may produce net tension in the walls [5].

Therefore the coupled walls are designed like the cantilever wall as per the provisions given in IS 13920:1993 (Clause 9) [42].

7.3.2 Design of Coupling Beam

In IS 13920 : 1993, formulae are given for calculating the area of diagonal reinforcement and condition for applicability of diagonal reinforcement. In this chapter the designing of coupling beam is discussed below.

7.3.2.1 Diagonal Reinforcement

The design procedure for the diagonal reinforcement is explained below:

• If the earthquake induced shear stress in the coupling beam exceeds (IS 13920:1993, Clause 9.5.1)

$$\frac{0.1L_b\sqrt{f_{ck}}}{D} \tag{8.1}$$

Where L_b is the clear span of coupling beam and D is the overall depth, the entire earthquake induced shear and flexure shall, preferably, be resisted by diagonal reinforcement.

 Angle between the diagonal reinforcement and longitudinal axis of the coupling beam, a can be calculated from following formula by trial and error [46].

$$\tan \alpha = \frac{D - \frac{2x}{\cos \alpha}}{L_{b}}$$
(8.2)

It should be noted that diagonal reinforcement is effective only if the bars are placed with a reasonably large inclination angle a. So, diagonally reinforced coupling beams are restricted to $a = 13^{0}$. Therefore, for beam with a geometry that results in a less than about 13^{0} , ACI 318-02 [43] doesn't permit diagonal reinforcement.

• The area of reinforcement to be provided along each diagonal in a diagonally reinforced coupling beam shall be (IS 13920:1993, Clause 9.5.2):

Chapter 7 Seismic Design of Coupled Shear Wall and Study of Its Foundation



FIGURE 7.1 FORCES IN DIAGONALLY REINFORCED COUPLING BEAM

$$2M_{\rm u} = V_{\rm u} \times I_{\rm s} \tag{8.3}$$

$$V_u = 2T_u \sin \alpha \tag{8.3a}$$

$$A_{sd} = \frac{V_u}{2f_v \sin \alpha}$$
(8.3b)

$$A_{sd} = \frac{V_u}{1.74 f_v Sin\alpha}$$
(8.3c)

Where V_u = the factored shear force. T_u and C_u are the tension and compression forces along diagonal. The design of a diagonally reinforced coupling is based on the assumption is that the shear force resolves itself into diagonal tension (T_u) and compression (C_u) forces. The equation 8.3b will be obtained by solving the equation 8.3 and 8.3a. The actual area of diagonal reinforcement is required as per equation 8.3b. The codal provision for the calculation of area of diagonal reinforcement is given in equation 8.3c, which is conservative [12, 18].

At least 4 bars of 8 mm diameter shall be provided along each diagonal. Each diagonal element is reinforced similar to a column consisting of longitudinal and transverse reinforcement. The minimum core dimensions measured to the outside of the transverse reinforcement not less than $b_w/2$ and $b_w/5$ [46].

• The diagonal bars of a coupling beam shall be anchored in the adjacent walls with an anchorage length of 1.5 times the development length in tension (IS 13920:1993, Clause 9.5.3).

7.3.2.2 Transverse Reinforcement

The design procedure for the transverse reinforcement is explained below:

- The spacing of ties or pitch of spiral shall not exceed 100mm (IS 13920:1993, Clause 9.5.2).
- The transverse reinforcement required around the diagonal bars to prevent buckling. The area of bar to be used for the transverse reinforcement is [6],

$$A_{te} = \frac{Af_y s}{16f_y 100}$$

$$(8.4)$$

Where s = the spacing of transverse reinforcement and A = the area of one bar of diagonal reinforcement.

7.3.2.2 Horizontal and Vertical Reinforcement

The design procedure for the horizontal and vertical reinforcement is explained below:

The intent of horizontal and vertical reinforcement is to contain or to hold the concrete outside the diagonal cores, in case the concrete is damaged by earthquake loading. Since the diagonal reinforcement is designed to resist the entire shear and flexure in the coupling beam, the additional horizontal and vertical reinforcement acts primarily as a basketing reinforcement to contain concrete that may spall [46]. The minimum reinforcement for the horizontal and vertical steel shall not be less than 0.25% of gross area to be provided.

7.4 REINFORCEMENT DETAILING FOR COUPLED SHEAR WALL

The typical reinforcement details for wall and coupling beam of coupled shear wall is given in FIGURE 7.2. The detailing of diagonal reinforcement is important in the coupling beam.



FIGURE 7.2 REINFORCEMENT DETAILS OF COUPLED SHEAR WALL

7.5 RESULTS AND DISCUSSION

All the design cases for coupled shear wall mentioned in TABLE 1.1 to 1.3 are modeled and analyzed using ETABS software. The member forces obtained form the analysis are designed considering all the parameters like depth and span of coupling beam and length of wall.

7.5.1 Beam Depth as parameter

The member forces in couple wall and the reinforcement details are given in TABLE 7.2. From the TABLE 1.1-1.3 and 7.2, it can be observed that as the depth of coupling beam increases, the axial force and shear in the coupled wall increases and the moment in the wall decreases. The nominal reinforcement is obtained form the design of coupled wall. The forces in the coupling beam and reinforcement details are given in TABLE 7.3. The shear and moment in the

coupling beam reduces with the increase in depth of coupling beam. The diagonal reinforcement is required more for the lesser depth of coupling beam.

7.5.2 Beam span as Parameter

When coupling beam span is considered as parameter the span of coupling beam increases the axial forces in the wall decreases and the moment and the shear increases (TABLE 1.1 to 1.3 and 7.2). The nominal reinforcement is obtained from the design of wall. With the increase in span of coupling beam, the shear and moment in the coupling beam increases. The reinforcement in the coupling beam increases with the increases in span of coupling beam (TABLE 7.3).

7.5.3 Wall Length as Parameter

When the length of wall increases, the axial force, moment and shear in wall increase (TABLE 1.1 to 1.3 and 7.2). The boundary element is required for the wall length of 2.0m. For the other parameters, the nominal reinforcements are obtained from design. The shear and moment in the coupling beam increase with increase in wall length. The reinforcement in the coupling beam increase with increase in length of wall (TABLE 7.3).

Prob.	Axial	Moment	Shear	Horizontal	Vertical	Reinf. In				
No.	forces (kN)	(kNm)	(kN)	Reinforcement	Reinforcement	B. E.				
			Be	am Depth as Parameter	·					
11	6885	1384	246	10 # @ 200 c/c	10 # @ 200 c/c	NR				
12	6961	1279	274	10 # @ 200 c/c	10 # @ 200 c/c	NR				
13	7008	1222	305	10 # @ 200 c/c	10 # @ 200 c/c	NR				
14	7043	1176	340	10 # @ 200 c/c	10 # @ 200 c/c	NR				
15	7074	1128	378	10 # @ 200 c/c	10 # @ 200 c/c	NR				
Beam Span as Parameter										
41	6885	1384	246	10 # @ 200 c/c	10 # @ 200 c/c	NR				
42	6865	1517	258	10 # @ 200 c/c	10 # @ 200 c/c	NR				
43	6818	1700	270	10 # @ 200 c/c	10 # @ 200 c/c	NR				
44	6804	1922	285	10 # @ 200 c/c	10 # @ 200 c/c	NR				
45	6805	2169	301	10 # @ 200 c/c	10 # @ 200 c/c	NR				
			W	all Length as Parameter						
71	5983	651	147	10 # @ 200 c/c	10 # @ 200 c/c	8 nos 16 #				
72	6885	1384	246	10 # @ 200 c/c	10 # @ 200 c/c	NR				
73	7749	2349	356	10 # @ 200 c/c	10 # @ 200 c/c	NR				
74	8545	3534	476	10 # @ 200 c/c	10 # @ 200 c/c	NR				
75	9341	4929	604	10 # @ 200 c/c	10 # @ 200 c/c	NR				

 TABLE 7.2 FORCES AND REINFORCEMENT SCHEDULE FOR COUPLE WALL

N.B.:- '#' represents TOR steel

				Stirrups for				
				Diagonal				
Prob.	Shear	Moment	Diagonal	Reinf.	l_d	Horizontal	Vertical	
No.	(kN)	(kNm)	Reinforcement.	(2-legged)	(mm)	Reinforcement.	Reinforcement.	
			Bea	am Depth as Para	neter		·	
11	289	68	8 nos. 20 #	8 # @ 100c/c	1150	6 nos. 10 #	8 # @ 125c/c	
12	183	60	8 nos. 12 #	8 # @ 100c/c	1000	6 nos. 12 #	8 # @ 125c/c	
13	128	51	4 nos. 12 #	8 # @ 100c/c	700	8 nos. 12 #	8 # @ 125c/c	
14	97	44	4 nos. 12 #	8 # @ 100c/c	700	10 nos. 12 #	8 # @ 125c/c	
15	78	39	4 nos. 12 #	8 # @ 100c/c	700	10 nos. 12 #	8 # @ 125c/c	
			Be	am Span as Paran	neter		·	
41	289	68	8 nos. 20 #	8 # @ 100c/c	1150	6 nos. 10 #	8 # @ 125c/c	
42	405	112	4 nos. 32 # +	× # @ 100₀/₀	1950	6 nos 10 #	× # @ 125 α/α	
42	403	112	4 nos. 20 #	8# @ 1000/C	1830	0 1108. 10 #	0π @ 1250C	
43	500	156	8 nos. 32 #	8 # @ 100c/c	1850	6 nos. 10 #	8 # @ 125c/c	
44	574	202	8 nos. 36 # +	8 # @ 100c/c	2050	6 nos 10 #	8 # @ 125c/c	
	574	202	4 nos. 25 #	0 // @ 1000/0	2030	0 1108. 10 #	0 // @ 1250/0	
45	636	250	12 nos. 36 #	8 # @ 100c/c	2050	6 nos. 10 #	8 # @ 125c/c	
			Wa	ll Length as Para	neter			
71	236	62	4 nos. 20 # +	8 # @ 100c/c	1150	6 nos 10 #	8 # @ 125c/c	
/1	230	02	4 nos. 16 #	0 m @ 1000/C	1150	0 1105. 10 #	0π @ 1250/C	
72	289	68	8 nos. 20 #	8 # @ 100c/c	1150	6 nos. 10 #	8 # @ 125c/c	
73	331	76	8 nos. 20 #	8 # @ 100c/c	1150	6 nos. 10 #	8 # @ 125c/c	
74	366	82	4 nos. 25 # +	8 # @ 100c/c	1500	6 nos 10 #	8 # @ 125.00	
/4	³⁰⁰ ⁸² 4 nos. 16 #		4 nos. 16 #	0 # @ 1000/C	1500	0 1105. 10 #	0 # @ 1250/0	
75	307	88	4 nos. 25 # +	8 # @ 100c/c	1500	6 nos 10 #	8 # @ 125 do	
75 397		88	4 nos. 20 #	0 # @ 100C/C	1500	0 1105. 10 #	0# @ 1250/0	

TABLE 7.3 FORCES AND REINFORCEMENT SCHEDULE FOR COUPLING BEAM

7.5.4 Comparison between Coupled Shear Wall and Shear Wall

The axial force, moment and shear in the shear wall are given in TABLE 7.4. From the comparison of different wall lengths of coupled shear wall and shear wall the points are to be noted as follows.

- The axial forces are more in CSW in compared to the SW. The increase in axial forces also increases the moment carrying capacity of wall.
- The moment in the SW is considerably higher than CSW. Therefore the requirement of reinforcement in SW is far higher than the CSW. To withstand such a high moment (TABLE 7.4), the large amount of steel reinforcement are required. The reason for the high moment in the shear wall is all the forces are concentrating at the base of wall. Due to redistribution of forces in CSW, the design forces in CSW is lesser than SW.

	Axial forces	Moment	Shear	Horizontal		Vertical		Reinf. In Boundary	
Prob. No.	(kN)	(kNm)	(kN)	Reint	forcement	Reinfo	orcement	Element (B.E.) (mm ²)	
а	3944	2747	80	10 #	@ 200 c/c	20 # 0	@ 100 c/c	(4+4) nos (25+20) #	
b	4640	4790	139	10 #	@ 200 c/c	16# 0	@ 150 c/c	(4+4) nos (20+16) #	
с	5301	7231	209	10 #	@ 200 c/c	12 # 0	@ 125 c/c	8 nos 16 #	
d	5906	10053	291	10 #	@ 200 c/c	10 # 0	@ 125 c/c	8 nos 16 #	
e	6521	13215	382	10 #	@ 200 c/c	10 # 0	@ 150 c/c	8 nos 16 #	

TABLE 7.4 FORCES A	AND REINFORCEMENT	SCHEDULE FOR	UNCOUPLED WALL

7.6 FOUNDATION OF COUPLED SHEAR WALL

All engineered construction resting on the earth must be carried by some kind of interfacing element called as a foundation. The loads from superstructure are transferred to the soil through foundation or substructure. Here the foundation of coupled shear wall is studied for 15-storey CSW. The parameters are considered as wall length, span and depth of coupling beam. The foundation is proportioned and designed for the support reaction obtained from analysis results. Combined footing is considered as the foundation system of CSW. The proportion and design approach followed for the combined footing is illustrated by a typical example. Then effect on responses of structure with the variation of safe bearing capacity of soil is also studied by modeling the soil as spring.

7.6.1 Typical example: Design of Foundation for 15-storey CSW

The following data are considered for the design of this example: Wall length = 3.0 mThickness of wall = 0.3 mSpan of coupling beam = 1.0 mDepth of coupling beam = 0.5 mGrade of Concrete = M25 Safe bearing capacity of soil = 250 kN/m^2 Support Reactions of CSW:

	Wor	king loads	Ultimate Loads		
Wall	Axial Moment		Axial	Moment	
No	(kN) (kNm)		(kN)	(kNm)	
1	3298	1022	4947	1533	
2	6885	1384	10327.5	2076	
Total	10183		15274.5		

TABLE 7.5 SUPPORT REACTION OF CSW

Proportion of footing:

The basic assumption is that, combined footing is a rigid member, so that the soil pressure is linear. The pressure will be uniform if the location of the load resultant (including column moments) coincides with the centre of area. Therefore,

$$\sum M_{wall1} = R \times \bar{x}$$
(8.5)

Where
$$R = 3298 + 6885 = 10183 \text{ kN}$$

And $\sum M_{wall1}$ = Moment @ wall 1

 $= 1022 + 1384 + 4 \times 6885$

$$= 10183 x$$

$$\Rightarrow \overline{x} = 2.94 \text{ m}$$

= Distance between center line of wall 1 to center of pressure of footing.

Assumed that distance between edge of footing to central line of wall 1 = 2.0 m Length of footing (L) = $2 \times (2 + 2.94) = 9.9$ m

Provided length of footing = 10.0 m

Assume weight of footing and earth above is as 5% of total weight.

Total load on the earth =
$$1.05 \times (3298 + 6885) = 10692.15$$
 kN

Width of footing = $\frac{10692.15}{10 \times 250}$ = 4.3 m

Calculation for depth of footing:

Net upward pressure per unit length =
$$\frac{(4947 + 10327.5)}{10} = 1527$$
 kN/m

(a) Bending moment criteria

Larger projection in longitudinal direction = 2.5 m

B.M. in longitudinal direction = 4773 kNm

Larger projection in transverse direction = 2 m

B.M. in transverse direction = 3055 kNm

B.M. at intermediate critical section:

Maximum shear force at central line of wall 1 (FIGURE 7.2),

 $V_1 = -1527 \times 2.0 = -3055 \text{ kN}; V_2 = 4947 - 3055 = 1892 \text{ kN}$

Maximum shear force at central line of wall 2 (FIGURE 7.2),

 $V_3 = -1527 \times 4.0 = -6110 \text{ kN}; V_4 = 10327.5 - 6110 = 4218 \text{ kN}$ Point of zero shear from central line of wall1 is, X = 1.24 mMaximum B.M. (at x = 1.24 m) = 1883 kNm Design bending moment = 4773 kNm Depth of footing from B.M. criteria = 567 mm(b) Two way shear criteria: $\tau_{_c}=\tau_{_c}'\times k_{_s}$, Where $\tau_{_c}'=0.25\sqrt{f_{_{ck}}}~$ = 1.25 N/mm² and k_s = 0.6 Permissible two way shear, $\,\tau_{_c}$ = 0.75 $\,\text{N/mm}^2$ Shear force at d/2 = 5583 kN For depth 0.57 m, Shear stress, $\tau_v = 1.1 \text{ N/mm}^2 > 0.75 \text{ N/mm}^2$ (NOT OK) Depth of footing is revised to 0.695 m, Shear stress, $\tau_v = 0.72 \text{ N/mm}^2 < 0.75 \text{ N/mm}^2$ (OK) Provided overall depth of footing is 0.8 m The Dimension of footing is (FIGURE 7.3) Length = 10m, Width = 4.3 m and Depth = 0.8 m**Reinforcement calculation:** (a) Longitudinal reinforcement Design moment in Longitudinal direction = 4773 kNm Required area of reinforcement, $A_{st} = 20034 \text{ mm}^2$ Minimum area of reinforcement, $A_{st} = 3870 \text{ mm}^2$ (b) Transverse reinforcement Design moment in Transverse direction = 3055 kNm Required area of reinforcement, $A_{st} = 12221 \text{ mm}^2$ Minimum area of reinforcement, $A_{st} = 3870 \text{ mm}^2$ Provide longitudinal reinforcement 20 t @ 100 c/c (FIGURE 7.3) Minimum steel is provided at the top of the footing, 16 🕁 @ 225 c/c One way shear check: Design shear force at d = 2673 kN

Shear stress $\tau_{\rm w} = 0.8 \text{ N/mm}^2$

Permissible shear stress from IS 456 : 2000, $\tau_c = 0.54 \text{ N/mm}^2$ (For percentage of tension steel = 0.65 %) Therefore, shear design is required.







SECTION X-X



FIGURE 7.3 FOUNDATION DETAILS FOR PROBLEM No. 11

7.6.2 Comparative study on Foundation of 15-storey CSW

The parameters considered for the study are wall length, span and depth of coupling beam. All these cases are designed for the DL+EQ load combination similar to example illustrated in 7.6.1 and the SBC is considered as 25 T/m². An EXCEL work sheet is prepared for the design of foundation. In TABLE 7.6 (SHEET No. 2), the support reaction, proportioning of footing, forces on foundation and reinforcement details are given. When beam depth as parameter, the differences in the sizes of footing is very less. But the foundation depth from B.M. criteria is increasing. The moment and shear in the foundation also increasing.

When beam span as parameter, the foundation depth from B.M. as well as shear criteria increases. The B.M. with in the foundation increases with increases in span. The shear in the foundation increases from span 1m to 2m and then decreases from 2.0m to 3.0m span. When wall length as parameter, the foundation depth from B.M. criteria increases and two way shear criteria decreases. The moment and shear in foundation is increasing significantly.

7.6.3 Effect of Soil Conditions on Coupled Shear Wall

The responses in 15-storey CSW and its foundation are studied for two soil conditions. For the two soil conditions the allowable bearing capacities are considered as, 15 T/m^2 and 25 T/m^2 . The soil is modeled as spring and modulus of subgrade reaction is the spring constant. The wall length, coupling beam span and depth are considered as 3.0, 1.0 and 0.5m respectively.

7.6.3.1 Modulus of Subgrade Reaction

The modulus of subgrade reaction (or subgrade modulus or subgrade reaction) is defined as,

$$k_{s} = \frac{\sigma}{\Delta\delta}$$
(8.6)

Where σ is the load intensity applied on the soil and δ is the settlement caused by this applied load. Joseph E. Bowles [45] has suggested the following expression for approximating k_s from allowable bearing capacity, q_a which is furnished by the geotechnical consultant.

$$k_s = 40 \times FS \times q_a$$
 (Unit- kN/m³) (8.7)

Chapter 7 Seismic Design of Coupled Shear Wall and Study of Its Foundation

SHEET No. -2 (TABLE 7.6)

Where q_a is in kN/m² and equation is based on that, the ultimate pressure is at differential settlement of 0.0254 m. The equation 8.7 can be explained for the factor of safety (FS) 3.0 as follows,

$$k_{s} = \frac{q_{ult}}{\Delta \delta} = \frac{FS \times \frac{q_{ult}}{FS}}{\Delta \delta} = \frac{FS \times q_{a}}{\Delta \delta} = \frac{FS \times q_{a}}{0.0254} = 40 \times FS \times q_{a} = 40 \times 3 \times q_{a} = 120 \times q_{a}$$

For allowable bearing capacity, $q_a = 15 \text{ T/m}^2 = 150 \text{ kN/m}^2$

 $k_s = 120 \times 150 = 18000 \text{ kN/m}^3$

Similarly for $q_a = 25 \text{ T/m}^2 = 250 \text{ kN/m}^2$ $k_s = 120 \times 250 = 30000 \text{ kN/m}^3$

7.6.3.2 Modeling of foundation

The 15-storey CSW is modeled in ETABS with foundation. The foundation is proportioned in rigid method for different SBC conditions.

The dimension of foundation is obtained as,

For SBC 15 T/m², Length = 12 m, Width = 6.0 m and Depth = 1.0 m For SBC 25 T/m², Length = 10 m, Width = 4.3 m and Depth = 0.8 m

The shell element is used for the foundation as well as CSW for modeling (FIGURE 7.4). The meshing size is 250×250 mm for wall and foundation. The springs are modeled below the foundation having subgrade modulus of 18000 kN/m³ for SBC 15 T/ m² and 30000 kN/m³ for SBC 15 T/ m².



The support conditions applied are, F_x , F_y and M_z restrained and M_x and M_y are free. The spring is applied in F_z direction.

7.6.3.3 Responses of CSW Foundation

The shear force along the longitudinal direction is calculated in conventional method and shear force obtained from ETABS results are shown in FIGURE 7.5 and 7.6. The FIGURE 7.5 shows the shear force diagram of the CSW foundation for SBC of 15 T/m². And shear force diagram of CSW foundation for 25 T/m² is shown in FIGURE 7.5.



In conventional method the shear forces are higher than the shear forces obtained from ETABS results. The settlement in the footing is also observed. The maximum settlement is 10.5 mm and differential settlement is 5.8 mm of

foundation for the SBC of 15 T/m^2 and deformed shape of the footing is shown in FIGURE 7.7 (a).



The maximum settlement is 9.2 mm and differential settlement is 3.5 mm of foundation for the SBC of 25 T/m² and deformed shape of the footing is shown in FIGURE 7.7 (b).



7.6.3.3 Responses of CSW

For the SBC 15 and 25 T/m², axial forces in Wall decreases with compared to CSW of fixed base (FIGURE 7.8 (a)). The moment in the wall significantly increases for SBC 15 and 25 T/m² as compared to fixed base. The moment in wall increases with decreases in SBC of soil (FIGURE 7.8 (b)). Similar to the moment, the shear in the wall also increases with decreases in SBC of soil (FIGURE 7.8 (c)). As it is observed from FIGURE 7.8 (a) that, the axial force decreases in SBC of soil, which means that the shear in coupling beam decreases (FIGURE 7.8 (d)). Due to decrease in shear of coupling beam, the axial forces in the wall decrease. The reason is that the axial force in wall also generated by coupling action.



FIGURE 7.8 RESPONSES OF CSW AND COUPLING BEAM (LOAD COMB DL+EQ)

The time period and modal mass participation of the CSW increase with incorporating more flexibility in the supporting system.

	Fixed Base	SBC 25 T/m ²	SBC 15 T/m^2
Time period of CSW			
(Second)	1.356	2.21	2.056
Modal mass participation			
(MODE 1) (%)	65.70	73.20	72.55

 TABLE 7.7 TIME PERIOD COMPARISON OF CSW

7.7 CLOSING REMARKS

The effects of various parameters on structural behaviour, seismic response and the structural designing aspects of coupled shear wall are discussed. The detailed information in IS13920-1993 to design the coupled shear wall is inadequate. In this paper the designing of coupled shear wall is illustrated.

Coupled shear walls are suitable for earthquake resisting structural systems. The ductile behaviour of CSW is much more significant than shear wall. The considerable amount of force concentration is reducing in CSW due to redistribution of forces as compared to shear wall. To look into the considerable difference in forces in between CSW and SW, it can be concluded that CSW is much more economical and earthquake resistant behaviour than the SW.

The design for the foundation of 15-storey CSW is carried out considering wall length, span and depth of coupling beam as parameters. The bending moments and shear forces with in the footing is studied for different parameters. Then the effect of soil conditions on the responses of CSW and its foundation also studied. The response of CSW and its foundation, for 15 and 25 T/m² allowable bearing capacity of soil is studied. It is observed form the analysis results that with decrease in allowable bearing capacity of soil the axial force in CSW decreases, moment in CSW increases and shear in coupling beam decreases. The shear forces obtained from ETABS results and calculated from conventional method is compared.

8.1 INTRODUCTION

Depending on different degree of coupling, coupled shear wall can behave in shear mode, flexure mode or flexural shear mode. For this reason the coupled shear wall may be considered as the generic high-rise structural system and the coupled wall theory may be considered as a generalized theory for shear-flexural cantilevers. When this generalized theory is implemented in real life projects, then its importance makes more sense. Therefore after studying the linear static, linear dynamic and nonlinear behavior of coupled shear wall considering different parameters, then the implementation of CSW concept in real life project is studied.

The live project is 51 storey (stilt + 50 floors) residential complex at simplex mills compound Mumbai. The owner of this project is GODREJ GREEN. The structural consultant for this project is STERLING ENGINEERING CONSULTANCY SERVICES PVT. LTD. The architects of the project are DP ARCHITECTS PTE LTD at Singapore and J.P. PAREKH & SON at Mumbai. In the construction of the project, the shuttering used for RCC work is a special type. The shuttering is known as MIVAN and made up of aluminum material. Concrete mix plant is set up at the site for the construction by GODREJ. The grade of concrete is used M30, M35 and M40 and the fly ash is used in this design mix of concrete.

8.2 PRIMARY DATA FOR ANALYSIS

The typical floor plan of the 51 storey building is shown in FIGURE 8.1 (SHEET No. 2). The important data required for the analysis are given below:

Storey heights –	3.0m (stilt), 4.2m (first floor) and 3.2m (other floors)
Live load –	$2.0 \ k\text{N/m}^2$ (For staircase and passage LL is $3.0 \ k\text{N/m}^2$)
Sunk in toilet slab –	75 mm
External walls –	230 mm thick
Internal walls –	115 mm thick
Seismic Zone –	III (As per IS 1893 (Part 1) : 2002)

7.

Chapter 8 Implementation of Coupled Shear Wall Concept

SHEET No. 3

Basic wind speed -44 m/sec (As per IS 875 (Part 3) : 1987)Foundation system -Pile foundationMinimum grade of concrete -M30

8.3 MODELING

The building is modeled in ETABS. The walls, slab and beams are modeled by using shell element, membrane element and line elements respectively. The plan and three dimensional view of the building model are shown in FIGURE 8.2 (a) and (b).



FIGURE 8.2 (a) PLAN OF 51-STOREY BUILDING MODEL IN ETABS, (b) 3-D VIEW OF BUILDING MODEL IN ETABS

8.4 LOAD CALCULATION

The dead load, live load earthquake load and wind load calculation for this building is given in this section.

8.4.1 Dead load

Floor finish (75 mm Thk.) = $0.075 \times 20 = 1.5 \text{ kN/m}^2$ Toilet sunk (75 mm) = $0.075 \times 20 = 1.5 \text{ kN/m}^2$ External wall (230 mm Thk.) = $0.23 \times 20 \times (3.2 - 0.8) = 11.96 \approx 12 \text{ kN/m}$ Internal wall (115 mm Thk.) = $0.115 \times 20 \times (3.2 - 0.8) = 5.98 \approx 6$ kN/m Stair case Riser (200 mm) = $(0.2/2) \times 25 = 2.5$ kN/m² Waist slab (175 mm Thk.) = $0.175 \times 25 = 4.375$ kN/m² Total = 6.875 kN/m²

Projected load in inclined slab = 8.0 kN/m^2



DL on Mid landing or floor beam due to staircase = 17.4 kN/m

8.4.2 Live load

Floor area = 2.0 kN/m^2 Staircase = 3.0 kN/m^2 LL on Mid landing or floor beam due to staircase = 9.2 kN/m

8.4.3 Seismic load

Z = 0.16, I = 1 and R = 3.0 Total seismic weight = 273143.4 kN (25 % of LL is considered) Time period of CSW building from IS 1893:2002,

 Z-Direction :
 Z-Direction :

 d = 19.2 m
 d = 27.8 m

 $T_a = \frac{0.09h}{\sqrt{d}} = 3.368 \sec$ $T_a = \frac{0.09h}{\sqrt{d}} = 2.8 \sec$
 $\frac{S_a}{g} = 0.3$ $\frac{S_a}{g} = 0.36$
 $A_h = \frac{ZIS_a}{2Rg} = 0.008$ $A_h = \frac{ZIS_a}{2Rg} = 0.010$

 Base Shear = 2185 kN
 Base Shear = 2622 kN

Level Wi×Hi² Height (m) Hi Wi (kN) Qiz (kN) Qix (kN) 3 3 4.87E+04 1 5408.8 0.0 0.1 4.2 2 7.2 5408.8 0.2 0.3 2.80E+05 3 3.2 10.4 5408.8 5.85E+05 0.5 0.6 4 0.9 1.1 3.2 13.6 5408.8 1.00E+06 5 3.2 16.8 5408.8 1.53E+06 1.3 1.6 6 1.9 2.3 3.2 20 5408.8 2.16E+06 7 3.2 23.2 5408.8 2.6 3.1 2.91E+06 8 3.2 3.3 4.0 26.4 5408.8 3.77E+06 9 5.0 3.2 29.6 5408.8 4.74E+06 4.2 10 3.2 32.8 5408.8 5.82E+06 5.1 6.1 3.2 36 5408.8 7.01E+06 6.2 7.4 11 3.2 39.2 7.3 8.8 12 5408.8 8.31E+06 13 3.2 42.4 8.5 10.3 5408.8 9.72E+06 14 3.2 45.6 5408.8 1.12E+07 9.9 11.9 15 3.2 48.8 5408.8 1.29E+07 11.3 13.6 16 3.2 52 5408.8 1.46E+07 12.9 15.4 55.2 17 3.2 5408.8 1.65E+07 14.5 17.4 5408.8 16.2 19.5 18 3.2 58.4 1.84E+07 19 3.2 21.7 61.6 5408.8 2.05E+07 18.0 20 3.2 64.8 5408.8 2.27E+07 20.0 24.0 21 22.0 26.4 3.2 68 5408.8 2.50E+07 22 3.2 71.2 5408.8 2.74E+07 24.1 28.9 23 3.2 74.4 5408.8 2.99E+07 26.3 31.6 24 77.6 5408.8 34.4 3.2 3.26E+07 28.6 25 3.2 80.8 5408.8 3.53E+07 31.0 37.3 40.3 26 3.2 84 5408.8 3.82E+07 33.6 27 3.2 87.2 5408.8 36.2 43.4 4.11E+07 28 90.4 46.6 3.2 5408.8 4.42E+07 38.9 29 93.6 5408.8 41.7 50.0 3.2 4.74E+07 30 3.2 96.8 5408.8 5.07E+07 44.6 53.5 31 3.2 100 5408.8 5.41E+07 47.5 57.1 32 103.2 50.6 60.8 3.2 5408.8 5.76E+07 33 3.2 106.4 5408.8 6.12E+07 53.8 64.6 34 3.2 57.1 68.5 109.6 5408.8 6.50E+07 35 3.2 112.8 5408.8 60.5 72.6 6.88E+07 36 3.2 116 5408.8 7.28E+07 64.0 76.8 37 81.1 3.2 119.2 5408.8 7.69E+07 67.6 38 3.2 122.4 5408.8 8.10E+07 71.2 85.5 39 90.0 3.2 125.6 5408.8 8.53E+07 75.0 40 3.2 128.8 5408.8 8.97E+07 78.9 94.7 41 3.2 82.9 99.4 132 5408.8 9.42E+07 42 3.2 135.2 5408.8 9.89E+07 86.9 104.3 43 91.1 109.3 3.2 138.4 5408.8 1.04E+08 44 3.2 141.6 5408.8 1.08E+08 95.3 114.4 45 3.2 144.8 5408.8 1.13E+08 99.7 119.6 5408.8 46 3.2 148 104.2 125.0 1.18E+08 47 3.2 151.2 5408.8 1.24E+08 108.7 130.4 48 3.2 154.4 5408.8 1.29E+08 113.4 136.0 49 3.2 5408.8 1.34E+08 157.6 118.1 141.7 50 3.2 160.8 5408.8 1.40E+08 122.9 147.5 164 2704.4 51 3.2 7.27E+07 63.9 76.7

TABLE 8.1 LATERAL LOAD DISTRIBUTION (EARTHQUAKE)

2.49E+09

2185.1

2622.2

273144

Total

8.4.4	Wind	load TA	ABLE 8.2	LAT	ERAL LO	OAD D	ISTRIB	UTION	(WINE))	
										Load in X	Load in Z
Level	Height	Vb	K1	K2	K3	Vz	Pz	Area X	Area Y	dir (kN)	dir (kN)
2	7.2	44	1	0.82	1	36.08	781.06	67.68	100.8	63	94
3	10.4	44	1	0.82	1	36.08	781.06	60.16	89.6	56	84
4	13.6	44	1	0.87	1	38.28	879.22	60.16	89.6	63	95
5	16.8	44	1	0.87	1	38.28	879.22	60.16	89.6	63	95
6	20	44	1	0.91	1	40.04	961.92	60.16	89.6	69	103
7	23.2	44	1	0.96	1	42.24	1070.53	60.16	89.6	77	115
8	26.4	44	1	0.96	1	42.24	1070.53	60.16	89.6	77	115
9	29.6	44	1	0.96	1	42.24	1070.53	60.16	89.6	77	115
10	32.8	44	1	1.02	1	44.88	1208.53	60.16	89.6	87	130
11	36	44	1	1.02	1	44.88	1208.53	60.16	89.6	87	130
12	39.2	44	1	1.02	1	44.88	1208.53	60.16	89.6	87	130
13	42.4	44	1	1.02	1	44.88	1208.53	60.16	89.6	87	130
14	45.6	44	1	1.02	1	44.88	1208.53	60.16	89.6	87	130
15	48.8	44	1	1.02	1	44 88	1208.53	60.16	89.6	87 87	130
16	52	44	1	1.02	1	44 88	1208.53	60.16	89.6	87	130
17	55.2	44	1	1.02	1	48.4	1405 54	60.16	89.6	101	150
18	58.4	 11	1	1.1	1	-10 18 1	1405.54	60.16	89.6	101	151
10	61.6	44	1	1.1	1	40.4	1405.54	60.16	89.0	101	151
20	64.8	44	1	1.1	1	40.4	1405.54	60.16	89.6	101	151
20	68	44	1	1.1	1	40.4	1405.54	60.16	89.0	101	151
21	71.2	44	1	1.1	1	40.4	1405.54	60.16	89.0 80.6	101	151
22	71.2	44	1	1.1	1	40.4	1405.54	60.16	89.0 80.6	101	151
25	74.4	44	1	1.1	1	48.4	1405.54	00.10	89.0 80.6	101	151
24	//.0	44	1	1.1	1	48.4	1405.54	00.10	89.0	101	151
25	80.8	44	1	1.1	1	48.4	1405.54	60.16	89.6	101	151
26	84	44	1	1.1	1	48.4	1405.54	60.16	89.6	101	151
27	87.2	44	1	1.1	1	48.4	1405.54	60.16	89.6	101	151
28	90.4	44	1	1.1	l	48.4	1405.54	60.16	89.6	101	151
29	93.6	44	l	1.1	l	48.4	1405.54	60.16	89.6	101	151
30	96.8	44	1	1.1	1	48.4	1405.54	60.16	89.6	101	151
31	100	44	1	1.1	1	48.4	1405.54	60.16	89.6	101	151
32	103.2	44	1	1.15	1	50.6	1536.22	60.16	89.6	111	165
33	106.4	44	1	1.15	1	50.6	1536.22	60.16	89.6	111	165
34	109.6	44	1	1.15	1	50.6	1536.22	60.16	89.6	111	165
35	112.8	44	1	1.15	1	50.6	1536.22	60.16	89.6	111	165
36	116	44	1	1.15	1	50.6	1536.22	60.16	89.6	111	165
37	119.2	44	1	1.15	1	50.6	1536.22	60.16	89.6	111	165
38	122.4	44	1	1.15	1	50.6	1536.22	60.16	89.6	111	165
39	125.6	44	1	1.15	1	50.6	1536.22	60.16	89.6	111	165
40	128.8	44	1	1.15	1	50.6	1536.22	60.16	89.6	111	165
41	132	44	1	1.15	1	50.6	1536.22	60.16	89.6	111	165
42	135.2	44	1	1.15	1	50.6	1536.22	60.16	89.6	111	165
43	138.4	44	1	1.15	1	50.6	1536.22	60.16	89.6	111	165
44	141.6	44	1	1.15	1	50.6	1536.22	60.16	89.6	111	165
45	144.8	44	1	1.15	1	50.6	1536.22	60.16	89.6	111	165
46	148	44	1	1.15	1	50.6	1536.22	60.16	89.6	111	165
47	151.2	44	1	1.15	1	50.6	1536.22	60.16	89.6	111	165
48	154.4	44	1	1.18	1	51.92	1617.41	60.16	89.6	117	174
49	157.6	44	1	1.18	1	51.92	1617.41	60.16	89.6	117	174
50	160.8	44	1	1.18	1	51.92	1617.41	60.16	89.6	117	174
51	164	44	1	1.18	1	51.92	1617.41	60.16	89.6	117	174
								Total		4922.31	7331.09

8.4.5 Load Combination

The load combinations for limit state of collapse and serviceability are given in TABLE 8.3 and 8.4. The earthquake as well as wind load is considered for the different load combinations.

Load Combination	Load Factors						
	D.L	L.L.	Wind	Earthquake			
COMB1	1.5	1.5	-	-			
COMB2	1.2	1.2	1.2 (x)	-			
COMB3	1.2	1.2	-1.2 (x)	-			
COMB4	1.2	1.2	1.2 (z)				
COMB5	1.2	1.2	-1.2 (z)				
COMB6	1.2	1.2	-	1.2 (x)			
COMB7	1.2	1.2	-	-1.2 (x)			
COMB8	1.2	1.2	-	1.2 (z)			
COMB9	1.2	1.2	-	-1.2 (z)			
COMB10	1.5	-		1.5 (x)			
COMB11	1.5	-		15 (x)			
COMB12	1.5	-		1.5 (z)			
COMB13	1.5	-		1.5 (z)			

TABLE 8.3 LOAD COMBINATIONS FOR LIMIT STATE OF COLLAPSE

TABLE 8.4 LOAD COMBINATIONS FOR LIMIT STATE OF SERVICEABILITY

Load	Load Factors							
Combination	D.L	L.L.	Wind	Earthquake				
COMB1	1	1	-	-				
COMB2	1	0.8	0.8 (x)	-				
COMB3	1	0.8	-0.8 (x)	-				
COMB4	1	0.8	0.8 (z)					
COMB5	1	0.8	-0.8 (z)					
COMB6	1	0.8	-	0.8 (x)				
COMB7	1	0.8	-	-0.8 (x)				
COMB8	1	0.8	-	0.8 (z)				
COMB9	1	0.8	-	-0.8 (z)				
COMB10	1	-		1 (x)				
COMB11	1	-		-1 (x)				
COMB12	1	-		1 (z)				
COMB13	1	-		-1 (z)				

8.5 RESULTS AND DISCUSSION

After performing the analysis of building, the obtained results are discussed in this section. The analysis is carried out by using ETABS.

8.5.1 Time period of 51-storey building

The formula proposed by Wallace and Moehle (1992) for the fundamental periods of structural wall buildings is (Ref. chapter 5),

$$T_{\text{wallace}} = 6.2 \frac{h_{\text{w}}}{(2D_{\text{w}} + L_{\text{b}})} n \sqrt{\frac{wh_{\text{s}}}{gE_{\text{c}}p}}$$

The datas for the time period calculation are, $h_w = 164 \text{ m}, h_s = 3.2 \text{ m},$ $E_c = 27386.13 \text{ N/mm}^2$ (M30 grade concrete is considered) $g = 9.81 \text{ m/sec}^2$ $2D_w + L_b = 19.2 \text{ m}$ n = 51Total weight = 266346 kN Wall area = 27.422 m², Plan Area = 344.2 m² $w = 15.2 \text{ kN/mm}^2, p = 0.08$ Time period from Wallace and Moehle's formula (T) = 4.068 sec Time period obtained from ETABS results = 4.044 sec

TABLE 8.5 COMPARISION ON TIME PERIOD OF 51-STOREY BUILDING

Wallace and Moehle	ETABS results	IS 1893 : 2002
4.068 sec	4.044 sec	3.368 sec

The time period of 51-storey building obtained from Wallace and Moehle and ETABS results are very much closer to each other (TABLE 8.5).

8.5.2 Base shear distribution in walls of 51-storey building

The base shear distribution in walls of 51-storey building is shown in FIGURE 8.4. Here one CSW consists of coupled wall (CW1 , CW 2, CW 3 and CW 4) and coupling beam (CB 1, CB 2 and CB 3) (FIGURE 8.1, SHEET 1). This CSW is contributing base shear in x-direction 5% and z-direction 31% (FIGURE 8.4). The other one consists of coupled wall (CW 5 , CW 6, CW 7 and CW 8) and coupling beam (CB 4, CB 5 and CB 6) (FIGURE 8.1, SHEET 1). This CSW is contributing base shear in x-direction 5% and z-direction 33% (FIGURE 8.4).

Chapter 8 Implementation of Coupled Shear Wall Concept

Figure 8.4 base shear distribution in wall

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8.5.3 Interstorey drift in 51-storey building

The interstorey drift index is defined as, the ratio between maximum difference of lateral deflection at top and bottom storey and the storey height [1]. The inter storey drift index, gives a measure of possible localized excessive deformation. The inter storey drift ratio for all floors are shown in FIGURE 8.5.



FIGURE 8.5 MAXIMUM INTERSTOREY DRIFT INDEX IN 51-STOREY BUILDING

The maximum interstorey drift along Z-direction is 0.0032 for the load combination DL+0.8LL+0.8WL_z (TABLE 8.3) at 51-storey level. The maximum inter storey drift along X-direction is 0.0011 for the load combination DL+0.8LL+0.8WL_x at 23-storey level. The interstorey drift index is maximum along Z-direction in compared to X-direction. Along x-direction, the DL+EQ_x load combination is critical from 43-storey to 51-storey. In all other cases the wind loads are critical. The Maximum permissible Interstorey drift index as per IS 1893-2002 (Clause 7.11.1) is 0.004 [39]. Here the maximum inerstorey of the building is with in permissible limit.

8.5.4 Lateral Deflection in 51-storey building

The tall structure is subjected to lateral deflections under lateral loads results the oscillatory movements. Motions that have psychological or physiological effects on the occupants, as a result an acceptable structure may become undesirable or even unrentable building. In structural aspects point of view, there is a possibility of coupling between lateral oscillations and bending of the structure. The coupling will produce unacceptable complex motions and accelerations [1]. Therefore to establish a limit for the lateral deflection of the structure is a major design decision. The lateral deflection can also be expressed as drift index. The <u>drift index is defined as the ratio of maximum deflection at the top of the building to the total height</u>. The Coull and Smith have given certain guidelines for the roof deflection limit, which is H/650 to H/350 [1]. In the practice structural consultants kept the permissible deflection limit for earthquake governing case is H/250 and for Wind governing case is H/500.

In the present study, the deflection of building in X and Z-direction at each floor level is shown in FIGURE 8.6. The maximum deflection in Z-direction is 0.3508m for load combination $DL+0.8LL-0.8WL_z$. The maximum deflection in X-direction is 0.1305m. Considering permissible limit for roof deflection H/500,

$$\frac{\mathrm{H}}{500} = \frac{164}{500} = 0.328 < 0.3508$$

The observed deflection is slightly more than the permissible in the present study of 51-storey building.



FIGURE 8.6 DEFLECTION AT FLOOR LEVEL IN 51-STOREY BUILDING

8.5.5 Responses of coupled wall in 51-storey building

The CW1, CW2, CW3 CW8 are the coupled walls in the 51-storey building. The CW1, CW2, CW3 and CW4 are combined to form one CSW and other one is the combination of CW5, CW6, CW7 and CW8. It is observed that the second one is contributing more shear in compared to first one. Therefore here the responses of CW5, CW6, CW7 and CW8 are discussed. The axial forces in the walls are shown in FIGURE 8.7(a). The axial forces in the CW5 and CW6 are in compression. The wall CW7 and CW8 are in tension.



FIGURE 8.7 RESPONSES IN COUPLED WALLS IN 51-STOREY BUILDING

The shear in the coupled walls is shown in FIGURE 8.7(b). The shear in the walls is critical at the lower intermediate storey. The moment in coupled walls are shown in FIGURE 8.7(c). The pattern of curve observed in the study of wall integrated with 51-storey building is showing similar kind of behavior, which are observed in the parametric study.

8.5.6 Responses of coupling beam in 51-storey building

The responses in the coupling beam are studied in the 51-storey building. The one CSW consists of coupling beam of CB1, CB2 and CB3. The other CSW consists of CB4, CB5 and CB6. The responses in coupling beams CB4, CB5 and CB6 are shown in FIGURE 8.8.

The moment and shear in coupling beams are shown in FIGURE 8.8(a) and (b). The moment and shear in CB4 are maximum of 358 kNm and 589 kN at the 9 and 8 storey of the building respectively. In the coupling beam CB6, the maximum moment and shear are 208 kNm and 359 kN at 3 and 4 storey respectively. The nature of curve in the coupling beam, which is an integrated part of the 51-storey building, is similar in nature with the parametric study cases.



FIGURE 8.8 RESPONSES IN COUPING BEAMS IN 51-STOREY BUILDING

8.6 CLOSING REMARKS

The responses of walls and coupling beams in the 51-storey building are studied. The behavior of coupled shear wall integrated with the 51-storey building is showing similar in nature with the behavior studied in the parametric study. The time period obtained from the ETABS analysis results and Wallace and Moehle's formula are very close to each other. The interstorey drift of the building is with in permissible limit. The roof deflection of the building is slightly exceeding the limit of H/500 criterion.

9.1 SUMMARY

The stiffness, strength, ductility and dissipation of energy of entire structure are focused at base of structural wall. In seismic event, the limited ductility of structural wall wasn't permitted to control its behavior. Therefore the coupled shear wall concept is come up as remedial measure. In inelastic behavior of CSW, the forces are redistributed throughout the structure instead of concentrating at the base like structural wall. The coupling action of CSW is expressed in terms of degree of coupling. The degree of coupling depends on the geometry of structure. The geometry of CSW includes, wall length, wall height (or number of storey), span and depth of coupling beam. The degree of coupling is not only control the elastic behavior but also inelastic behavior of CSW. The numbers of plastic hinge formation, which is the source of dissipation of energy, are also depending on degree of coupling.

Therefore a parametric study is carried out to study the behavior of CSW and it's foundation for different geometry. The behavior of CSW includes the elastic static, elastic dynamic and inelastic static behavior. The parameters of the study are wall length, wall height (or number of storey), depth and span of coupling beams. The wall length is varying form 2.0 to 6.0m; number of storey is 5 to 30; depth of coupling beam is 0.5 to 1.5m and span of coupling beam is 1.0 to 3.0m. The responses are studied as axial force, moment and shear in wall, moment and shear in coupling beam and degree of coupling. Ninety numbers of cases are studied for elastic behavior of CSW.

The dynamic properties of CSW, time period and modal mass participation are studied. Time period obtained from the study of ninety cases are compared with O. Chaalla's formula, Wallace and Moehle's formula and codal provision given in IS 1893 : 2002. Failure pattern in different geometry of CSW and effect of degree of coupling upon the plastic hinge formation are studied in inelastic behavior. The 45 cases are studied for 10, 15 and 20 storey CSW to observed failure pattern.

Seismic design of 15 storied CSW and its foundation is carried out considering wall length, depth and span of coupling beam as parameter. The effects of soil condition on the responses of CSW and its foundation are studied considering allowable soil bearing capacity 15 and 25 T/m². Finally the implementation of CSW concept is important for the structural reality point of view. Therefore 51-storey building, which is a live project in Mumbai, is studied with incorporating the CSW concept. The responses in CSW, the base shear distribution in all walls and drift in the building are studied.

9.2 CONCLUSIONS

Based on the study carried out on coupled shear wall in different area following conclusions are drawn:

Use:

 a) CSW can be used as earthquake resistance structure in new construction as well as can be used for seismic rehabilitation of existing structure due to desirable seismic behavior.

Modeling:

b) Shell elements are more suitable than the line element for modeling of coupling beam. Because the line element under estimates the shear in coupling beam.

Elastic Static Behavior:

- c) The CSW concept is similar to the earthquake resistance philosophy, "strong column and weak beam".
- d) Axial force increases and bending moment decreases in the wall of CSW due to coupling action.
- e) Degree of coupling depends on geometry of the coupled shear wall.
- f) Higher depth of coupling beam at lower storey gives more degree of coupling and lower depth of coupling beam at higher storey gives more degree of coupling.

Elastic Dynamic Behavior:

g) Time period of CSW building from Wallace and Moehle formula and ETABS results are close agreement.
h) Time period provision given in IS 1893 : 2002 needs to incorporate the dimension of CSW elements for CSW building.

Inelastic Static Behavior:

- Good proportioning of CSW can give desired failure pattern, in which the flexural plastic hinges are formed at the end of coupling beam first and then at base of walls.
- j) More coupling action dissipates more energy by formation of plastic hinges.
 For the higher degree of coupling, the response reduction factor (R) for CSW building can be increased in compared to the current codal provision.

Seismic Design:

- k) The diagonal reinforcement should be provided in coupling beam for better shear and flexural strength.
- CSW concept gives more economical design in comparison to uncoupled structural wall.

Foundation:

m) With reduction in allowable bearing capacity of soil, the axial force decreases and moment increases in wall; shear in coupling beam and degree of coupling decreases.

9.3 FUTURE SCOPE OF WORKS

The present work can be extended to study the followings.

- $_{\odot}$ $\,$ Parametric study on core type coupled shear wall.
- Coupled shear wall Frame interaction in highrise structures.
- Computer program for the plastic hinge properties of coupled shear wall.
- Parametric study on ductile demand, ductile capacity and suitable response reduction factor for coupled shear wall building in Indian region.
- Parametric study on soil-structure interaction of coupled shear wall considering site specific response spectrum.
- Analytical study on dynamic and inelastic behavior of coupled shear wall can be verified by displacement controlled experimental work.

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APPENDIX – A

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APPENDIX – B LIST OF PAPERS PUBLISHED

- Kirti Sundar Dash & P.V. Patel, "Coupled Shear Wall-A earthquake resistant structural system", *Proceedings of National conference on Earthquake Disaster: Technology and Management (EARTH'06)*, MNNIT, Allahabad, 11-12 February, 2006, pp. III-10 – III-13.
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