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EXPERIMENTAL VERIFICATION OF SEISMIC EVALUATION OF RC FRAME BUILDING DESIGNED AS PER PREVIOUS IS CODES BEFORE AND AFTER RETROFITTING BY USING STEEL BRACING

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ABSTRACT

An experimentally obtained pushover curves of a ¼ size RC frame models with and without infill wall and steel bracing have been used to calibrate the non-linear analytical model of the frame. The pushover testing has been carried out on three non-ductile frame models namely bare frame(BF), infilled frame (INF) and a steel braced (SBF) frame under quasi-static condition. The non-linear analytical model is further extending for the seismic evaluation and retrofitting of a 4-storied 2D frames using infill wall and steel bracing. In this context; firstly a 4-storied 2D RC frame structure has been analyzed and designed using different versions of IS: 456 and IS: 1893. Re-evaluation of these frames has been carried out to with masonry infill and steel bracing as retrofitting scheme using pushover analysis. The different pushover parameters of the frames before and after retrofitting have been compared.

Keywords: Pushover testing; retrofitting; seismic vulnerability; in-filled frame; steel bracing; X-bracing

1. INTRODUCTION

Recent earthquakes in many parts of the world have revealed the issues pertaining to the seismic vulnerability of existing buildings. The existing building structures, which have been designed and constructed according to earlier codal provisions, do not satisfy requirements of the current seismic code and design practices. Many reinforced concrete buildings in urban regions lying in active seismic zones, may suffer moderate to severe damages during future ground motions. Therefore it is essential to mitigate unexpected hazards to property and life of occupants, posed during future probable earthquake. The mitigation of hazards is possible by means of seismic retrofitting of inadequate existing building structures. There are a number of techniques available for enhancing earthquake resistance but how effective they are remains to

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be answered. The available measures have rarely been adequately verified for their effectiveness through experimental testing.

A pushover testing has been carryout on ¼ scale RC model with or without infill and steel bracing under quasi-static condition for the verification of analytical non-linear pushover curves of same model in SAP 2000 in which the force-deformation relationships of individual elements are developed based on FEMA 356. The non-linear analytical model is further used for the seismic evaluation and retrofitting of a 4-storied 2D frames using infill wall and steel bracing. In this context firstly a 4-storied 2D RC frame structure, assumed to be located in Zone IV has been analyzed and designed using different versions of IS: 456 and IS: 1893 one by one in order of their occurrence starting from the year 1970 up to year 2002. One frame is also analyzed and designed considering without any seismic loads. Then non-linear static (pushover) analysis has been carried out using software package SAP2000 to find out their existing capacity. In the second phase; the 4-storey frames (which have been designed as per different previous Indian codes and evaluated for strength and ductility and if found seismically deficient) are re-evaluated with masonry infill and steel bracing as retrofitting scheme using pushover analysis. The amount of retrofitting will depend upon the actual capacity of the original frame that may be analyzed as a bare or infilled and the effectiveness of retrofitting will depend upon the final capacity of strengthened structure. The different pushover parameters of the frames before and after retrofitting have been compared.

2. EARLY STUDIES

Numerous analytical studies and numerical modeling of steel braces used as retrofitting techniques of RC frame with various configuration and different slenderness ratio have been carried out in past [1,22,21,7, 26]. But a few studies are concentrated in which experimental behavior of steel brace has been investigated. Bush et al. [2] conducted some experimental investigations on behavior of RC frames strengthened with steel bracing system. Maheri and Sahebi [20] recommended using direct connections between the brace elements and RC frame without the need for an intermediary steel frame. In experimental work showed the ability of this bracing system to enhance the strength capacity of RC frames. In a continuation of the previous work [19,6] further conducted experimental investigations on pushover response of scaled RC frames; braced with both diagonal bracing and knee bracing systems.

The analytical and experimental behavior of masonry infilled frames has also been extensively studied in the last four decades in attempts to develop a rational approach for design of such frames. A few concerned reference studies are [23,3,8]. Saneinejad and Hobbs [24] developed a method based the diagonal strut approach for the analysis and design of steel or concrete frames with concrete or masonry infill walls subjected to in-plane forces. The method takes into account the elasto-plastic behavior of infilled frames considering the limited ductility of infill materials. Madan et al. [18], Chao et al. [4], have also been evaluated the seismic performance of masonry-infilled RC frames.

3. EXPERIMENTAL VALIDATION OF NON-LINEAR STATIC PUSHOVER ANALYSIS

Three ¼ scale RC frame models namely bare frame (BF), infilled frame (INF) and a steel braced (cross bracing) frame (SBF) of identical dimensions and reinforcement details have been constructed as per Indian Standard and tested under monotonic pushover loading. All three units are 1200 mm in height, measured from column base to the top of the beam, and the span length between centre lines of the columns are 1260 mm as shown in Figure 1.

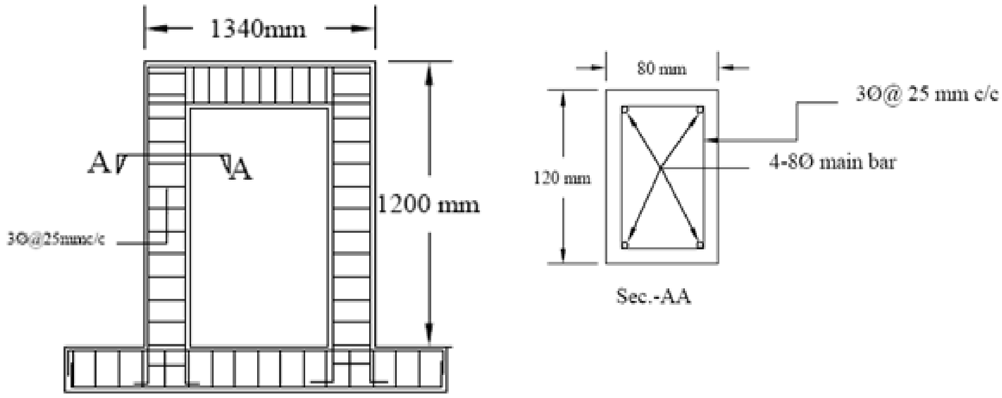


Figure 1. Reinforcement details of the RC bare frame model under pushover loading

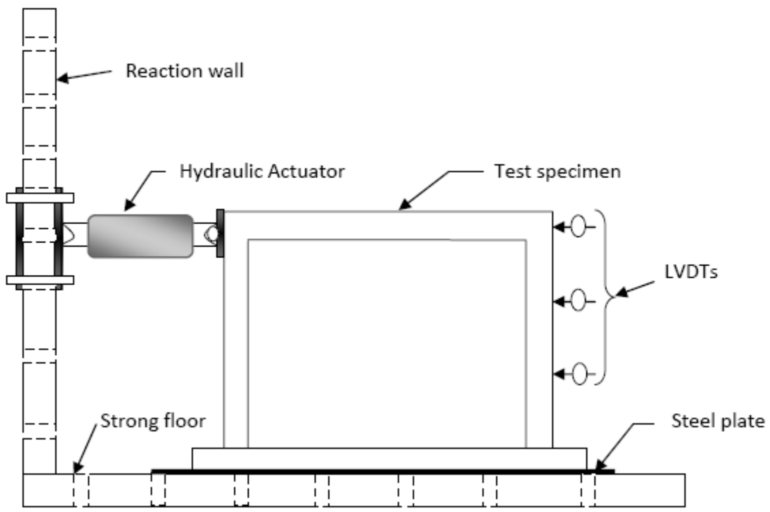


Figure 2. The pushover test setup of BF model

Both beams and columns in each frame had a cross section of 120 mm×80 mm. The main reinforcement of columns and beams are 8 mm dia. Fe-415 grade steel bars. To ensure that the primary damage would occur in the columns, the column reinforcement have been provided by rectangular hoops 3mm dia. (Fe-250 grade steel) spaced at 30 mm on centers with 90^0 hooks at both ends, as observed in many non-ductile concrete structures.

The models have been constructed on steel base plates connected to the platform with shear keys. To ensure displacement of the models along the loading direction only, the models have been supported from two opposite directions with the help of frame consisting of bearings. The horizontal, in-plane load has been applied to the frame using a displacement-controlled actuator. The displacement controlled loading is applied with the help of servo-hydraulic actuator at a very slow rate to eliminate material strain rate effects. The arrangements of LVDTs are used for measuring the responses shown in Figure 2.

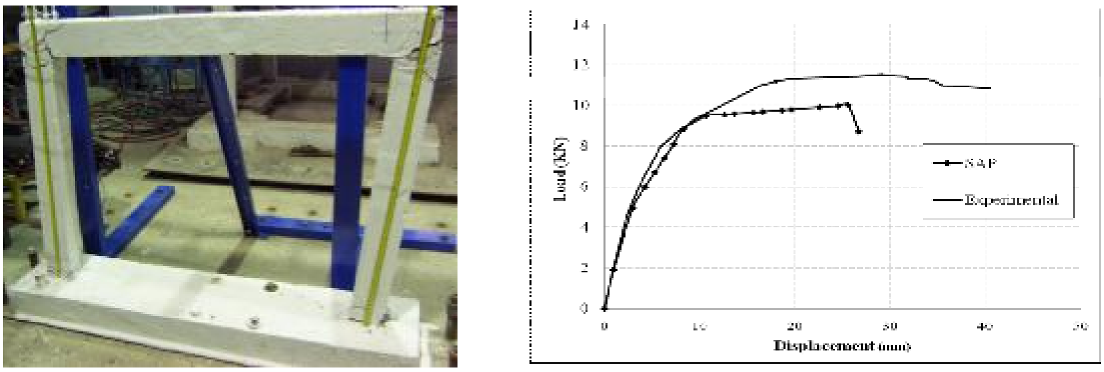


Figure 3. The BF model after pushover testing and its pushover curves

In the case of INF model, ½ scale clay brick having dimensions 120mm X 60 mm X 40 mm with 1:6 cement sand mortar have been used. The infill is added within the full panel between columns and beam. Figure 3 and Figure 4 show the BF and IBF model after pushover testing and obtained pushover curves, respectively.

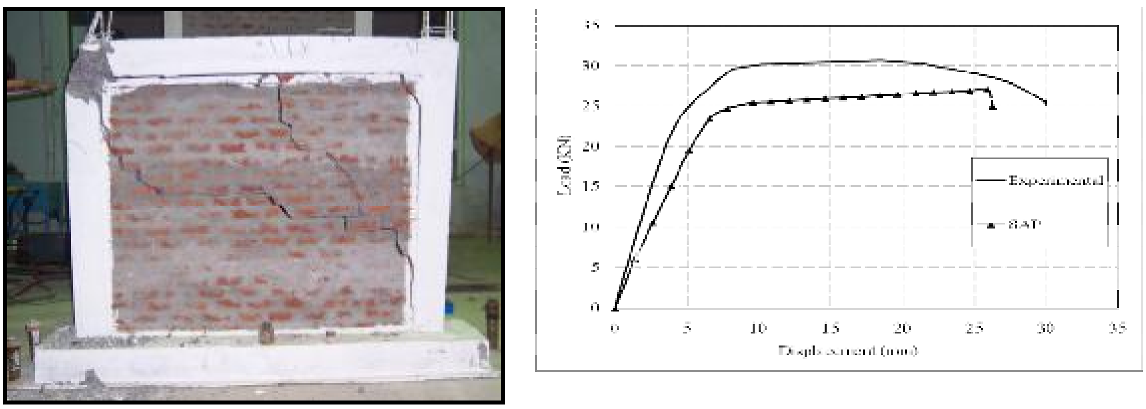


Figure 4. The IBF model after pushover testing and its pushover curves

The SBF model consists of RC frame with concentric steel bracing of X-pattern. The size of bracing is 2ISA30x30x5 @ 2.2 kg/m as per IS 800 and connected back to back at the spacing of 5 mm. The cross bracing derives its primary benefit from the interconnection of two diagonals, which cuts down their unsupported buckling length in compression. The Figure 5 shows the SBF model after pushover testing and obtained pushover curves.

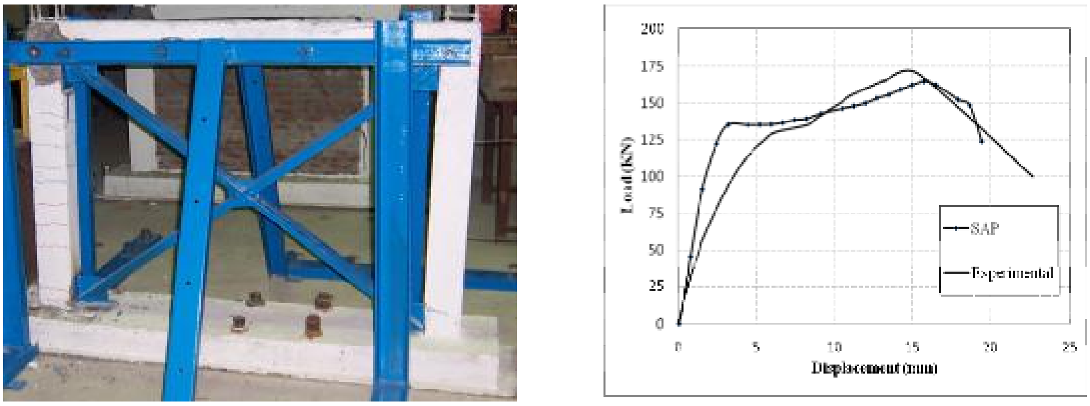


Figure 5. The SBF model after pushover testing and its pushover curves

An analytical model for pushover analysis has also been created for each tested model in SAP 2000. Beams and columns are modeled as nonlinear frame elements with lumped plasticity at the each end of frame element. In this analysis, the elastic stiffnesses of beams and columns in the frames are calculated based on the recommendations of FEMA 356 [5]. Since no external vertical load is applied on models, the compression force due to gravity load in any frame column is well under $0.3A_g f_c'$. Therefore, flexural and shear rigidities of $0.5E_cI_g$ and $0.4 E_cA_g$ were applied to all beams and columns, respectively.

To perform a step-by-step force displacement response analysis of buildings with infilled frames (INF), a modeling for infill is required. For masonry infill is modeled as the compression strut as recommended by FEMA 356 for the calculations of strengths and effective stiffness of the infill panels. The infill is modeled as single strut element with possibility of forming axial hinge, Figure 6. FEMA 356 gives the following equation for the calculation of the width of the equivalent compression strut that represents the in-plane stiffness of a solid un-reinforced masonry infill panel before cracking:

$$a_1 = 0.175(l_1 h_{col})^{-0.4} r_{\inf}$$

where

$$l_1 = \left[\frac{E_{me} t \sin 2q}{4E_c I_c h_{\inf}}\right]^{1/4}$$

Where:

$$\lambda_1 = \left[\frac{E_{me} t \sin 2q}{4E_c I_c h_{\inf}}\right]^{\frac{1}{4}}$$

and

h_{col} = Column height between centre lines of the beams; $h_{\inf}$ = Height of the infill panel; E_{fe} = Expected modulus of elasticity of the frame material; E_{me} = Expected modulus of elasticity of the infill material; I_{col} = Moment of inertia of the column; $L_{\inf}$ = Length of the infill panel; r_{inf} = Diagonal length of the infill panel; t_{inf} = Thickness of the infill panel and equivalent strut; θ = Angle whose tangent is the infill height-to length aspect ratio; l_1 = Coefficient used to determine equivalent width of the infill strut

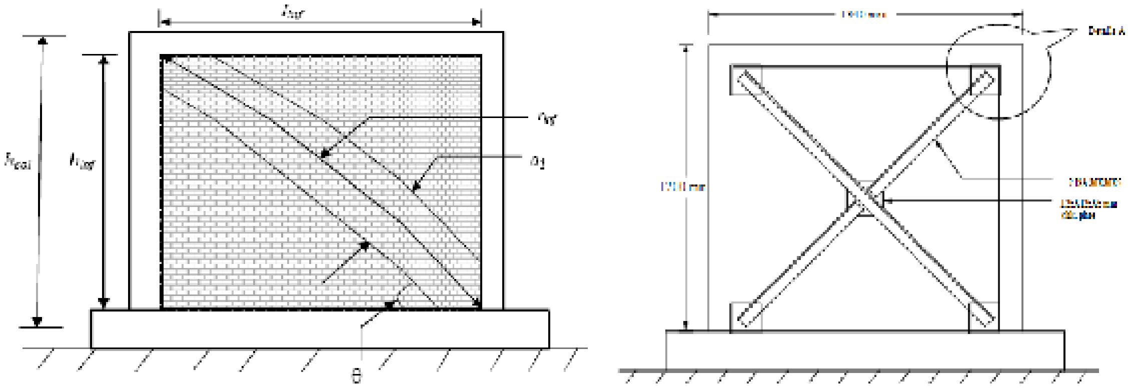


Figure 6. Compression strut analogy (FEMA, 2000)

In case of steel braced frame; the steel braces are modeled as an axially loaded member, hinged at both ends. Additional joints (nodes) are created at intersection point of diagonal bracing. The hinge property assigned as P (axial) hinge is given in FEMA356.

The experimental and analytical pushover curves of bare frame, infilled frame and steel braced frame are also compared in Figure 3, 4 and 5. The nonlinear force-displacement relationship between base shear and displacement is replaced with an idealized relationship to calculate the effective lateral stiffness, K_e, and effective yield strength V_y of the different frames as per FEMA 356 which is shown in Figure 7. This relationship shall be bilinear, with initial slope K_e and post-yield slope α. The line segments on the idealized force-displacement curve have been located using an iterative graphical procedure that approximately balances the area above and below the curve. The effective lateral stiffness, K_e, is taken as the secant stiffness calculated at a base shear force equal to 60% of the effective yield strength of the structure. Table 1 is summarized the results of pushover parameters of experimental and analytical frame models.

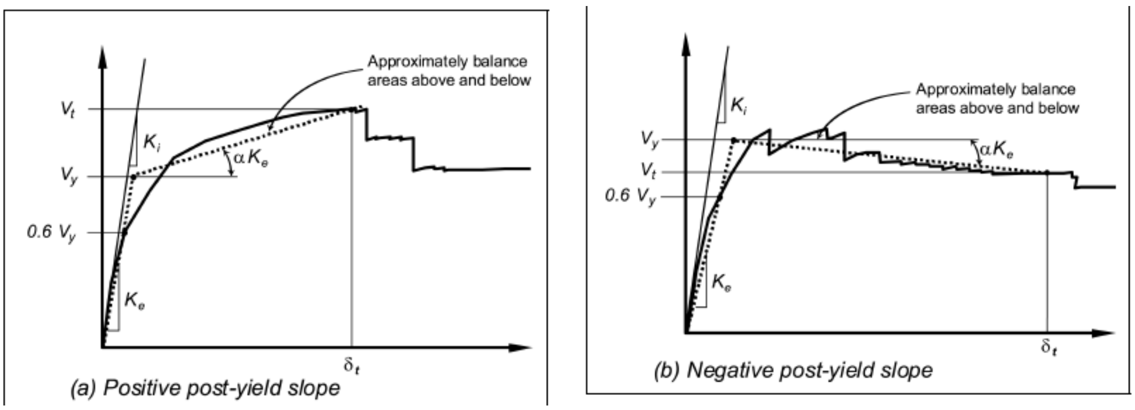


Figure 7. Idealized force-displacement curves [5]

Table 1: Comparison of the pushover parameters of experimental and analytical frame models

Result type	**Experimental results**			**SAP results**		
Specimen types	**Bare frame (BF)**	**Infilled frame (INF)**	**Steel braced frame (SBF)**	**Bare frame (BF)**	**Infilled frame (INF)**	**Steel braced frame (SBF)**
Time period (s)	0.034	0.0219	0.0082	0.035	0.019	0.0066
Yield load (KN)	9.5	27.5	110	8.5	24.25	124
Ultimate load (KN)	11.53	30.7	170	10.01	27.08	165.29
Ductility	5.33	3.89	3.75	5.12	3.98	3.64
Effective stiffness (kN/m)	1727	5790	29330	1700	3731	35428
Post-yield stiffness (kN/m)	83.7	232.7	5172.4	73.12	145.9	4440.8

4. EVALUATION OF THE SEISMIC CAPACITY OF AN EXISTING 4 -STOREY RC FRAME BUILDING

A single bay 4 storey (G+3) 2D frame with storey height of 3.5 m and slab thickness 150 mm has been considered for this study, Figure 8. The frames have been analyzed and designed as per loads shown in Table 2 with different versions of the Indian codes [9-17] as given in Table 3. This gives a comprehensive picture of the seismic vulnerability of this structure in the light of the latest version which helps to provide retrofitting measures for those frames having inadequate capacity in comparison to latest code requirements.

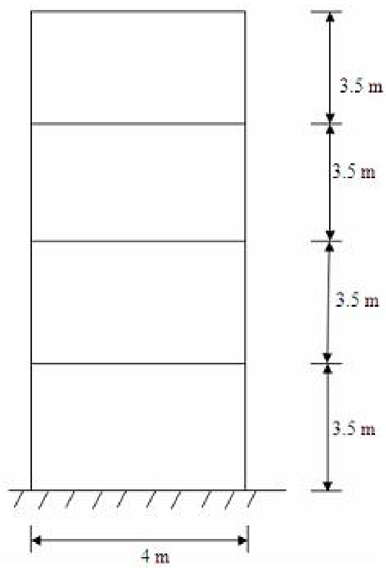


Figure 8. A 4-storey frame under consideration

Table 2: Load calculation of the frame

Sequence	Load at roof level (kN/m)		Load at floor level (kN/m)		Total earthquake load in terms of base shear (kN)
	DL	LL	DL	LL	
1.	11.5	3.0	8.7	8.0	Not considered
2.	11.5	3.0	8.7	8.0	18.62
3.	11.5	3.0	8.7	8.0	22.28
4.	11.5	3.0	8.7	8.0	35.65
5.	11.5	3.0	8.7	8.0	44.6

Assume dead and live load on roof = 5.75 kN/m^2 and 1.5 kN/m^2; at floor level = 4.35 kN/m^2 and 4.0 N/m^{2}and; weight of infill wall = 17.5 kN/m

Table 3: Five sequences used for analysis and design of RC frames

Sequence	Year	Concrete code	Seismic code	Design theory
1.	1964	IS:456-1964	N.A.	WSM
2.	1970	IS:456-1964	IS:1893-1970	WSM
3.	1975	IS:456-1964	IS:1893-1975	WSM
4.	1984	IS:456-1978	IS:1893-1984	LSM
5.	2002	IS:456-2000	IS:1893-2002	LSM

After designing and detailing with different code, nonlinear static (pushover) analysis has been carried out using SAP2000 [25] to find out its existing capacity. The pushover analysis consists of the application of gravity loads and a representative lateral load pattern. The lateral loads are applied monotonically in a step-by-step nonlinear static analysis. Two types of nonlinearity, considered for modeling, are geometric nonlinearity and material non linearity. The capacity curves of the 4 storey 2D frame designed as per different codes are obtained as shown in Figure 9. All the curves show similar features. They are initially linear but start to deviate from linearity as the beams and the columns undergo inelastic actions. When the buildings are pushed well into the inelastic range, the curves become linear again but with a smaller slope. These curves could be approximated by a bilinear relationship. It is clear from the plotted curves that the capacities of frames designed with only gravity (DL+LL) loading [Seq. 1] and the frames designed according to earlier codes [Seq. 2 to Seq.4] are less than the capacity of the frame designed according to present day code i.e. IS: 1893 (Part 1): 2002.

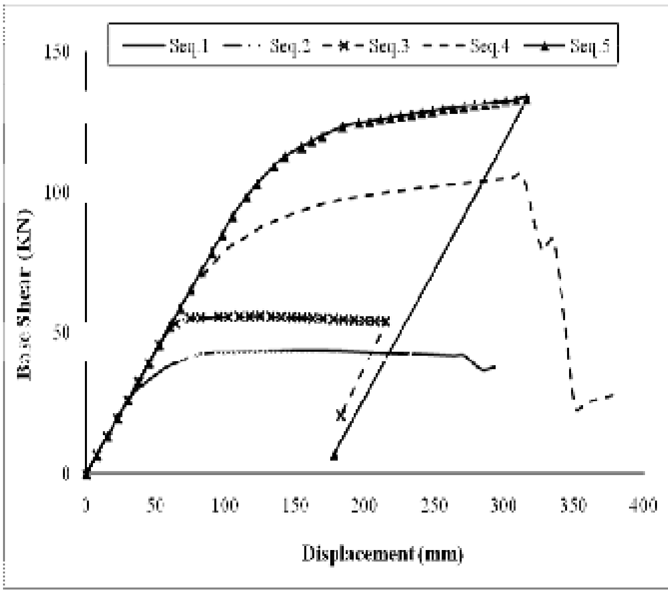


Figure 9. Pushover curves of 4-storey RC frame building designed with different IS codes

The nonlinear force-displacement relationship between base shear and displacement is replaced with an idealized bi-linear relationship [as discussed earlier] to calculate the different parameters. Different pushover parameters including strength and ductility are extracted from the capacity curves for different frame which are tabulated in Table 4. It is concluded that the existing building in seismic zone IV, designed and constructed using previous Indian standards is found inadequate to withstand the revised present day code. The maximum base shear calculated at the roof level of the 4- storey frame according to IS :1893 (Part-I) -2002 increases upto about 2.40, 2.0 and 1.25 times as compare to the values in 1970, 1975 and 1984 respectively.

Table 4: Different pushover parameters extracted after idealization of pushover curves

Design sequences	**Sequence 1**	**Sequence 2**	**Sequence 3**	**Sequence 4**	**Sequence 5**
Yield base shear (kN)	39	54	54	86	116
Ultimate base shear (kN)	43	55.9	55.9	105.6	133
Effective stiffness (N/mm)	866.7	900	900	860	859.3
Post-yield stiffness (N/mm)	36.7	29.7	29.7	92.0	94.4

5. RE-EVALUATION OF THE SEISMIC CAPACITY OF RETROFITTED RC BUILDING USING STEEL BRACING TECHNIQUE

The 4- storey frames which have been designed as per different previous Indian codes is re-evaluated for strength and ductility with masonry infill and steel bracing as retrofitting scheme using nonlinear static (pushover) procedure as shown in Figure 10. The infill masonry used for analysis having compressive strength (f_m) equal to 2.5 MPa and modulus of elasticity (E_m) equal to 1500 MPa by assuming E_m= 600.f_m. The thickness of masonry infill is assumed to be of equal to 250 mm. The steel bracing used is of cross pattern (X-bracing) which is more commonly used. The cross bracing derives its primary benefit from the intersection of the two diagonals, which cuts down their unsupported buckling length in compression. The size of steel brace used is of size 2ISA 60x60x8 connected back to back at a spacing 8 mm.

The most important step in the nonlinear static (pushover) analysis of a structure is to create an appropriate mathematical model that will adequately represent its stiffness, mass distribution and energy dissipation so that its response to earthquake could be predicted with sufficient accuracy. The frames have been modeled and analyzed using software SAP 2000 on the similar lines of analytical modeling of experimental test model. Beams and columns are modeled as frame elements with centreline dimensions. Supports at the base are assumed to be fixed. Two types of nonlinearity have been considered in modeling i.e. *geometric nonlinearity* and *material non linearity*. Geometric nonlinearity is provided in the form of P-Δ effects of loading. Material nonlinearity is provided in the form of plastic hinges in the frame elements. In the analysis M3 pushover hinges are assigned at both ends of beam elements (at locations of plastic hinge formations). PMM pushover hinges are assigned to columns at both ends. The infill was modeled as single strut element with possibility of forming axial hinge. The width of the equivalent compression strut has been calculated as per guidelines of FEMA 356. Steel bracing members (double angle back to back) are modeled as truss member. X bracing (cross bracing) system has been considered. In the cross pattern of steel bracing, additional joints (nodes) are created at intersection point of diagonal braces. The connection between steel brace and frame have been made rigid by providing end length offset with rigid zone factor 1, i.e. the entire connected zone has been made rigid. The pushover curves of the 4- storey 2D frame with infill wall and steel bracing are obtained as shown in Figure 11.

Figure 10. Re-evaluation of seismic capacity of 4-storey RC frame with infill wall and steel bracing

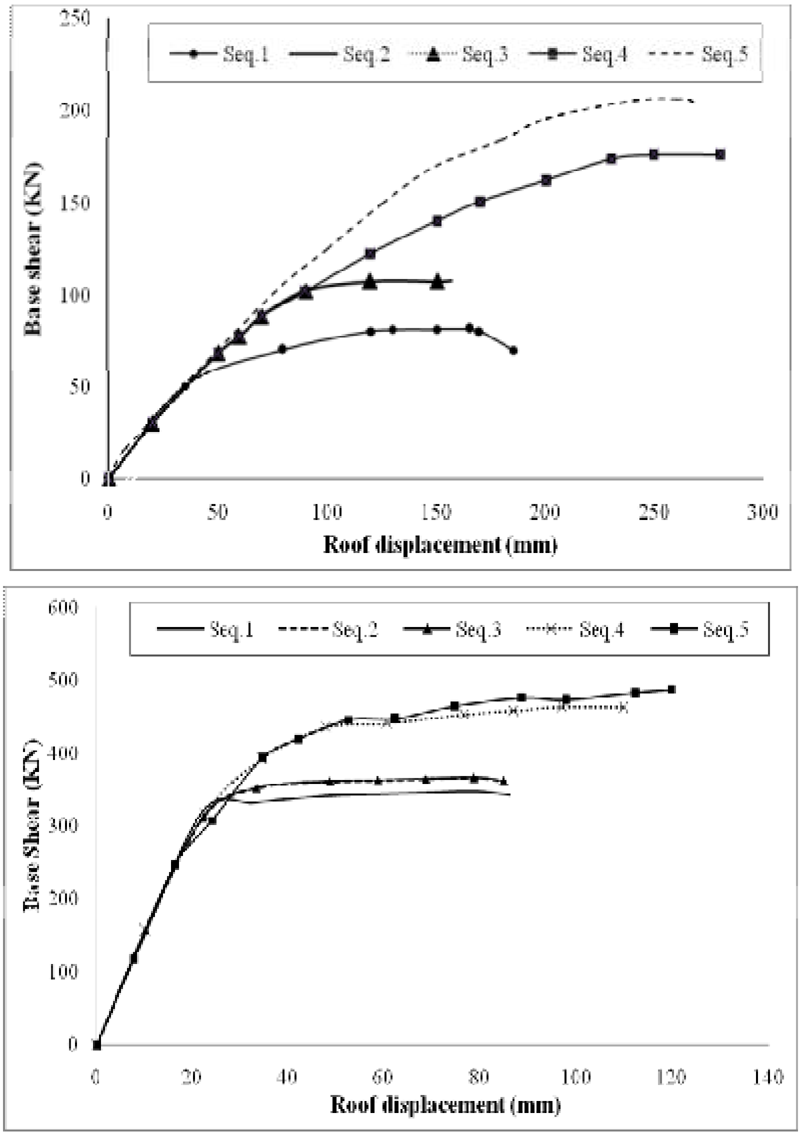
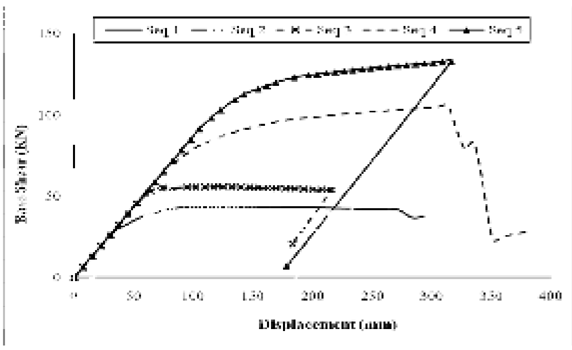


Figure 11. Pushover curves 4-storeyed RC frame infilled frame with infill wall and steel bracing

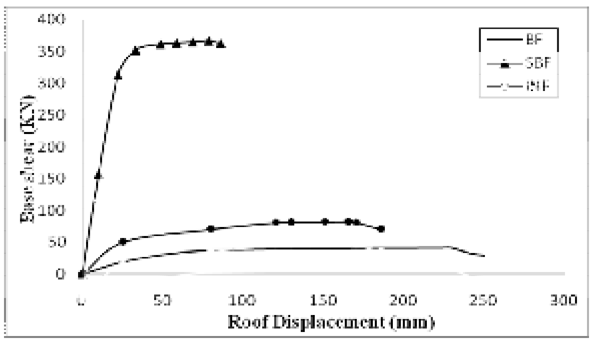
The nonlinear force-displacement relationship between base shear and displacement of the retrofitted frames is replaced with an idealized bi-linear relationship to calculate the different parameters. Different pushover parameters including strength and ductility are extracted from the capacity curves for different frame are tabulated in Table 5. The capacities of the frames designed as per different sequences (Seq.1 to Seq.5) before retrofitting (as bare frame) and after retrofitting (using steel bracing) are compared in Figure 12. By comparing the curves it can be concluded that by adding steel bracings the lateral strength of ordinary moment resisting frame can be enhanced in the lateral strength of the system sufficiently and adequately.

Table 5: Different pushover parameters of frame retrofitted with infill wall and steel bracing

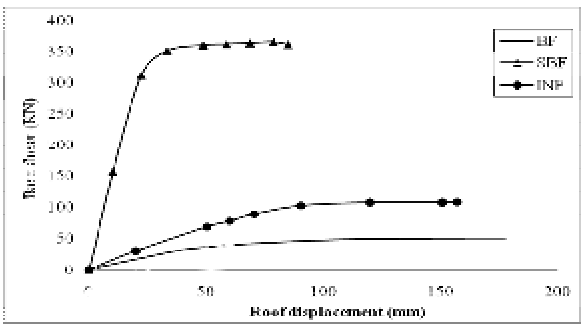
Design sequences	Sequence 1		Sequence 2		Sequence 3		Sequence 4		Sequence 5	
	infilled	**braced**	**infilled**	**braced**	**infilled**	**braced**	**infilled**	**braced**	**infilled**	**braced**
Yield base shear (KN)	65	330	95	340	95	340	125	390	165	395
Ultimate base shear (KN)	80	342	107	366	107	366	178	463	206	485
Ductility	2.40	3.54	2.41	3.59	2.41	3.59	2.8	4.07	2.82	4.62
Effective stiffness (N/mm)	1400	15000	1401.5	15454	1401.5	15454	1250	15000	1320	15192
Post-yield stiffness (N/mm)	130.4	214.3	130.4	456	130.4	456	294.4	698	294.4	957



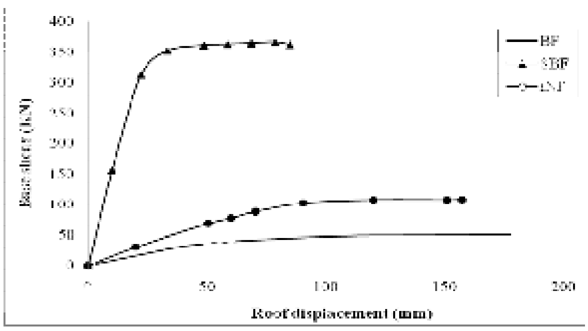
Comparison of pushover curves before retrofitting under different sequences



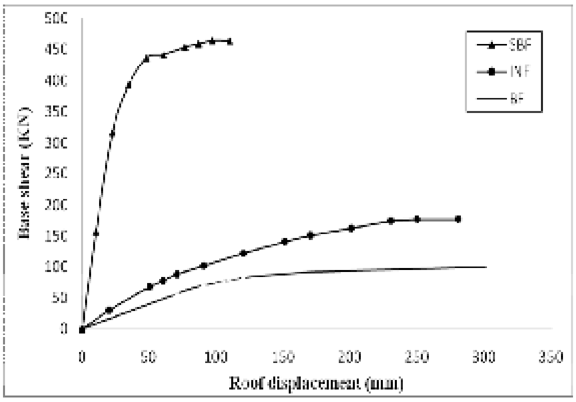
Comparison of pushover curves before and after the retrofitted of the frame designed as per Sequence 1



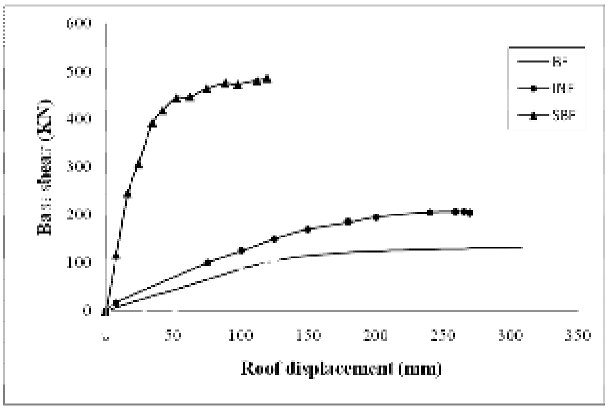
Comparison of pushover curves before and after the retrofitted of the frame designed as per Sequence 2



Comparison of pushover curves before and after the retrofitted of the frame designed as per Sequence 3



Comparison of pushover curves before and after the retrofitted of the frame designed as per Sequence 4



Comparison of pushover curves before and after the strengthening of the frame designed as per Sequence 5

Figure 12. Pushover curves of 4-storey RC frame building designed with previous IS code as bare frame (BF), infilled frame (INF) and steel braced frame (SBF)

6. SUMMARY AND CONCLUSIONS

An experimentally and analytical study has been carried out on single storey RC model. A seismic evaluation of 4 -storey 2D frame designed with previous IS codes has also been carried out to investigate the effect of retrofitting technique. On the basis of this study the following conclusions are drawn;

1. The existing 4-storey RC frame building in seismic zone IV, designed and constructed using previous Indian standards is found inadequate to withstand the present day code requirement. The maximum base shear as per seismic code IS 1893 in 1970, 1975 and 1984 is lower than the 2.40, 2.0 and 1.25 times base shear calculated as per existing IS:1893 (Part-I) respectively.
2. The experimental pushover analysis of the frame model shows that there is an increase in *effective stiffness*, *yield load and ultimate load* of about 3.4, 2.9 and 2.7 times respectively due to inclusion of infill wall whereas the above three parameters increases about 17, 11.6 and 14.7 times respectively due to addition of steel bracing.
3. The analytical pushover analysis of the 4 -storey frames also shows that there is a increase in *effective stiffness, yield load and ultimate load* of about 1.5, 1.6 and 1.8 times respectively due to inclusion of infill wall whereas the above three parameters increases about 16, 4 and 5 times respectively due to addition of steel bracing.
4. For fully confined infill, the equivalent strut model specified in FEMA 356 gives a reasonable prediction on both un-cracked stiffness and lateral strength of masonry infilled panel frame.

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