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SEISMIC CAPACITY EVALUATION OF EXISTING REINFORCED CONCRETE BUILDINGS IN TURKEY (EFFECT OF RETROFIT)

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SUMMARY

Following the 1999 Kocaeli earthquake, seismic evaluation and retrofits of existing buildings have been promoted in the Republic of Turkey. This paper describes seismic capacities of ten existing reinforced concrete buildings in Istanbul evaluated by the Japanese Seismic Capacity Evaluation Standard. Since six of the buildings have been strengthened, the seismic capacities of both original and strengthened buildings were evaluated and the effect of strengthening was examined. The buildings were from two to twelve stories. The methods of strengthening were jacketing of columns and/or adding new reinforced concrete walls. The buildings were strengthened to conform to the 1997 Turkish Seismic Design Code, which is considered to be similar to the Japanese Seismic Design Code in seismic capacity level.

The Japanese Seismic Capacity Evaluation Standards consists of the first, second and third screening method. The second screening method, which evaluates the buildings seismic capacity based upon the strength and ductility of columns and walls assuming a strong beam concept, is mainly used. However, since the failure modes of the strengthened buildings were expected to be beam failure types, because their failure mechanisms might be changed from column and wall failure types to beam failure types as a result of heavily strengthening columns and walls. Therefore, in order to verify the results of the evaluation, a building, which was one of the typical Turkish current buildings, was chosen and a pushover analysis and a dynamic response analysis were carried out.

Major findings are; 1) All but one of the original buildings were evaluated vulnerable by the Japanese seismic safety criteria 2) Seismic capacities of six strengthened buildings have been improved much and evaluated as safe. 3) A pushover analysis and a dynamic response analysis of a typical Turkish building showed similar seismic performance and failure mode to those obtained by the seismic evaluation. 4) The failure mode of the strengthened typical building was beam failure type by pushover analysis. 5) The maximum story drift angles by the dynamic response analysis were less than 1/100 to scaled El Centro NS and Taft EW ground motions. Therefore, the seismic performance of the strengthened building could be similar to those of Japanese high-rise buildings.

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INTRODUCTION

A strong earthquake of Magnitude 7.4 occurred in western Turkey, on August 17, 1999. The earthquake produced the collapse of many reinforced concrete buildings. Following the earthquake, seismic evaluation and retrofits of existing buildings have been promoted in the Republic of Turkey. This paper describes seismic capacities of ten existing reinforced concrete buildings in Istanbul. Six of the buildings have been strengthened. In the seismic evaluation of both the original and the strengthened buildings, the Japanese Seismic Capacity Evaluation Standards, JBDPA[1], were applied. A pushover analysis and dynamic response analysis were also carried out for one typical Turkish current building to verify the results of the seismic evaluation.

OUTLINE OF THE BUILDINGS

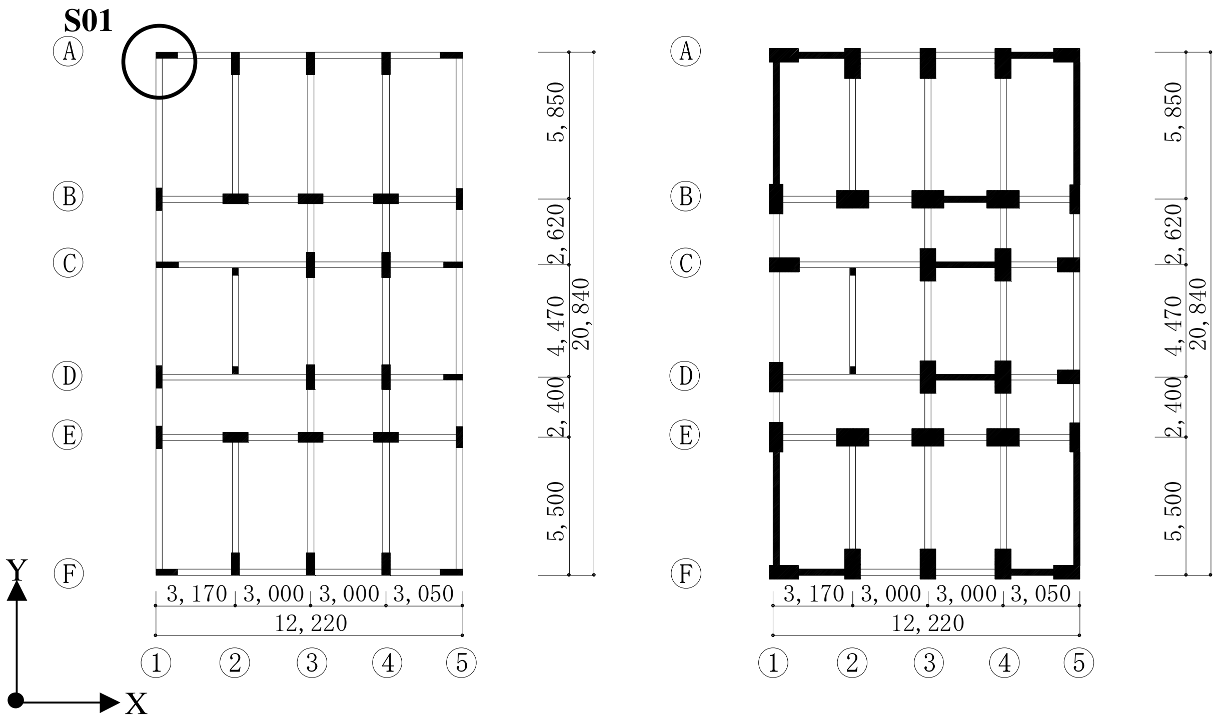
Outline of the buildings are shown in **Table 1**. Since the original of No.2 building had typical characteristics of current buildings in Turkey, it was chosen for detailed analyses.
The characteristics of the No.2 building including the strengthening method are as follows; 1) The floor plan and the elevation of the No.2 building are shown in **Figure 1** and **Figure 2**. 2) The column cross sections were flat and no shear wall was provided. 3) The building was strengthened by jacketing of columns and adding new reinforced concrete walls. 4) The new reinforced concrete walls added at X3-4 of YC, YD-frame and the corner of the floor were placed on each floor of the building and those added at X3-4 of YB-frame were placed on the 1st to 4th floor. 5) The columns placed at X1 and X5 of YB, YC frame, and those at YA and YE of X3 frame were jacketed on the 1st and 2nd floor. The columns placed at X2, X3 and X4 of YB and YE frame were jacketed on the 1st to 4th floor. 6) The other columns are jacketed on each floor.
An example of jacketing of a column is shown in **Figure 3**. The slash marked area shows the original column. 10-ϕ16 as main reinforcement steels and ϕ8@250 as hoop reinforcement steels are arranged in the original column. 19-ϕ20 as main reinforcement steels and ϕ10@100 as hoop reinforcement steels are arranged in the jacketing of the column.

Table 1 Outline of each building

Building No	Number of Stories			Af		w		W	
	Basement	Ground Story	Penthouse			Original (assumed)	Strengthened	Original	Strengthened
No.1	1	3		each F.	392.6	0.90	0.98	353	385
No.2		**7**		**each F.**	**254.7**	**0.90**	**1.44**	**229**	**367**
No.3		7		each F.	781.7	0.90	1.12	704	876
No.4		7		each F.	243.4	0.90	1.42	219	346
No.5		3		1F	649.5	0.90		585	
				2F	375.2			338	
				3F	120.7			109	
No.6		2		each F.	326.0	0.90		293	
No.7	1	5		each F.	205.3	0.90	1.05	185	216
No.8		7		each F.	355.7	0.90	1.37	320	487
No.9	1	12		each F.	328.4	0.90		296	
No.10	2	5	1	1F	353.4	0.90		318	
				2-5F	432.0			389	

Af: Floor area (m^2), w: Unit weight per one square meter ($tonf/m^2$), W: weight per story (tonf)

In the evaluation of the strengthened buildings, original steel bars are neglected. The compressive strength of concrete of the original and the strengthened buildings are 110kg/cm^2 (10.8N/mm^2) and 250kg/cm^2 (24.5 N/mm^2), respectively. The yield strength of steel bar of the original and the strengthened buildings are 2200kg/cm^2 (215.7 N/mm^2) and 4200kg/cm^2 (411.9 N/mm^2), respectively.



< the original building > < the strengthened building >

Figure 1 The first floor plan of No.2 building

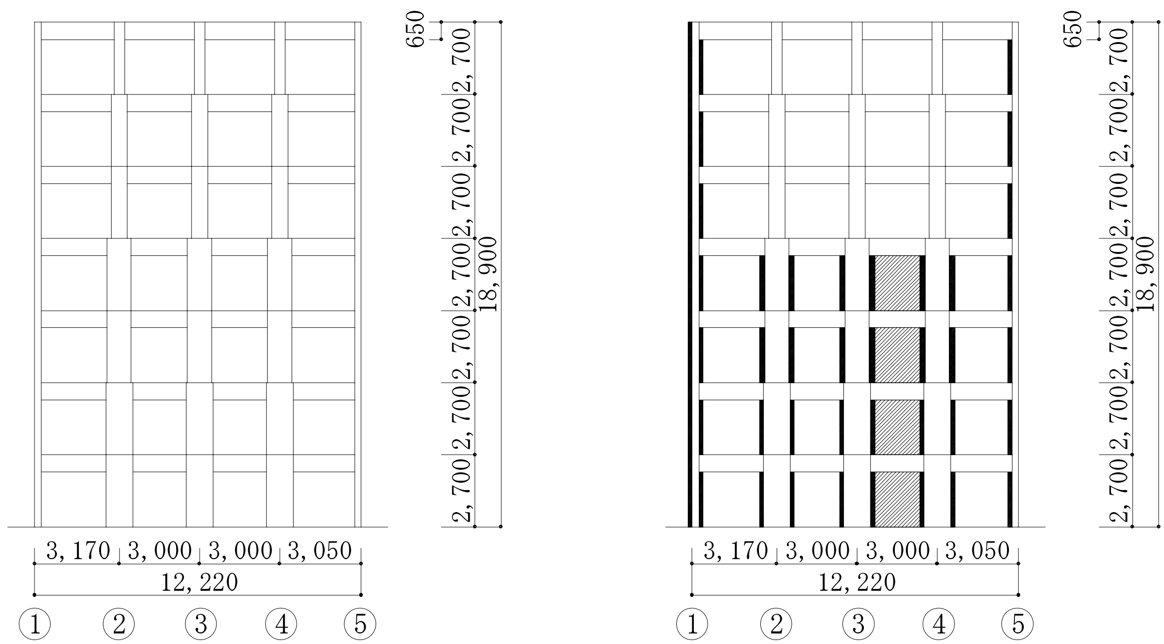


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Figure 2 The elevation of No.2 building (B-Frame)

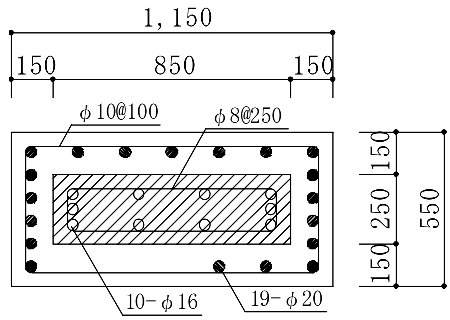


Figure 3 The size and steel bars arrangement of S01 column

METHOD OF SEISMIC PERFORMANCE EVALUATION

The buildings were strengthened to conform to the 1997 Turkish Seismic Design Code. This code is considered to be similar to the Japanese Seismic Design Code in required seismic capacity levels. Therefore, the seismic capacities of the buildings were evaluated by the Japanese Seismic Capacity Evaluation Standard, JBDPA [1].

The Standard consists of three different levels of procedures; first, second and third level procedures. The first level procedure is the simplest and most conservative. Only the strength of concrete and the sectional areas of columns and walls are considered to estimate the seismic capacity, and the ductility is neglected. In the second and third level procedures, ultimate lateral load carrying capacity of vertical members or frames are evaluated using material and sectional properties together with reinforcing details based on the field inspections and structural drawings. The second level procedure evaluates the building seismic capacity based upon the strength and ductility of columns and walls assuming a strong beam concept. The third level procedure considers the strength of beams in addition to the strength of columns and walls to evaluate the seismic capacities of buildings, which are expected to be beam failure types.

According to the Standard, the seismic performance index of a building is expressed by the *Is*-index, as shown in Eq. (1)

$$Is = E_0 \times S_D \times T \qquad \text{Eq. (1)}$$

E_0 is a basic structural index calculated from the product of strength index (C), ductility index (F), and story index (ϕ), i.e., $E_0 = \phi \times C \times F$.

S_D-index and T-index are reduction factors accounting for the disadvantages in the seismic performance of structures. S_D-index is a modifying index for unbalanced distribution of stiffness both in the horizontal plane and along the height of the structure, resulting from irregularity and complexity of structural configuration. T-index is for the deterioration of strength and ductility due to age after construction, fire and/or uneven settlement of the foundation.

The criterion of the seismic performance of a building is expressed by the Is_0-index, as shown in Eq. (2)

$$Is_0 = E_S \times Z \times G \times U \qquad \text{Eq. (2)}$$

E_S is a basic structural judgment index, which is modified by reduction factors due to seismic zonings, grounds condition and building uses; Z-index, G-index and U-index.

In the Standard, the value of E_S is provided as the demand criteria of the each level procedure. The recommended level of Is_0 is 0.8 in the first screening method and 0.6 in the second and third screening method, when all reduction factors are assumed to be unity. This criterion is based on the study of the past earthquake damaged buildings. Past experiences of 1968 Tokachi-oki, 1978 Miyagi-ken-oki and other earthquakes reported that buildings with Is_2 indices higher than 0.6 could escape from moderate or severe damage. Even in the case of the 1995 Hyogo-ken Nambu Earthquake, most buildings with *Is* indices higher than 0.6 could escape from severe damage, Okada [3]. The required seismic performance indices higher than Is_0 can be considered to correspond with the level of seismic capacity required in the Japanese Seismic Design Code.

Additionally, the standard recommends that the strength index $C_T \times S_D$ should be higher than 0.3.

The comparison between *Is*-index and Is_0-index and the value of strength index $C_T \times S_D$ evaluate whether the building has the seismic capacity to prevent major structural damage or collapse. A building is evaluated vulnerable in the case the value of *Is*-index is less than one of Is_0-index. A ductile building is evaluated vulnerable in the case the value of $C_T \times S_D$ index is less than 0.3 even though *Is*-index is relatively high.

APPLICATION OF SECOND LEVEL PROCEDURE

To evaluate the seismic performance of the buildings, the following assumptions are adopted;

1) The criteria of the seismic performance of buildings are 0.6 as the value of Is_0-index and 0.3 as the value of $C_T \times S_D$ index.
2) The steel bars are embedded in the jacketing columns, and the connection of column and beam is not damaged.

Table 2 shows the evaluation results of both the original and the strengthened buildings. The values of Is_2 and $C_T \times S_D$ are the values of the first story of the buildings, Tago [4].

Is_2 values of all but one of the original buildings were less than 0.6, and $C_T \times S_D$ values of the No.5 and the No.6 were less than 0.3. Therefore, all but one of the original buildings were judged vulnerable. Six of the buildings have been strengthened to conform to the 1997 Turkish Seismic Design Code. The Is_2 values and $C_T \times S_D$ of those buildings were calculated also for strengthened buildings. The values of Is_2 and $C_T \times S_D$ were higher than the criteria. Therefore, all of the strengthened buildings were evaluated as safe.

The strength indices (*C*) and ductility indices (*F*) in X and Y directions of original buildings are shown in **Figure 4**. The ductility performances of the No.1, 2, 3, 4, and 8 building were low, and the strength performances of the No.5 and No.6 building were remarkably less than that required.

The strength indices (*C*) and ductility indices (*F*) in X and Y directions of strengthened buildings are shown in **Figure 5**. The points of the values of both the original (◯) and the strengthened (●) buildings were plotted in the figure. Since shear walls were provided and columns were jacketed, the strength performance of the buildings was improved.

Table 2   The second seismic performance indices

Building No.	Direction	Original			Strengthened		
		Is	$C_T \times S_D$	Judgment	*Is*	$C_T \times S_D$	Judgment
1	X	0.34	0.34	×	1.08	1.08	O
	Y	0.51	0.40	×	1.20	1.20	O
2	X	0.40	0.40	×	0.91 [0.61]	0.91 [0.76]	O
	Y	0.44	0.44	×	1.11 [0.76]	1.11 [0.95]	O
3	X	0.37	0.37	×	0.78 [0.44]	0.31 [0.55]	O
	Y	0.38	0.38	×	1.17	1.17	O
4	X	0.48	0.48	×	0.95	0.95	O
	Y	0.51	0.51	×	0.93 [0.60]	0.93 [0.75]	O
5	X	0.39	0.13	×			
	Y	0.22	0.07	×			
6	X	0.41	0.14	×			
	Y	0.38	0.13	×			
7	X	0.67	0.31	O	1.04	0.42	O
	Y	1.08 [0.7]	1.08 [0.87]	O	0.90 [0.57]	0.90 [0.71]	O
8	X	0.34 [0.25]	0.34 [0.31]	×	0.74 [0.46]	0.74 [0.58]	O
	Y	0.46	0.26	×	0.70 [0.44]	0.70 [0.54]	O
9	X	0.17 [0.23]	0.17 [0.28]	×			
	Y	0.13 [0.24]	0.07 [0.29]	×			
10	X	0.61	0.61	O			
	Y	0.48	0.48	×			

[   ] : In case of the structure with Extremely Brittle Column

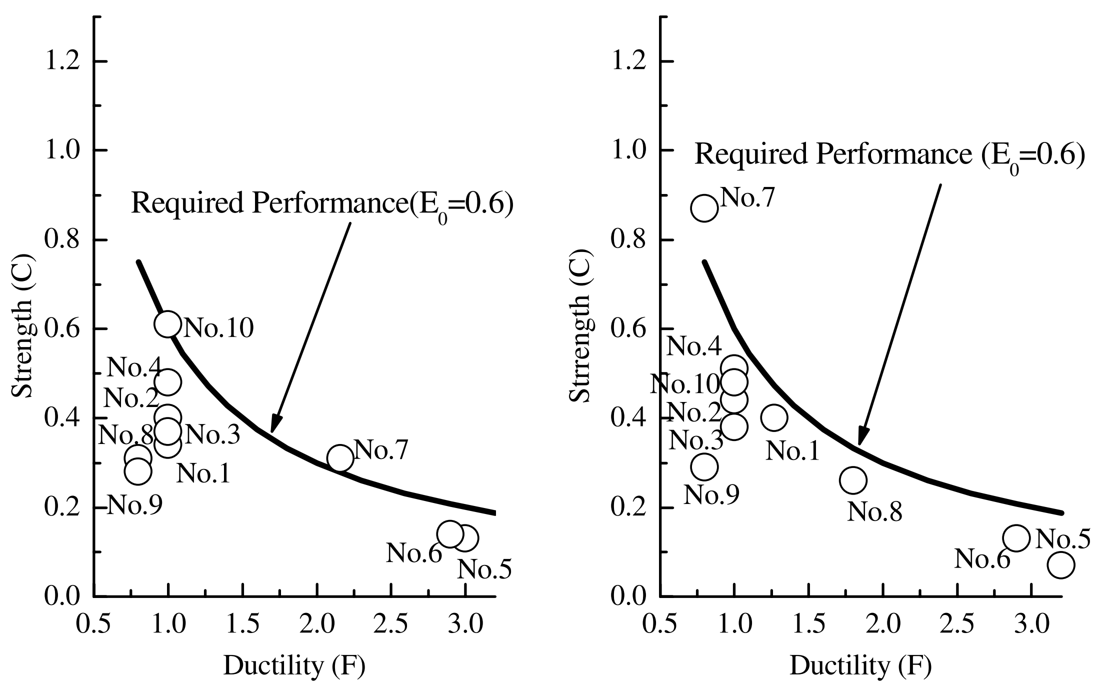


Figure 4   The seismic performance in strength (C) and ductility (F) of the original buildings

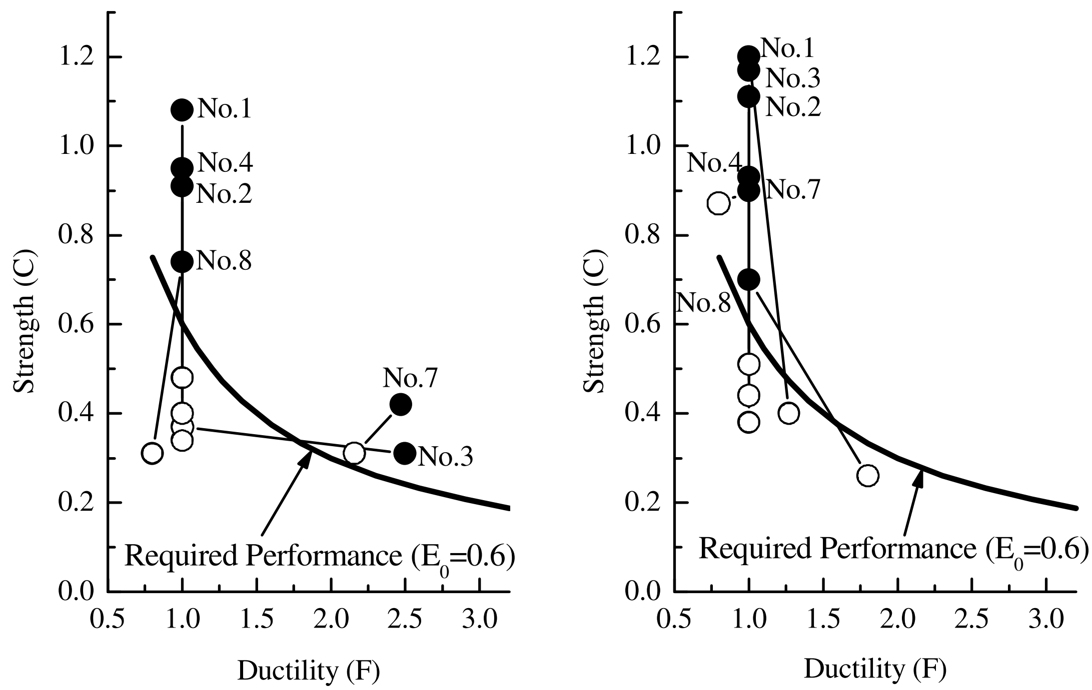


Figure 5 The seismic performance in strength (C) and ductility (F) of the strengthened buildings

PUSHOVER ANALYSIS AND RESPONSE ANALYSIS

The pushover analysis and dynamic analysis were performed in order to assess the more realistic response and to identify the possible failure mechanisms. The frame models of the strengthened No.2 building, which has typical characteristics of the ten buildings evaluated, were analyzed.

Modeling of the structure in response analysis

To analyze the frame models, the assumptions are as follows:

1) The three-dimensional structure was substituted for the two-dimensional model, which was composed of each frame connected in parallel based on the assumption of a rigid floor.
2) The columns, beams and walls were substituted for the linear elements with the rigid zones, and the behavior of the members was analyzed by elasto-plastic rotational springs at both ends of those elements and a shear spring at the center of them in consideration of the deformation by moment and shear force.
3) The elasto-plastic rotational springs have hysteresis characteristics of linear reducing-stiffness type and the shear springs have that of origin oriented type.
4) The model has proportional damping to the instantaneous stiffness, and the damping ratio of the 1st mode is 5%.

Pushover analysis

In pushover analysis, the approximate manner of the structure behavior was estimated by nonlinear static analysis under monotonously increasing lateral loading.

The relationship between story drift and story shear at each story is shown in **Figure 6**, **7**. The relationship shows similar behavior in both X and Y direction.

The formulation of yield hinges in structural elements when maximum story drift angle is equal to 1/100 is traced in **Figure 8**, **9**. Those yield hinges were formulated at most beams and unstrengthened columns in both directions. The failure mechanism of the building proved to be a beam failure type.

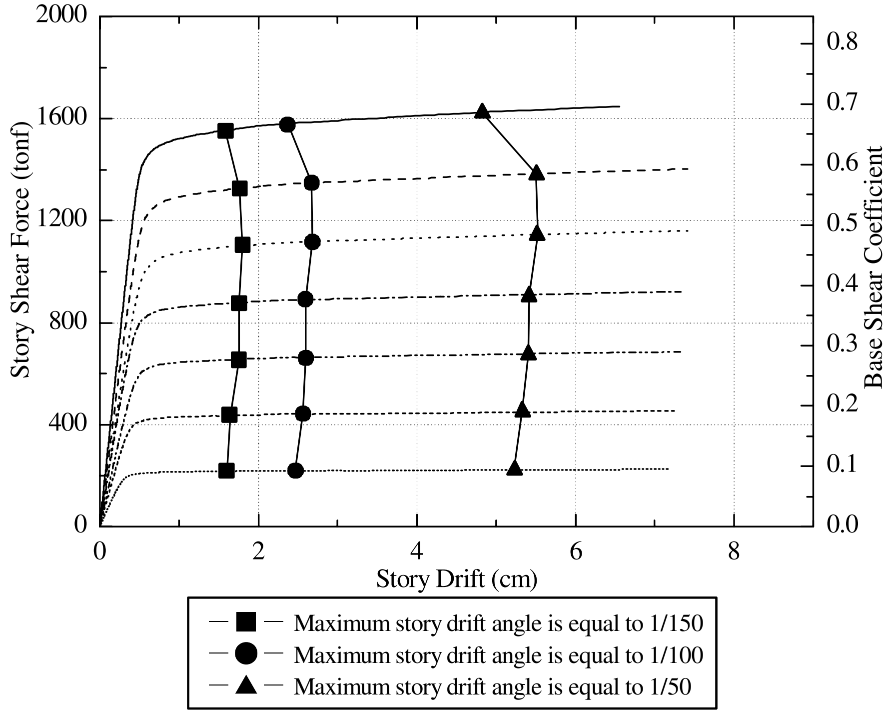


Figure 6 The relationship between story drift and story shear force (X direction)

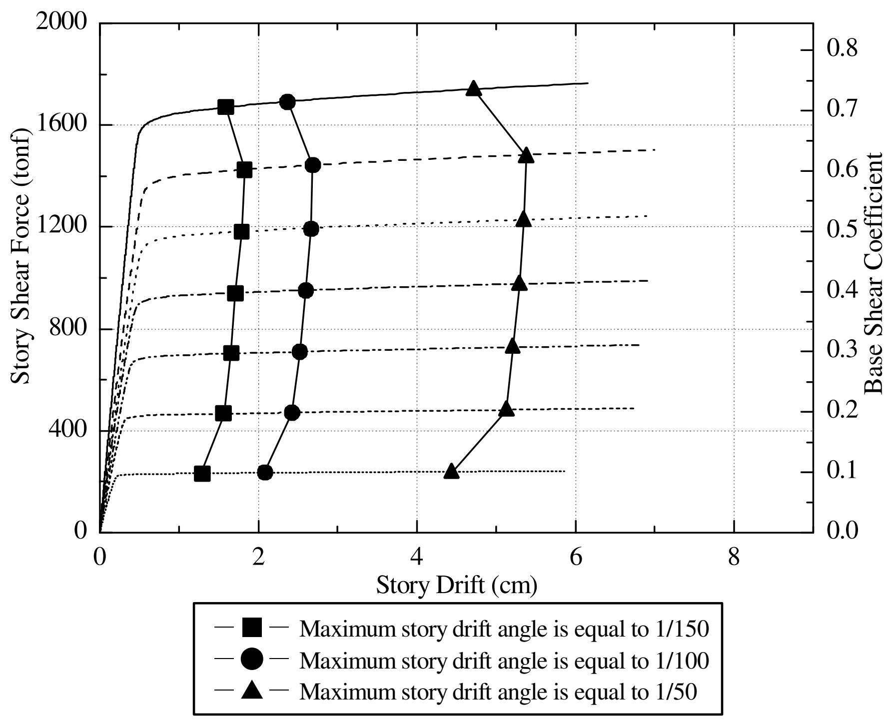


Figure 7 The relationship between story drift and story shear force (Y direction)

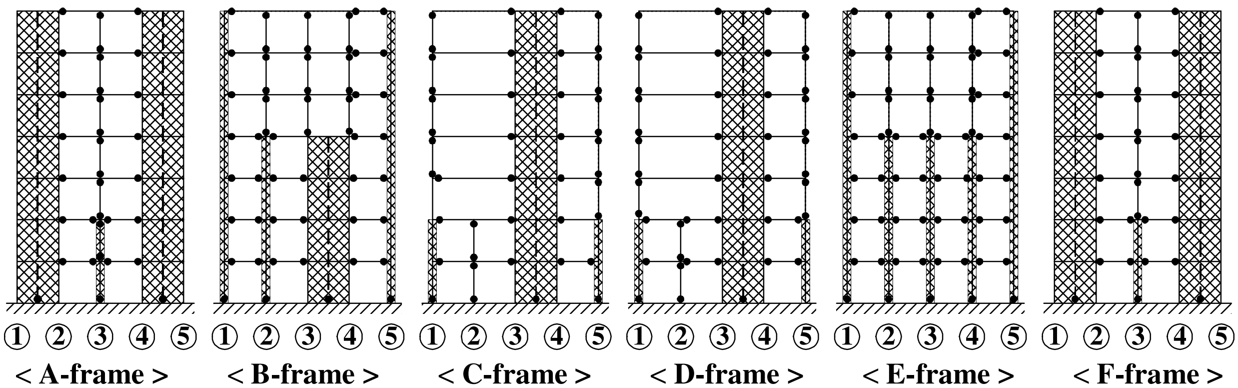


Figure 8 The occurrence of yield hinges (X direction)

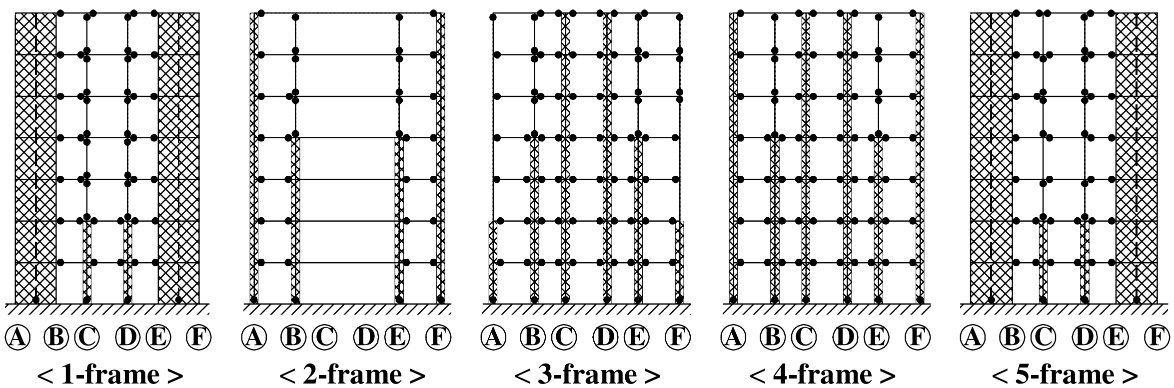


Figure 9 The occurrence of yield hinges (Y direction)

Dynamic analysis

The dynamic analysis was used for predicting the realistic response of the building subjected to strong ground motions. The analysis was performed using two strong motion records, El Centro NS and Taft EW, respectively recorded during Imperial Valley earthquake at El Centro (1940, M=7.1) and the California earthquake at Kern county (1952, M=7.8), scaling their maximum velocity to 50cm/sec, as usually used for the design of high-rise buildings in Japan. The earthquake response spectrums and 1st predominant periods of the building in both directions are shown in **Figure10**.

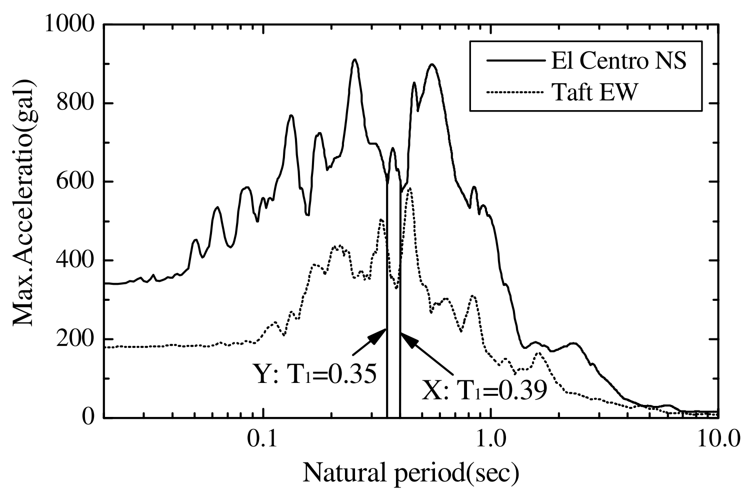
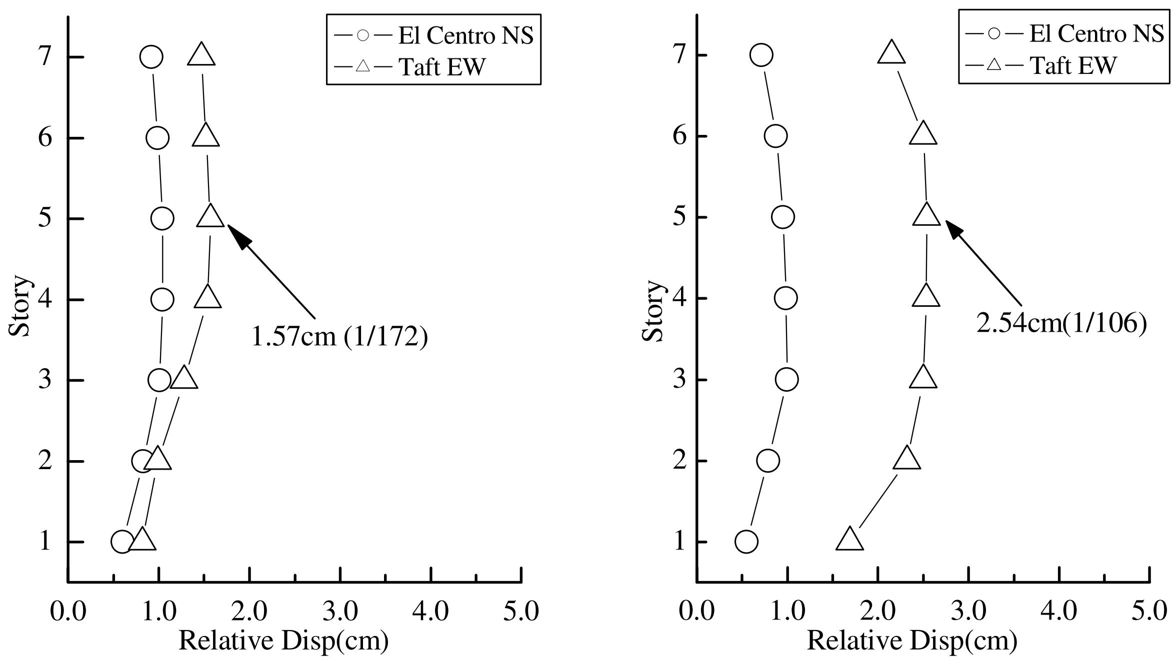


Figure10 The earthquake response spectrums

The yield hinges mechanism obtained by the response analysis was similar to the pushover analysis on the whole. The number of formulated hinges in X direction was around 70% of those in pushover analysis. In Y direction, the yield hinges were formulated at a part of the added walls. The yield hinge formulation may be attributed to the stress concentration in a few walls.
The maximum relative story displacement at each story in X direction and Y direction is shown in **Figure 11**.
The result showed that the maximum values of the response were 1.57cm in X direction and 2.54cm in Y direction and the story drift angles were respectively 1/172 and 1/106.
Since the stiffness at each story was improved, the story drift was controlled due to multi-story shear walls being added in the building, and the maximum story drift angles were less than 1/100 which is usually required for high-rise buildings in Japan. Therefore, the results showed the effect of the retrofit.



< X direction > **< Y direction >**

Figure 11 The relative story displacement

CONCLUSIONS

The major findings in this study are summarized as follows:
1) All but one of the original buildings were evaluated vulnerable by the Japanese seismic safety criteria.
2) Seismic capacities of six strengthened buildings have been improved much and evaluated as safe.
3) A pushover analysis and a dynamic response analysis of a typical Turkish building showed similar seismic performance and failure mode to those obtained by the seismic evaluation.
4) The failure mode of the strengthened typical building was beam failure type in pushover analysis.
5) The maximum story drift angles by the dynamic response analysis were less than 1/100 to scaled El Centro NS and Taft EW ground motions. Therefore, the seismic performance of the strengthened building could be similar to those of Japanese high-rise buildings.

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