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Seismic Performance Evaluation of Hybrid Coupled Shear Walls in High Rise Buildings

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ABSTRACT: ***The efficiency of coupled shear walls in high rise buildings is the key factor for the performance of overall structure during seismic hazards. The coupling beams in a coupled shear wall dissipate seismic energy and act as a fuse between two shear walls and attain maximum damage in earthquake. The objective of this research is to observe the non-linear analytical behaviour of coupling beams by using ETABs and to select appropriate section from conventional reinforced concrete coupling beams, steel link beams and composite steel – concrete coupling beams in a high seismic zone. From the non-linear static analysis it isclearly observed that coupling beams behaves as a shear dominated beams rather than flexure. The steel link coupling beams deformed in a more ductile way as compared to the RC coupling beams and composite steel-concrete coupling beams. The concept of Braced Coupled Shear walls is also presented in this comparative study***

Keywords: RC Coupling Beams, Steel links, steel-concrete composite link, PBD Performance Based Design, non - linear pushover analysis, plastic hinges.

I. INTRODUCTION

In this modern era of construction technology, high rise buildings are the only option left for structure engineers due to less ground space available for construction. For building High Rise Structures, designers needs to analyse various lateral load resisting systems and their effect on the overall performance of the structure during seismic events and high wind guests. The buildings with core shear wall are always being the functional choice for resisting the torsional effects and overall deflection of the building. In thisresearch, the behaviour of Special Moment Resisting Frame (SMRF) with Core Shear Wall is taken into design consideration. The flexural deformation of moment resisting frame (MRF) and shear wall is relatively opposite which in the combination behave as a perfect sway resisting mechanism (Bungale S. Taranath). The large stiffness of these core shear wall reduces the response factors i.e. Maximum Storey Displacement, Storey Drift and allows the building to not exceed the displacement limits and storey drifts in the codalprovisions. But due to architectural limitations, the door openings to be assigned for vertical functionality of the building as a lift service continues till the overall height of the core shear wall reduces the relative stiffness of the core shear wall. Hence core shear walls are always constructed as coupled shear walls with coupling beams.

The entire shear wall is divided into two wall piers joined by coupling beams which can be conventional RC Coupling Beams, Steel links or composite link coupling beams. These coupling beams undergo large inelastic deformation or can be called the deformation controlled elements in performance bases designs. The performance of these coupling beams must be checked with non-linear analysis. For static non-linear analysis, Pushover Analysis can be performed. Non Linear Dynamic analysis can be performed using Non-Linear Time History Analysis.

The linear elastic response spectrum analysis can't be a right choice for observing the performance of these coupling beams due to high shear demands and large axial forces distributed by the walls piers from the ground acceleration.

1.1 Characteristics & Benefits

The coupling effect revealed these main benefits (Pauley & Priestly, 1992):

- The main function of coupling beams is to transfer the high shear from one wall pier to the joined wall pier. Many Coupling beams are designed as flexural members with shear confinement which further leads to diagonal tension failure. To overcome the diagonal tension failure, diagonal bars are provided with proper confinements which are either in compression or in tension over the full length.
- These coupling beams are the primary source of dissipating seismic energy and are known for the large inelastic deformations or deformation controlled elements.
- The span to depth ratio of these coupling beams is relatively low hence their performance is largely affected by high shear forces.

(Khatami & Zehrai, 2011) conducted non-linear analysis on the behaviour of steel link beams, composite link beams in coupled shear wall and RC coupling beams and found composite link beams asmuch more efficient source of energy dissipation over RC coupling beams and steel link beams. (Harries et al. 2000) shows importance of composite coupling beam designed with built up steel shapes and so called hybrid coupled shear wall which can be preferred in high seismic zones. He included the effect of embedded length of composite beam in wall piers in the performance of the coupled shear wall. (Chairunnisa et al. 2016) brings the idea of steel truss coupling beams and detailed the

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connection difficulties for coupling beams and choose to design coupling beams as shell elements to obtain the design forces and concluded shell-frame elements are efficient and appropriate for the non-linear analysis of coupled shear wall. But later on (Nofal & Elsayed, 2017) depicts the shell elements gives the accurate design forces. (Bhunia et al. 2003) conducted a large study on behaviour of coupled shear walls under non-linear static analysis and discussed completely about the type of reinforcement used in coupling beams and its ductile deformation analytically and found behaviour of coupling beams should be governed by shear while following the standards from FEMA 356 and ATC 40 and concluded that base of coupled shear wall can be a pinned restraint because it shows better non-linear behaviour rather than fixed base conditions.

The depth of coupling beams is always being a frequently discussed parameter in previous researches as depth of coupling beam decides the angle of inclination (α) of diagonal reinforced bars (Pauley & Priestly, 1992). If the angle of inclination is low the diagonal reinforcement can't be able to transfer large axial forces into wall piers adequately and remains in flexure which is not the relevant behaviour of coupling beams. In conventional RC Coupling beams, the top and bottom rebar undergoes tension or compression simultaneously and leads to the diagonal cracks. According to IS13920:2016; if $\tau ve > 0.1\sqrt{fck}$ (Ls/D) then diagonal reinforcement is recommended to resist earthquake produced shear in coupling beams. However guidelines provided by Indian Codes are very limited for the practical behaviour of coupling beams and there is no any description provided for the performance based designs (PBD). An updated context is needed in Indian standards regarding performance based designs.

As per FEMA 356, ATC 40 coupling beams are dominated by shear rather than flexure if $\phi \leq 2$ *or* $Lb/Db \leq 4$. The plastic moment capacity and yield moment capacity of the coupling beams must be kept low such that the rotations in coupling beams are greater than the plastic moment capacity so that coupling beams can dissipate much seismic energy.

1.2 Diagonally Reinforced Coupling Beams

The coupling beams having such a low span to depth ratio smaller or equalto 2.0 can be designed with diagonal reinforcement for fulfilling the demand of high shear forces and sustaining the diagonal tension and compression distributed by the two joined wall piers. The diagonal reinforcement intersects at mid span with no moment resistance. While in conventional reinforced coupling beams the compression forces are transferred by concrete whereas longitudinal reinforcement doesn't pay any significant role. Due to large axial forces tension reinforcement yield and cracks are developed and further increase in axial forces leads to the overall failure of the coupling beams. Hence diagonal reinforcement is effective in high seismic zone and for low span to depth ratio coupling beams.

1.3 Steel Link Coupling Beams

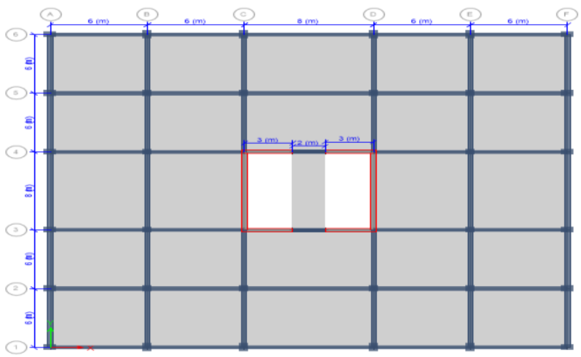
Steel link coupling beams are designed with I-Section which is embedded into the wall piers up to a desired depth of embedment for adequate shear capacity. Shear studs are designed with suitable depth for better bonding with concrete. But for the better flexibility the embedment length is kept low. If the embedment length is considered large then damage to the joined wall is much during seismic event. It was observed during analysis that plastic hinges are formed in the base of the coupled shear wall. The steel link beams are designed with low depths and meets the demands of architectural aspects for opening sizes very well. Also the stiffness provided by steel link beams is greater than conventional reinforced coupling beams as per the extensive research work done previously on steel link beams.

1.4 Composite Steel – Concrete Beams

The benefits of composite steel – concrete coupling beams is observed in resisting the web and flange buckling of the steel encasement. When composite steel – concrete coupling beams are provided with reinforcing bars the web and flange thickness can be kept minimum without using web stiffeners. The hysteretic response of steel – concrete composite couplings is very stable. The connection strength between wall and coupling beams is generally dependent on the reinforcement of wall boundary element, if the wall boundary element is conventionally reinforced it may not develop adequate strength but if the boundary element is reinforced with steel column having reinforcing bars around and proper confinement then the whole composition behaves very well and damage to the boundary element can be permitted inside permissible limits and similarly with the coupling beams.

II. MODELLING PROCEDURE

A regular geometry for a commercial office building (G+20) Special Moment Resisting Frame (SMRF) with Coupled Shear Walls in the core situated in seismic Zone V having zone factor 0.36 and soil type medium stiff and Importance Factor (I) 1.5is modelled by using ETABs 2015 which is a modern structural analysis software package used worldwide for performing both linear and non-linear, static as well as dynamic analysis. The methodology adopted in this research is Pushover Analysis which is a non-linear static method of seismic analysis and comes under performance based design procedure.



PLAN

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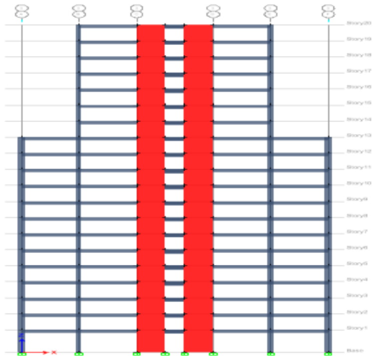

ELEVATION

Figure 1: Configuration of Analytical Model

Table 1: Dimensional Properties of Various Structural Members

Member Dimension	Unit : mm
Periphery Columns	600 x 800
Inner Columns	600 x 600
Walls: W1, W2	300
RC Coupling Beams: LB1	300 x 1200
Steel- Concrete Composite Link Beams: LB2	350 x 850
Steel Links Coupling Beams LB3	ISMB 600
Braces	ISMB 500
RC Beams	300 x 650
RC Slab	150
Depth of the wall (Lw)	3200
Length of Coupling beams(Lb)	2000
Bottom Storey Height(GF)	4500
Storey Height (N)	3200

Gravity loading given by IS875 (Part 1) for selfweight of structural elements and imposed loads on floor according to IS875 (Part 2) is followed appropriately. A floor finish load of 1.5KN/m² and imposed load of 3KN/m² is distributed uniformly on RC Slabs and wall load of 6.5KN/m (AAC Blocks 300mm thickness having density 550-650 kg/m³) is applied on beams. Also live load reductions are included according to the IS 875 (Part 2). But this research is mainly focused on the behaviour of coupling beams only so the rest of structural elements is considerd elastic throughout the analysis procedure. The concrete (M40 grade) and steel (Fe500) were considerd for columns and (M30 Grade) for beams for achieving the strong column weak beam effect. The poisson's ratio is taken 2.0 of concrete with unit weight 25KN/m³ and poisson ratio of steel is 3.0 with unit weight 78.5KN/m³. The role of stiffness modifiers is absent from Indian and International codes for coupling beams. Only few International Standard Construction Codes provide stiffness modifiers in case of coupling beams given in Table 2

Table 2: Recommended Stiffness Modiefiers according to various International Codes

Member	IS1893:2016	NZS 3101: Part 2 : 2016 Ultimate Limit State	NZS 3101 Part2 : 2016 Serviciability Limit State	FEMA 356	NBC2004	Euro Code
Coupling beams (L / H ≤ 4)	n/a	0.60Ig(diagonally reinforced)	0.75Ig	n/a	0.4Ig (without diagonal rein forcement) 0.25Ig (diagonally reinforced)	0.5Ig

Although Indian Standards IS1893:2016 and IS16700:2016 has given some provisions related to the stiffness modifiers for cracked section for beam 0.35Ig, columns 0.7Ig, slabs 0.25Ig and walls 0.7Ig but still missing the exact criteria for designing coupled shear walls. The results of response spectrum analysis observed can,t be the right choice for observing the behaviour of coupling beams in coupled shear walls.

2.1 Pushover Analysis (Non Linear Static)

Pushover analysis are performed by assigning non linear plastic hinges which can be moment rotaion or moment – curvature hinges or can be user defined. In these analysis more priority is given to the shear deformation controlled hinges because coupling beam are shear dominant elements so assigning moment rotation or moment curvature hinges is not appropriate choice, although reinforced coupling beams are analysed for flexure controlled hinges and acceptance criteria is checked with accordance to FEMA356 and ATC40 given in Table 3. ETAB's itself calculates target displacement according to the FEMA356 as per the overall height of the building

Table 3: Acceptance Criteria Given by FEMA356 for Non Linear Flexure Contolled Elements

Codes	Type of Coupling Beam	Condition Shear/$t_w l_w \sqrt{f_c'}$	Plastic Rotation Capacity (Radians)		
			IO	LS	CP
FEMA 356	Conventional longitudnal reinforced with conforming transverse reinforcement	≤3	0.010	0.020	0.025
		≥6	0.005	0.010	0.015
	Diagonal Reinforcement	n/a	0.006	0.018	0.030
	Shear dominant steel coupling beam	-	0.005	0.11	0.14
ATC40	Conventional longitudnal reinforced with conforming transverse reinforcement	≤3	0.006	0.015	0.025
		≥6	0.005	0.010	0.015
	Diagonal Reinforcement	n/a	0.006	0.018	0.030

III. PERFORMANCE EVALUATION

Performance evaluation is done on the basis of plastic hinge behaviour. The backbone curve obtained from the non linear static analysis for different types of coupling beam sections are obtained and it is observed that the plastic hinges assigned to conventional RC Coupling are gone to the collapse prevention (CP) state as shown in fig.3. which is the least damage state considerd in non linear analyis. The displacement found in coupled shear walls for steel link beam coupled shear wall is 12.35cm, 12.42cm for composite link beams and 18.34cm for conventional reinforced concrete coupling beams. In this research only the inelastic deformation of coupling beams and coupled walls are taken into consideration and rest of the srructural elements are considerd elastic throughout the analysis.

The backbone curve show the exact non-linear behaviour of a hinge and shows the strength and deformation relationship

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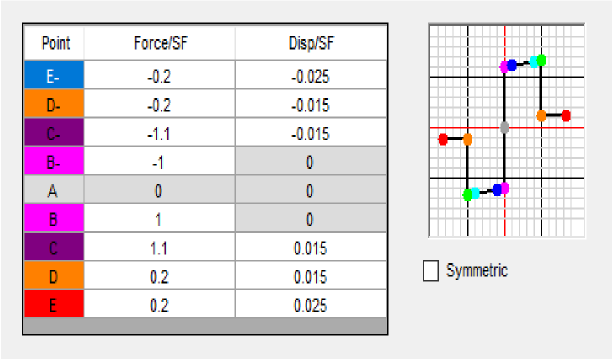
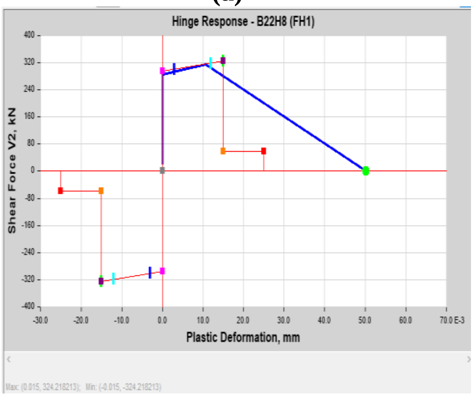


Point	Force/SF	Disp/SF
E-	-0.2	-0.025
D-	-0.2	-0.015
C-	-1.1	-0.015
B-	-1	0
A	0	0
B	1	0
C	1.1	0.015
D	0.2	0.015
E	0.2	0.025

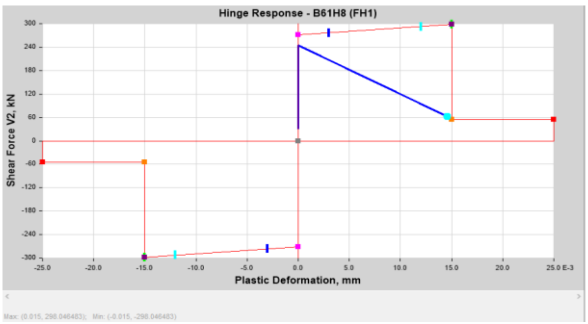
(a)



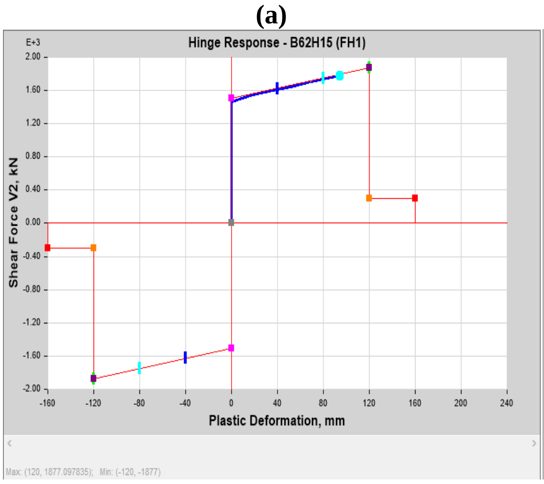
(b)

Figure 2: Backbone Curve of Plastic Hinge Formed in Convention RC COUPLING beam (a) Acceptance Criteria for Hinge (b) Hinge Response

Whereas a composite steel-concrete link beam performs better for the same storey's coupling beam, plastic hinges assigned remains in Life Safety (LS) state for the relatively equivalent amount of shear force as shown in figure



(a)

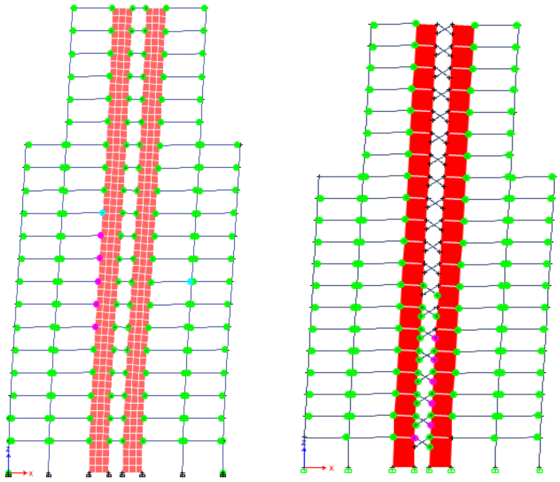


(b)

Figure 3: (a) Hinge Response for Steel – Concrete Composite Link Coupling Beam

(b) Hinge Response for Steel Link Coupling Beams

The steel link coupling beam analysis gives the most ductile behaviour over rest of the coupling beam section used in non-linear static analysis for a high shear (V2) of 1877KN and attains a plastic deformation of 120mm and remains in Life Safety (LS) while conventionally reinforced concrete beams failed the acceptance criteria according to FEMA356 and ATC40. So in storeys having high shear forces which fails the plastic hinge mechanism formed in coupling beams can be designed a steel link having a composite column as a boundary element.



(a) (b)

Figure 4:(a) Plastic Hinges Formation in Steel Link Coupling Beam (b) Plastic Hinges Formation in Braced Coupled Shear Wall

Table 4: Axial Forces in Braces from Analysis of Braced Coupled Shear Walls

Story	P	U1 Plastic	Hinge State	Hinge Status
	(KN)	(mm)		
Story1	-102.8506	-0.05	D to E	A to IO
Story1	6006.2055	2.976	B to C	A to IO
Story2	-99.0239	-0.07	D to E	A to IO
Story2	6082.1908	4.325	B to C	A to IO
Story3	-92.0553	-0.07	D to E	A to IO
Story3	6095.5056	5.229	B to C	A to IO
Story4	-87.5738	-0.07	D to E	A to IO
Story4	6098.612	5.031	B to C	A to IO
Story5	-82.6628	-0.05	D to E	A to IO
Story5	6079.5336	4.216	B to C	A to IO
Story6	-77.6982	-0.03	D to E	A to IO
Story6	6030.7755	2.794	B to C	A to IO
Story7	-72.6792	-0.009	B to C	A to IO
Story7	5935.611	0.771	B to C	A to IO

Note – All negative forces are tensile forces and positive are compressive.

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Table 5: Hinge State of Composite Steel- Concrete Link Beam

Story	V2	U2 Plastic	Hinge State	Hinge Status
	(KN)	(mm)		
Story1	193.0447	0.038	A to B	A to IO
Story2	277.2511	0.044	D to E	LS to CP
Story3	247.0322	0.042	A to B	A to IO
Story4	284.7153	0.05	>E	LS to CP
Story5	227.915	0.04	A to B	A to IO
Story6	206.4501	0.039	A to B	A to IO
Story7	292.5610	0.05	D to E	LS to CP

Table 6: Hinge State of Steel Link Coupling Beams

Story	V2	U2 Plastic	Hinge State	Hinge Status
	KN	mm		
Story1	1579.3454	29.595	B to C	A to IO
Story2	1644.4489	50.831	B to C	A to IO
Story3	1692.2555	66.17	B to C	A to IO
Story4	1760.2232	84.396	B to C	IO to LS
Story5	1786.1065	92.695	B to C	IO to LS
Story6	1804.621	96.833	B to C	IO to LS
Story7	1810.8725	98.832	B to C	IO to LS

IV. RESULTS AND CONCLUSIONS

- Through a vast hinges response recorded from non - linear static analysis it was observed that assignment of plastic hinges in coupled shear walls exhibits important properties of coupling beams and overall behaviour.
- Steel link beams shown adequate strength and ductility over concrete link beams and huge amount of shear is observed by the steel links beams. The shear controlled plastic hinges assigned to conventional RC beams was found in Collapse prevention state which is considered most damageable state in performance based design (PBD) whereas composite steel-concrete coupling beams remains in Life Safety (LS) and steel link beams performed enormous shear absorption but still plastic hinges formed gone to the Intermediate Occupancy (IO) which means core shear wall can be operated during seismic hazards and not in case of RC coupling beams.
- The elastic analysis of coupled shear walls can't be adopted because coupling beams undergoes large inelastic deformations in designing coupled shear walls. Braced coupled shear walls also can be a better option in high seismic zone as the diagonal braces distributed maximum axial forces from one wall pier to the joined wall pier by means diagonal braces.

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