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A STUDY ON EFFECT OF SHEAR WALL STREGTHENING ON OPEN GROUND STOREY FRAMED BUILDINGS

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Abstract: Due to overwhelming population rise in the urban locations and an exponential increase in usage of vehicles, the need for parking spaces has risen manifold. Parking spaces were usually provided in open but due to occupation of the open lands by human settlements and lack of open spaces, the role of open ground storey (OGS) framed buildings is irreplaceable. An OGS building has no walls on the ground floor; a structure built to cater the needs of a parking facility just below an Institution or Organization. OGS has an advantage that you can utilize the same space as a parking site. OGS helps in effective space management and utilization for manyfold purposes and has proved to be a great advantage in a time when open space is scarcely available. Usually the earthquakes affect such buildings at large. In this study, we dealt with strengthening of OGS buildings using shear wall to sustain the shocks, taking minimum damage. As a result of strengthening the OGS frame with shear wall, we observed an increase in shear capacity of the OGS frames. A four storey building was analyzed in this research work using Kent and Park model. Pushover Analysis is done for the nonlinear response calculation in accordance with IS1893(2002). In this study we found that on strengthening the OGS frame with shear wall, the shear capacity of the frame increased drastically. The cost of construction falls towards the costlier side but the safety is greater after the use of shear wall in strengthening.

***Index Terms* – Open Ground Storey, Shear Wall, Strengthening**

I. INTRODUCTION

The idea of open ground storey (OGS) building has been introduced mainly because of the need for parking in urban localities. Due to the special feature of providing parking facility in the ground storey of this building, a large number of open ground storey buildings have been built and accommodated especially for residential purposes throughout the different cities of the country. In actual sense, when the columns of a reinforced concrete building are left open without providing any masonry infill wall as partition wall in between them to have parking area in the ground storey then this type of structure can be treated as open ground storey or soft storey building. The most important issue can be verified that the ground storey is quite flexible in nature in comparisons to the other upper storeys of this building. This literally means that the relative storey drift of the ground storey is quite larger with respect to the other upper storeys of such buildings when subjected to earthquake loads. Consequently, the ground storey is exceptionally weak against other upper storeys to resist large earthquake forces usually present at the ground storey of the building. The study after Bhuj earthquake happened in 2001 at Ahemdabad has explicitly mentioned that open ground storey building is unsafe and highly vulnerable to earthquake shaking. Due to the presence of masonry infill wall in upper storeys made them much stiffer than the ground storey. Therefore, it creates difference of stiffness between the ground storey and upper storeys of open ground storey building. Thus, the horizontal drift of ground storey is quite large relatively and the upper storeys of this building displaces like a single block. Subsequently, if the columns of ground storey are not strong enough to resist large horizontal loads like earthquake forces and are not provided adequate ductility then they may get highly damaged which may lead to the catastrophic collapse of such buildings.

II. LITERATURE REVIEW

2.1 BASIC PHILOSOPHY

Murty and Jain (2000) conducted experiments on RC frames with masonry infill based on cyclic tests. It was observed that masonry infill provides significant lateral stiffness, energy dissipation capacity and ductility. With the help of some arrangement by providing reinforcement in the masonry infill, it was anchored into the column of the frame to improve effectively the out of plane response of masonry infill.

Davis et al. (2004) studied the seismic performance of two typically existing buildings situated in moderate seismic zones of India by performing linear static analysis, response spectrum analysis and nonlinear pushover analysis. In one building irregularity in plan and vertical irregularity like soft storey were found and another building was symmetric in nature. The equivalent strut method was used to modelled infill walls.

Kaushik et al. (2007) conducted experiments on unreinforced masonry infill for obtaining compressive stress- strain behaviour. Nonlinear stress-strain curves had been obtained for bricks, masonry, mortar six control points had been plotted on the stress-strain curves of masonry, which were used to define the performance limit states of the masonry infill.

Pujol et al. (2008) tested a full scale three-storey structure having infill brick walls under displacement reversals. Results were compared of this test with the results of the same building without having infill walls. In the first test, at the slab-column junction, structure showed a punching shear failure. Infill walls prevented the slab collapse and effectively increased the strength and stiffness of the structure. The experimental results were calibrated to match the numerical model of the test structure. Numerical simulations suggested that the measured drift capacity was not reached even during strong motion.

Mulgund (2011) designed five RC frame buildings with masonry infill walls as per IS code in order to consider the effect of masonry infill under same seismic condition because while designing of RC frame buildings usually do not consider the effect of masonry infill. The present work dealt with a study of RC frames subjected to dynamic loading with different arrangement of masonry infill walls. The results were extracted and compared for both bare frame and bare frame with infill walls. Finally, conclusions were derived and put forward in accordance of with IS code.

Prakashvel et al. (2012) conducted a work to study the seismic bahaviour of soft storey or open ground storey building under seismic loading and their problems to make them earthquake resistant to check the catastrophic collapse of such buildings. An attempt had been made to evaluate the seismic performance of open ground storey buildings with the help of shake table test.

Lopes (2001) described a comprehensive test in order to study the seismic performance of reinforced concrete walls subjected to extreme conditions and a shear failure was observed. To conduct this experiment, a test setup was designed to impose beam behaviour and low shear ratio was maintained during the test had been described in this work. Finally, observations were made and some special features described that were failure mode dependent.

Rana et al. (2004) performed a nonlinear static analysis of a 19-storey reinforced concrete building with total area of 430,000 Sq ft. located in San Francisco. The building was typically designed as per 1997 Uniform Building Code with shear walls as a lateral resisting system to check the provisions and guidelines of the Life Safety performance level when subjected to design earthquake and results were presented in this work.

Martinelli et al. (2013) studied the capability of two distinctive of fiber beam-column finite elements to simulate the dynamic behaviour of a shear wall using shake table test.

III. METHODOLOGY

The methodology followed to accomplish the objectives are given below.

1. A broad literature review on the use of shear walls in the frame buildings.
2. Selection of typical 4-storey Open Ground Storey RC frames.
3. Linear static analysis of RC frames without considering the effect of masonry infill as per Indian Standard.
4. Designing of RC shear wall and ground storey and/or first storey columns of the four storey OGS building with various multiplication factors.
5. Modelling of the selected frame buildings to capture nonlinear behaviour.
6. Performance comparison of the buildings in terms of nonlinear static pushover curves.
7. Comparative cost analysis of each strengthening scheme.

3.1 Assumptions Made

Following are the assumptions of the present study.

1. The OGS frames are assumed to be symmetric in plan and hence plane frames are considered. Torsional effects are neglected.
2. The shear wall considered in the study is provided throughout the height of the building as studies [4] suggests the same.
3. The interaction effect of the soil-foundation structure is neglected.
4. Distributed plasticity element is employed for nonlinear modelling of RC members.

3.2 Models Involved in The Study for Nonlinear Behaviour

1. Mander et al. (1988) Concrete Model
2. Menegotto–Pinto Steel Model (1973)
3. Infill Wall Panel Element (Crisafulli, 1997)
4. Inelastic Force-Based Frame Element

3.3 Geometry and Modelling Used in The Work

All the necessary details of beams and columns are given here in Table 1 for the present work. For capturing nonlinear behaviour, Kent and Park model is employed for the modeling of beams and columns of the reinforced concrete frame. The lateral distribution of force for the nonlinear pushover analysis to compute R for each frame is suggested as per IS 1893 (2002).

Table 3.1 Necessary Details

Frame	Floor	Member	Width(mm)	Depth(mm)	Reinforcement Details
2-Storey	Beam	1–2	300	550	3–25φ + 2–20φ (top) +
					2–25φ + 1–20φ (bottom)
	Column	1–2	400	400	8–25φ (uniformly distributed)
5-Storey	Beam	1–4	350	650	6–25φ (top) + 3–25φ (bottom)
	Column	1–4	550	550	12–25φ (uniformly distributed)
7-Storey	Beam	1–4	350	650	6–25φ (top) + 3–25φ (bottom)
	Column	1–4	650	650	12–25φ (uniformly distributed)
	Beam	5–8	350	650	6–25φ (top) + 3–25φ (bottom)
	Column	5–8	550	550	12–25φ (uniformly distributed)

$$Q_i = V_d \times \frac{W h_i^2}{\sum_{i=1}^{n} W h_i^2} \quad \text{....... 1}$$

Where, Q_i is the equivalent lateral force on the i^{th} floor, W_i the seismic weight of the i th floor, h_i the height up to the i^{th} floor, and n is the total number of storeys.

3.4 Comparison of Pushover Curves

The comparison of nonlinear pushover curves has been made here. From the analysis, it can be seen that the pushover curves are much closer to the results of published literature taken here for the validation of modelling approaches used for the present study.

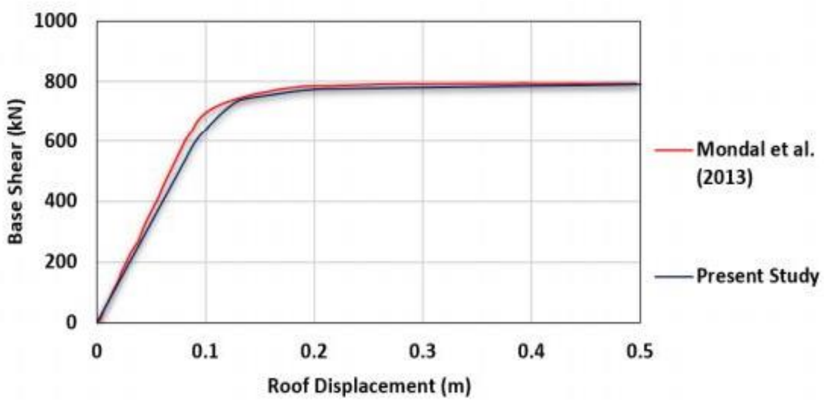

Fig. 3.1 Pushover Curves for Two Storey Frame

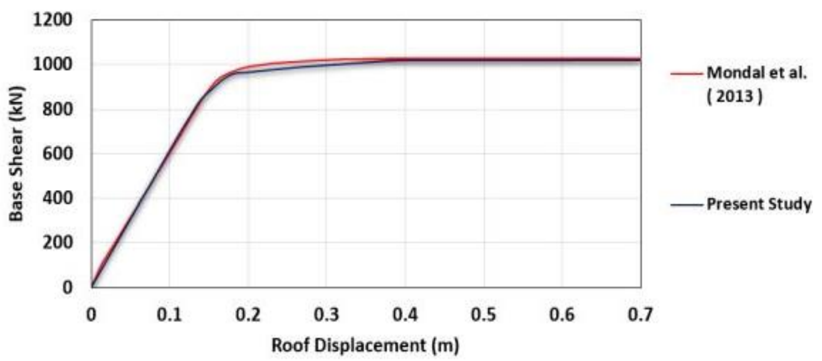

Fig. 3.2 Pushover Curves for Four Storey Frame

3.5 Geometry of The Frames Used

All the frames considered in the present study are explained below.

1. The elevation of bare frame, B 1.0 is provided in Fig. 3.3a
2. The elevation of fully infilled frame, F 1.0 in which the infill wall is considered in all the storeys is provided in Fig. 3.3b
3. The elevation of OGS frame, O 1.0 in which the the multiplication factor of 1.0 is used for the design of ground storey Columns, is provided in Fig. 3.3c
4. The elevation of OGS frame strengthened with shear wall, OS 1.0 is provided in Fig. 3.3d
5. The elevation of OGS frame re-designed with IS 13920 (1993) incorporating shear wall, OSR 1.0 is provided in Fig. 3.3e
6. The elevation of OGS frames designed with MF of 1.5, 2.0, 2.5, 3.0 in the ground storey alone are shown in Figs.3.4a to 3.4d

respectively.

7. The elevation of OGS frames designed with MF of 1.5, 2.0, 2.5, 3.0 in both the ground storey and first storey are shown in Figs.3.5a to 3.5d respectively.

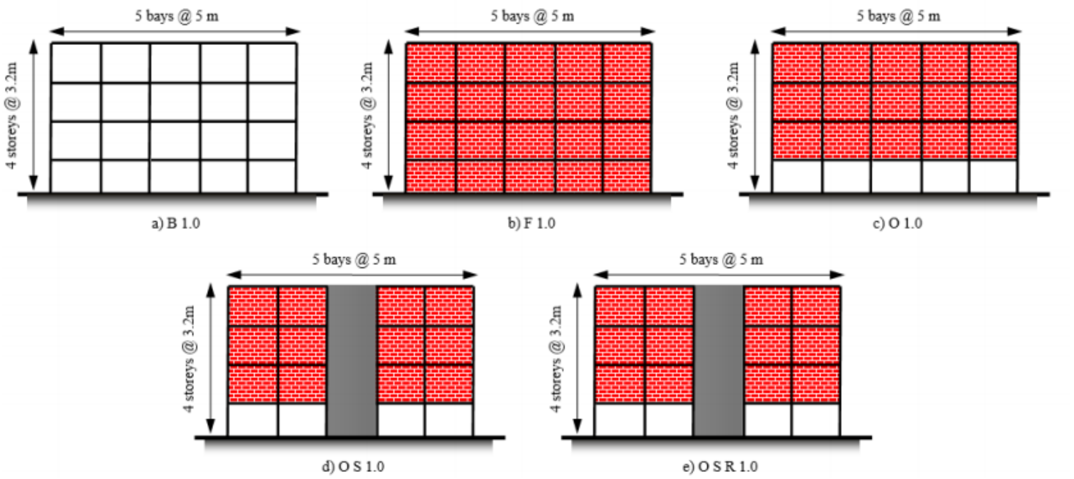

Fig 3.3 4-Storey RC Frames with One Multiplication Factor (Fundamental Models).

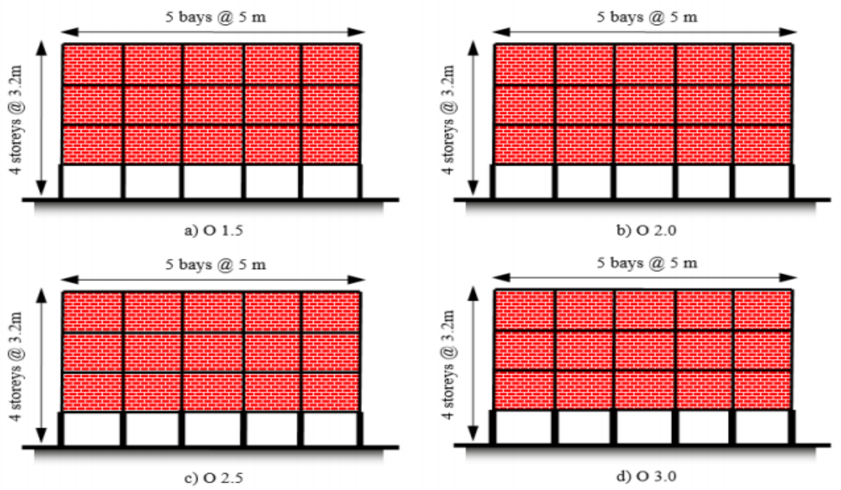

Fig. 3.4 4-Storey RC Frames with different Multiplication Factor for Ground Only.

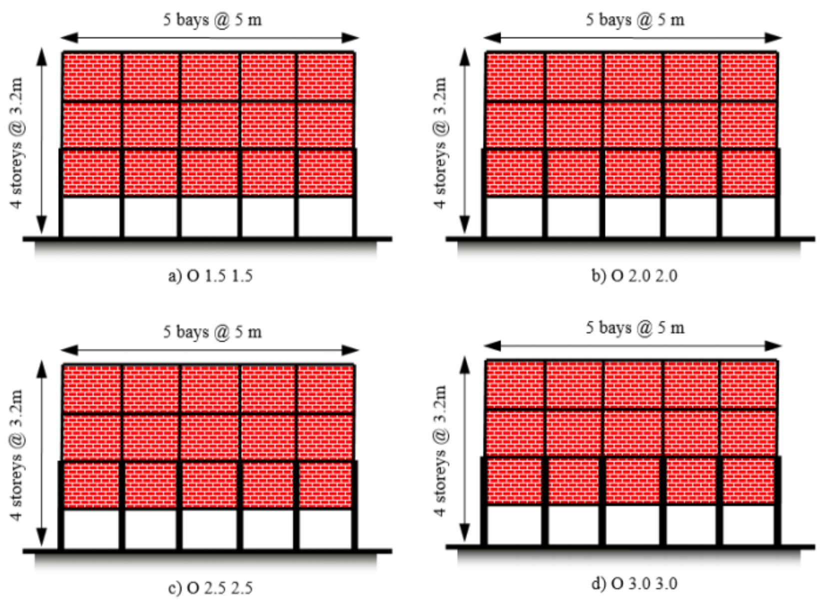

Fig. 3.5 4-Storey RC Frames with different Multiplication Factor for Ground and First Storey Only

3.6 Performance Criterion

In the era of performance-based engineering, it is foremost that investigators and engineers are equipped with identifying the instants at which different performance limit states (e.g. non-structural damage, structural damage, collapse) will be reached. The following Performance criteria are defined for the present work.

1. **Performance limit (PL 1)** - Yielding of steel is identified by checking for (positive) steel strains larger than the ratio between yield strength and modulus of elasticity of the steel material. A value of 0.0038 has been assigned for this parameter.

2. Performance limit (PL 2) - Spalling of cover concrete can be monitored by checking for (negative) cover concrete strains larger than the ultimate crushing strain of unconfined concrete material. At present, a value of -0.005 is taken.

3. Performance limit (PL 3) - Crushing of core concrete can be confirmed by checking for (negative) core concrete strains larger than the ultimate crushing strain of confined concrete material. Herein, a present value of -0.02 is adopted.

IV. RESULTS AND DISCUSSION

4.1 Performance Comparison

Pushover curves of all the frames are shown in Fig. 4. It can be seen that the base shear capacity of the frames OS 1.0, OSR 1.0 and F 1.0 are significantly higher than B 1.0 and O 1.0. The initial stiffness of frames OS 1.0, OSR 1.0 and F 1.0 are also higher than that of remaining frames.

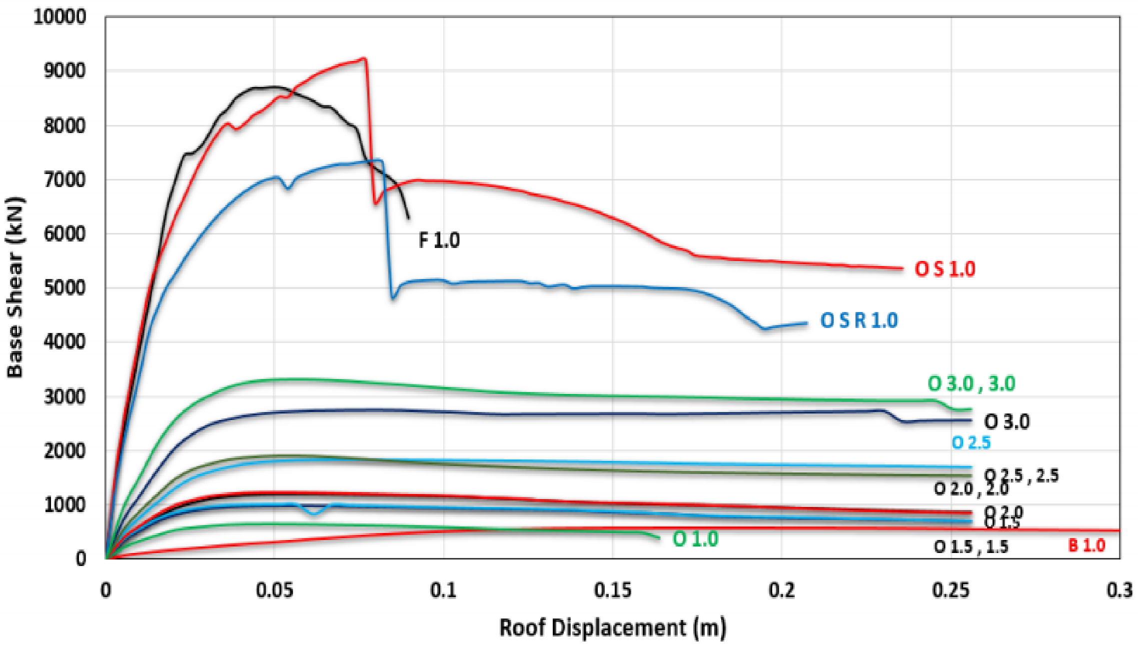

Fig. 4.1 Pushover curves of all the 4 Storey Frames

4.2 Maximum Base Shear Capacity and Lateral Displacement Capacity

The maximum base shear capacity and lateral displacement capacity of all the four storey frame models obtained from pushover analysis are tabulated in Table 2. Following are observations based on the maximum capacities.

1. After providing reinforced concrete shear wall to the open ground storey frame, the base shear capacity significantly increases by 92.85 % along with that lateral displacement capacity increases by 39.74 % under seismic loading.
2. With the application of various multiplication factor to the ground storey of an open ground storey frame ranging from 1.5 to 3.0 with an interval of 0.5 increment. The base shear capacity gradually increases from 34.15 % to 76.15% and the corresponding lateral displacement capacity improves from 9.80 % to 43.90 % respectively in a regular manner respectively.
3. By employing distinct multiplication factor to the ground and first storey both of an open ground storey frame running from 1.5,1.5 to 3.0,3.0 with an interval of 0.5,0.5 increment each. The base shear capacity gradually increases from 34.49 % to 80.14% relatively the lateral displacement capacity improves from 2.12 % to 17.85 % respectively in a regular manner.
4. In the course of present work, a bare frame has the lowest base shear capacity around 11.47 % and the highest lateral displacement capacity of 74.45 % with respect to open ground storey frame.
5. At the end, a full masonry infill wall frame has the base shear capacity of 92.45 % almost equivalent to the base shear capacity of the RC shear wall with the lateral displacement capacity of 4.17 % in opposite to an open ground storey frame.
6. While keeping constant the cross- section and reinforcement steel of shear wall and without altering the details of masonry infill wall and after slightly

Table 4.1 Maximum Base Shear Capacity and Lateral Displacement Capacity

Frame Model	Base Shear (kN)	% increase in Base Shear Capacity	Roof Displacement (m)	% increase in Roof Displacement Capacity
B 1.0	583.74	11.49	0.360	74.47
O 1.0	658.47	-	0.092	-
F 1.0	8703.62	93.81	0.098	4.34
O S 1.0	9209.37	92.59	0.037	39.95
O S R 1.0	7358.37	91.37	0.152	41.15
O 1.5	999.58	34.09	0.102	9.16
O 2.0	1205.15	46.93	0.112	17.18
O 2.5	1848.39	65.81	0.144	36.24
O 3.0	2755.71	77.33	0.164	43.18
O 1.5,1.5	1005.83	36.98	0.096	2.24
O 2.0,2.0	1218.39	47.48	0.102	9.26
O 2.5,2.5	1911.60	65.12	0.112	17.90
O 3.0,3.0	3309.38	81.28	0.112	17.63

4.3 Cost Analysis

Material and labour cost for each frame are computed. A ratio of maximum base shear to total cost for each frame is calculated. Details of the cost for each frame is provided in the Table 4.2.

Ratios of maximum base shear to total cost for each frame are presented in Fig. 4.1 to find out effective and economical frame under earthquake action. It can be seen that the ratio of more for F 1.0, which means that this frame is more economical. However, this frame may not serve the purpose of parking right in the ground storey of the building. Out of all other frames that may provide the parking space in the ground storey, OS 1.0 and OSR 1.0 are the most economical frames. From the cost analysis, it is found that the maximum base shear to cost ratio of OGS frames strengthened with shear wall is about 9 times more than that of OGS frame.

In case of OGS frame re-designed with shear wall, the ratio is about 8 times more than that of OGS frame. After strengthening Open Ground storey buildings by applying various schemes of multiplication factors in line with the approach proposed by IS 1893 (2002), the maximum base shear to cost ratio is only about 3 times than that of OGS frames.

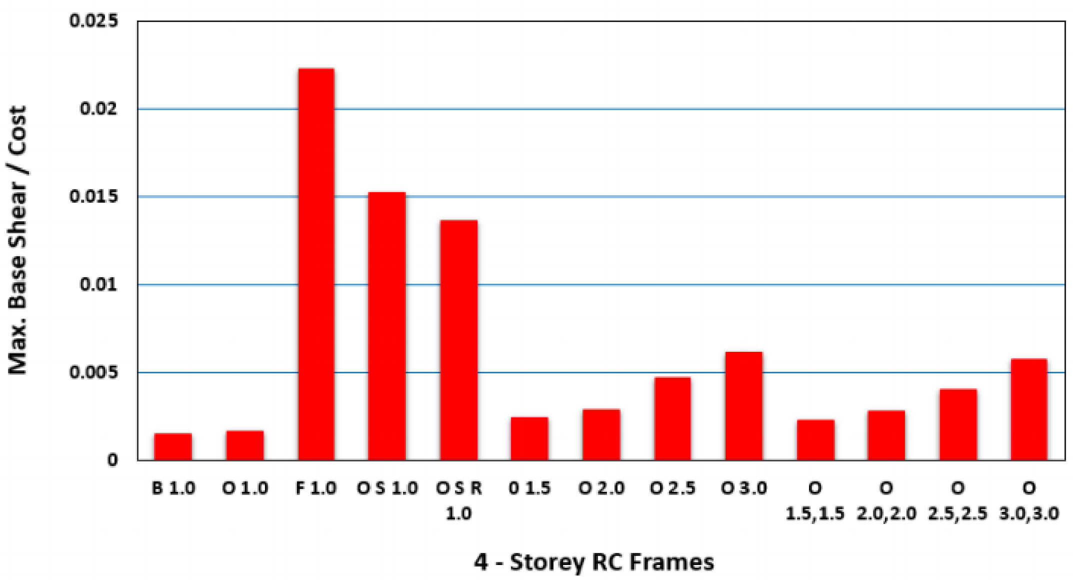

Fig 4.2 Comparison of Maximum base shear to cost ratio of all frames

Table 4.2 Cost analysis of RC frames with Shear wall and various MF

Frame Storey	Concrete (cu m)		Reinforcement Steel (Kg)		Total cost (Rs.)
	Column	Beam	Column	Beam	
B 1.0	9.454	10.43	1845.43	1584.76	389854
O 1.0	9.442	10.85	1848.32	1584.76	389854
F 1.0	9.426	10.84	1845.82	1584.76	389854
O S 1.0	9.408 + 11.942	8.054	4182.82	1260.42	608558
O S R 1.0	6.912 + 11.965	7.923	3693.67	1128.47	538175
O 1.5	10.514	10.043	2066.81	1581.48	413038
O 2.0	10.565	10.056	2066.84	1584.63	413021
O 2.5	11.396	8.056	2235.74	1256.50	393448
O 3.0	13.958	7.85	2742.55	1253.64	448750
O 1.5,1.5	11.659	10.23	2284.63	1574.87	436216
O 2.0,2.0	11.625	10.45	2284.30	1574.87	436249
O 2.5,2.5	13.390	9.95	2623.35	1561.36	472424
O 3.0,3.0	18.258	9.83	3584.76	1542.54	575126

The objective of the present study was to compare the performance of OGS buildings strengthened with shear walls. Two different cases of providing shear wall in the OGS frames are considered. In the first case the shear walls are provided as a strengthening option whereas in the second case, the OGS frame is re-designed with shear wall. The OGS frames with Shear walls are considered as strengthening schemes in Four storey OGS frames and nonlinear static pushover analyses are conducted. In addition to that, the study includes the OGS frames with the ground storey columns strengthened by different schemes of multiplication factors.

V. CONCLUSIONS

The following are the major conclusions from the present study-

5.1 For OGS frames strengthened with shear wall

1. The maximum capacities of base shear and roof displacement of the OGS frame strengthened with shear wall is increased by about 93% and 40% respectively.
2. The maximum capacities of base shear and roof displacement of the OGS frame strengthened with shear wall is increased by about 5% and 37% respectively compared to a RC frame infilled in all storeys.

5.2 Cost Analysis

1. The maximum base shear to cost analysis ratio for OGS frames strengthened with shear wall is more by about 9 times that of OGS frame.
2. The maximum base shear to cost analysis ratio for OGS frames re-designed with shear wall is more by about 8 times that of OGS frame.
3. The strengthening schemes in line with IS code procedure of applying multiplication factor could achieve only a maximum base shear to cost ratio of 3 times that of OGS frames.

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