Seismic performance evaluation of strengthened R.C buildings

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This work examines the limits given by modern international codes for a performance-based evaluation of strengthened buildings. Four strengthening techniques of RC buildings are presented. Two of these techniques consider the main concepts of free spanning virendeel system, which is used as a retrofit technique of existing RC buildings. The other two techniques are considered by adding steel bracing with visco-elastic dampers or adding shear walls to the RC buildings. Structures with different heights of 8, 12, 16, and 20-stories at the condition of nominal deterioration represented by the appropriate hysteretic parameters are considered in this study. The performance evaluation is based on comparing the overall structural damage of both the original and retrofitted structures. The overall structural damage is represented by the overall damage indices, the maximum inter-story drift ratios, and the overall drift ratios. Time history dynamic analysis is conducted using the modified IDARC-computer program; In which, the zero-element length connection needed for two of the considered techniques is developed. It is found that the 2% limit for maximum overall drift ratio of existing buildings, which is recommended by National Earthquake Hazard Reduction Program "NEHRP" is over optimistic and the appropriate limit is 1.35%. Limits of maximum inter-story drift ratios of existing and strengthened buildings just before failure are recommended as 2.75% and 4% for existing and strengthened buildings respectively.

الهدف الرئيسي لهذه البحث هو دراسة إمكانية تقييم التلف للمنشآت الخرسانية القائمة والمدعومة عند تعرضها للزلازل. وللتقويه تم أستخدام مجموعتين من الأنظمة للتدعيم• المجموعة الأولى تشمل الأنظمة المقاومة للأحمال الجانبية وتمثل بعدد ثلاثة أنظمة مقترحة حيث يعتمد النظامين المقترحين الأول والثانى على أستغلال نظرية الفرانديل والنظام الثالث المقترح يعتمد على أضافة حوائط القص• أما المجموعة الثانية فقد تم التدعيم بإضافة شكالات معدنية مائلة تنتهى بمخمد للطاقة وذلك للتقليل من القوة المؤثرة ويزيد من درجة الخمد للمنشأ مدعوم. ولتوضيح مدى تأثير تلك الأنظمه المقترحة للتدعيم فقد تم تطوير البرنامج -IDARC Version4 بأدخال وصلة "Zero-Element Length Connection" وكذا مقارنة النتائج مع نتائج برنامج -DRAIN 2DX. وقد تضمنت الدراسة مقارنة للمعاملات المحددة تبعا لأحدث الكودات العالمية بين كلا من المنشأت الأصليه وأنظمة التدعيم المقترحة متمثلة فى القيم القصوى للأنحراف الجانبى بين الأدوار وكذلك الأنحراف الكلى للمنشأ وتلك العوامل التي تعكس السلوك الإنشائي لكل مبنى• وكذا تقييم درجة التلف التى ترتبط بالغرض من التدعيم تبعا لطريقة التصميم المقترحة لتحقيق مستوى أداء محدد وذلك لشدة زلزالية متوقعة للمنشأ وفى هذا البحث تم أعتبار معامل للتلف يراعى القيم القصوى للأزاحات الجانبية والطاقة الممتصة المتراكمة وكذا التلف الناتج عن الأجهادات المتكررة والتى تزيد عن أجهاد الخضوع.وقد بينت النتائج أن نسبة ٢% والمستخدمة كحد أقصى للأنحراف الكلى والمحددة تبعا لبعض الكودات العالمية مبالغ فيها للمنشآت الاصلية وتتناسب المنشآت المدعومة وقد أظهرت النتائج ان نسبة ١,٣٥% هى الافضل ملائمة لحالات التدهور العادية للمنشآت الاصلية. كما بينت النتائج أن النسب الملائمة للأنحراف الجانبى بين الأدوار هى ٢,٧٥% و٤% للمنشآت الأصلية والمدعومة على التوالى.

Keywords: Free-span virendeel, Seismic behavior, Damage indices, Rehabilitation codes

1. Introduction

There are several techniques for improving seismic withstands capacity of existing R.C structures. These techniques are divided into two types: a) Seismic resistant techniques which include addition of shear walls, bracing, and/or considering the main concepts of Free Spanning Virendeel System (FSVS) [1] to increase the lateral resistance and redundancy of existing buildings; b) Seismic reduction techniques which includes installation of a diagonal bracing with added supplemental damping.

The FSVS is originally developed and used by P.V. Banavalkar [2, 3] in constructing high-rise modern buildings in USA. Norwest Center [4] constructed in Minneapolis, USA is an ideal candidate for a spine structure consisting of composite super columns and FSVS. The FSVS consists of five to six stories of free spanning virendeel girder anchored by the main columns, which are going uninterrupted to the foundations. The vertical stubs are rigidly connected to the continuous horizontal beams, which in turn are rigidly connected to the main columns. The lateral resistance of the system can be divided into two parts. In the first part, resistance is provided by the frame system, without the stubs, which is termed the frame action. In the second part, shear resistance is also provided by the stubs and the beam assembly, and is termed the virendeel action.

Reinhorn et al. [7] evaluated the seismic damageability of low rise R.C building for the Memphis area of USA. The Reinhorn and Valles damage indices considered in this study indicates that the structure can withstand an earthquake with PGA of 0.1g with repairable damage. But, an earthquake with a PGA of 0.2g or greater could cause the building to collapse. Shehata et al. [8] Presented a study in which the program IDARC is used. The story drift ratio and damage indices were compared for different heights of buildings subjected to nine earthquakes to cover a wide range of different durations and different frequency contents. The obtained results indicate that the frequency content of the earthquakes play a significant rule on the seismic response of buildings.

In 1998, Elkordi et al. [5] developed two rehabilitation techniques and used DRAIN-2DX program [6] in performing the nonlinear push-over static and time history dynamic analyses. The first rehabilitation technique considers the main basis of the FSVS and dual system. In this technique, the vertical stubs added to the existing (bare) buildings are connected by a hinge. This hinge is treated as a zero-element length connection in the horizontal direction. In modeling this zero-element length connection, the shear-force, shear-deformation relationships were represented by bilinear curves and three behavior options were assumed concluding inelastic unloading, elastic unloading, and inelastic unloading with gap. The second rehabilitation technique was developed to overcome the failure problems of the first one that resulted from the unlimited vertical displacement at specific locations. At the same time, it increases the redundancy of the rehabilitated system. In this technique, the hinge is treated as a translational spring in both the vertical and the horizontal directions. In this study the comparisons were conducted on the basis of maximum lateral displacements, inter-story drift ratios, and base shear ratios between the original and two rehabilitated buildings. Damage indices and slip control parameters were not included causing major limitations in such study.

In 2003, Elkordi et al. [9], proposed a modification for the well known IDARC program-version4 [10] that succeeded to model the translational spring element in the horizontal and/or the vertical directions. In this study, the proposed model of translational spring was based on two relationships. The first one was the shear force–shear strain relationship under monotonic loading. The second relationship was the unloading and reloading branches of hysteresis loops under cyclic loading. The primary curve of translational spring was the tri-linear model, which was established using well-defined cracking, yield, and ultimate loads and was defined as the envelope curve for the hysteretic relationship. This curve was used to define the boundary of shear strength for the purpose of modeling. The crack loads, yield loads, and ultimate loads are determined based on the section properties of the hinge connection. In this work, a comparison between the results of the 2003 study and the work done by Elkordi et al. 1998[5], using the computer program DRAIN-2DX [6] was made. It showed that the results of the two programs were in a very good agreement. The study concluded that the modification made to the IDARC computer program is a very good tool to simulate the zero-element length connection. In this study only two techniques of strengthening were used considering the main concepts of FSVS. In addition, a damage evaluation for the existing buildings and these two rehabilitation techniques was conducted by Elkordi et al. [11] using the overall damage indices. It was concluded that drift ratios and damage indices can evaluate and predict the degree of damage for the different limit states of structures. Based on this evaluation, it was possible that one can decide which design or retrofit options can be implemented.

In the present study, additional two techniques commonly used are presented to strengthen existing RC buildings. The first strengthening is achieved by installing a diagonal bracing with visco-elastic dampers to increase the damping capacity of the building. The second one is achieved by adding shear walls to increase stiffness and reduce drift and damage of existing RC buildings. The evaluation of the strengthened and existing buildings is conducted by comparing: a) the values of overall damage indices of Reinhorn and Valles [12] to a limit value of one representing loss of building, b) the maximum inter-story drift ratios according to the standard of rehabilitation registered by Federal Emergency Management Agency "FEMA-356" [13], and c) the overall drift ratio in accordance with National Earthquake Hazard Reduction Program (NEHRP-1985) [14].

2. Assessment of damage state

2.1. Drift ratio

The overall drift which is defined as the roof displacement divided by the building height is sometimes used to evaluate seismic performance of buildings. The BSSC "Basic Seismic Safety Council" [15] specifies the maximum inelastic drift ratio to be 2% for framed office buildings. Thus, a value of 2% is considered as the threshold of extensive damage in most buildings in accordance with NEHRP-1985 [14].

2.2. Damage index

Damage indices are usually used to indicate how close is the maximum response to the maximum ultimate capacity of the structure under monotonic loading. The fatigue based damage model introduced by Reinhorn and Valles [12] and considered in IDARC-version4 [10] is used for this study. It was developed on the basis of maximum structural response considerations and a low-cycle fatigue rule. The index is defined as:

$$DI = \left(\frac{\delta_m - \delta_y}{\delta_u - \delta_y}\right) \frac{1}{\left(1 - \frac{E_h}{4(\delta_u - \delta_y)F_y}\right)} . \qquad (1)$$

Where; δ_m is the maximum deformation; δ_y is the yield deformation capacity; δ_u is the ultimate deformation capacity and determined from empirical formulas derived from experimental data [10]; F_y is the yield force capacity; and E_h is the cumulative dissipated hysteretic energy.

Overall damage indices estimate the overall state of the structure. These parameters reflect damage condition of the entire structure. For establishing the story damage index $(DI)_{story}$, a weighing factor is considered based on the energy absorbed by the elements and determined as follow:

$$DI_{\ story} = \sum (\lambda_i)_{component} \ (DI_i)_{component} ;$$
$$(\lambda_i)_{component} = \left[\frac{E_i}{\sum E_i}\right]_{component} . \qquad [2]$$

Where λ_i are the energy weighing factors; and E_i are the total absorbed energy by the component or story "i". According to FEMA-Standards [13], the building performance can be described qualitatively in terms of: a) the safety afforded by building to occupants during and after the event; b) the cost and feasibility of restoring the building to pre-earthquake condition; and c) the length of time in which the building can be removed from service status to repair case status. These performance characteristics are directly related to the extent of damage that would be sustained and represented by damage indices. Recently, the Vision 2000 Committee of Structural Engineers Association of California "SEAOC-1995" [16], has envisioned a performance-based overall design process that consists of three phases termed as the conceptual phase, the numerical phase, and the implementation phase. Amador et al. [17] presented a performance-based design procedure including numerical design methodologies. In this proposed procedure, qualitative definition of the desired behavior of the building for different levels of ground motion could be determined through the use of damage indices. This quantification leads to establishing limits to the maximum demands of all response parameters.

Thus, to assess the seismic performance of different rehabilitated building structures, some parameter such as the overall drift ratios, maximum inter-story drift ratios and overall damage indices are presented in this study. The overall drift ratios and maximum inter-story drift ratios are measures of lateral displacement of the structures. And, damage indices parameters consider both maximum inelastic response and dissipation of energy during the input motion. In this study, the input acceleration records are scaled to achieve the specified peak ground accelerations. The choice of time step of the nonlinear analysis may cause numerical instabilities especially in the case just before failure; and hence results in extremely large values of the damage indices (DI >> 3.0) of some or all elements. The recommended time step used for response analysis by Park et al. [10] is 0.005 sec. In the present study, a value of 0.002 sec is used as the time interval for the input data. Failure conditions are determined as follow: 1. Limit value of overall damage indices should not exceed 1.0 [12]. 2. The values of maximum inter-story drift ratios should not exceed 4.0% (FEMA-356). And 3.The overall drift ratios should not exceed 2% (NEHRP-specification). Hence, to examine the limits given by modern codes taken into account the choice effect of the considered time step at higher dynamic responses, the results of different buildings loaded to a value of 3.0 as a limit to ODI (in accordance to IDARC-program for nonlinear time history dynamic analysis); is also examined.

3. Examined buildings

The seismic performance of four RC buildings with different heights of 8, 12, 16, and 20-stories respectively, is assessed. Each building has three bays with a span of 4.5m and story height of 3m. Interior frames are selected to conduct this analysis. In the design procedure, building materials are assumed to be 250 kg/cm^2 concrete and Grade 36/52 steel reinforcement. Five structural systems, each of 8S, 12S, 16S, and 20S-buildings are considered concluding the existing buildings and four strengthened cases. The four bare buildings are labeled as 8R, 12R, 16R and 20R for the 8, 12, 16 and 20-stories buildings respectively. These existing buildings are designed according to the Egyptian Code [18]. The First Strengthening Techniques (FST) consists of vertical steel elements, stubs, connected to the horizontal strengthened girders of the original structural system in the middle bays fig. 1. The translational spring connection between these vertical stubs capable of transmitting only the horizontal shear and insures that the vertical load transfers only to the main columns and hence the vertical stubs act as shear membrane only. The 8, 12, 16, and 20-stories buildings in which this strengthening technique is used are labeled as 8F, 12F, 16F and 20F respectively. In the second strengthening technique fig. 2, and in addition to the horizontal translational spring, a vertical translational spring is used in the middle bays to control the unlimited vertical displacements resulting at the connection. This procedure increases the redundancy and is termed as SST. The 8, 12, 16 and 20-stories buildings in which this strengthening technique is used are termed as 8S, 12S, 16S and 20S, respectively. A third strengthening technique considered a shear wall addition to the building. This procedure increases the redundancy and is termed as 8W, 12W, 16W and 20W for the 8, 12, 16, and 20 story buildings respectively fig. 3. The fourth strengthening technique considers the additional of diagonal bracings with Visco-Elastic (VE) dampers to the building figs. 4-1 to 4-4". This procedure is termed as 8V, 12V, 16V and 20V for the 8, 12, 16, and 20-story buildings respectively.

4. Method of analysis

IDARC-program does not include the translational spring connection, and hence a

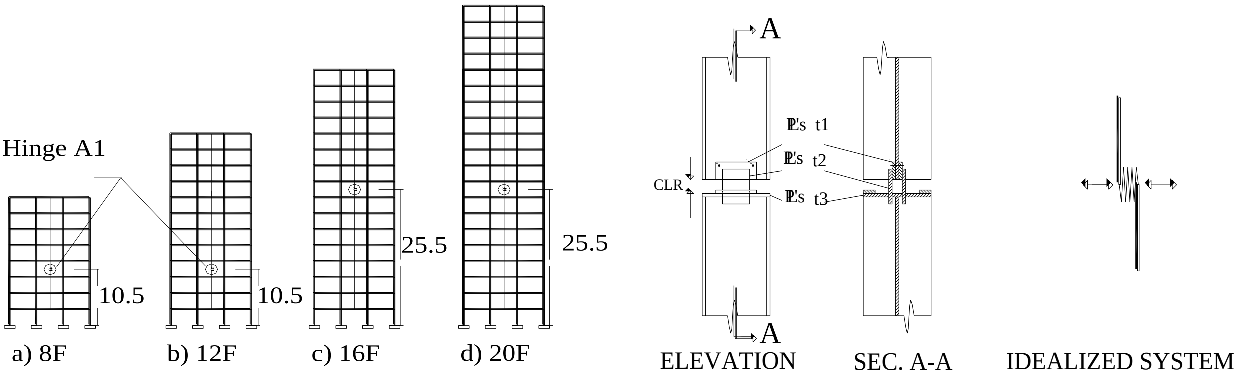

Fig. 1. Elevation of FST and details of hinge A1 of FST (Elkordi et al.-1998).

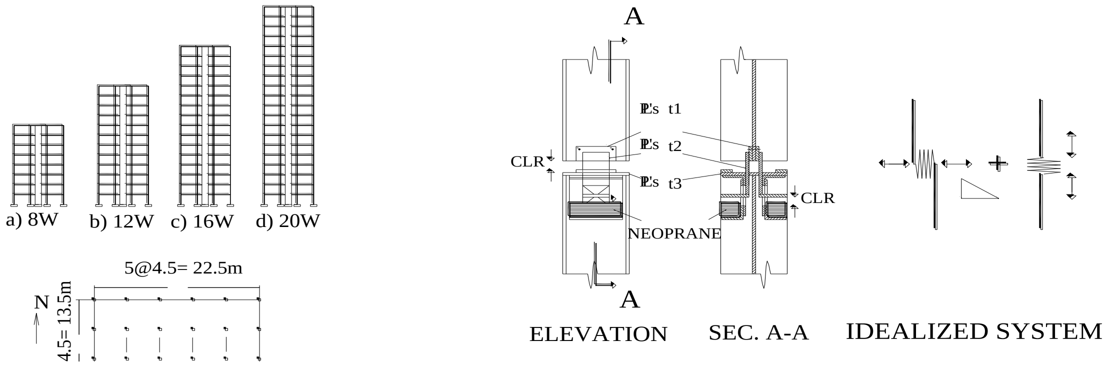

Fig. 2. Elevations of SST and details of hinge A2 (Elkordi et al.-1998).

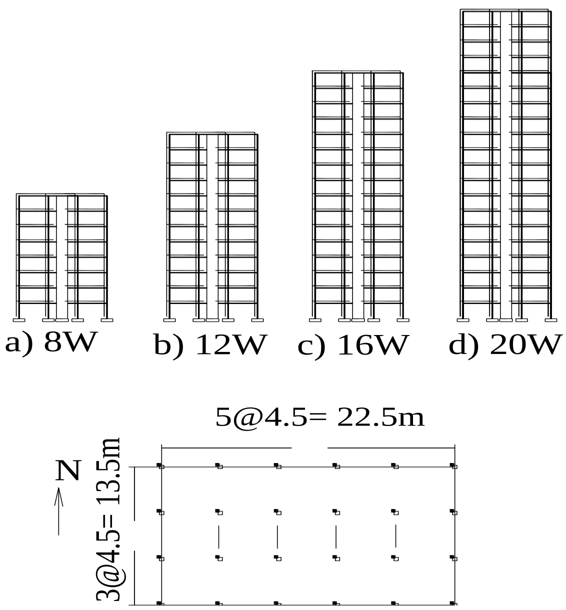

Fig. 3. Plan and elevations of buildings with shear walls.

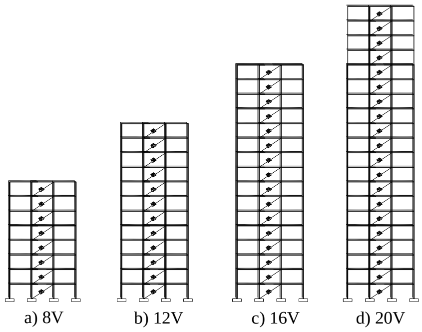

Fig. 4-1. Buildings with viscous dampers.

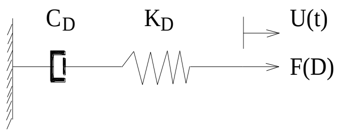

Fig. 4-2. Maxwell model for VE-damper [10].

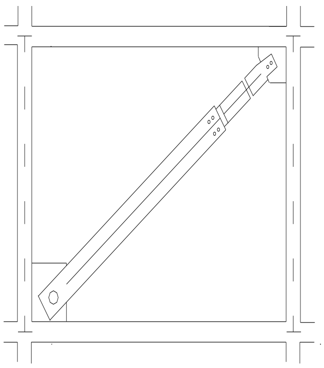

Fig. 4-3. Installation of VE damper [10].

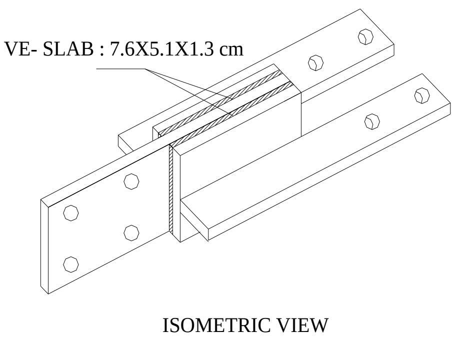

Fig. 4-4. VE-damper constrained layer shear damper [19].

modification has been made to the program by the authors to simulate a translational zero-element length connection. The used property of the translational spring is a tri-linear curve in a conjunction with three hysteretic parameters to control the unloading and reloading cycles. In addition, the VE-damper is modeled in IDARC-program with an axial diagonal element, and the dynamic stiffness is calculated considering Maxwell model. The Maxwell model consists of a damper and a spring in series fig. 4-2.

In 1993, the effectiveness of strengthened buildings with VE-dampers and friction dampers is evaluated by comparing the response of the bare buildings and strengthened ones with energy dissipators by Aiken et al. [19]. The study presented a summary of the results of 1/4 scale 9-story steel structure using the acrylic copolymer 3M VE shear damper. The VE dampers were added to the Moment Resisting Frame (MRF) in a diagonal bracing providing the steel structure with a specified level of damping (10% of critical). It is concluded from the obtained results that VE dampers have no activation force level as been for friction dampers and thus they dissipate energy and reduce drifts and deformations of MRF for all levels of earthquake excitations. However and from response comparisons for ELC, Taft, and Miyagi time history records, drifts and story accelerations in buildings using Viscous Dampers were reduced by 60% over those of the bare buildings. Also, the VD models experienced no yielding in any of earthquake tests.

In 1996, Chang et al. [20] also analyzed results of shaking table studies to examine the effect of strengthening on the inelastic behavior of two identical 2/5-scale three story steel structures considering the conditions of bare frames, and frames with visco-elastic dampers. The visco-elastic dampers role was to increase the hysteretic damping in the structure. These dampers were designed to provide the test structure with two levels of damping as 8% and 15% of critical damping ratios at an ambient temperature of 28°C. Results concluded from this study suggested that a VE damped structure with sufficiently large damping may remain elastic under strong earthquake ground motions. However, with the smaller design damping ratio (8% of critical), the test structure dissipated the seismic input energy through both viscous and hysteretic damping under strong earthquake ground motions. On the other hand, and with the larger design damping ratio (15% of critical), the structure remained nearly elastic under the same strong earthquake by dissipating the seismic input energy primarily through deformation of the VE dampers.

In the present study, the characteristics of the VE-dampers considered by Chang et al. [20] and based on the modal strain energy method is used. The damper is designed for the following parameters: 1) Design temperature: 28°C; (2) Design damping ratio: 15%; (3) Design damper strain: 60% at 0.5% story drift, corresponding to the maximum elastic story drift subjected to the design lateral force; (4) VE material with shear storage modulus G' = 0.06 kN/cm^2 at the shear strain of 60% and frequency of 1.6Hz of the VE dampers. The damper storage stiffness, K', is estimated to be 3.5KN/cm. The VE damper comprises two layers of VE material fig. 4-4 and the dimensions of the damper's layers thickness are 2x7.6 x 5.1x 1.3 cm.

The hysteretic curve considered in modeling the beams, columns, and walls uses three parameters in conjunction with a tri-linear curve to establish the rules under which inelastic loading reversals takes place. A variety of hysteretic properties can be achieved through the combination of the tri-linear envelope and the three parameters, henceforth to be referred to as stiffness degrading parameter, strength degrading parameter (energy-controlled), strength degrading parameter (ductility-based), and slip or crack closing parameter or HC (α), HBE (β), HBD (β), and Hs (γ) respectively. These parameters are assumed to be 10, 0.1, 0.1 and 1.0 to represent the case of nominal deterioration condition (Park et al. [10]) as shown in fig. 5.

Table 1 depicts the fundamental periods and weights of the existing buildings and strengthened ones. The periods of seismic resistant buildings (in particularly the cases of shear wall) are lower than the original buildings. This indicates that the strengthened buildings are stiffer than original ones and hence it is expected that strengthened buildings may sustain higher levels of overall deformation than the original buildings but may suffer from higher stresses than the un-strengthened buildings.

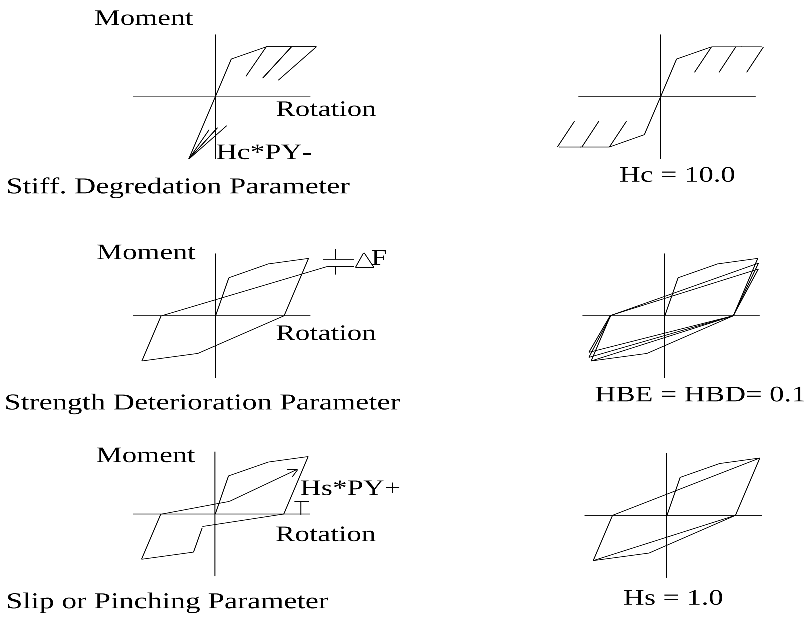

Fig. 5. The deterioration parameters for the three parameters hysteretic model at the case of nominal deterioration (Park et al. [10]).

Table 1
Periods of buildings

Building	Case	Weight (tons)	Fundamental period (sec)
8-Story	8R	375.72	0.69
	8F	375.75	0.60
	8S	376.71	0.61
	8V	375.72	0.69
	8W	392.59	0.37
12-Story	12R	579.83	0.97
	12F	579.83	0.89
	12S	580.83	0.88
	12V	579.83	0.97
	12W	605.70	0.71
16-Story	16R	792.46	1.35
	16F	792.46	1.22
	16S	793.46	1.22
	16V	792.46	1.35
	16W	827.33	1.04
20-Story	20R	1025.16	1.59
	20F	1025.16	1.48
	20S	1026.16	1.48
	20V	1025.16	1.59
	20W	1069.04	1.31

5. Discussion of the dynamic analysis

In the nonlinear time history dynamic analysis of each building, two standard earthquake acceleration records are used; the S00E component of El-Centro earthquake "ELC", and the S69E component of Taft earthquake "TAFT". To differentiate between the obtained results from each record for the existing and strengthened buildings, a character E or T is used and will proceeds the mentioned labels of buildings to represent ELC or TAFT obtained results, respectively. For example, E8R, and T8R represents un-strengthened 8-story building case when subjected to ELC and Taft earthquake records respectively.

The acceleration records are scaled so that the Peak Ground Acceleration (PGA) is set to be 0.1g and increased at 0.05g intervals until failure occurs. The modified IDARC-computer program concluding the zero-element length connection is selected to perform the inelastic dynamic time history analysis. The damping coefficient considered is 2% for existing and strengthened buildings. For the cases of additional bracing ended with VE-dampers, the damping ratio is 15%. The hysteretic model used in the modified-IDARC program is capable of modeling strength deterioration and pinching effect in addition to stiffness degradation.

Figs. 6 to 9 and table 2 depict the strengthening evaluation represented by the strengthening effect on the relation between PGA(g), and Base Shear Ratios (BSR), Overall Drift Ratios (ODR(%)), maximum Inter-story Drift Ratios (IDR(%)) and the Overall Damage Indices "ODI" of existing buildings and strengthened ones, respectively, till loss of building "ODI = 1.0 ".

5.1. Relation between peaks ground acceleration (PGA) and damage (ODI)

Fig. 8 shows the relationship between the Peak Ground Acceleration (PGA) and the overall damage indices "ODI" of the bare and strengthened buildings when subjected to the scaled ELC and TAFT earthquakes. It is obvious that the Overall Damage Indices (ODI) increase almost linearly with the PGA. Thus, the strengthened techniques are effective in increasing the lateral resistance of original buildings and sustain higher levels of PGA at the same level of damage from those of existing ones.

Table 2
The values of PGA, BSR and ODR (%) of existing and strengthened buildings
and the increase of strengthened ones to those of existing buildings "ODI = 1.0 [10]".

Case	Elc eq.						Taft eq.					
	PGA(g)		BSR		ODR(%)		PGA(g)		BSR		ODR(%)	
	value	Ratio	value	Ratio	Value	Ratio	value	Ratio	value	Ratio	value	Ratio
8R	0.43		.1394		.572		0.52		.1740		.639	
8V	0.68	58.1	.2037	46.1	.793	38.4	0.82	65.9	.2531	45.4	.815	27.6
8F	0.72	67.4	.2658	90.7	.781	36.4	0.73	40.4	.3105	78.4	.848	32.7
8S	0.74	72.1	.2731	95.9	.787	37.5	0.74	42.3	.3448	98.1	.865	35.4
8W	0.60	39.5	.4909	252.	.734	28.3	0.69	32.7	.4804	176.	.680	6.4
12R	0.38		.1043		.502		0.51		.1262		.473	
12V	0.79	107.9	.1882	80.5	.780	55.4	0.60	18.0	.1651	30.9	.801	69.1
12F	0.58	52.63	.1763	69.0	.718	43.0	0.70	37.3	.2114	67.6	.780	64.6
12S	0.60	57.90	.1794	72.1	.727	44.8	0.72	41.2	.2167	71.7	.750	58.5
12W	0.63	65.79	.2276	118.	.669	33.3	0.60	17.6	.2836	125.	.601	27.0
16R	.329		.0832		.422		0.38		.0925		.423	
16V	0.61	85.41	.1473	77.0	.686	62.6	0.55	44.7	.1183	27.9	.653	54.4
16F	0.48	45.9	.1239	48.9	.577	36.8	0.60	57.9	.1571	69.9	.475	12.5
16S	0.48	45.9	.1248	49.9	.576	36.7	0.60	57.9	.1576	70.4	.483	14.2
16W	.445	35.3	.1427	71.5	.482	14.4	0.49	28.9	.1663	79.9	.434	3.0
20R	.432		.1077		.463		0.41		.0928		.358	
20V	0.68	57.4	.1628	51.1	.776	67.4	0.58	41.5	.1302	40.4	.696	94.2
20F	0.60	38.9	.1551	43.9	.590	27.1	0.69	68.3	.1629	75.6	.634	76.9
20S	0.61	41.2	.1559	44.7	.591	27.6	0.68	65.9	.1642	77.0	.620	73.0
20W	0.57	31.9	.1795	66.7	0.50	8.0	0.62	51.2	.1687	81.9	.592	65.1

The increase of lateral resistance represented by PGA to level of damage represented by ODI of strengthened buildings from those of existing buildings till loss of building "ODI = 1.0[12]" are depicted in table 2. However, the level of damage is independent of the type of strengthening except the shear walls cases until about a value of 0.6 damage index and after that there is a significant divert between the strengthened buildings and the existing ones. This observation indicates that the strengthening of the structural system is very efficient after the moderate state of damage (Park et al. [22]) and also viscous dampers additions play an important role in reducing damage of original buildings. In addition, The increase of lateral resistance represented by PGA, ODR, BSR and the values of IDR (%) to level of damage represented by ODI of strengthened buildings from those of existing buildings till collapse occurs ODI ≤ 3.0 [12] are also depicted in table 3.

From these results, two conclusions are drawn. The first one is that the first "FST" and second "SST" strengthening techniques are effective in reducing the damage of original buildings and hence they are recommended to mid-to high rise buildings subjected to severe seismicity. The second conclusion is that considering the addition of viscous dampers technique is more effective for mid to high rise buildings than that other cases. Since, at the same level of damage, the VD buildings can sustain higher levels of PGA fig. 6. The increasing of PGA levels for VD buildings is ranging from 57.0 to 108% for ELC results and from 18% to 57.0% for TAFT results. The difference between the ELC and TAFT obtained results may be attributed to differences in intensities, amplitudes of records and also frequencies between the records and studied cases. From fig. 6, VD-buildings that represent seismic reduction techniques are recommended for lower level of damage at the same PGA than the seismic resistant ones.

Table 3 depicts the same results to a level of damage indices ODI ≤ 3.0. From these results, the effectiveness of VD buildings in resisting higher PGA for mid-to high rise buildings is obvious when compared with lower ones. The FST and SST are the best ones can sustain higher levels of PGA within the considered levels of damage. The increase ratios compared with existing buildings are ranging from 103% to 146% for ELC results and from 60% to 123% for TAFT results. The FST results is the better for higher ones than that SST results as depicted in table 3.

5.2. *Effect of strengthening on Base Shear Ratio (BSR)*

The results of dynamic analysis shown in fig. 7 and depicted in tables 2 and 3 are presented in the form of the effect of strengthening for the existing buildings considering the base shear ratios "BSR" (base shear divided by the building weight) for 8, 12, 16, and 20-story buildings when subjected to ELC and TAFT-earthquakes.

From the fig., it is noticed that the strengthened techniques are very effective in increasing the lateral strength of existing buildings in particularly the cases of shear walls addition. Figure shows also that up to the yield, the existing and strengthened systems demonstrate almost the same level of damage at the same base shear force.

Reviewing fig. 7 and table 2, it should be noted that both of first FST and second "SST" strengthening buildings are very effective in increasing the lateral strength than those from existing buildings. In addition, it is noticed that the lateral strength of the additional bracing technique with VE-dampers in increasing the lateral strength of existing buildings at the same level of damage indices is small comparing to some other techniques as shear wall additions. The increase of BSR

Table 3
The values of PGA, BSR and ODR(%) of existing and strengthened buildings
and the increase of strengthened ones to those of existing buildings ODI ≤ 3.0 [10].

Case	Elc eq.						Taft eq.					
	PGA(g)		BSR		ODR (%)	IDR (%)	PGA(g)		BSR		ODR (%)	IDR (%)
	Value	Ratio	Value	Ratio	Value	Val.	Value	Ratio	Value	Ratio	Value	Val.
8R	.625		.1712		1.326	2.71	.713		.1740		1.05	2.57
8V	.715	14.4	.2062	20.4	1.517	3.10	0.82	14.5	.2531	45.4	.815	3.25
8F	1.35	116.	.3272	91.1	1.991	3.32	1.14	59.9	.3105	78.4	1.66	2.93
8S	1.28	105.	.3320	93.9	2.00	3.46	1.18	65.5	.3448	98.1	1.62	1.82
8W	1.39	122.	.6678	290.	1.754	2.01	0.99	38.9	.4804	176.	1.84	2.09
12R	0.64		.1668		1.159	3.06	0.73		.1262		.886	2.01
12V	0.93	45.3	.2270	36.1	1.676	3.44	0.80	9.6	.1651	30.9	2.00	3.47
12F	1.47	130.	.2908	74.4	1.988	2.94	1.48	103.	.2114	67.6	1.87	2.68
12S	1.30	103.	.2814	68.8	1.673	2.86	1.63	123.	.2167	71.7	2.00	3.47
12W	1.25	95.3	.3650	119.	1.660	3.04	1.27	73.3	.2836	125.	1.54	2.03
16R	.672		.1586		1.142	2.25	0.71		.0925		.930	1.86
16V	1.01	50.3	.2030	28.0	1.941	2.8	0.85	19.7	.1183	27.9	1.82	3.77
16F	1.39	107.	.2393	50.9	1.733	3.11	1.36	91.5	.1571	69.9	1.48	2.79
16S	1.38	105.	.2455	54.8	1.648	3.37	1.37	93.0	.1576	70.4	1.50	2.83
16W	0.90	33.9	.2679	68.9	1.238	1.88	1.04	46.8	.1663	79.9	1.17	2.78
20R	0.56		.1292		1.212	2.66	0.71		.0928		1.12	2.24
20V	1.09	94.6	.2241	73.4	1.92	2.78	1.08	52.1	.1302	40.4	1.67	3.3
20F	1.46	161.	.2341	81.2	1.679	3.13	1.27	78.9	.1629	75.6	1.22	2.35
20S	1.38	146.	.2374	83.7	1.520	3.06	1.23	73.2	.1642	77.0	1.20	2.33
20W	.926	65.4	.2429	88.0	1.95	3.51	1.15	61.3	.1687	81.9	1.15	3.02

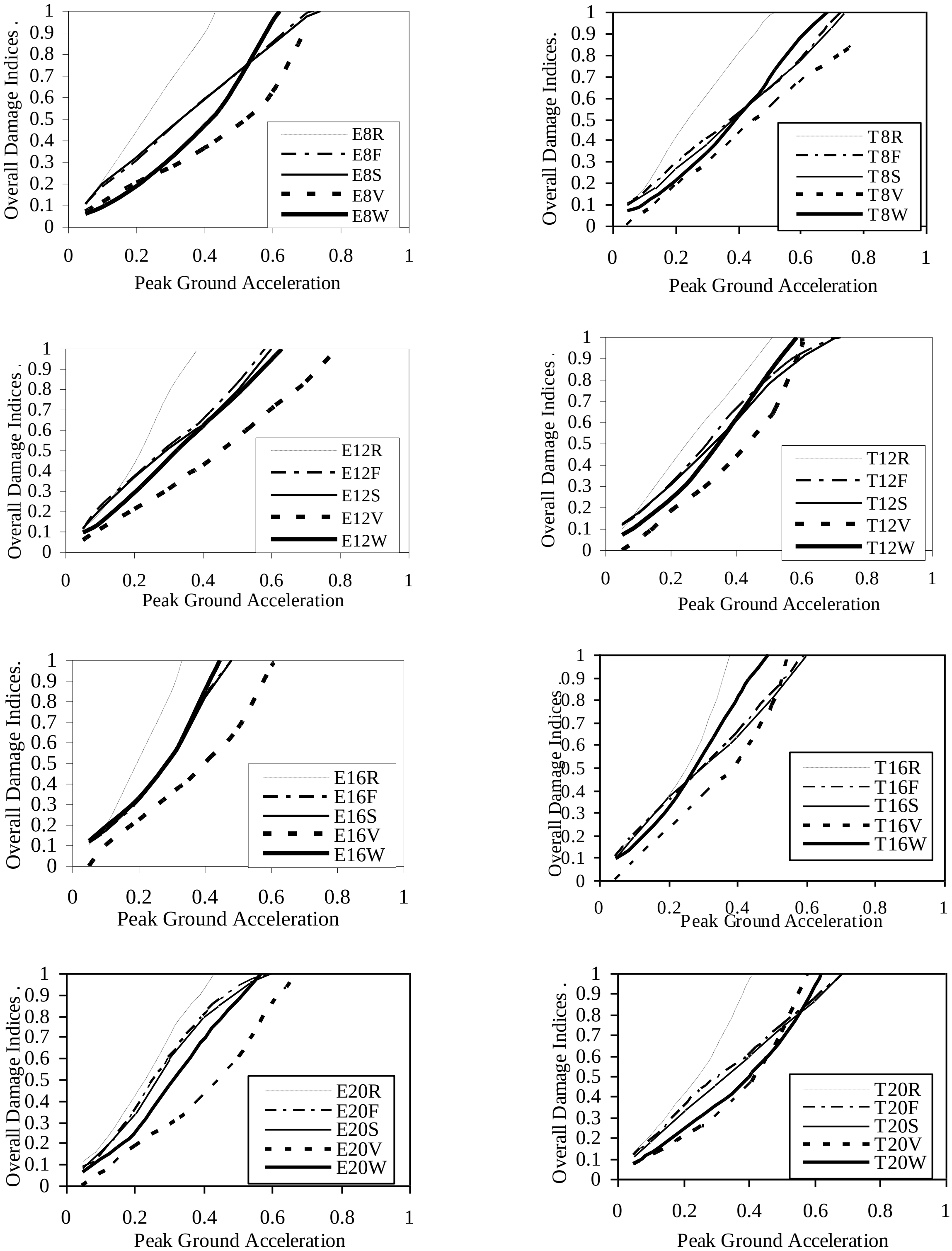

Fig. 6. Relationship between the overall damage indices and peak ground acceleration of 8, 12, 16, and 20-story buildings.

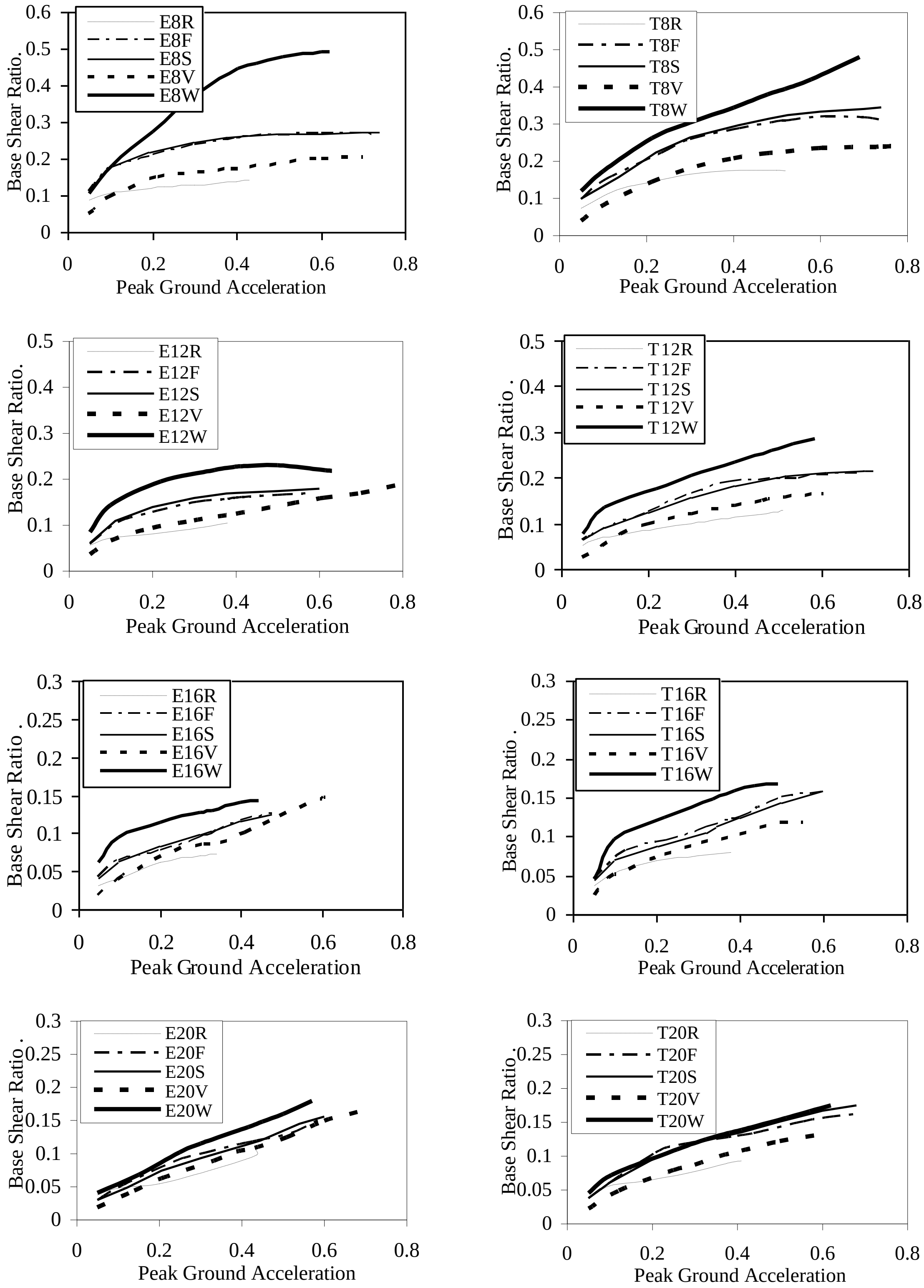

Fig. 7. Effect of strengthening techniques on the base shear ratios for 8, 12, 16, and 20-story buildings.

for VD buildings is directly proportional to the heights of existing buildings and ranging from 46% to 81% for ELC results and from 28% to 45% for TAFT results. Hence, the additional bracings with VE-dampers are effective for higher buildings than shorter ones. Based on the obtained results, the shear wall addition technique is more effective (in particularly for mid-rise buildings for 8S and 12S) in increasing the lateral resistance represented by BSR than that from the other cases of strengthening at the same level of damage table 2 and this is attributed to the increasing in mass, stiffness and also redundancy. Hence, it is obvious that the seismic resistant techniques are more effective especially the case of shear wall addition than the additional bracing technique with VE dampers in increasing the lateral strength of existing buildings at the same level of damage indices.

5.3. Effect of strengthening on drift ratios

The NEHRP [14] specifies a value of 2% as the maximum inelastic overall drift ratio "ODR". The results shown in Fig. 8 show the effect of strengthening for the existing buildings considering the overall drift ratio "ODR" for the 8, 12, 16, and 20-stories buildings. It is noticed that the strengthened techniques are effective in reducing the lateral response from those of existing buildings at the same level of damage for all cases of strengthening. As can be seen from Figure 8, there is an increase in the overall drift ratio of the strengthened buildings from those of existing buildings when subjecting to ELC and TAFT earthquake records respectively, can be recognized. However, the response of different cases is different in some cases in particularly as been for shear walls buildings "fig. 8" and this is may be attributed to differences in frequencies of records and considered cases. Also, it is obvious that the 2% limit for maximum drift ratio is over optimistic for bare buildings and not to the strengthened ones and hence values of 1.35% and 2.0% "table 3" under the nominal deterioration condition are the appropriate for original and strengthened buildings respectively. From the obtained results shown in fig. 8 and depicted in table 2, the appropriate limits for original and strengthened buildings limits at the nominal deterioration condition are 0.75% and 1.0% respectively. The figure shows also that the strengthened techniques reduce the overall drift ratios than the existing ones at the same level of damage and hence sustained more damage when subjected to higher levels of ODR. The obtained results are in good agreement with the comparative evaluation of seismic assessment applied to a 32S-building conducted by Memari et al. [24].

Fig. 9 shows that there is a linear relation between the PGA and maximum inter-story drift ratios of the structural systems when subjected to scaled ELC and TAFT earthquakes. Both the seismic resistant and reduction strengthening techniques increased the lateral resistance of existing buildings at the same level of maximum inter-story drift ratios. In addition, at the same level of PGA, the inter-story drift ratio is higher for the existing buildings than those from strengthened ones. The range of maximum inter-story drift sustained by strengthened cases may be higher than those of the original cases. According to the obtained results, it is obvious that the 4% rule of thumb for maximum inelastic drift ratio according to FEMA 356 is over optimistic for existing R.C frames. Hence, limits of 2.75%, and 4% might be more appropriate for the existing buildings, and strengthened ones respectively table 3.

6. Conclusions

Four strengthening techniques to existing R.C buildings are presented. Two of them are developed based on Free Spanning Virendeel concepts. The other two are implemented by installation of shear walls or diagonal steel bracing with visco-elastic dampers. Two ground motion records are used to conduct the time history analysis; namely ELC and TAFT records. Evaluation of these techniques considering nonlinear time history dynamic analysis is carried out. The results of the dynamic analyses show that the values of overall damage indices and overall or maximum inter-story drift ratios vary significantly. The following conclusions and recommendations are made.

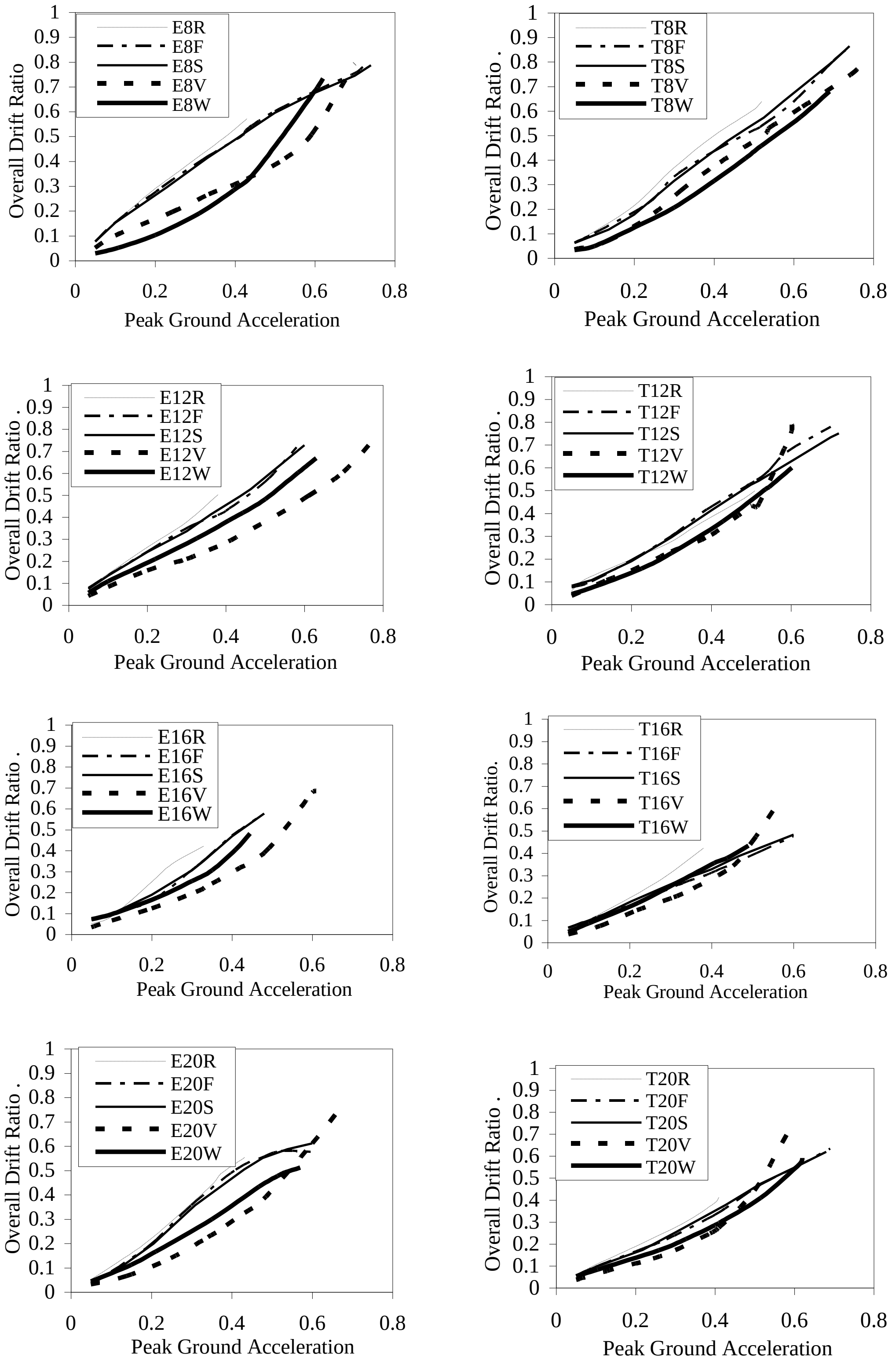

Fig. 8. Effect of strengthening on the overall drift ratios of 8, 12, 16, and 20-story buildings.

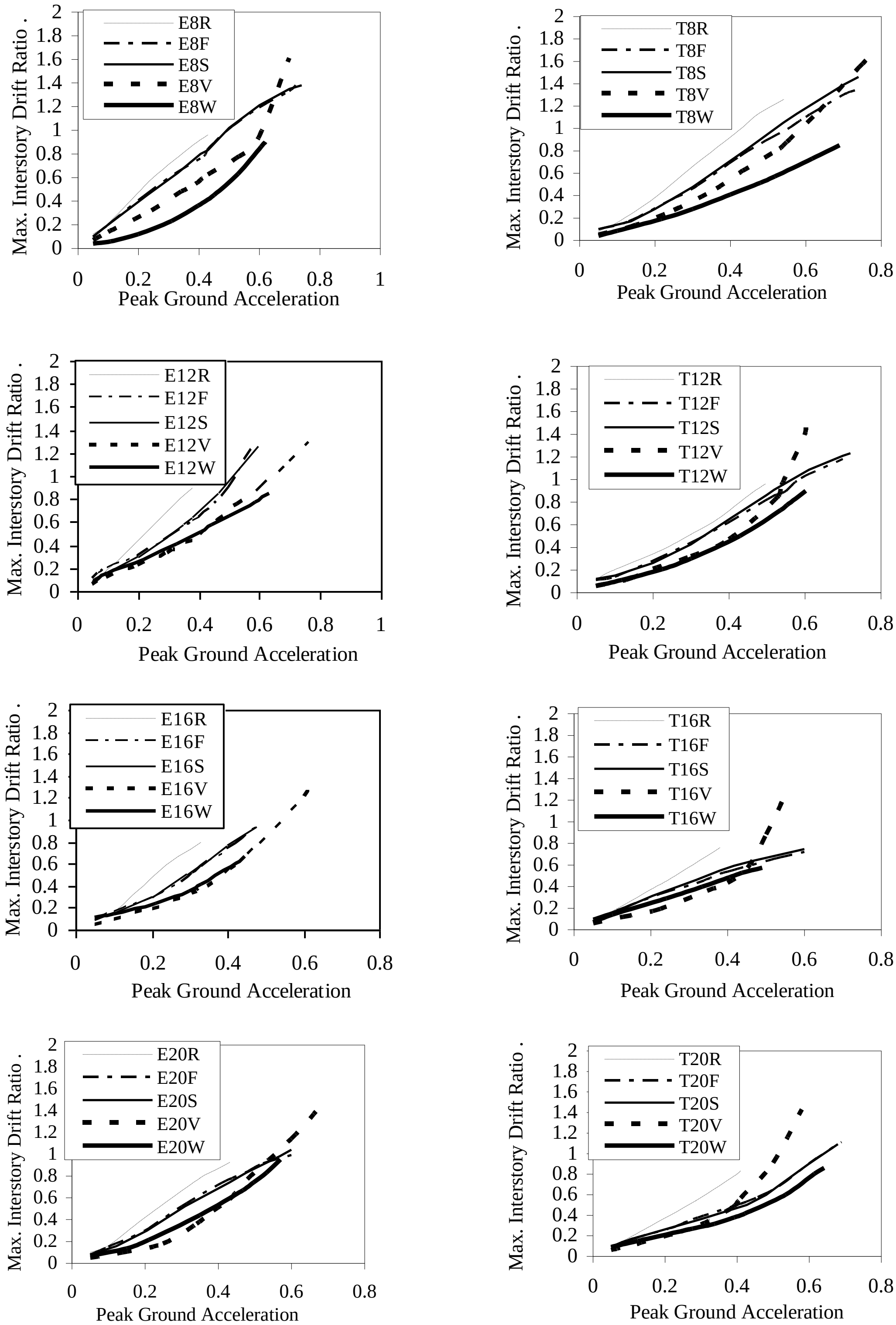

Fig. 9. Relationship between the peak ground acceleration and the maximum inter-story drift ratios of 8, 12, 16 and 20-story buildings.

1. The 2% rule of thumb set by National Earthquake Hazard Reduction Program "NEHRP" for maximum overall drift ratio is over optimistic for existing buildings. The study concluded that the appropriate values of existing and strengthened framed RC structures are 1.35% and 2.0%.
2. The 4% rule of thumb for maximum inelastic drift ratio according to Federal Engineering Management Agency standards for rehabilitation "FEMA-356" is properly for seismic resistant techniques and seismic reduction techniques of framed RC structures. However, value of 2.75% might be more appropriate for existing framed RC structures.
3. The seismic resistant techniques especially the cases of shear wall additions are more effective than the seismic reduction techniques in resisting severe earthquakes with increased PGA. In addition, the resistance of seismic reduction techniques is directly proportional to the height of buildings with increased ratio than the existing buildings ranging from 12% for 8-story to 63% for 20-story buildings with added bracing ended with visco-elastic dampers.
4. Strengthened structures and its performance-based design should be concluded in the Egyptian Codes of rehabilitation and also the damage indices should be carefully studied in the numerical phase for any design of new or rehabilitated buildings to satisfy a target performance objective.

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