SEISMIC PERFORMANCE OF COUPLED SHEAR WALL BUILDINGS WITH REPLACEABLE ENERGY DISSIPATION DEVICES

A DISSERTATION

Submitted in partial fulfilment of the

Requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

EARTHQUAKE ENGINEERING

(With Specialization in Structural Dynamics)

by

NARTU MANOJ KUMAR

DEPARTMENT OF EARTHQUAKE ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY ROORKEE
ROORKEE-247667, INDIA

MAY, 2016

ACKNOWLEDGEMENT

I wish to express my deep sense of gratitude and indebtedness to my learned and enlightened guide **Dr. Yogendra Singh**, Professor, Department of Earthquake Engineering, Indian Institute of Technology Roorkee, for his instinctive and careful guidance and continued encouragement in the completion of this dissertation work.

I am extremely grateful to my family and to my friends for their endless support and encouragement, and for always believing that I can succeed in my endeavours.

I am highly indebted to all other members of the Department for their continuous support, valuable suggestions and encouragement throughout this dissertation work. I thank all members, who helped directly or indirectly in bringing this work to this present form.

(Nartu Manoj Kumar)

CANDIDATE'S DECLARATION

I hereby declare the work which is being presented in this dissertation entitled, **"Seismic Performance of Coupled Shear Wall Buildings with Replaceable Energy Dissipation Devices",** in the partial fulfilment of the requirements for the award of the degree of **Master of Technology** in **Earthquake Engineering**, with specialization in **Structural Dynamics**, submitted in the Department of Earthquake Engineering**,** Indian Institute of Technology Roorkee is an authentic record of my own work carried during the period from May 2015 to May 2016 under the supervision of **Dr. Yogendra Singh,** Professor, Department of Earthquake Engineering, IIT Roorkee.

The matter embodied in this Dissertation has not been submitted by me for the award of any other degree or diploma of this Institute or any other University/Institute**.**

Place: Roorkee
Date: (**Nartu Manoj Kumar)**

CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

(Dr. Yogendra Singh)
Professor
Dept. of Earthquake Engineering
Indian Institute of Technology Roorkee
Roorkee-247667.

ABSTRACT

Performance of a structure in an earthquake is governed by its capacity to dissipate energy. In mid- to high-rise buildings, coupled shear walls act as efficient lateral load resisting and energy dissipation. Coupling beams, in coupled shear walls are designed to yield prior to yielding of walls. This provides higher energy dissipation capacity to coupled shear wall system as compared to uncoupled shear walls. Thus, the performance of coupled shear wall system is dependent on the energy dissipation capacity and ductility of the coupling beams. Various researchers have proposed different types and design of coupling beams, such as RC, steel, composite beams. Recently different types of fuses in coupling beams have been used and such systems are termed as Hybrid Coupled Wall systems. The use of steel as coupling beam provides high degree of ductility, acts as good energy dissipation system and has the added advantage of ease of construction compared to diagonally reinforced beam. Fuses in coupling beams are energy dissipation devices which have similar advantages as those of steel beam and can be replaced easily. Literature on different coupling beams consists with experimental evaluation of performance but little emphasis is there on use of beams in design to take advantage of higher dissipation capacities.

The scope of this work consists in numerical modeling and design of a typical 15 storey building with coupled shear wall system, using guidelines of ACI 318, Euro-code 8, IS code and other available literature on design of Hybrid Coupled Wall systems. Design recommendations for coupling beams are reviewed such that they yield well before the walls yield. Design considerations for ductile performance of walls in coupled system is also reviewed. Placement of shear walls of different sizes and various locations is carried out and associated difficulties are discussed. The performance of building with different types of coupling beams is evaluated using nonlinear static procedures as per ASCE 41. From this study, it is observed that the Steel coupling beams with fuses display the maximum ductility as compared to other systems. The diagonally reinforced coupling beam displayed higher overstrength capacity. Model with steel beams showed less ductility compared to other models due to formation of flexural hinges which have less ductility compared to shear hinges.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

Chapter 1
INTRODUCTION

1.1 General

Since the beginning of 19^{th} century, mid- to high-rise buildings are constructed to cater urban population demand. Now, they are norm of urban topography, and also signify prosperity and serve as important landmarks. With advancement of engineering and construction technologies, the profile of tall buildings continues to evolve in form as well as in height making tall buildings more vulnerable to lateral loads.

The design of mid to high-rise building usually governed by wind induced lateral vibration and loading, and basic approach is that building should remain elastic when subjected to wind loads. Design of buildings for seismic loads requires a basically different approach, as it is based on inelastic deformation capacity of the structure. Inelastic deformation behavior is due to material nonlinearity, which is considered in design by using force reduction factors that are dependent on material ductility. Major drawback of above approach is during any major seismic event, large amounts of energy is being imparted to the structure and the way in which the structure dissipates energy imparted determines level of damage, there is no control over the way in which the energy is dissipated. The emerging philosophies based on capacity design is to control the inelastic response. In capacity design approach, elements with lower strength and having large inelastic deformation capacities are designed, adjacent elements are designed to remain elastic. This way only the desired elements are yielded and provides stable means of energy dissipation to the structure.

For mid-to high-rise structures, there several lateral load resisting systems one such type is shear wall system. There are several variations of the system such as shear wall, coupled shear wall, shear wall–frame dual system. Shear wall is simply a structural wall designed to resist vertical loads and lateral loads. Coupled walls are series of walls connected by beams and if the walls used in conjunction with frame designed to resist lateral loads, such systems are called as dual systems. Even though all the systems has shear wall as its main component in resisting lateral load, the mode of resistance and performance under seismic loads is different.

1.2 Shear walls

The term shear wall is designated to the structural walls from the fact that the wall resists the shear force generated due to lateral loads. Shear walls provide effective lateral resistance to building due to their high in plane stiffness. They also provide ductility to the structure through formation of plastic hinges. Due to such features they are widely used for mid to high rise buildings. The location of shear walls in general depends upon design and functional requirement. To improve torsional resistance in the structure the walls may be located at the periphery of the building. Shear wall fails in flexure or shear depends upon dimensions of wall, for midrise building with wall dimension such that length of the wall is less than height of the wall; it behaves as cantilever beam in which flexure mode of failure is predominant. The slab at story level acts as fixity preventing wall segments against buckling. When length of wall is more than height of the wall, the wall has large moment carrying capacity compared to shear capacity of wall; hence shear mode of failure is possible. In shear wall only system the frames acts as gravity loads resisting system and total lateral loads are resisted by the shear wall.

1.3 Coupled shear walls

Openings in shear wall are functional requirement such as doors and windows. The openings in walls look like walls are connected to each other by beams in other words coupled together. Efficient lateral load resisting system with a good ductile response is achieved when openings in the shear walls are arranged in regular and rational pattern. The load resistance of the shear wall system is improved due to frame action caused by the coupling beams. Fig 1 shows two cantilever wall piers are connected in between by the coupling beams. Due to frame action of the system, compression and tensile forces are generated in the wall piers. The magnitude of the forces generated is equal to sum of shear forces developed in the coupling beams. The total overturning moment from the lateral loads (M_T) is resisted by both the walls (M_1 and M_2) and also due to coupling. The degree of frame action is expressed by term known as Coupling Ratio (CR) it is defined as the ratio of moment resisted due to frame action of coupling beams to the total overturning moment. If the value of CR is 0, it represents that no frame action exists and the wall behave as two isolated cantilever walls and if the value of CR is 1 it signifies that the walls act as single solid cantilever wall.

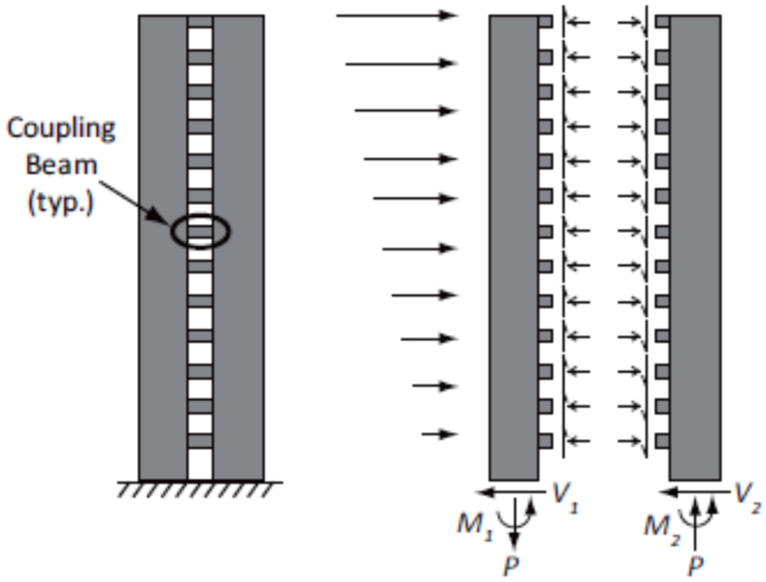

Figure 1.1 *Coupled shear wall*

1.4 Wall-frame dual system

Lateral force resistance is provided by the combined contribution of frames and structural walls, such systems are called as Dual systems. Under the influence of lateral loads in a Dual system, frame primarily deforms in a shear mode and structural walls in flexural deformation. At lower storey levels shear walls and the frame tend to share resistance due storey shear forces but they tend to oppose each other at higher storeys. Fig 1.2 shows the mode of resistance for different wall lengths of wall for 13 storey building (Priestley et al., 1992). Dynamic response characteristics and formation of Plastic hinges during a major seismic event influences the mode of sharing resistance and is quite different from the predicted elastic analysis. Common practice to design of such systems is to allocate a portion of lateral force to the frames and remainder to walls and each of which are then independently designed.

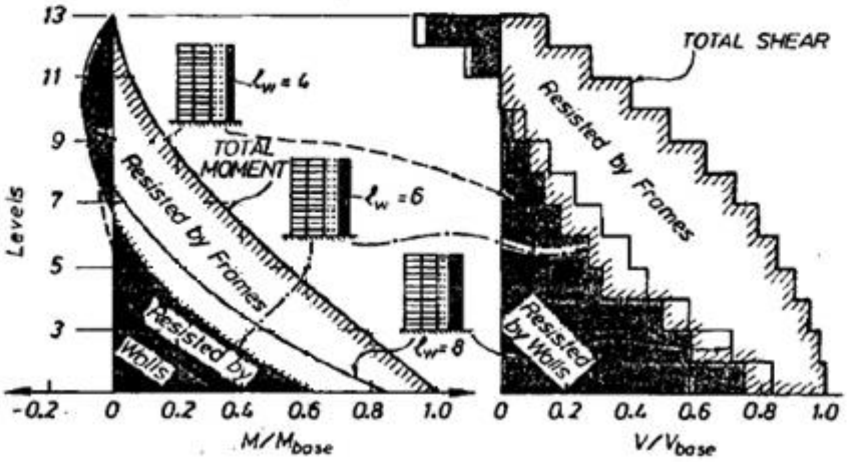

Figure 1.2 *Moment and shear resisted by frame and wall (Priestley* et *al., 1992)*

1.5 Seismic performance evaluation

Nonlinear static analysis also known as Pushover analysis is used to evaluate the inherent capacity of the structure for a seismic load. In this method lateral loads are applied in predefined pattern on numerical simulated model and are increased gradually to desired roof displacement. Due to increment of loads the simulated model starts yielding and formation of hinges takes place. The plot of roof displacement and Base shear is defined as capacity curve of the structure. This plot is used to evaluate the performance of the structure for various seismic hazards. This analysis provides an estimate of strength and ductility of the structure.

The ASCE 41 & ATC-40 documents have developed modelling and analysis procedures, acceptance criteria for nonlinear static analysis. These documents define force deformation properties of plastic hinges for beams, columns, walls etc. The force displacement behaviour is defined as A, B, C, D and E as shown in Fig 1.3 and acceptance criteria is designated as IO, LS and CP. The IO, LS & CP stand for Immediate occupancy, Life safety and Collapse prevention respectively.

The capacity spectrum which is obtained from capacity curve using following relations (1.1) and (1.2) where α_1 and PF_1 are the modal mass coefficients and participation factor for first mode of the structure, respectively. V_1 is the base shear, W_1 is weight of the structure.

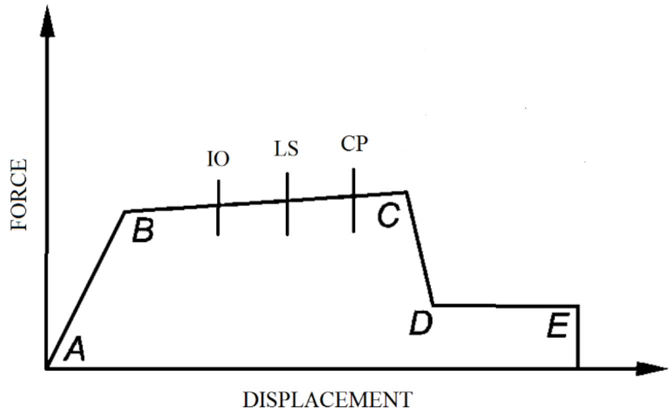

Figure 1.3 *Force-displacement curve defined for the plastic hinge in pushover analysis*

ϕ_{1roof} is roof level amplitude of the first mode. The earthquake demand represented in the response spectrum. Every point in the response spectrum has unique spectral acceleration, spectral displacement, spectral velocity and period. In Fig 1.4, plot between the spectral acceleration vs. spectral displacements of earthquake demand is called demand spectrum. The value of spectral displacement is obtained from Equation 1.3, where, T_i is initial time period of the structure.

$$S_{ai} = \frac{V_1}{W \alpha_1} \tag{1.1}$$

$$S_{di} = \frac{\Delta_{roof}}{\phi_{1,roof} PF_1} \tag{1.2}$$

Conversion of capacity spectrum and demand spectrum in a same coordinate system allows imposing both on a plot. The intersection of these curves gives the performance of the structure for given hazard level. At the intersection of these curves the total damping provided by the structure and demand damping should be same.

$$S_{di} = \frac{T_i^2}{4\pi^2} S_{ai} g \tag{1.3}$$

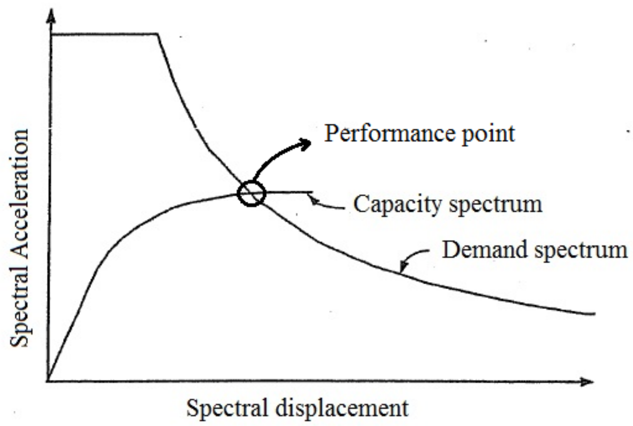

Figure 1.4 *Typical demand and capacity spectrum (ATC-40)*

1.6 Objectives

The aim of this dissertation work is to compare the seismic performance of a 15 storey representative coupled shear wall building with different types of coupling beams. The specific objectives are as following:

1. To study the behaviour of different types of coupling beams and their performance in past earthquakes and from experimental studies.

2 Review of available design procedures for RC coupled shear wall buildings and hybrid coupled wall buildings.

3. To develop a step-by-step procedure for design of coupled shear wall system using IS code.

4. To design a 15 storey building using the developed procedure, and to evaluate the seismic performance of the building using nonlinear static procedure as per ASCE 41.

1.7 Scope of present work

In this present study a 15 storey building having a coupled shear walls as lateral load resisting system is designed using the procedure laid down in subsequent chapters. The building is a designed as dual-frame system with the base shear resisted by the wall is kept above 70% and the CR around 60%. The building is designed with Diagonally reinforced concrete coupling beams (DRCCB), Steel coupling beam (SB) and Steel coupling beam with fuse (SB Fuse) and seismic performance of the building is evaluated using nonlinear static procedure.

1.8 Organization of Dissertation

This Dissertation is organized in the following chapters:

Chapter 1: This chapter is introductory chapter describes the behaviour of shear walls, coupled shear walls and frame-shear dual systems. This chapter also outlines the objectives identified and scope of present work.

Chapter 2: In this Chapter, types of coupling beams, comparison of provisions of ACI 318, IS 13920, Euro code 8 and NEHRP guidelines for design of coupled shear wall buildings. This chapter also discuss modelling of nonlinear hinges for coupling beams.

Chapter 3: This chapter discusses step-by-step procedure for proportioning and design of coupled shear wall buildings for code based performance objective.

Chapter 4: In this chapter presents the results of the numerical study carried out to investigate the seismic behaviour of coupled shear wall building with diagonally reinforced, steel and steel with a fuse coupling beams and its performance using nonlinear static procedure.

Chapter 5: Conclusions drawn from the parametric study, in line with the objectives identified for present study are presented.

Chapter 2

LITERATURE REVIEW

2.1 Behaviour of coupled shear walls

Coupled walls are a common form of lateral load resisting structure in residential and commercial multi-storey buildings. The advantage of coupled wall system is that it has lateral stiffness that is greater than the lateral stiffness of individual wall piers. Thus if a single shear wall has to be provided in place of coupled wall the dimensions would be significantly larger than coupled shear wall.

The structural response of coupled walls is complex because it is made of several components that exhibit different ductility properties. Fig 2.1 shows the idealized lateral force - deformation response and frame like responses of the coupling action provided by the beams. The coupling beams undergo significant inelastic deformation in order to allow the structure to achieve lateral yield strength. As the system continues to deform laterally in ductile mode, the wall ductility is smaller than the coupling beams and the beams should be designed to satisfy the high ductility demands imposed upon them. If the coupling beams are incapable of such ductility demands it leads to significant decrease in lateral stiffness and the system acts as two individual wall piers.

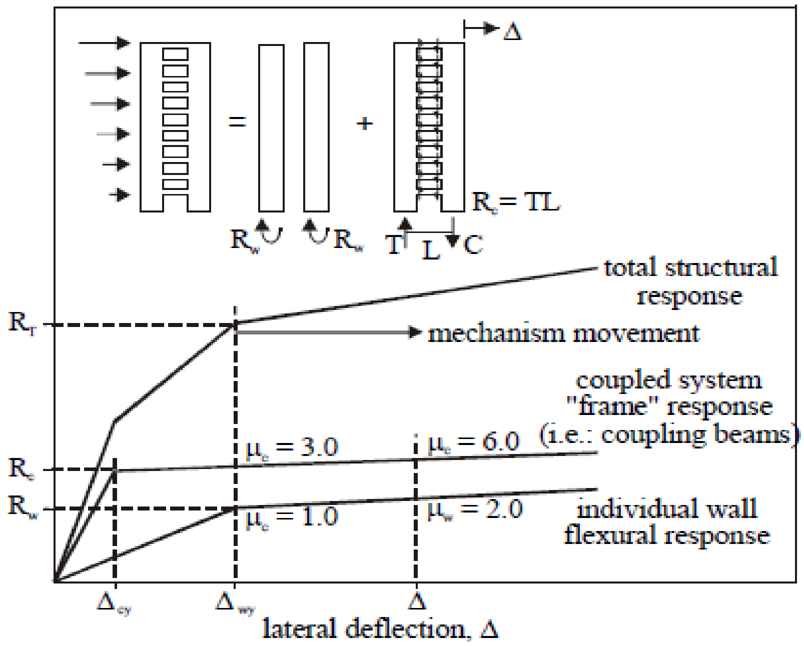

Figure 2.1 *Ductility of Walls, Coupling beams and Structure (El-Tawil et al., 2007)*

From the above discussion in better structural performance of the system the coupling beams reach their plasticity before the system achieves it plastic capacity. The damage is progressive and system evolves from behaving as coupled wall system to behaving as group of linked wall piers. The evolving behaviour is represented by progressive reduction of degree of coupling in elastic stage to degree of coupling in plastic stage. If the lateral stiffness of the system is considered as function of CR, the effect of CR reducing from a higher value to lower value results in increase in structural flexibility and demand on the wall piers.

2.2 Coupling Beams

2.2.1 Conventionally Reinforce Concrete Coupling Beams (CRCCB)

The main purpose of coupling beams between coupled walls due to seismic action is to transfer the shear force from one wall to other and for the coupling beams having length to depth ratio, l_b/h_b greater than 4, it is designed as CRCCB accordance with ACI–318 provisions using traditional longitudinal and transverse reinforcement. These beams are easy to construct but in comparison with diagonally reinforced coupling beams they have less energy dissipation capacity. In CRCCB, both the top and bottom reinforcement may simultaneously undergo tension leading to diagonal tension crack. During a seismic event several reversals of moment takes place and the flexural cracks at the boundaries interconnect and sliding shear failure takes place (Priestley et al., 1992).

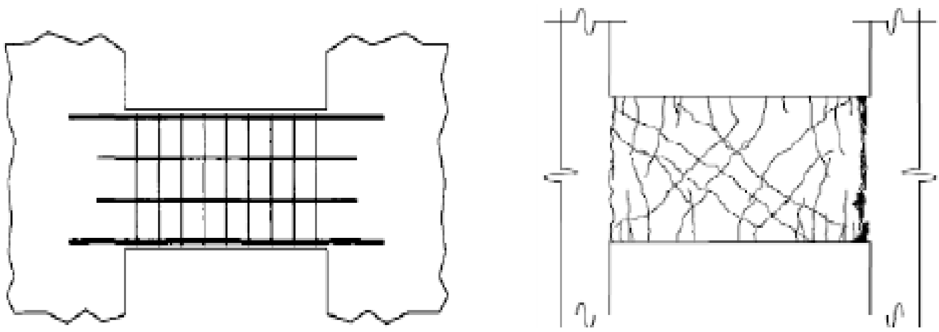

Figure 2.2 *CRCCB and shear slip crack (Priestley et al., 1992)*

2.2.2 Diagonally Reinforced Concrete Coupling Beams (DRCCB)

The reinforcement pattern of coupling beams requirements given by ACI -318 for a beam l_b/h_b ratio less than 2 shall be reinforced with two intersecting groups of diagonally placed bars symmetrically about the mid-span. For coupling beams having l_b/h_b ratio in between 2 and 4 shall be permitted to be designed with two intersecting groups of diagonally placed bars or other provision given ACI-318. For beams with ratio greater than 4, the diagonal reinforcement will not be effective in resisting the applied forces because of the very low angle of reinforcing bar inclination, α.

The primary reinforcement of DRCCB consists of two groups of diagonal bars placed symmetrically about the mid span of the beam. Each group is treated like a compression member having a minimum of 4 bars enclosed by transverse reinforcement. In addition, conventional longitudinal and transverse reinforcement are used to confine the entire beam section as shown in Fig 2.2. The Shear and moment capacities of DRCCB are provided entirely by the diagonal reinforcement. The shear strength of a DRCCB is determined by the following equation ACI-314.

$$V_u = 2A_{vd} f_y \sin\alpha \leq 10\sqrt{f_c^{'}} A_{cw} \tag{2.1}$$

Where A_{vd} is the area of steel in one diagonal bar group, A_{cw} and is the gross area of the concrete coupling beam resisting shear. Neither transverse reinforcement nor concrete contribute to the shear strength of $10\sqrt{f_c{}'}A_{cw}$ the DRCCB. The limit of is based on the observation that DRCCB remain ductile to at least this limit. The practical conditions lay another limit of shear stress equal to $6\sqrt{f_c{}'}$.The main drawback of DRCCB is that they are difficult to construct and the trade-off between steel area, A_{vd} and angle of inclination, α results in an inability to design a code compliant beams in many cases. The failure is due to opening of single dominant diagonal shear crack which occurs after maximum energy dissipation takes place.

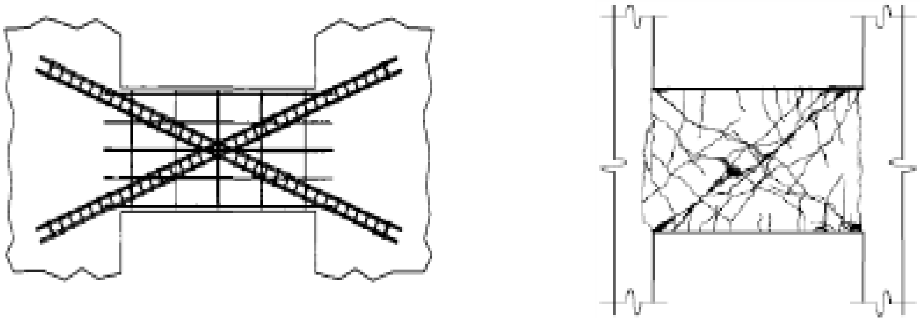

Figure 2.3 *DRCCB and diagonal shear crack (Priestley et al., 1992)*

2.2.3 Steel Coupling Beams (SB)

Steel Coupling beams are used in coupled wall systems as an alternative to reinforced concrete beams these systems are known as Hybrid coupled walls. Steel beams are advantageous when, due to the architectural restrictions, the available depth for the coupling beams is limited and when large ductility demand is present as a result of seismic loads. When subjected to cyclic loads, it has been shown experimentally that steel beams have superior stiffness and energy dissipation capacity over diagonally reinforced coupling beams. In response to seismic excitation steel coupling beams are expected to dissipate energy in a manner that is similar to the response of shear links in eccentrically braced frames. Shear links, and coupling beams are categorized as short, intermediate and long (ASCE 41). When architectural constraints permit short coupling beams that dissipate energy through shear distortions are preferred to longer coupling beams that dissipate due to flexure hinge rotation. Mechanisms that involve inelastic shear deformation in steel coupling beams are generally more ductile than those involving flexure related plastic hinge deformations.

To ensure full capacity of the beam is developed, steel coupling beams must be embedded appropriately in the adjacent wall piers. The coupling forces can be transmitted entirely through the interaction that occurs between the embedded beam and the wall as shown in the detail in figure 2.3

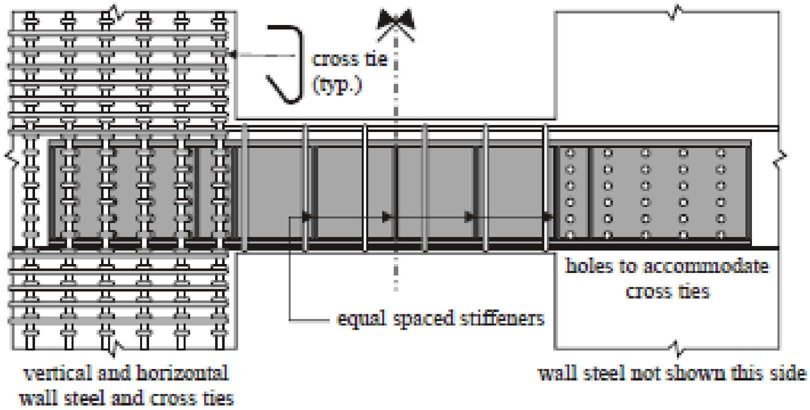

Figure 2.4 *Steel coupling beam (El-Tawil et al., 2007)*

2.2.4 Steel Coupling Beams with Fuses

Coupling beams are easily damaged in case of seismic event and are often used as energy dissipation part in structures. The reparability of coupling beams is a serious issue. This became necessary for traditional anti collapse design to be repairable design. It can be done by some replaceable fuse in coupling beams. These beams were first investigated by Fortney, Shahrooz and Rassati. The fuse acts as replaceable weak link where inelastic deformations get concentrated while the remaining components of the system are to remain elastic. In Fig 2.6 Steel coupling beam with fuse is shown. Fortney examined fuse with 50% and 70 % of the shear capacity of beam. Slip critical connections were used to prevent damage to the main section in event of failure will affect the intended behaviour of the structure. The major drawback was the failure in fillet welds used in built up I sections and failure of connection due to tear in web of the fuse. Yun Chen and Xilin Lu developed fuse beam that avoid such failures shown in Fig 2.5. The span of coupling beam should be same as conventional coupling beams with two non- yield segment and fuse segment which is made up of I section with diamond shape hole in middle cross. The flexural beam capacity and shear bearing capacity of the non- yield segment should be designed for an amplification factor times the shear and moment of fuse section. The fuse is designed for the forces intended to yield like conventional coupling beam. The fuse section designed for shear according to the formula given below.

$$\tau = \frac{V S_T}{2 t_w I_T} \tag{2.2}$$

Where, V is the shear in the hole section, t_w is the thickness of the web, S_t is the area moment of T section and I_t is the moment of inertia.

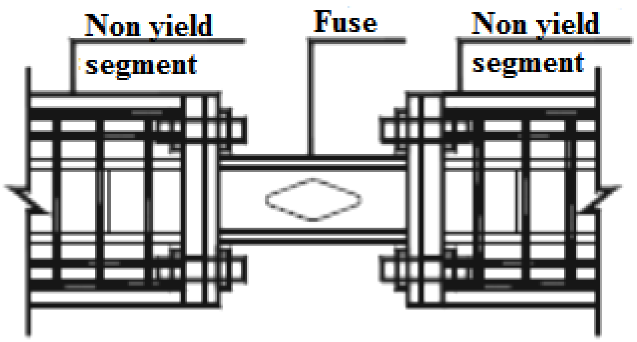

Figure 2.5 *Fuse proposed by Yun Chen and Xilin Lu (2012)*

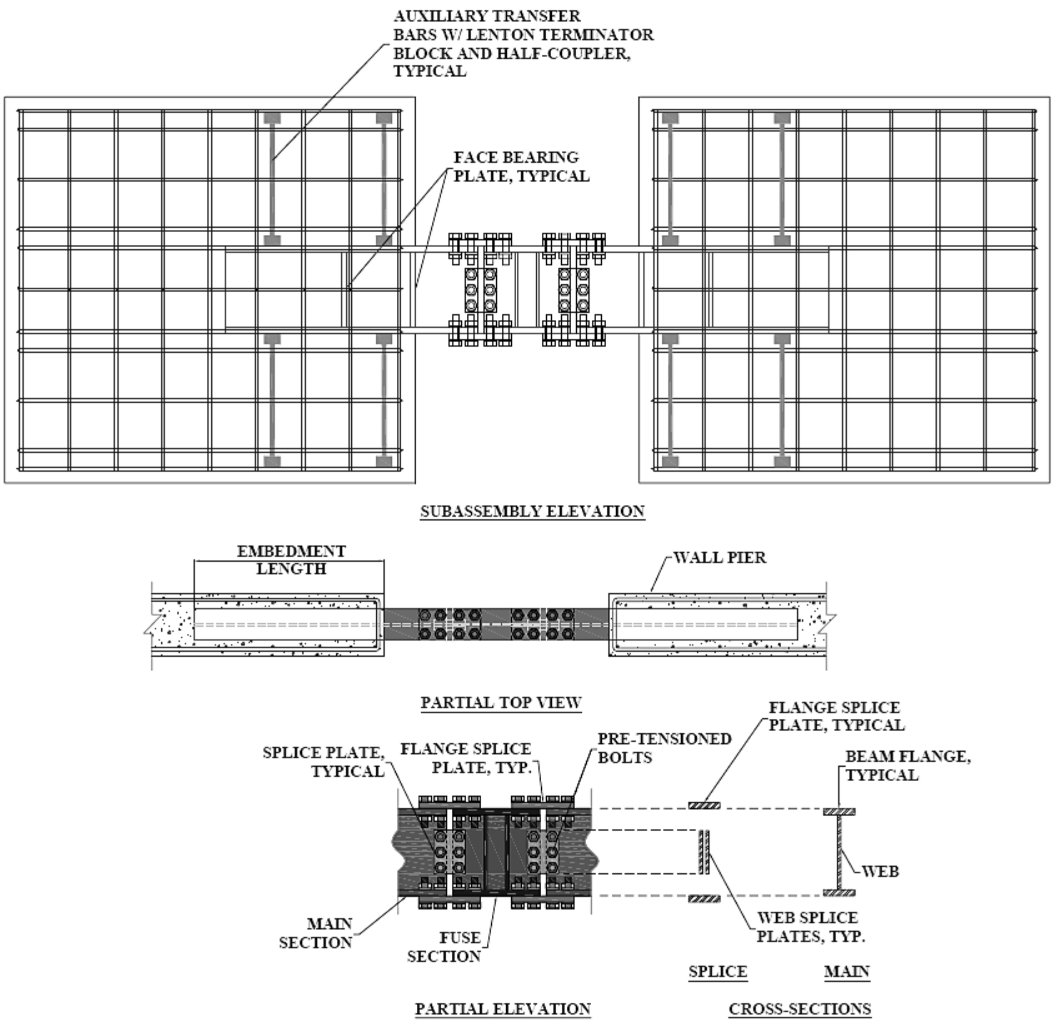

Figure 2.6 *Fuse proposed by Fortney (El-Tawil et al., 2007)*

2.3 CODE RECOMMENDATIONS FOR DESIGN OF COUPLED SHEAR WALLS

A Comparative study of recommendations of ACI-318, Euro code 8, IS code and NHERP guidelines on design of coupled shear walls have been done. The provision are given in Table 2.1.

2.4 COUPLING RATIO (CR)

As discussed previously coupling ratio CR also known as degree of coupling, it is defined as the ratio of moments resisted by walls through frame action of coupling beams i.e. the total shear force in coupling beams times the centre to centre distance of the walls to the total moment resisted by the coupled wall system.

$$CR = \frac{L\sum V}{L\sum V + M_1 + M_2} = \frac{L\sum V}{M_T} \tag{2.3}$$

Where,

CR = Coupling ratio,

L = Centre to centre distance between walls,

$\sum V$ = Sum of all coupling beam forces,

M_1= Moment at the base of compression wall,

M_2= Moment at the base of tension wall,

M_T= Total overturning moment resisted by the coupled shear wall system.

CR is calculated at the bottom of the wall, when the system undergoes plastic mechanism. Here, Plastic mechanism implies the coupling beams should have yielded and maintain their plastic rotation capacity and wall piers yield at their base. Coupled walls are significantly stiffer than individual components. Fig 2.7 illustrates the effects of uniform coupling beams over individual wall piers for triangular loading (El-Tawil et al., 2007). The plot of normalized roof deflection with CR shows that for small change in coupling ratio the stiffness is improved substantially at smaller CR's but at higher CR's there is little change.

Table 2.1 *Comparison of code provisions*

Code of Practice **Specification**	**NEHRP Guidelines**		**ACI 318 (2014) Design**		**IS Design**	**EURO code(2&8)**
Stiffness Modifiers	Coupling beams (CB)	Shear walls (SW)	CB	SW	N.A.	CB &SW
Flexure	0.15*EIg* (ATC72)	0.7*EIg* uncracked. 0.35*EIg* Cracked.	General beams 0.35*EIg*	For walls 0.7*EIg* uncracked 0.3*EIg* cracked.	N.A.	50 % of uncracked elements. For both flexure and shear unless more accurate analysis of cracked elements is performed.
Shear	0.4*EA* for $l/d \geq 2$ 0.1*EA* for $l/d<1.4$	Ranges from 0.05*GA* to 0.1*GA*	$1.0A_g$	$1.0\ A_g$	N.A	N.A.
Coupling beams **Based on aspect ratio and shear**	Cited from ACI 318		$l/d \geq 4$ must satisfy requirements of beams.		$\tau_{ve} > 0.1\sqrt{fck}(\frac{l}{d})$, the entire Earthquake induced shear, bending moment and axial compression shall be taken by diagonal reinforcement.	N.A.
Shear design	Same as ACI		$V_n = 2Avd.\ f_y\ .\sin\alpha \leq 10\sqrt{f'}.\ A_{cw}$		$A_{st}=V_n/(1.74\ f_y\ \sin\alpha)$	$V_n < 2A_{st}\ f_y\ \sin\alpha$

Wall Aspect ratio	*hw/lw* >2 behave like slender walls	*hw/lw* >2 designed as provisions for columns for *lw/bw* >2.5	*hw/lw* < 1 squat walls	If *l/b* > 4 to be called as wall.
	hw/lw <0.5 behave like diagonal strut where horizontal and vertical reinforcement resist shear.	*hw/lw* <2 designed as walls	1<*hw/lw*<2 intermediate walls	
	In-between defined as intermediate walls		*hw/lw* >2 slender walls.	
Wall min thickness	8in. practical lower limit. 12 in. for construction and performance. 10 in. for boundary elements. 14 in. in case of coupling beams and 16 in. for diagonally reinforced coupling beams.	A min thickness of 12in.	Min thickness of wall should not be less than 150mm and for coupled structural wall building 300mm in any seismic zone.	Wall thickness Max(0.15,hs/20) Min thickness of boundary elements should not be less than 200 mm. length of confined part does not exceed max of *2bw* and 0.2*lw*. *bw* should not be less than *hs*/15. If length of confined part exceed above, thickness should not be less than *hs*/10.
Moment	Design using strain and equilibrium compatibility.	Design using strain and equilibrium compatibility.	As per Annex A of IS13920.	Design as column as recommended in euro code 2

Shear	ACI , design shear strength is at least the shear developed in wall due to flexure capacity	$V_n = Acv\ (\alpha\lambda\sqrt{f'}+\rho t\ f_y)$ α = 3 for *hw/lw* <1.5 And 2 for *hw/lw* >2 and varies linearly.	$A_h = V_{us}/(0.87f_y(d/s_v))$	The increase in shear force after yielding at the base of a primary seismic wall, shall be taken into account. Code based design envelope and magnification factor.
Special boundary elements	Same as ACI	$C \geq lw\ 600\ (du/hw)$, where c is Neutral axis depth. Special boundary elements are required at edge if stress exceeds 0.2fc' and depth of boundary element is kept until stress drops to 0.15fc'	Special boundary elements are required at edge if stress exceeds 0.2fck and depth of boundary element is kept until stress drops to 0.15fck.	N.A.
Inter story drift	based on occupancy category (ii) 0.020h (iii) 0.015h (iv) 0.010h as per ASCE 7	As per ASCE 7	0.004h from IS 1893 part1	Drift= 0.010*h/v* V is reduction factor based on class of buildings
Coupling Ratio (CR)	N.A.	N.A.	N.A.	25%

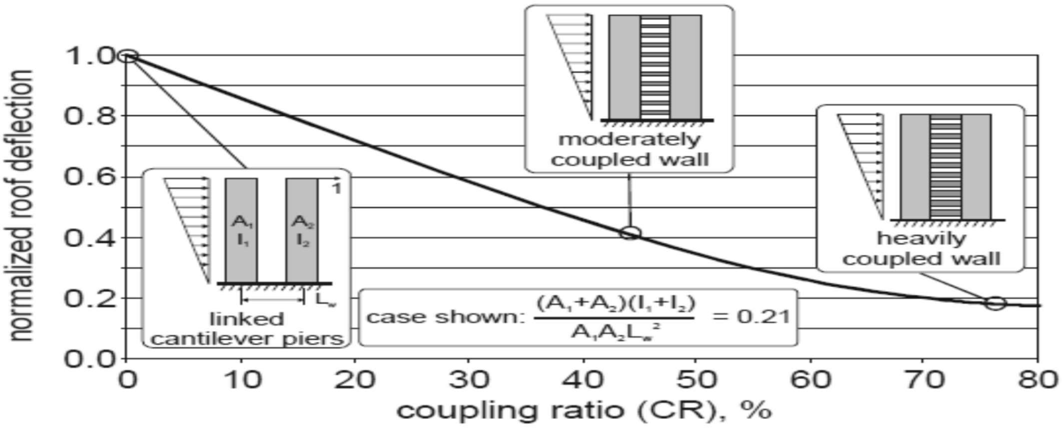

Figure 2.7 *Plot of roof deflection and coupling ratio (El-Tawil et al., 2007)*

CR also varies with lateral displacements, a lateral load is gradually increased on to a lightly coupled 12 storey shear wall. The system being elastic initially, CR increases as the load in coupling beams increases and reaches peak when the coupling beams yield. After yielding CR drops since the contribution of beams to CR does not increase while contribution of walls increases and reaches a minimum when both wall piers yields, on further displacements the CR starts to increase due to hardening of the beams.

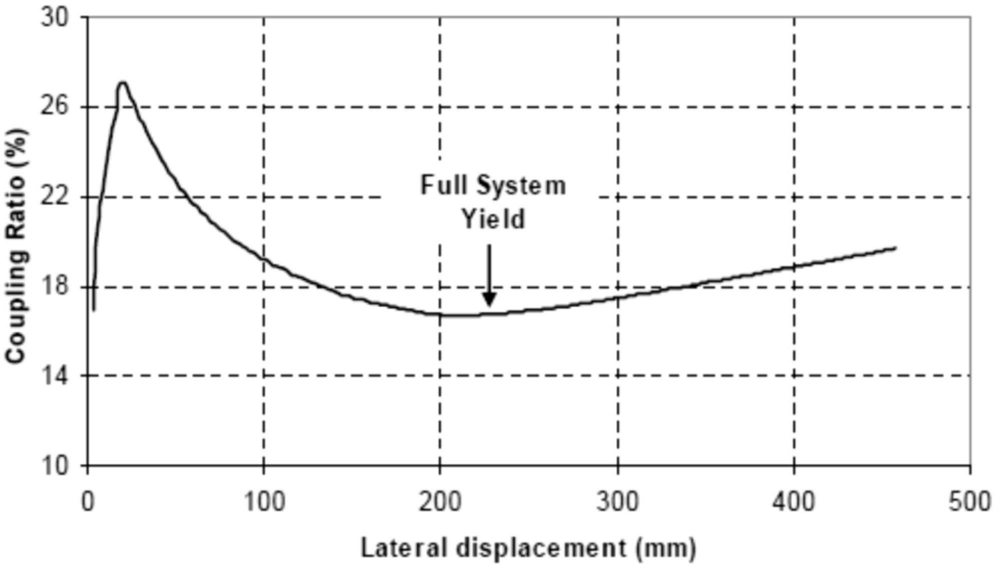

Figure 2.8 *Variation of CR with lateral displacement (El-Tawil et al., 2007)*

The performance of wall depends upon its P-M interactions, CR influences axial load in the wall piers significantly, decreasing the ductility of the wall substantially and causes crushing failure (). It also effects the wall piers in tension, increase in tensile forces in wall piers leads to less moment carrying capacity and ductility. Thus the system with large CR may suffer earlier crushing failures compared to lower CR's and systems with very low CR's shows poor performance. For better performance and economically sound design the CR should be in the range of 30-45% as recommended El-Tawil et al. (2007).

2.5 MODELLING OF ELEMENTS

Modelling of elements to predict the force deformation behaviour of the system. The beams and columns of the building are modelled using frame elements that depicts the deformation behaviour in both moment and shear. Walls and coupling beams behaviour can be modelled using following models,

- Equivalent frame model
- Multi spring model
- Finite Element model

In Equivalent frame model, generally the wall behaviour is modelled using wide column analogy, where the frame element is placed at the centre of the wall and connected to beams by rigid elements. These rigid elements are beam elements with very high rigidity. In linear analysis the elements should account for reduction in stiffness for cracking.

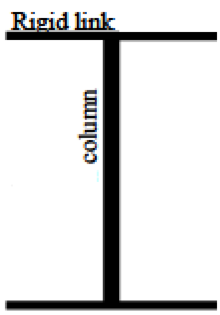

Figure 2.9 *Wide column model*

In multi spring models, the behaviour of the wall modelled using a number of series /parallel springs to simulate the axial, shear and bending behaviour of wall panels, while rigid elements are used to represent the physical wall.

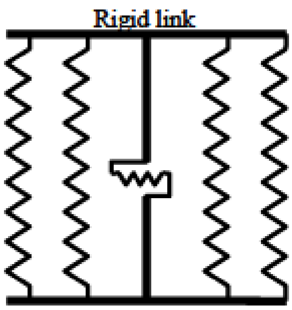

Figure 2.11 *Multi spring model*

In Finite element model, the shear walls are modelled using shell element, layered shell element, plain stress and plain strain elements. The elements in linear analysis should account for expected effect of cracking.

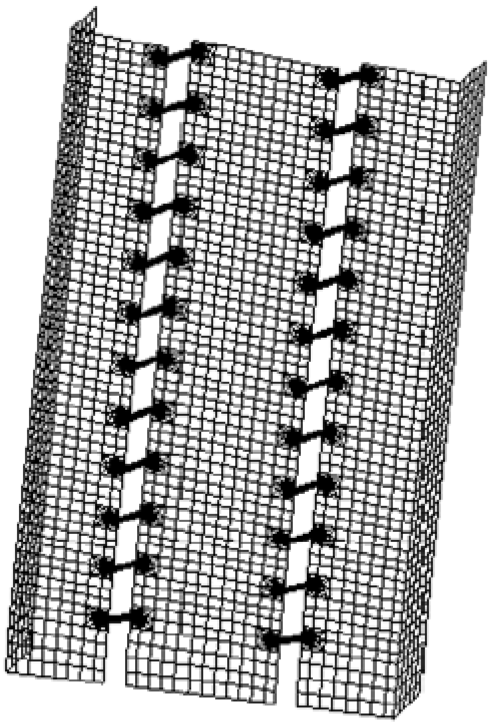

Figure 2.10 *Finite element model*

2.6 Nonlinear modelling

The nonlinear modelling of the building is done after linear modelling and design. It is done to evaluate the performance of the structure using nonlinear static procedures and nonlinear dynamic procedures. Beam are modelled with M3 hinges, columns and shear walls shall be modelled with interacting P-M2-M3 hinges. The hinges properties are as per ASCE 41. Coupling beams are designed to fail in shear, in case of steel beams and steel beams with fuses nonlinear model is similar to nonlinear modelling of EBF shear link. ASCE 41 discuss only about flexural modelling of diagonally reinforced coupling beams. Available literature on modelling of shear hinge is proposed by Hindi.

Hindi (2004) proposed a theoretical model to predict monotonic load deformation behavior of diagonally reinforced coupling beams. The model is based on assumption that all the loads acting on the coupling beam is resisted by strut action of the diagonal reinforcement, that is due to diagonal compression and tension. The diagonal tensile force (T) and diagonal compressive force (C) at any diagonal strain can be computed by relation given below.

$$T = A_s f(\varepsilon_s) \tag{2.3}$$

$$C = A_s f(\varepsilon_s) + A_c f(\varepsilon_c) \tag{2.4}$$

where, A_c is the area of concrete core within the diagonal compression reinforcement; $f(\varepsilon_s)$ and $f(\varepsilon_c)$ are the steel and concrete stresses, ε_s and ε_c are the concrete and steel strains along the diagonal reinforcement. The strains along the diagonal reinforcement in proposed model is assumed same. As shown in Fig the total shear resistance V of coupling beam at any diagonal strain is given by the equation.

$$V = (T + C)\sin\alpha \tag{2.5}$$

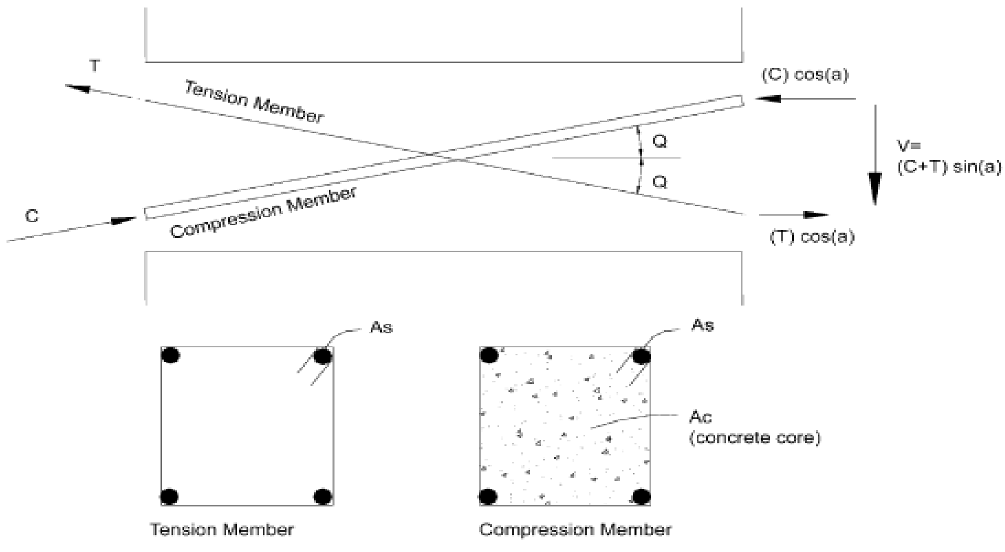

Figure 2.12 *Model proposed by Hindi (2004).*

2.6 Summary

In this chapter literature on the available design guidelines has been reviewed. The design of coupled shear wall is based on philosophy that the coupling beams should yield prior to yielding of the shear wall. The coupling beams should have large ductility, they should maintain their plastic strength till the walls reach their plastic strength limit. CR plays major role in implementing this philosophy. Design of walls and coupling beams is similar across the codes. Euro code 8 suggests a lower limit on CR but other codes do not recommend any limit. However, none of the codes recommend a cap on CR.

Chapter 3

DESIGN METHODOLOGY

3.1 General

The design of coupled shear wall building has several challenges and guidelines for design of coupling walls is limited, from the review of code provisions shows that CR an important parameter in the design of coupled shear walls is neglected. CR is evaluated, when the plastic mechanism of wall occurs; evaluation of CR is possible only after nonlinear analysis and since most of codes are based on liner analysis and design not much emphasis is given. There is alternative to this by evaluating $CR_{elastic}$ which is evaluated at the lateral design load. Generally CR increases till the coupling beams yield and decreases till wall reaches yield. Thus $CR_{elastic}$ is kept little on higher side to account for this effect usually around 1.2-1.4 times the desired range. Recommended values of CR for better performance of structure and economical design are given as 0.3 to 0.45. The following is step by step procedure for design of coupled shear wall building based on philosophy that coupling beams should yield before the yielding of walls.

3.2 Preliminary design

Step 1. Select the type of structural system to design, such as dual system or wall system. Euro code 8 suggests definition of such structural system such as,

a) Wall system: at least 60% of base shear due to lateral load is resisted by the walls.

b) Wall equivalent dual system: at least 50% of base shear due to lateral load is resisted by the walls.

c) Frame equivalent dual system: at least 50% of base shear due to lateral load is resisted by frames.

ACI 318, IS 13920 suggest that in a dual system, the frame system shall be designed to resist at least 25% of lateral loads.

Step 2. The location of the walls position may be functional requirement. For higher torsional resistance walls should be located at periphery of the building.

Step 3. The preliminary member sizes of beams, columns, walls are based on architectural constraints and experience. The sizes of walls should be kept a minimum of 300 mm in a coupled shear wall system. The openings in walls or the length of coupling beam is based on functional requirement and architectural constraint.

3.3 Procedure for Modeling & Analysis.

Step 1. Construct a linear elastic model according to modeling guidelines as follows. Walls shall be modeled as Equivalent frame model, multi spring model or finite element model. Coupling beams shall be modeled using elements that depict flexure and shear behavior. Reduced section properties are to be used to account for cracking and loss of stiffness due to slippage etc. Different codes recommend various reduced properties stiffness properties. Following are the recommended provision for design.

a) Stiffness for beams is $0.35EI_g$ in flexure and for columns it is $0.3EI_g$.

b) Stiffness for walls at possible cracked sections are $0.3EI_g$ in flexure and $0.7EA_g$ in shear; stiffness for uncracked wall it is $0.7EI_g$ in flexure and $1.0EA_g$ in shear.

d) For RCC coupling beams effective stiffness is taken as $0.3EI_g$ and shear stiffness is taken as $0.25GA_g$. In case of steel beams the stiffness $0.6EI_g$ is taken in flexure.

Step 2. Modelling of Diaphragm is done as per ASCE 41, to account for in-plain behavior of slab.

Step 3. Perform Equivalent Lateral Force Analysis (ELFA) or Modal Response Spectrum Analysis (MRSA) to evaluate design forces and global deformations.

Step 4. The member sizes of the model should be proportioned to satisfy drift recommendations of code; IS 13920 recommends Inter story drift 0.004 time's story height at unfactored applied lateral load.

Step 5. Evaluating $CR_{elastic}$, it is evaluated at the base of walls using the equation (2.3) from the forces obtained at applied factored lateral load. The member sizes are iterated until the choice of $CR_{elastic}$ is in desired range.

3.4 Detailed design

Step 1. Design of coupling beams

i) Diagonally reinforced coupling beam

a) Shear

Redistribution of shear forces in coupling beams along the height is allowed up to 20% provided that sum of beam shear capacities exceeds the sum of the maximum beam shears obtained from analysis. El-Tawil et al., 2007 suggests the redistribution of forces for system with lower CR should be done as per Figure 4.1(a) and for higher CR it should be done as per figure 4.1(b), where V_n and V_f are redistributed shear and shear obtained from analysis respectively.

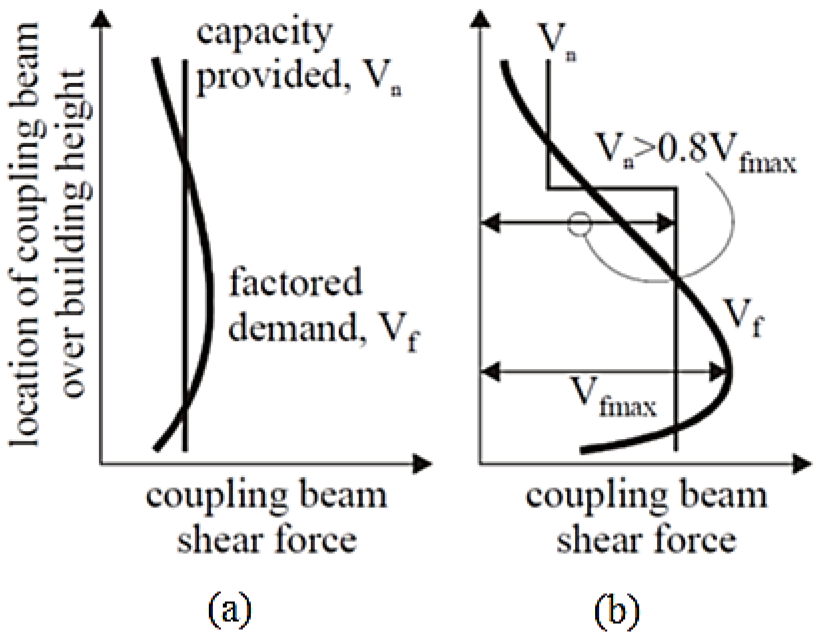

Figure 3.1 *Vertical shear distribution in coupling beams; Low CR and High CR (El-Tawil et al., 2007)*

The shear stress calculated for the shear force obtained from redistribution satisfies $\tau_{ve} > 0.1\sqrt{f_{ck}}\ (l/d)$, then the redistributed forces in coupling beams are designed to resist by diagonal reinforcement calculated by relationship given below. If the shear stress calculated is less, then it designed as per guide lines devised in IS 456.

$$A_{vd} = \frac{V_n}{1.74 f_y \sin\alpha} \tag{4.2}$$

where,

V_n = Redistributed shear force.

A_{vd} = Area of diagonal reinforcement provided.

α = Angle of the diagonal reinforcement.

b) Moment

The capacity of members to be checked for the redistributed moments. All the moments generated due to earthquake should be resisted by the diagonal reinforcement. The moment at the critical section is resisted by the diagonal reinforcement as shown in Fig 4.2.

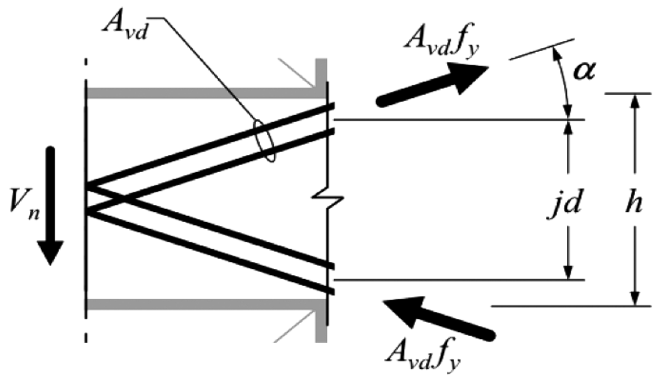

Figure 3.2 *Free body diagram diagonally reinforced coupling beam*

The following relationship is obtained for moment capacity of the section.

$$M_d = jd.A_{vd} f_y \cos\alpha \tag{4.3}$$

where,

M_d = Moment capacity.

jd = Lever arm at critical section, center to center distance between the diagonal reinforcement.

ii) Steel beam

a) Shear

Steel beams are designed for redistributed shear forces as above given redistribution curve. The beam is design force should be less than 0.6 times shear capacity of the member for complete plastic failure of member in moment. The shear capacity of member is calculated as per code provision of IS 800.

b) Moment

Moment obtained from analysis is also redistributed as per above given redistribution, the members are designed for the redistributed moments. The moment capacity of members are calculated as per design procedure laid in IS 800.

c) Embedment length

The embedment length L_e, required to develop the expected coupling beam shear strength V_e, calculated using model given by Mattock and Gaafar. The embedment L_e is determined by relation given below.

$$V_e = 4.05\sqrt{f_c^{'}}\left(\frac{b_w}{b_f}\right)^{0.66}\beta_1 b_f L_e\left[\frac{0.58-0.22\beta_1}{0.88+\dfrac{g}{2L_e}}\right] \tag{4.4}$$

where,

V_e = Expected coupling beam shear

L_e= Embedment length of the beam

g= clear length of the beam

$f'c$ = cylindrical compressive strength of concrete in Mpa.

β_1= ratio of average compressive strength to the maximum compressive strength defined as per ACI 318, usually the value is taken as 0.8

b_w= web thickness of the coupling beam

b_f= flange width of the coupling beam

iii) Steel beam with fuse

a) Shear

The basic design purpose of steel beam with fuse is that it should yield in shear, similar to EBF link. The shear capacity of fuse element is designed for the redistributed forces and the non-yielding portion is designed for γ_b=1.4 factor times the fuse element design force, in order for the member to remain elastic.

b) Moment

The design moment of the member fuse and non-yielding portion is designed for the factored redistributed moments obtained. The moment capacity of members is calculated same as of steel beam.

Step 2. Design of wall piers

a) Axial

Minimum thickness of coupled shear wall is 300mm. To ensure ductile behavior for preferred plastic mechanism the walls should be stronger than the coupling beams. In order to achieve this, wall over strength factor γ_w is applied to wall forces. It is defined as the ratio of sum of nominal shear capacities of coupling beams to sum of design forces determined for factored load loading. Additional factor R_y considers for expected material strength and strain hardening, that is

$$V_e = R_y . V_n \tag{4.4}$$

The axial load for design of wall is maximum load obtained from factored gravity load combinations plus sum of the expected shear capacities of coupling beam. An example load case is given below for shear wall only system.

$P = 1.2DL + 1.2LL + \Sigma V_e$ in compression wall.

$P = 0.9DL - \Sigma V_e$ in tension wall.

In case of shear wall dual system where beams are rigidly connected to walls, since the axial forces on the wall is also dependent on the frame action of the adjacent beams, the following modified load cases are applied for the design.

$P = 1.2DL + 1.2LL + 1.2\ (\gamma_w R_y)\ EQ$ in compression wall pier.

$P = 0.9DL + 1.5(\gamma_w R_y)\ EQ$ in tension wall pier.

There is limit to account for detrimental effects of beam over strength and ensure axial stability of wall in compression the normalized axial force should be less than 0.35.

b) Flexure

To design the wall in flexure, the bending moment obtained from analysis is modified as per Figure 4.3.The moment at the base of the wall is shifted to a height

a_i to account for the effect of diagonal tension on internal flexural reinforcement. The tension shift considered is equal to length of wall l_w (Priestley et al., 1992). In the figure 4.3 (a) shows design moment envelope for wall system and figure 4.3 (b) shows moment envelope for dual system, in dual system there is change in sign of moments along height, the values obtained are taken positive as shown by dotted lines.

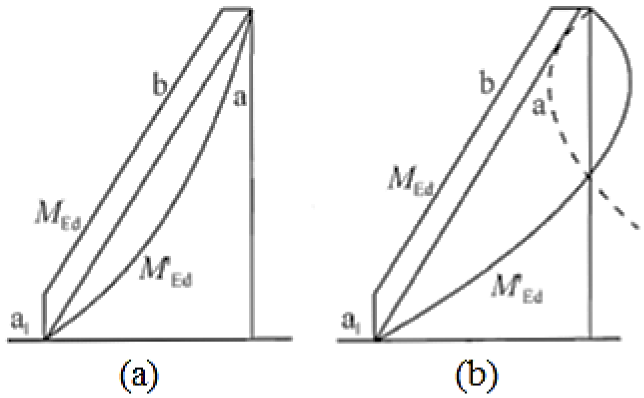

Figure 3.3 *Design moment envelope, Left: Wall system, Right: Dual systems (Euro code 8)*

where,

a_1 = tension shift

a, M'_{Ed}= Bending moment from analysis.

b, M_{Ed} = Bending moment for design.

Wall shall be designed for worst combination of axial load and moment using design procedure as per Annex A of IS13920.

c) Shear

The increase in shear forces after yielding of slender wall in flexure shall be considered in design of wall in shear. General practice according to Euro code 8 is to increase the obtained forces from analysis according to equation (4.5) and incase of dual system the design envelope to be followed is given in figure below for the modified force obtained.

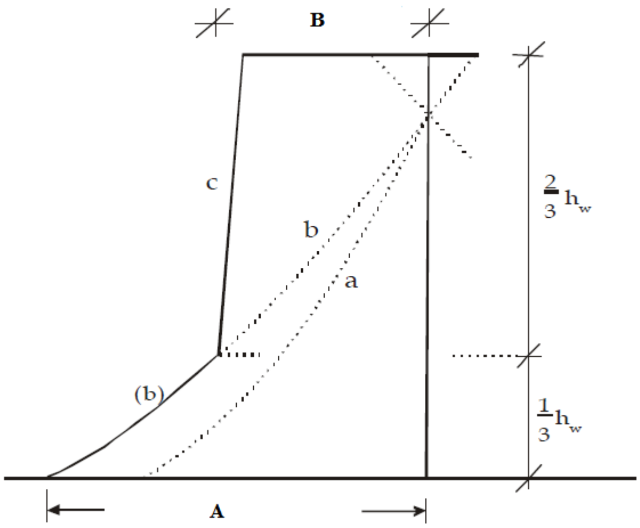

Figure 3.4 *Design shear envelope (Euro code 8)*

where,

a = shear diagram from analysis

b = magnified shear diagram

c = design envelope

A = shear at the base of the wall

B = shear at the top of the wall is taken as A/2.

The magnified shear V_{wd} is calculated from the relation given below,

$$V_{wd} = \varepsilon . V_{we} \tag{4.5}$$

$$\varepsilon = R.\sqrt{\left(\frac{\gamma_{Rd}.M_{Rd}}{R.M_{Ed}}\right)^2 + 0.1\left(\frac{S_a/g(T_c)}{S_a/g(T_1)}\right)^2} \tag{4.6}$$

R = Response reduction factor as per IS 1893

M_{Rd} = Design flexural capacity of the wall.

M_{Ed} = Design Bending moment of the wall.

γ_{Rd} = Overstrength factor to account for steel strain-hardening, if accurate data is not available it may be taken equal to 1.2.

T_1 = Fundamental period of vibration of the building in the direction of shear force V_{we}.

T_c = Upper limit period of the constant spectral acceleration region of design spectrum.

$S_a/g\ (T)$ = Ordinate of the elastic response spectrum.

For the obtained design shear force the shear reinforcement is calculated as per design provisions of IS13920.

c) Boundary elements

Boundary elements are provided along the vertical boundaries of the wall, it is provided when extreme fibre compressive stress of the wall exceeds $0.2f_{ck}$ due to gravity and earthquake loads. It is discontinued when the stress decreases to $0.15f_{ck}$.

3.5 Summary

In this chapter, a step-by-step procedure has been laid for proportioning and design of members for buildings with coupled shear walls. The procedure is laid down according to design recommendations of IS code. The parameter CR has been given due importance in design. Design of coupled shear walls is based on philosophy that coupling beams in coupled shear walls should yield prior to wall. This is achieved by considering overstrength of coupling beam in design of walls. Walls are designed to yield in flexure and this is achieved by designing the walls for magnified shear forces.

Chapter 4

NUMERICAL STUDY

4.1 General

A numerical study is performed based on design procedure laid down in the previous chapter to validate the design objectives. The building considered is a 15 storied Reinforced concrete building having a floor area of 625 sq.m on each floor. The building is 25 m symmetric in both directions with 48m in height, the plan area is show in Figure 4.1. For all structural elements, M40 grade concrete is used. Sizes of the beams and columns are kept 400x600 mm and 700x700 mm respectively. The slab thickness is taken as 150 mm. The external infill thickness is taken as 230 mm and internal as 150 mm. The storey height is taken as 3.2 m.

In this chapter, proposed methodology in modelling and design has been illustrated using a 15 storey RC coupled shear wall building. The lateral load resistance is provided by the coupled shear wall system. In this study the example building is designed using different coupling beams first with Diagonally Reinforced concrete coupling beam, Steel beams and Steel beams with fuses using the proposed methodology. The building is designed for IS code based design response spectrum for zone V DBE condition. The importance factor is considered as 1 while the response reduction factor is taken as R=5. Dead load, imposed load are taken as per IS: 875 (Part I & II). The seismic performance is evaluated using nonlinear static procedure laid down in ASCE 41-13.

4.2 Linear modelling of the building

The building has been modelled as 3D space frame model using ETABS V15.2.2. For simulation of behaviour under gravity and seismic loading the beams and columns have been modelled using frame element with six degree of freedom system at each node. To consider the effects of slab rigid diaphragm constraint has been provided .Walls are modelled as equivalent frame models using wide column analogy. Coupling beams are modelled as member depicting both flexure and shear deformation can be modelled as recommended by ASCE 41. The foundation of building is considered as fixed support. The loads of slab are distributed on to the frame elements according to the yield line theory. Following load combinations given in IS: 1893-2002 (part1) are used to design:

1.5(DL+LL)

1.2(DL+LL±EQ)

1.5(DL±EQ)

0.9DL±1.5EQ

4.2.1 Placement of coupled shear walls

Coupled shear walls were placed at different location in plan depicted in the following figures and the problems associated with configurations is discussed.

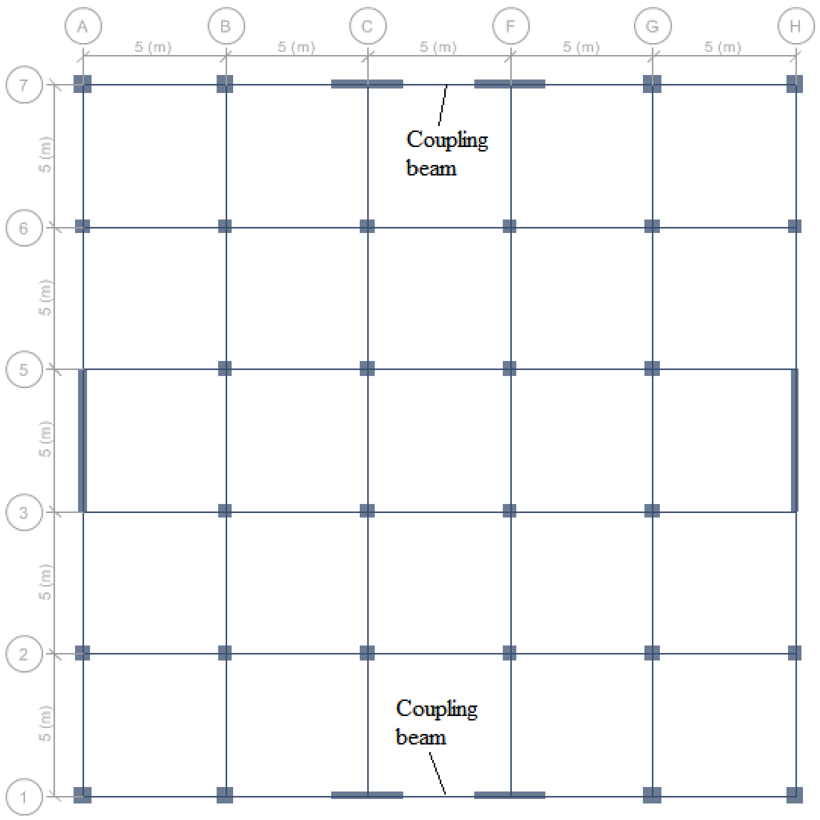

Figure 4.1 *Plan view of the Model 1*

Configuration shown in Fig 4.1 is modelled with coupling beam of size 300x 600 mm and walls of size 2500 mm. The percentage of shear resisted by the walls is 65% and $CR_{elastic}$ is 0.53. In this configuration wall pier is subjected to tensile forces for applied load combinations and concrete compression members are not designed for tensile forces and thus this configuration is modified.

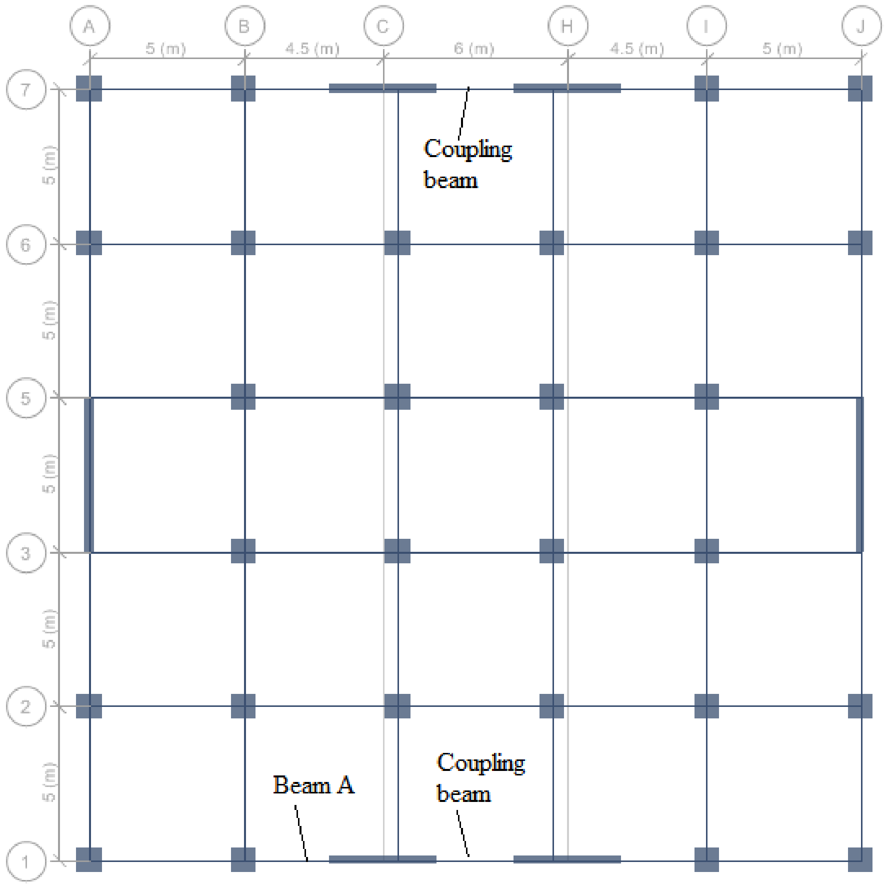

Figure 4.2 *Plan view of the Model 2*

Configuration shown in Fig 4.2 is modelled with coupling beam of size 300x 800 mm and walls of size 3500 mm. The percentage of shear resisted by the walls is 61% and $CR_{elastic}$ is 0.59. Beam A as marked in Fig 4.2 cannot be designed in shear as shear forces exceeds the shear capacity of beam.

Configuration shown in Fig 4.3 is modelled with coupling beam of size 300x 1200 mm and walls of size 3500 mm. The percentage of shear resisted by the walls is 65% and $CR_{elastic}$ is 0.58. Beam B as marked in Fig 4.2 cannot be designed in shear as shear forces exceeds the shear capacity of beam and wall pier subjected to tensile forces as discussed in model 1.

Configuration shown in Fig 4.4 is modelled with coupling beam of size 300x 600 mm and walls of length 2600 mm. The percentage of shear resisted by the walls is 72% and $CR_{elastic}$ is 0.73. In this configuration does not show any difficulty in design.

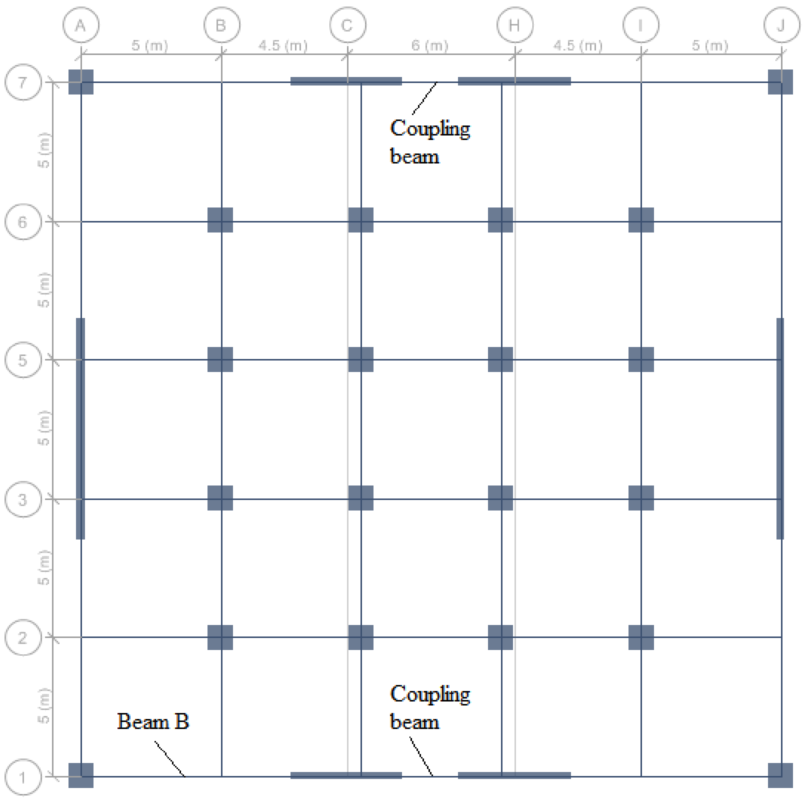

Figure 4.3 *Plan view of the Model 3*

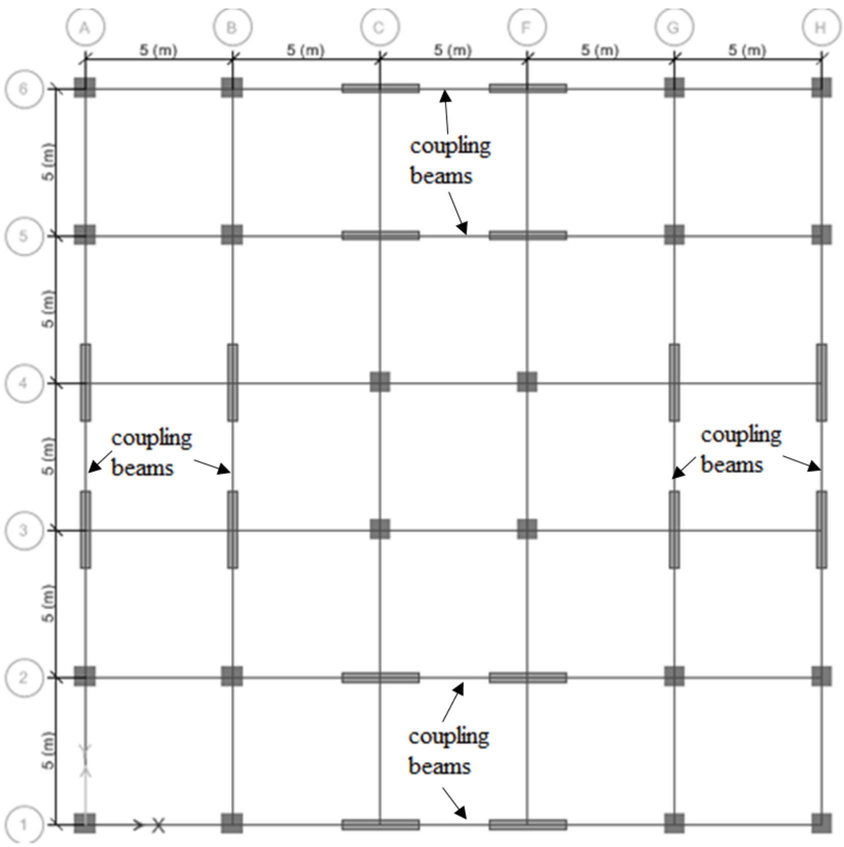

Figure 4.4 *Plan view of the Model 4*

. The location of coupled shear walls is symmetric in both the direction as pointed out in the plan area. The length of the wall is 2.6m and 300 mm thick coupled with coupling beam of depth 600 mm separated by centre to centre distance of 5 m. The clear length of coupling beam is 2.4 m. The model with steel beam and steel beam with fuse is modelled with ISMB 500 section.

4.2.2 Modal analysis

The analysis of the model has been done based on cracked section properties recommended as per given procedure. The columns are modelled with flexural rigidity of $0.3E_cI_g$ and shear rigidity of $0.4E_cA_w$, beams are modelled with $0.35E_cI_g$ and shear rigidity of $0.4E_cA_w$. The shear walls are modelled same as the columns in flexure with $0.3E_cI_g$ with shear rigidity of $0.7E_cA_w$. Diagonally reinforced coupling beams are modelled with reduced flexural stiffness of $0.3E_cI_g$ and reduced shear stiffness of *0.25GA*, Steel coupling beams are modelled with stiffness reduced to $0.6E_sI_g$ to take it into effect of post yield reduced stiffness. The results obtained from vibrational analysis are as given below.

Table -4.1 *Vibration characteristics of buildings*

Model Description	Direction	Period of Vibration (sec)	Modal Mass participation
Model with DRCCB	Longitudinal	2.01	0.7718
	Torsional	1.702	-
Model with SB and SB Fuse	Longitudinal	2.018	0.7719
	Torsional	1.705	-

Note: DRCCB=diagonally reinforced concrete coupling beam, SB= steel beam.

The vibrational characteristics of the three models are similar, that can be used for comparison of the buildings for performance. Before proceeding to design of the building drift checks are performed to satisfy serviceability criteria based on IS: 1893-2000 (Part 1) recommendations.

4.2.3 Design base shear

The buildings are designed to resist earthquake, it is done by applying a lateral equivalent load calculated as per IS: 1893-2000 (Part 1). The code recommends period on type of the geometry of the building, this results in a minimum force that should be applied to design a building. The design base shear of the proposed models is corrected for the code based design base shear and the results are given below.

Table 4.2 *Design base shear*

Model	Base shear (kN)	% Resisted by walls	Type of system
Model with DRCCB	9304	71.2	Wall, dual
Model with SB and SB Fuse	9155	71.9	Wall, dual

Table 4.2 *Member sizes, (%) of reinforcement*

Member	Member size (mm)	Reinforcement (%)
Beam	400x600	0.29-1.8
Column	700x700	0.8-1.29
Shear wall	300x2600	0.4-1.28
Coupling beam (DRCCB)	300x600	2.38 (diagonal reinforcement)
Coupling beam (SB)	ISWB 500	N.A

Table 4.3 Drift check

Storey level	Drift (<0.4%) Model with DRCCB X Direction (%)	Drift (<0.4%) Model with SB X Direction (%)
Level 15	0.054	0.053
Level 14	0.072	0.071
Level 13	0.094	0.94
Level 12	0.116	0.116
Level 11	0.137	0.137
Level 10	0.156	0.157
Level 9	0.174	0.176
Level 8	0.190	0.193
Level 7	0.205	0.209
Level 6	0.218	0.224
Level 5	0.229	0.235
Level 4	0.233	0.24
Level 3	0.226	0.23
Level 2	0.195	0.199
Level 1	0.105	0.107

There is a decrease in the base shear between the model with DRCCB and model with SB, for similar stiffness the mass of concrete building is larger than the building model with SB. After correction of the design base shear the type of building is evaluated based on the percentage of the base shear resisted by the walls. Given in table 4.3. The percentage of the force obtained in both the systems is above 60%, the systems can be designed for both wall systems and wall equivalent frame dual system.

4.3 Nonlinear Modelling

The nonlinear modelling of the building is done only after the linear analysis and design of the building is performed. The nonlinear modelling is done using ETABS v15, using nonlinear material properties for beams, columns and shear walls. The nonlinear modelling of the beams are done by using M3 deformation controlled hinges, for columns and shear walls modelling is done using P-M2-M3 interacting hinges from ASCE 41. The nonlinear modelling of the DRCCB is done using Hindi's model and the modelling parameters are shown in Table 4.6. The nonlinear modelling of the steel beam and steel beam with fuse are modelled as per ASCE 41, nonlinear properties for steel beam with fuse are same as shear link in EBF.

Table 4.5 *Nonlinear modelling parameters*

Member (Typical)	Modelling parameter					
	a	**b**	**c**	**IO**	**LS**	**CP**
Beams (ASCE 41)	0.02	0.03	0.2	0.005	0.02	0.03
Columns (ASCE 41)	0.012	0.012	0.2	0.005	0.010	0.012
Walls (ASCE 41)	0.005	0.012	0.6	0.003	0.009	0.012
DRCCB (Hindi model)	0.0027	0.0081	1.04	-	-	-
SB (ASCE 41)	0.09	0.11	0.6	0.01	0.09	0.11
SB with fuse (ASCE 41)	0.0028	0.0036	0.6	0.0031	0.057	0.07

4.4 Nonlinear Static Analysis

The performance of the model with DRCCB, SB and SB with fuse is evaluated by performing nonlinear static pushover analysis using ETABS v15 software. Pushover analysis is performed as discussed previously in Chapter 1. The load patter as recommended by ATC-40 should be proportional to first mode if the contribution of first mode is greater than 75%. In this study the contribution is around 77.2%, so lateral load proportional to first mode is used. Figure 4.2 shows the capacity curves of the considered models.

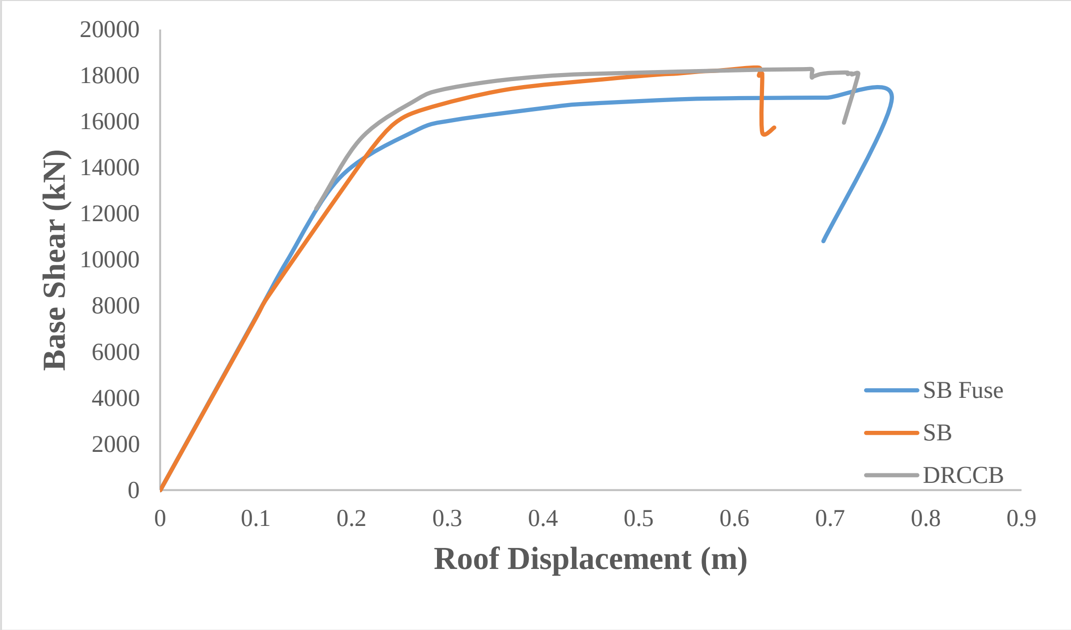

Figure 4.5 *Capacity curve of the models*

4.5 Evaluation of CR

As discussed earlier CR is evaluated at the base of the wall when the plastic deformation takes place in the wall. It is possible only after nonlinear analysis is performed. $CR_{elastic}$ is calculated at laterally applied load i.e. the equivalent earthquake load obtained from response spectrum analysis.

Table 4.6 *Values of CR for different models*

Model	$CR_{elastic}$	CR	$CR_{elastic}$/ CR
Model with DRCCB	0.73	0.66	1.10
Model with SB	0.73	0.60	1.21
Model with SB Fuse	0.73	0.68	1.07

4.6 Hinge patterns

The Figures 4.3 and 4.4 show the hinge pattern obtained from pushover analysis at the maximum displacement. From the hinge pattern shown the intended yield pattern is observed. In the model with DRCCB beams from storey 1 to 9 are DRCCB and 10 to 15 storey designed as conventional beam. The DRCCB forms shear hinge while the conventional beams formed flexural hinges. The model with steel beams yielded in flexure as expected since the coupling beams act as EBF link and for the given length of beam the EBF link yields in flexure. In order to improve the performance of model with steel beam, a fuse has been introduced, so that it yields in shear from the hinge pattern observed coupling beams with fuse yields in shear.

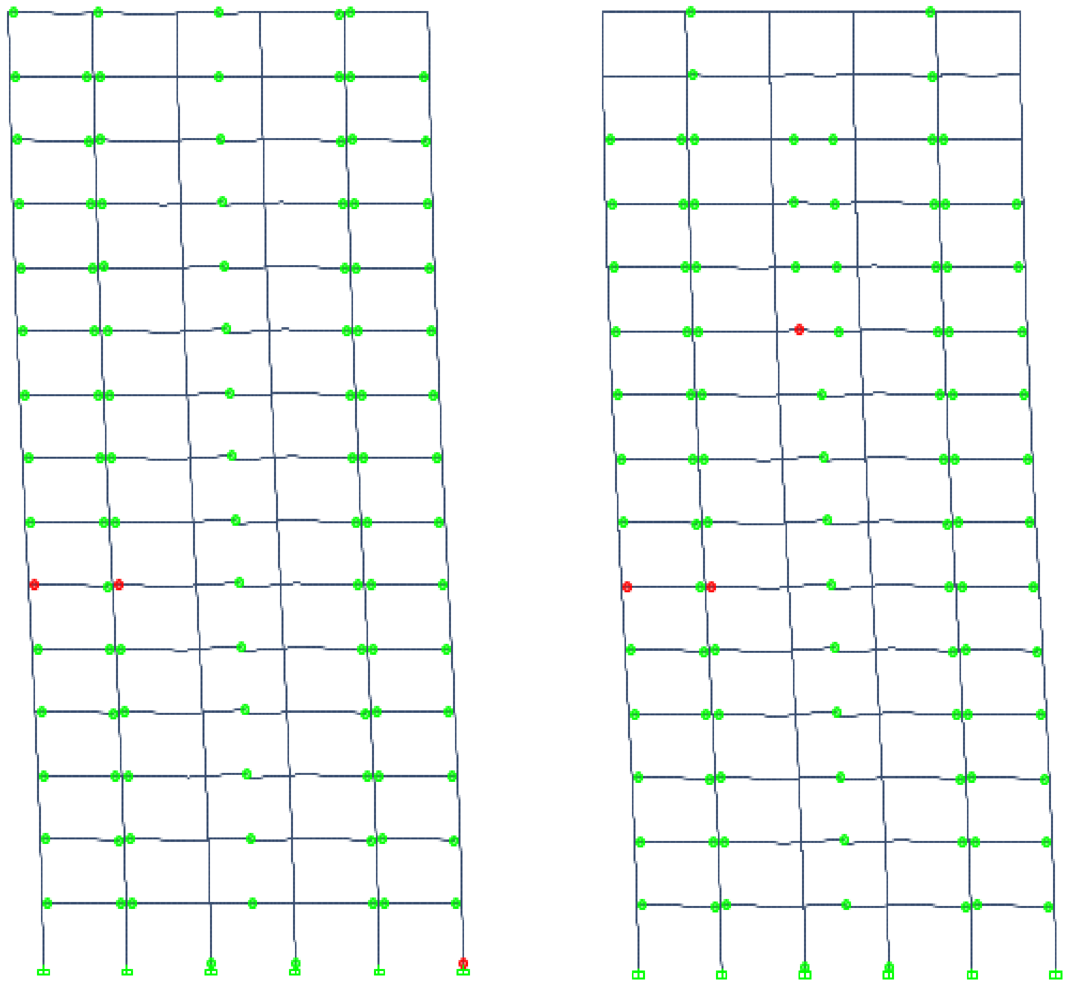

Figure 4.6 *Hinge pattern in models with steel coupling beam with fuse and DRCCB*

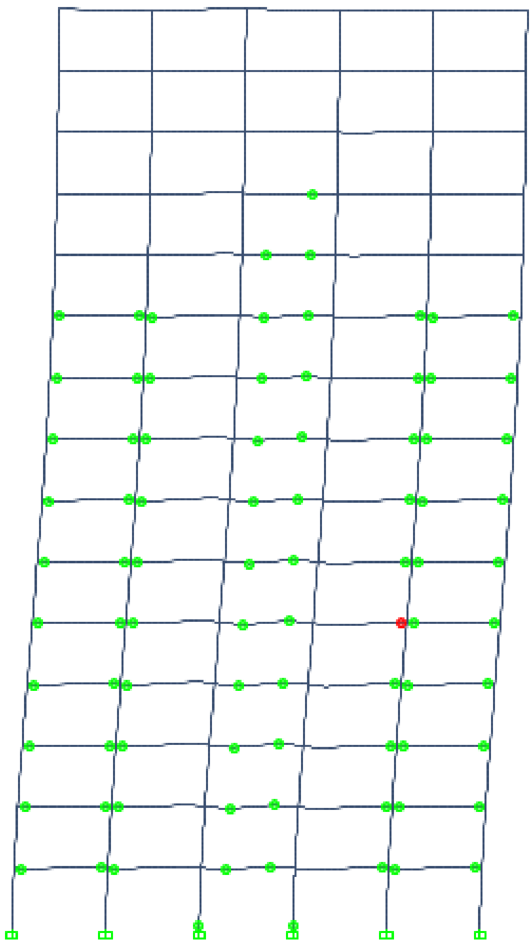

Figure 4.7 *Hinge pattern in model with steel coupling beam*

4.7 Performance Evaluation using Displacement Modification Method (ASCE-41)

In order to determine earthquake induced displacement the displacement modification method has been used. According to the displacement to the displacement modification method the target displacement is calculated as given below

$$\Delta_t = \frac{C_0 C_1 C_2 S_a T_e^2 g}{4\pi^2} \tag{4.1}$$

Where, C_0 is a factor to relate spectral displacement of single degree of freedom system to spectral displacement of multi degree of freedom system.

C_1 is a factor to relate spectral displacement of single degree of freedom system to spectral displacement of multi degree of freedom system.

For periods greater than 1.0 sec C_1 is to be taken as 1 and for periods less than 0.2 sec, C_1 is given by equation 4.3. C_1 need not be taken greater than the value at T= 0.2sec.

$$C_1 = 1 + \frac{R - 1}{a T_e^2} \tag{4.2}$$

Where, a = A Factor for Site Class

T_e= Effective period of the structure for direction under consideration in seconds.

R= Ratio of elastic strength demand to yield demand coefficient calculated in accordance with equation 4.3.

C_2= A factor to consider the effect of pinched hysteresis loop, strength and stiffness degradation. For periods greater than 0.7sec C_2 is to be taken as 1.0

$$C_2 = 1 + \frac{1}{800} \frac{(R - 1)^2}{T_e^2} \tag{4.3}$$

S_a= Spectral acceleration, at the effective fundamental period and the damping ratio of the building in the direction under consideration

g= Acceleration due to gravity

The strength ratio R should be determined as

$$R = \frac{S_a}{V_y / W} . C_m \tag{4.4}$$

Where, V_y= Yield strength calculated by converting the actual force deformation curve into an equivalent bilinear curve

W= Seismic Weight of the Building

C_m = Effective modal mass participation factor calculated for fundamental mode using the Eigen value analysis. C_m shall be taken as 1 if fundamental period T, is greater than unity.

The Table 4.7 shows the capacity curve parameters converted into equivalent bilinear curves such that the area under the two curves is same and initial straight line should intersect the original capacity curve at 0.6V_y.

Table 4.7 *Capacity curve parameters*

Model Description	Yield Base shear (kN)	Ultimate Base shear (kN)	Yield Displacement (mm)	Ultimate Displacement (mm)	Ductility Capacity (μ)	Over strength (Ω)
Model with DRCCB	17597	18064	234.7	714.5	3.04	1.89
Model with SB	17373	17476	244.2	641.7	2.63	1.89
Model with SB Fuse	15917	17042	211.7	764.6	3.61	1.73

Table 4.8 *Performance Levels for Seismic Hazard levels for DBE and MCE*

Model Description	Performance point for DBE (mm)	Performance Level (DBE)	Performance point for MCE (mm)	Performance point (MCE)
Model with DRCCB	149.7	IO	299.5	LS
Model with SB	143.8	IO	287.5	LS
Model with SB Fuse	151.1	IO	302.3	LS

Chapter 5

CONCLUSIONS

The behavior of different types of coupling beams has been studied in this dissertation work. A study of available literature on design of coupled shear walls has been performed. A comparative study about various code provisions for design of coupled shear wall buildings has also been performed. A numerical study has been performed on a 15 storey representative coupled shear wall building with different types of coupling beams viz. diagonally reinforced coupling beams, steel beams and steel beams with fuses. The performance of the buildings has been evaluated using non-linear static procedure. Following are the main conclusions of the study:

1. A step–by-step design procedure has been proposed for design of coupled shear wall buildings. It is based on the philosophy that coupling beams should yield before the walls yield.
2. Coupling Ratio (CR) is an important parameter in behavior of coupled wall system, due emphasis should be given to selection of appropriate value of CR in design.
3. The displacement capacity of the example building has been found to be dependent on ductility capacities of coupling beams, walls and frame members. The ductility of models with steel beam and steel beam with fuse, is governed by the ductility of frame, whereas in case of model with DRCCB, it is governed by the ductility capacity of coupling beams.
4. The ductility capacity obtained for the coupled shear wall models with steel coupling beams, steel coupling beams with fuses and DRCCB are 2.63, 3.65 and 3.04, respectively. The model with steel coupling beams has lower ductility since the hinge formation is in flexure whereas the model having steel coupling beams with fuses, exhibit higher ductility as expected due to yielding in shear.
5. The overstrength of the different models varies from 1.73 to 1.89. The model with DRCCB has higher overstrength because of higher factor of safety in design for RC members as compared to steel beams.

6. The performance of all the considered building models, evaluated using displacement modification method is IO for DBE hazard level and LS for MCE hazard level.

REFERENCES

1. ASCE (2013)." Seismic Rehabilitation of Existing Buildings." *American Society of Civil Engineers.* ASCE/SEI 41-13, Reston, VA.

2. ATC (1996). "ATC-40: Seismic Evaluation and Retrofit of Concrete Buildings," Volume 1, Applied Technology Council, Redwood City, California, U.S.A.

3. BIS (1987a). "IS: 875 (Part 1)-1987-Indian Standard Code of practice for design Loads (other than Earthquake) for Buildings and Structures, Part 1: Dead Loads-Unit Weights of Building materials and Stored Materials (Second Revision)." Bureau of Indian Standards,New Delhi.

4. BIS (1987b). "IS: 875 (Part 2)-1987-Indian Standard Code of practice for design Loads (other than Earthquake) for Buildings and Structures, Part 2: Imposed Loads (Second Revision)." Bureau of Indian Standards,New Delhi.

5. BIS (2000). "IS: 456-2000-Indian Standard Plain and Reinforced Concrete Forces-Code of Practice (Fourth Revision)." *Bureau of Indian Standards,New Delhi.*

6. BIS (2014)."IS:13920-2014-Indian Standard Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces-Codes of Practice." Bureau of Indian Standards, New Delhi (DRAFT).

7. Chen, Yun, and Xilin Lu (2012). "New Replaceable Coupling Beams for Shear Wall Structures." *15th World Conference on Earthquake Engineering*, Lisbon, 2583: 24-28.

8. El-Tawil, Sherif, and Kent Harries (2007). "Recommendations for Seismic Design of Hybrid Coupled Walls." *Structural Engineering Research Frontiers*: 1–10.

9. Hindi, Riyadh A., and Midhat A. Hassan (2004). "Shear capacity of diagonally reinforced coupling beams." *Engineering structures* 26.10: 1437-1446.

10. Liao, W., and Goel, S(2014). "Performance-Based Seismic Design of RC SMF Using Target Drift and Yield Mechanism as Performance Criteria." *Advances in Structural Engineering* 17(4): 529–42.

11. Moehle, J.P, Hooper,J.D., Fields,D.C., and Gedhada, R. (2012). "Seismic Design of Cast-in-Place Concrete Special Structural Walls and Coupling Beams A Guide for Practicing Engineers." *NEHRP Seismic Design Technical Brief* (6).

12. PEER/ATC (2010). "Modeling and acceptance criteria for seismic design and analysis of tall buildings (PEER/ATC- 72-1)", Applied Technology Council in cooperation with the Pacific Earthquake Engineering Research Center. Redwood City, CA: PEER.

13. Paulay, T and Priestley, M.J.N.(1992). *Seismic Design of Reinforced Concrete and Masonry Buildings*.

14. Standard, British (2005). "Eurocode 8: Design of structures for earthquake resistance-Part 1: General rules, seismic actions and rules for buildings." BS EN 1998-1:2004, Management centre, Brussels.