Performance-Base Design and Analysis of Infilled Reinforced Concrete Frames Retrofitted with Prestressed Cables

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ABSTRACT: The main objective of current study is to develop an improved seismic design retrofit technique for deficient/old R/C frames based on using Prestressed cable bracings and also including effects of masonry infill walls. Cyclic behavior of different structural configurations was analyzed by considering two phases of analytical studies. First, behavior of single-bay R/C frame models was investigated and analytical modeling results verified. In second phase, five story frames were retrofitted and analyzed by using pushover and non-linear dynamic analysis. Variable parameters included effect of infill walls, cable area, prestressing load and different earthquake records. Lateral capacity curves were obtained and then used to determine required design variables. Furthermore, a preliminary design process was proposed based on behavior of old R/C buildings. The results showed that the models could reasonably simulate the structural behavior. Also, the retrofitting method was able to restrain deformations and increase strength substantially, resulting in protection of brittle RC buildings during strong earthquakes.

Keywords: Retrofit; Performance-base; Prestressed cables; Reinforced concrete, Infill wall, Nonlinear analysis.

1. Introduction

Many non-ductile and seismically deficient concrete structures have suffered severe damages or even collapsed during recent earthquakes. Most of the damages caused by excessive ductility demands and on the other side, lack of ductility mostly caused by inappropriate seismic detailing [2]. Replacing these deficient structures with new buildings would be expensive and time consuming. This indicates the need for evaluating the seismic condition and performance of existing buildings. Also, it shows the importance of designing economical and effective retrofitting plans with easy implementation as an affordable approach to seismic risk mitigation.

While developing different retrofitting techniques, the contribution of infill walls to lateral resistance and performance of infilled frames has been shown experimentally and analytically in the past by different researchers including Mainstone 1971 [9]; Mehrabi et al. 1996 [10]; Ehsani et al. 1999 [5]; Michael L. Albert et al. 2001 [11]; Saatciouglu et al. 2005 [15]; Hashemi et al. 2005 [8]; Binici et al. & Yuksel et al. 2006 [14]; Erdem et al. 2006 [6], Altin et al. 2007 [2] and others. Overall results indicated the strengthening potential of infill walls.

Specifically, application of prestressed cables for retrofitting R/C structures has been studied in 2005 at the University of Ottawa in Canada during an experimental research program on seismic retrofit strategies. The program included retrofitting non-ductile concrete frames with diagonal pre-stressing, strengthening with fiber-reinforced-polymer sheets and active control of buildings. According to an article by M. Saatciouglu [15] diagonal pre-stressing showed good results in terms of strength and ductility. However, the need for further analytical investigations to create a simplified retrofitting design method still exists.

The objective of the current study is to analytically determine the effects of using diagonal prestressed cables as a retrofitting method in non-ductile concrete frames with and without infill walls. Results of this research should enable the structural engineers to perform retrofit design of deficient reinforced concrete frames.

2. Design strategy & modeling

Since the old structures are considered as non-ductile, frame elements will experience significant strength decays beyond approximately 1% to 2% lateral drift ratio based on the previous experimental data [15]. Therefore, to achieve the best results in this retrofitting method, lateral drift ratio should be restrained at around 1% which is a reasonable value.

Based on different previous observations [13, 14] the frame and infill wall are expected to act against lateral loads together. Accordingly, the strength of the retrofitted models can be considered as the summation of the lateral strength of the frame, the infill walls and the retrofitting elements. The modeling of the masonry infill walls is based on a simplified macro model which considers that the lateral system behavior changes to an interaction of a flexural frame and a shear dominant wall. This leads to developing a diagonal compression strut between the opposite compression corners as specified by Paulay and Priestley [12]. Its compression capacity, Df, can be calculated from the following equation:

$$D_f = \frac{\tau_0 D_i t}{1-\frac{\mu h}{L}} = \frac{0.03 f'_m D_i t}{1-\frac{0.3h}{L}} \quad (1)$$

Where D_i = infill diagonal length; τ_0 = shear stress between mortar and masonry; h = Infill height; L = Infill length; t = Infill width; and f'_m is compressive strength of masonry brick units.

Accordingly, Infill walls were modeled as truss line elements with assigned axial nonlinear hinges, developing during loading. Also masonry elasticity modulus was calculated as suggested by Hamid et al. [7] and equivalent width of the compressed diagonal strut is considered based on the Mainstone's formula [9] as follows:

$$W_s = 0.16 D_i (\lambda_h h)^{-0.3} \sin 2\theta; \left\{ \lambda_h h = \frac{\pi h}{2\alpha} = h \sqrt[4]{\frac{E_i t \sin 2\theta}{4 E_f I_f}} \right\} \quad (2)$$

Where Di = infill diagonal length; h = Infill height; θ = diagonal strut angle; If is moment of inertia of the surrounding frame member; Ei and Ef are the elastic modulus of the infill and the surrounding frame member (beam or column); and α is a dimensionless relative stiffness parameter to determine the contact lengths of the wall.

High strength ASTM grade 270, 7-wire cables with 15.24 mm nominal diameter and 140 mm2 area were used for retrofitting. They were modeled as diagonal ties, using diagonal truss elements with zero compressive strength. Prestressing loads were also applied in some models as this was expected to improve the strength of the whole system [15].

To achieve realistic results, behavior of R/C frame components was determined considering the potential of failure due to different loading actions including flexural, shear, and axial. Rebar development length and splicing problems were also considered in a macroscopic approach. Member plastic rotations were calculated from chord rotations as suggested in FEMA 356 [3], taking into account axial force interaction (P-M-M) effects in columns which where modeled using a plastic hinge beam-column element. Development of the shear inelastic mechanisms was also considered in analysis as it was expected to have an important role in specifying the ultimate strength of structure [16]. In all mechanisms, the plasticity was assumed to be concentrated at the ends of the members.

3. Analytical simulation of a single bay retrofitted R/C frame

Eight different half-scaled single story and single-bay assemblies were designed according to ACI 318-1963 code [1]. Analytical models were labeled concisely using different notations based on their properties, including “B” as “Bare frame”, “W” as “Masonry infill wall”, “C” as “Cable”, “P” as “Prestressed” and the last number in labels indicates the level of prestressing load at each bay diagonal (kN), if any. Model “1BW” was taken as reference, representing the characteristics of an unretrofitted frame-wall assembly. Each half-scale model consists of two 25×25 cm columns and a 25 cm width and 35 cm depth beam as they were originally at the first story of a 5-story building. Retrofitted models included two prestressed cables along each diagonal. The infill wall, consisting of double layers of hollow clay brick, replaced with a 28.25 cm equivalent width and 15 cm depth (7.5 cm for each layer) strut according to Mainstone’s formula. Furthermore, important properties of materials are summarized in Table 1.

Table 1. Material Property

Material	Description			Specifications
Concrete	4000psi			f'_c=27.579 Mpa
Rebar	ASTM A992			f_y=413.68 Mpa, f_u=448.159 Mpa
Clay brick	24.5×9×7.5 cm units			f_c=19.62 Mpa
Brickwork	Mortar combination			$f'm$=1.177 Mpa* E_i=21000 kg/cm^2
	Cement	Lime	Sand	
	1	2	8 to 9	
Cables	ASTM A416 Grade 270			f_y=1689.9052 Mpa, f_u=1861.5846 Mpa

* Compressive strength assuming a safety factor of 3.5.

All analytical models were subjected to gravity design loads and then to increasing lateral push-over loading as illustrated in Fig. 1. The retrofitted frames were analyzed considering two different levels of prestressing load. The maximum prestressing load was selected around 50% of the total capacity of the selected cables. It was considered based on the assumption that retrofitting cables should reach their maximum capacity at around 1% lateral drift ratio. This level corresponds to maximum capacity of the retrofitted experimental model (Fig. 2) and can be considered as structural life safety performance level [3].

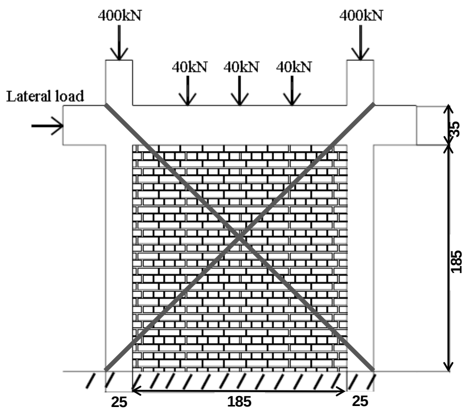

Figure 1. General view of Single-bay frame with masonry infill wall and applied loads.

The experimental hysteretic pushover curves and analytical curves for the models 1BW & 1BWPC150 are shown in Fig. 2. The results of the pushover analysis successfully simulated the previously available experimental capacity curves and were capable of demonstrating the expected semi ductile behavior of the models [3].

Analysis results for different models in terms of base shear versus roof displacement are plotted in Fig. 3. The results of parametric study in retrofitted R/C frames showed that the prestressed cables improved initial stiffness substantially. The retrofitted frames were able to reach high base shear levels at the same drift values for unretrofitted models with & without considering infill wall effects. As illustrated in Fig. 3, diagonal prestressed cables increased lateral load resistance more than 100% in case of 1BWPC150 versus 1BW which resulted in sufficient lateral bracing forces to control displacements. Also, by comparing the curves between 1B and 1BW, it is obvious that the Infill walls had a great share in improving stiffness and lateral strength of infilled frames.

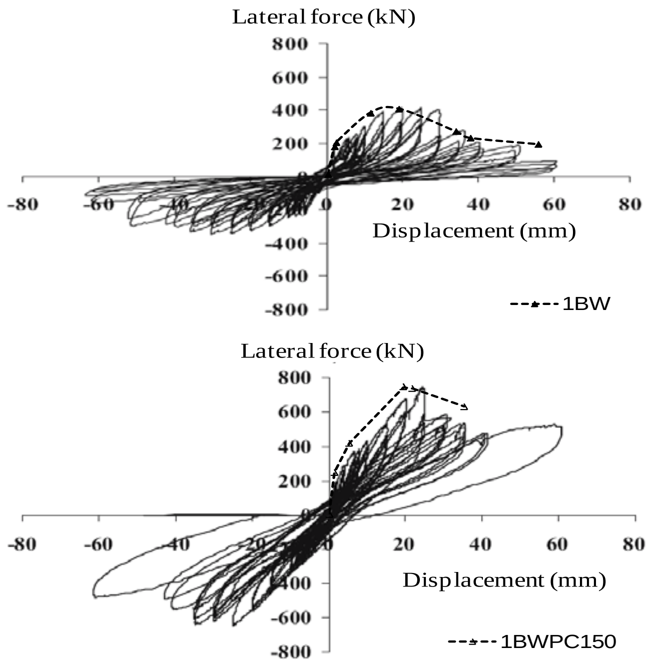

Figure 2. Comparison of analytical and experimental [15] capacity curves.

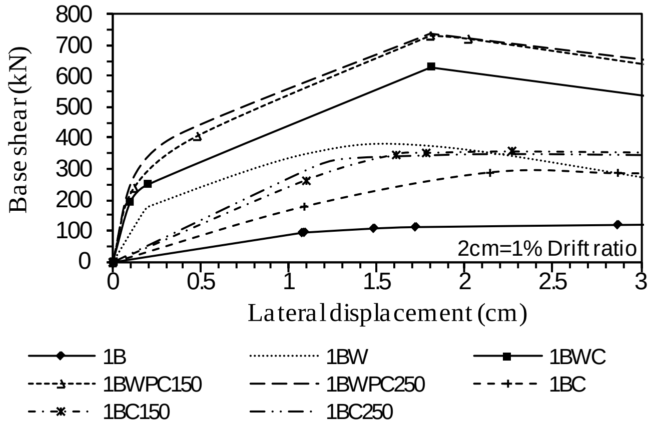

Figure 3. Pushover curves for the different single-bay models with & without masonry infill-wall.

4. Multistory frames

A 5-story R/C residential building with a 30×18 m rectangular plan was designed based on the old ACI 318-1969 code [1] to represent a seismically deficient structure with members designed to carry mainly gravity loads. The plan consists of 3 by 5 bays with the length of six meters each and 4 m height stories. Ten different models of the building were analyzed following a parametric study program applying nonlinear pushover analysis in order to estimate the effects of retrofitting on the lateral resistance of the structures. Each model was named according to its specifications by using the previously described notations. Furthermore, the added middle number (2 or 4) stands for the number of cables along each middle bay diagonal (Fig. 4). In retrofitted models, the external frames were diagonally prestressed in the middle bays along the height of the structure. Masonry walls were modeled as diagonal struts. Diagonal tension resistance of infill walls was neglected and the effective width of the masonry was calculated equal to 77.88 cm based on the Mainstone’s formula. The nonlinear response of equivalent strut was assumed similar in both lateral directions. The strut capacity was calculated from Eqn. 1, Df = 48.193 kN, and was assigned to nonlinear axial hinges along the truss elements. Effects of insufficient confinement due to poor seismic detailing were considered in all models. Loading consisted of 650 kg/m2 and 200 kg/m2 as dead and live loads. Also, two levels of prestressing load including 100 and 125 kN per cable were considered, and the maximum value specifies that cables should reach around their maximum capacity at approximately 1% drift ratio. For lateral loading, the uniform and the dynamic load patterns are applied to each model according to requirements by FEMA 356 [3] during two separate pushover analysis cases. The analysis is conducted in the short direction of plan to generate lateral capacity curves and also included P-Delta effects.

The structural performance level was defined as life safety (S-3) [3]. Building global drift levels were set to 2% transient and 1% permanent based on FEMA 356 [3] which also approved to be realistic according to experimental results [14]. BSE-2 hazard level (2%/50 year) was primarily applied to all analysis results. Also, to achieve a more usual Basic Safety Objective (BSO) performance level, the spectral acceleration is reduced to BSE-1 hazard level (2/3 BSE-2) and the results recalculated. The NEHRP design response spectrum for BSE-2 hazard level including Soil-Structure Interaction (SSI) effect is shown in Fig. 6. The design short-period response acceleration, Sxs = 1.5g and the spectral acceleration at the period of 1 second, Sx1 = 0.78g were selected based on the NEHRP Site Class C [3].

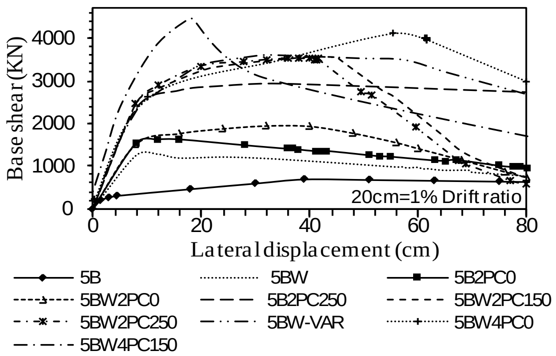

Figure 4. Pushover curves for different structural models.

Capacity curves in terms of base shear versus roof displacement are plotted in Fig. 4. Prestressed cables improved initial stiffness and the retrofitted structures were able to resist high base shear levels at low drift ratios. The deficient structures, 5B & 5BW, were unable to resist current levels of design base shear equal to 2070 kN for Vancouver based on NBCC-2005 [17] and 2920kN for Tehran based on Iranian standard No. 2800 [18]. In case of 5BW, the structure yielded and collapsed after reaching a lateral capacity around 65% of NBCC-2005 or 45% of standard No. 2800 design base shear. The stiffening effects of infill walls significantly improved the structural performance and characteristics nearly the same level in all infilled models. Model 5BW2PC0 was retrofitted with two 15mm diameter cables diagonally at middle bays, along the height of the structure. This resulted in 75% increase in base shear at the target displacement compared to 5BW. The results for prestressed models, 5BW2PC150 & 5BW2PC250, also showed substantial improvements in stiffness and base shear capacity at the yield point (Fig. 4). Also, they had approximately the same behavior in terms of initial stiffness and maximum base shear because of creation of the similar compression force in equivalent infill wall struts. In models with uniform cable distribution, sudden collapse of the first floor was observed due to the flexural and especially shear failure in columns (Fig. 5). As a solution to this problem, the uniform distribution of cables substituted with a reverse triangular distribution pattern in number of cables in 5BW-VAR (Fig. 5). In this way the gradual strength reduction of the capacity curve postponed successfully till almost 3% (60 cm) lateral drift. Additional analysis has been done to investigate effects of different parameters on capacity curves. Doubling the number of cables in 5BW4PC0 & 5BW4PC150 increased stiffness in both structures and resulted in 65% & 30% additional lateral capacity in each model compared to 5BW2PC0 & 5BW2PC150.

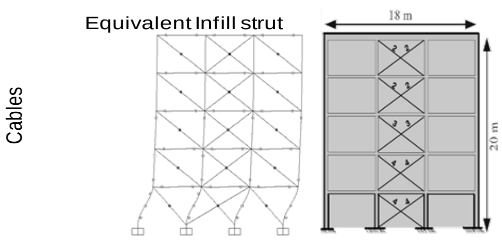

Figure 5: Sudden collapse of columns by using a uniform cable pattern (left), proposed reversed triangular cable pattern (right).

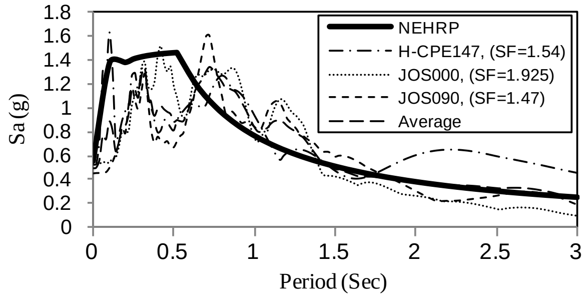

Figure 6. NEHRP response spectrum (S_S=1.5g & S_1=0.6g) and 5% damped response spectra of scaled motions.

In bare frame models, effectiveness of the retrofitting method was also verified. In case of 5B2PC250, the maximum base shear was equal to 2944.317 kN versus 705.899 kN for the unretrofitted model (5B).

Capacity curves converted to an equivalent SDOF by using displacement modification technique [3] to estimate the maximum global displacements (performance points) and R-factors. In the process the effective period, Te, calculated from the initial period, Ti, by applying graphical procedure of FEMA 356 [3]. The performance points for retrofitted frames were clearly between 1% to 2% lateral drift. This indicated that the rehabilitation design meets the acceptance criteria for the selected “Enhanced Rehabilitation Objective” and “Basic Safety Objective”. In case of latter, maximum roof displacements were found clearly around 1% drift. The values obtained from pushover analysis are summarized in Table 2.

Table 2. Analytical results for multistory structures, BSE-2 hazard level, S_S=1.5g & S_1=0.6g

Model name	T_e(s)	S_a(g)	Base shear at yield	Expected reactions at δ_t*	
			V_y(kN)	V(kN)	D(cm)
5B	2.1924	0.3542	705.899	570.341	12.35
5BW	1.908	0.3903	1321.7798	1057.42	40.6
5B2PC0	1.8047	0.4297	1834.6809	1850.583	44.076
5B2PC150	1.4371	0.5387	2648.4634	2931.665	34.650
5B2PC250	1.433	0.5402	2669.1637	2937.618	34.592
5BW2PC0	1.4674	0.5277	2142.9575	2914.543	36.42
5BW2PC150	1.463	0.5292	2872.908	3484.721	34.45
5BW2PC250	1.4611	0.5299	3032.2643	3512.470	33.94
5BW-VAR	1.428	0.542	3023.5666	3603.723	36.5

* δ_t= Calculated target displacement based on FEMA 356 [3]

R-factors were also determined for the two earthquake hazard levels defined as BSE-2 and BSE-1 (Ss = 1g & S1 = 0.4g) and illustrated in Table 3. Based on the results, the minimum R-factor was obtained by applying a reverse triangular arrangement of cables over the height of the structure.

Table 3. Obtained R-factors

Model name	**R**-factor		SF_R	
	BSE-2[1]	BSE-1[2]	(Exact)	(Approx.)
5B	5.5976	4.5477	**0.812**	**0.979**
5BW	6.9132	5.1075	**0.738**	**0.737**
5B2PC0	5.4842	4.3573	**0.794**	**0.726**
5B2PC150	4.7642	3.5721	**0.749**	**0.704**
5B2PC250	4.7406	3.5607	**0.751**	**0.705**
5BW2PC0	6.0566	4.4934	**0.742**	**0.714**
5BW2PC150	4.3135	3.3042	**0.766**	**0.715**
5BW2PC250	4.1356	3.1187	**0.754**	**0.715**
5BW-VAR	4.0247	3.4818	**0.865**	**0.712**

1-(S_S=1.5g & S_1=0.6g) / 2-(S_S=1g & S_1=0.4g)

5. Design process

Rehabilitation measures shall be designed in accordance with the specified hazard level and the selected rehabilitation objectives. As indicated in the previous section, the most prevalent design parameters in the current retrofit method can be considered as: 1. total required area of strands at each floor, 2. approperiate level of prestressing load and 3. proper arrangement of cables along the height of structure. According to pushover analysis results, a simplified design procedure devised for preliminary design of a retrofitting system based on determining equivalent linear static seismic forces. In this procedure, the effective period of structure (Te) can be approximated equal to linear period of structure (Ti), which is slightly lower and can be calculated from a modal analysis or formulas proposed by different seismic codes. Since the design approach is mainly focused on keeping the brittle structure near to the elastic region, the effect is small and results would not be unrealistic. Since the retrofitting would make a dramatic increase in the yield base shear strength, Vy, it also decreases the R-factor and increases the level of lateral forces. Therefore the retrofitted structure behavior is considered different from the unretrofitted structure. Also, to generalize the design procedure, R-factors could be found for any other considered response spectrum relatively by applying a scale factor. The following simple formula can be used for this purpose. However, this is an approximate method because of slight differences in Vy when using different response spectra and caused by graphical procedure of coefficient method [3].

$$SF_R = S_a(T_i) \big/ S_{a(old)}(T_i) \quad (3)$$

Sa (Ti) can be derived from selected spectrum related to any considered seismic hazard level and Sa(old) (Ti) is obtained from the considered NEHRP [3] spectrum here (Fig. 6). The exact and approximate values of SFR for current models are illustrated in Table 3.

The lateral strength of the retrofitted structure can be considered as the summation of lateral strength of the unretrofitted structure, V(unretrofitted), and the horizontal component of the prestressed diagonal cables, Vc, [16].

$$V_{(retrofitted)} = V_{(unretrofitted)} + V_c \quad (4)$$

Accordingly, the proposed design steps are as follows:

1. Determine fundamental lateral period of the structure.

2. Determine elastic pseudo-acceleration Sa as a ratio of gravitational acceleration (g), for the first mode considering seismic hazard level and structural performance.

3. Compute effective lateral yield strength (base shear at yield) for the retrofitted (Vyr) and unretrofitted structure (Vyur) as following:

$$V_{y(r|ur)} = C_m \frac{S_a}{R} W \quad (5)$$

Where W is effective seismic weight of the building and Cm is effective mass coefficient. Recommended R-factors can be taken as 4 or 3.5 for retrofitted structure (e.g. for 5BW-VAR) and 6.9 or 5.1 for unretrofitted structure including infill walls for two considered hazard levels (BSE-2 or BSE-1).

4. Determine the lateral load applied at any floor level j for retrofitted (Frj) and unretrofitted structure (Furj) from equations (6) & (7) in which wj & hj are weight & height of story j and k is determined based on FEMA 356 [3] section 3.3.1.3.2.

$$F_{(r|ur)j} = C_{Vj} V_{y(r|ur)} \quad (6)$$

$$C_{vj} = \frac{w_j h_j^{\,k}}{\sum_{j=1}^{n} w_j h_j^{\,k}} \quad (7)$$

5. Determine required area of prestressed cables at each floor level, Acj, based on the difference between total lateral shear forces for retrofitted and unretrofitted structures from equation (8) in which fpy represents yield tension capacity of cables and θ is the angle between cables and horizontal beam.

$$A_{cj} = \frac{\sum_{j}^{n} (F_{rj} - F_{urj})}{f_{py} \cos\theta} \quad (8)$$

Because of the limited global drift demand level [15], applying prestressing loads can reduce number of needed cables significantly. But to prevent cables from yielding under excessive tension, caused by strong seismic forces, the initial prestressing load should be limited to a certain level corresponding to the limited drift demand level. As mentioned before, the failure of the weak columns is to be set at 1% lateral drift ratio. Subsequently, the prestressing loads for each strand at each story could be determined from the following equation:

$$F_{pi} = F_{pu} - 0.01 h_i \frac{A_c E_c}{L} \cos\theta \quad (9)$$

Where Fpu = fpuAp and fpu represents ultimate tension capacity of cables. Ac, Ec and L are section area, modulus of elasticity and length of each cable.

6. Nonlinear dynamic evaluation of retrofitted structure

In following section, nonlinear dynamic behavior of the retrofitted structure “5BW-VAR” is investigated under three selected earthquake records. This would evaluate the results of the pushover analysis by using the nonlinear time-history method. The ground motion records with similar site conditions and magnitude were selected and scaled to be representative of a specific hazard level. In current research, the ground motions were intended to depict the design-level motions for a building located approximately at 11 km distance from a fault rupturing with strike-slip mechanism and magnitude Ms with values of 6.6 and 7.4. The soil at the site corresponds to NEHRP Site Class C. The records characteristics are specified in Table 4.

Table 4. Characteristics of selected ground motion records

Earthquake	M+	Record	PGA(g)	PGV(cm/s)	PGD(cm)	DCFR*(km)
Imperial, Valley 15/10/1979-1417 PST	6.6	IMPVALL/ H-CPE147	0.169	11.6	4.25	27.1
Landers, 06/28/92, 11:57:34.1 UTC	7.4	LANDERS/JOS000	0.274	27.5	9.82	11.6
	7.4	LANDERS/JOS090	0.284	45.2	14.51	11.6

* Distance closest to fault rupture, + Magnitude

The IMPVALL/ H-CPE147 record were scaled based on the site specifications by using attenuation relationship presented by Campbell [19]. All records were also scaled to stand for design-level motions at building site as required by the NEHRP Recommended Provisions [3]. The scaled 5% damped elastic response spectra of the ground motions and the specified scale factors (SF) are demonstrated in Fig. 6. Analysis results including roof drift displacements and obtained base shear diagrams are illustrated in Fig. 7 to Fig. 9.

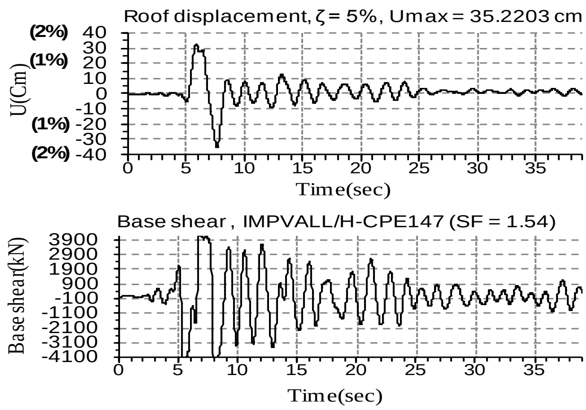

Figure 7. Nonlinear response-history analysis result for retrofitted model “5BW-VAR”, ground motion IMPVALL/ H-CPE147 scaled by factor 1.54.

Comparison of the pushover curve for “5BW-VAR” with the results of the nonlinear dynamic analyses supports the results of pushover analysis in terms of predicted maximum roof displacements and base shear forces. The retrofitted structure could withstand intense motion and acceleration. It is also recommended to apply a general safety factor on number of cables to take into account the unanticipated nature of earthquakes and complication of movements. Additional nonlinear analysis with different time-history functions might be needed for more validation of the method.

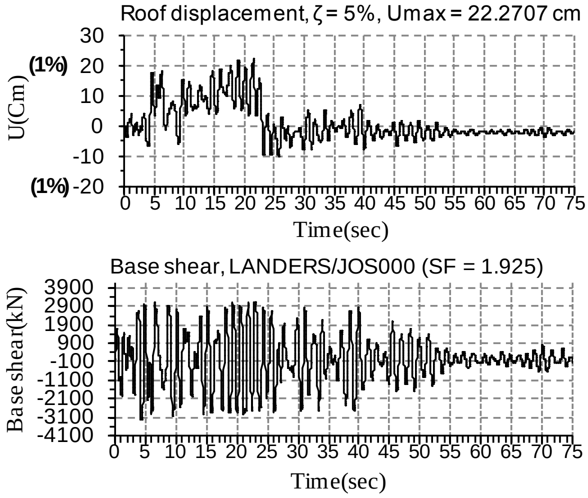

Figure 8. Nonlinear response-history analysis result for retrofitted model "5BW-VAR", ground motion LANDERS/JOS000 scaled by factor 1.925

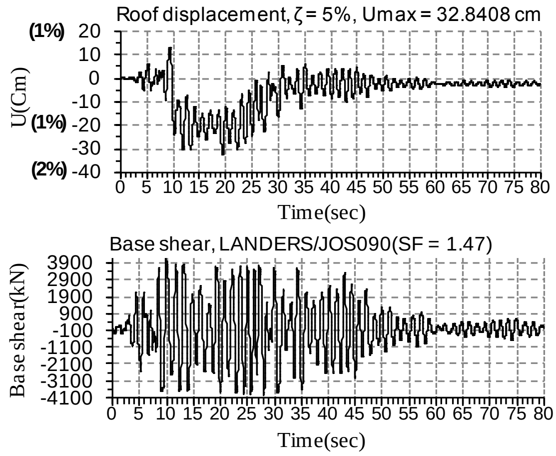

Figure 9. Nonlinear response-history analysis result for retrofitted model "5BW-VAR", ground motion LANDERS/JOS090 scaled by factor 1.47.

7. Conclusions

Based on the results, diagonal prestressing method could substantially increase stiffness of deficient R/C frames especially with considering the effect of masonry infill walls. As a result the seismic resistance of the old structures can be increased more than 100%. Deficient concrete structures with old design and inappropriate seismic detailing would not withstand lateral drifts more than 2%. Especially, lack of confinement and insufficient splice length at beam-column connections would significantly reduce the lateral strength. On the other hand, by using a proper material distribution, multistory structures retrofitted by prestressing technique would withstand plastic deformations more than 3%, enhancing the structural behavior. To design a proper retrofitting plan, demand response spectrum and also deformation acceptance criteria should be determined. The important variables in designing the retrofitting system would be number of cables, cable arrangement and level of applied prestressing forces. A simplified design method was proposed for preliminary design purposes according to pushover analysis results. The design method is based on determining linear static design seismic forces. Determined R-factors were used to calculate base shear yield values. In case of determining cable arrangement, a reversed triangular pattern in number of cables along the elevation was proposed. Functioning of designed structure was approved by conducting nonlinear static and dynamic analysis under different earthquake motion records. The retrofitted structure could successfully withstand lateral seismic forces based on the defined hazard level and the level of structural performance.

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