INVESTIGATION OF INFILL WALL EFFECT FOR THE SEISMIC PERFORMANCE OF A MODERATELY DAMAGED RC BUILDING

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ABSTRACT :

In order to investigate the infill wall effect in reducing the earthquake damage, a four-story RC building in Yalova province in north-western part of Turkey, which has experienced moderate damage during the August 17, 1999 Kocaeli Earthquake and strengthened afterwards, a set of non-linear dynamic analysis is carried out in this research. Structural performances for both before and after strengthening states are evaluated, also taking into account the existence of infill walls for the as-built and strengthened states of the building. Pushover analyses are carried out to establish the capacity and demand curves for the structures, selecting the target displacements considering the Turkish Earthquake Resistant Design Code-2007. Utilizing DRAIN-2DX computer program, dynamic analyses are performed using an earthquake ensemble of 7 recorded strong ground motions as well as a simulated spectrum compatible accelerogram. It is assumed that, bilinear hysteretic model represents the nonlinear force-displacement relationship of structural members of frames, while all infill walls are modeled as compression struts. Envelopes for generalized and relative displacements and time variations of overturning moments and roof displacements with respect to base-shear are determined and these demands are compared for the two states of the structure. It is clearly exhibited that inclusion of infill walls led more realistic results, not only for estimation of occurrence of plastic hinges but also consistency of analysis and site inspection of damaged structure.

KEYWORDS: Reinforced-concrete structures, Earthquake damages, Nonlinear dynamic analysis, Performance evaluation, Infill wall effect, Kocaeli earthquake

1. INTRODUCTION

Seismic safety of structures is commonly realized during the design stage by means of reducing the risk of casualties and assets considering the unpredictability of the earthquakes. Other than the safety regulations in most of the earthquake resistant design codes, economical issues are also taken into account by introducing a life-safety performance level for the structures to be designed. Hence, structural behavior is mostly inconsiderable when the intensity of the design earthquake is exceeded.

Within the concept of performance-based design, structural performance of a system subjected to seismic loads, is determined by the evaluation of expected damage levels. Various design codes and standards indicate the determination of the performance and capacity of the structures by using nonlinear static analysis procedures based on establishing the pushover curves as defined in ATC-40 (1996), FEMA356 (2000), etc. Similarly, this concept is also considered in the latest version of the Turkish Earthquake Resistant Design Code (TERDC, 2007), by introducing various damage and performance levels for structures subjected to earthquakes having different probability of exceedence. In these or similar procedures, earthquake forces, which will be monotonically increased after each static analysis step, are defined to be distributed height-wise along the structure. Nonlinear static analysis is agreed to be satisfying when a target displacement is encountered and commonly, this target displacement is determined from the analysis of an equivalent SDOF system although the pushover analyses procedures might differ from each other.

The main objective of this study is to evaluate the performance of a four-story reinforced-concrete (RC) building structure, considering two cases; as-built and strengthened states by performing a set of non-linear dynamic analyses. Existence of infill walls for each state of the structure is also investigated as another parameter affecting the structural behavior. Consistency of the latest TERDC recommendations with the nonlinear analyses results will be illustrated.

2. STRUCTURAL PROPERTIES OF THE MODERATELY DAMAGED RC BUILDING

A four-story RC frame building, which has suffered moderate damage and strengthened afterwards by adding high-wise shear-walls and jacketing of all columns at the first story level, is investigated considering each state of the structure. Story heights are 3.0m and similar for the entire building. The original structural material is determined as C14 (f_{ck}=14 N/mm^2) concrete and S220 (f_{yk}=220 N/mm^2) type of structural steel and the materials for jacketing and new shear-walls are C30 concrete and S420 steel. Beams have the dimensions of 20/60 cm/cm while columns in the first two stories vary from 25cm×50cm and 25cm×60cm. In the upper two stories, dimensions of the entire columns decrease to 25cm×40cm. Characteristic properties of the structural elements are tabulated in Hasgür and Taskin (2008) and the layout plans for before and after strengthening cases are shown in Figure 1. It has to be emphasized that only the analysis results that are performed on relatively weak *y-y* axis will be exhibited herein the paper.

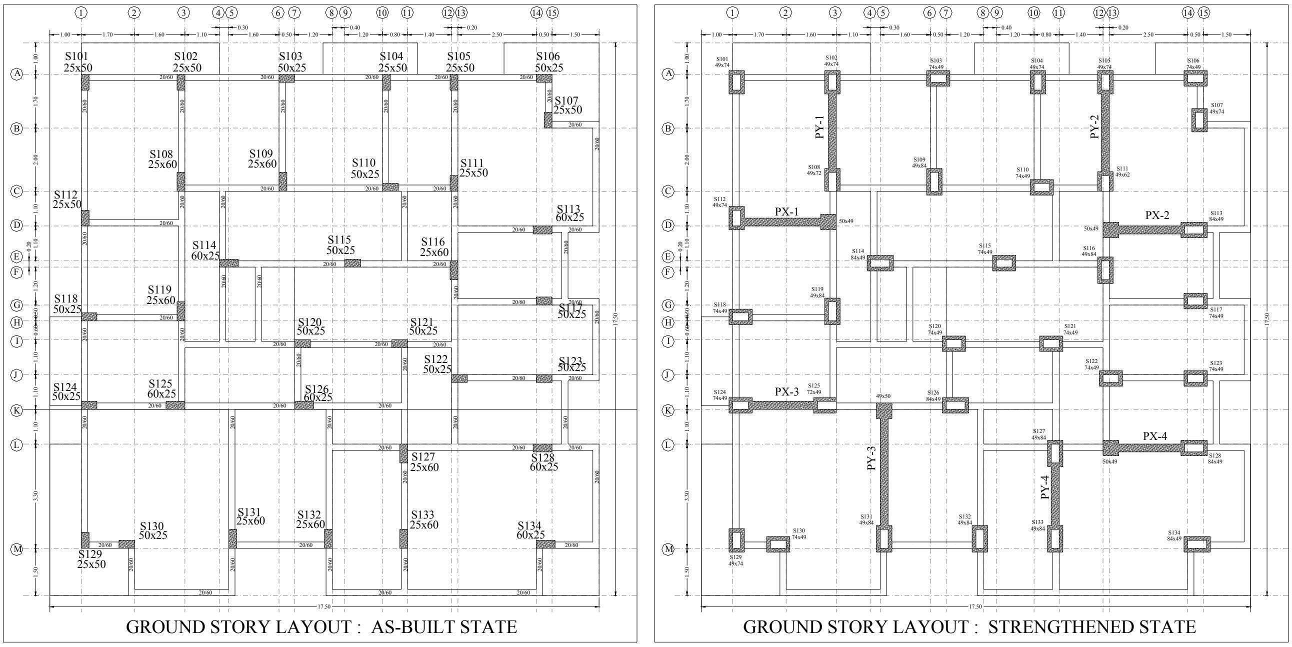

Figure 1 Ground story plans for the: as-built (left); and strengthened (right) states

3. CHARACTERISTICS OF STRONG MOTION DATA

The four-story RC building in Yalova province will be investigated by means of the demands concerned with maximum story drifts, base shear and overturning moment capacities, ductility demand and the locations of plastic hinges. Depending on the lack of strong motion records within the region, a series of earthquake time histories that might represent the characteristics of either the earthquake event or the site itself are considered. Herein the study, Sakarya-EW, Yarimca-EW and -NS, Düzce-EW and -NS strong motions records of August 17, 1999 Kocaeli Earthquake and Bolu-EW, Düzce-EW strong motions recorded during the November 12, 1999 Düzce Earthquake are selected to establish an ensemble of ground motions, (*official web-site of ERC; Ministry of Publicworks; Gen.Dir.of Disaster Affairs: www.deprem.gov.tr*). Considering the length of each acceleration-time history, 'strong motion' portion of longer records are exposed and used; consequently, the amount of the output became manageable. In order to demonstrate that such a shortening procedure does not cause any significant change on the characteristics of the record, engineering intensities are calculated and compared for the original and cropped motions. The below Table 3.1 tabulates the characteristics of the earthquake ensemble.

Table 3.1 Engineering Intensities of the Strong Motion Records

	Ground Motion Records		Engineering Intensities						
			a_{max} (mG)	a_{eff} (mG)	t_{eff} (sec)	$SI_{0.2}$ (cm)	I_{RS}	I_{RMS}	I_{EAP}
Kocaeli EQ	Sakarya EW	Clipped	407	195	14.6	80.8	343.5	6.9	84.7
		Unclipped	407	195	44.0	80.8	352.3	0.9	256.7
	Yarimca EW	Clipped	322	110	32.0	83.9	315.8	7.9	100.7
		Unclipped	322	110	33.2	95.0	321.0	2.4	92.3
	Yarimca NS	Clipped	230	125	31.1	92.2	319.9	8.0	101.3
		Unclipped	230	125	31.9	107.7	323.7	2.4	88.7
	Düzce EW	N/A	374	218	12.1	138.7	296.4	10.9	45.6
	Düzce NS	N/A	315	175	11.9	97.5	266.3	9.8	51.2
Düzce EQ	Bolu EW	Clipped	806	433	9.0	155.3	396.4	11.3	59.9
		Unclipped	806	433	9.0	155.3	397.2	7.1	60.5
	Düzce EW	N/A	514	240	10.9	157.8	436.5	16.9	39.8

a_{max} in Table 2.1 represents the peak acceleration recorded during the earthquake. When engineering intensities are evaluated, it should be emphasized that the ensemble consists of destructive motions with mostly long effective durations. As an alternative to recorded motions, a simulated strong motion has been generated by means of iteratively used Fourier transform pairs. Considering a *Z*2 class local soil, the design spectrum values with 5% damping are obtained by iteration in the frequency domain until a convergence of ±1 cm/s^2 acceleration spectrum value is encountered. To figure out the spectral characteristics and site conditions, absolute acceleration response spectra of the strong motion ensemble and the simulated ground motion are computed and exhibited in Figure 2. Spectral acceleration axis is normalized with respect to *g*. It is observed that Düzce and Yarimca sites exhibits rather soft soil conditions when compared to Bolu region. Same figure shows a very successful coverage of the design spectrum when compared to the normalized acceleration spectrum of the simulated motion.

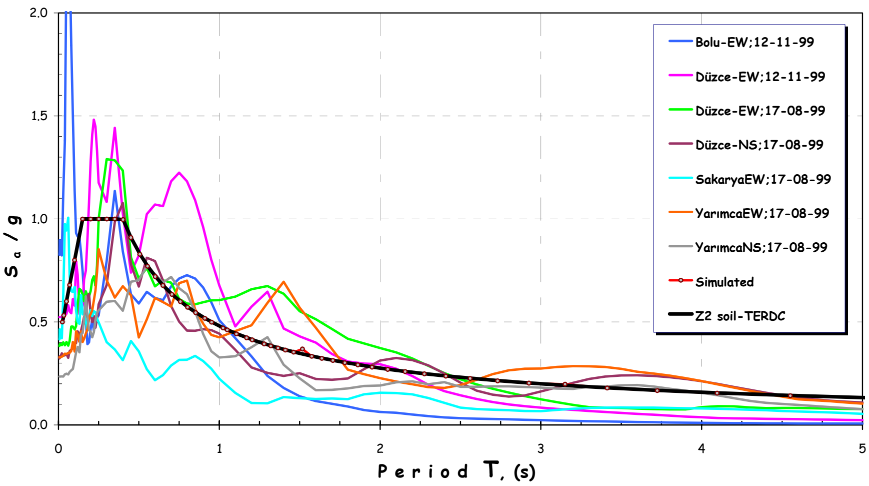

Figure 2 Comparison of the elastic response spectra of the strong motions and design spectrum

4. NON-LINEAR DYNAMIC AND STATIC ANALYSES

The structural system is modeled as planar frames, which consists of non-linear elements connected at nodes of beams and columns. It is assumed that, bilinear hysteretic model represents the nonlinear force-displacement relationship of structural members, which are subjected to dynamic loading; therefore, assigned according to this assumption as the input of DRAIN-2DX, (1993). Theoretically in this behavior, the first linear part of force-deformation relationship represents cracked-section behavior and then reaches to the yielding point. After yielding occurs, the following linear branch takes into account the strain-hardening effect. Unloading stiffness is the same as the first branch stiffness.

For illustrating the infill wall contribution in the structural response, infill walls are later introduced in the structural system as a second solution case. Locations of the infill walls are captured from the architectural drawings and the structural model is formed as in Al-Chaar and Lamb (2002). According to the method, hollow brick infill walls are modeled considering two diagonal compression struts having width a, depth t, diagonal length D, wall height h, story height H and diagonal slope angle θ as in Eqn. 4.1:

$$a = 0.175D\left(H\,\frac{E_w t \sin 2\theta^{11/4}}{4E_c I_c h} \right)^{-0.4} \tag{4.1}$$

Here E_w and E_c represent the modulus of elasticity for infill walls and concrete, respectively. I_c is the moment of inertia for the neighboring column. Following Figure 3 shows the hysteretic relations for structural and non-structural elements.

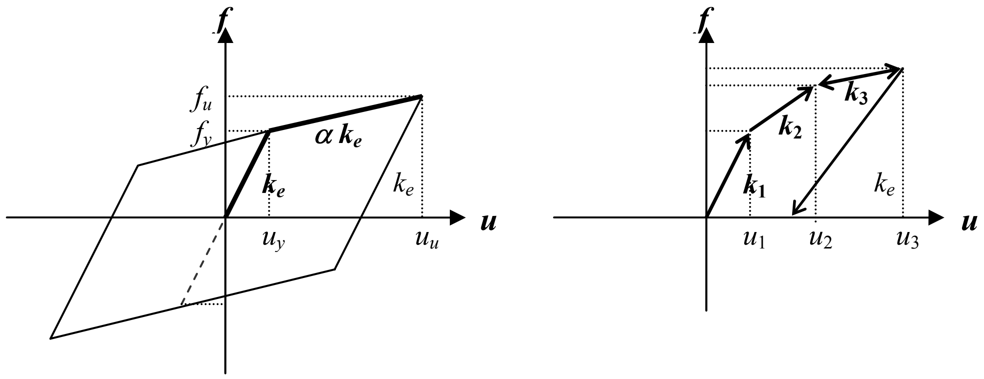

Figure 3 Hysteretic models for: beams, columns and shear walls (left); infill walls (right)

4.1. NL Dynamic Analysis without Infill Walls

Considering a constant damping ratio of ξ=5% and a post-yield stiffness ratio of α=3%, non-linear dynamic analyses are carried out for Δt=0.002 s subjected to the earthquake ensemble above. Calculations are performed for as-built (BS) and then strengthened (AS) states of the building; for each, existence of infill walls are ignored (w/o walls) or considered (w/ walls) during the modeling making a total solution of four sets.

According to the analysis results, natural vibration periods are calculated to be T_1=0.624s, T_2=0.226s, T_3=0.144s and T_4=0.099s for (BS) and T_1=0.260s, T_2=0.062s, T_3=0.026s and T_4=0.016s for (AS), respectively. Envelopes for generalized displacements and story drifts are exhibited in Figure 4, (Yilmaz, 2006).

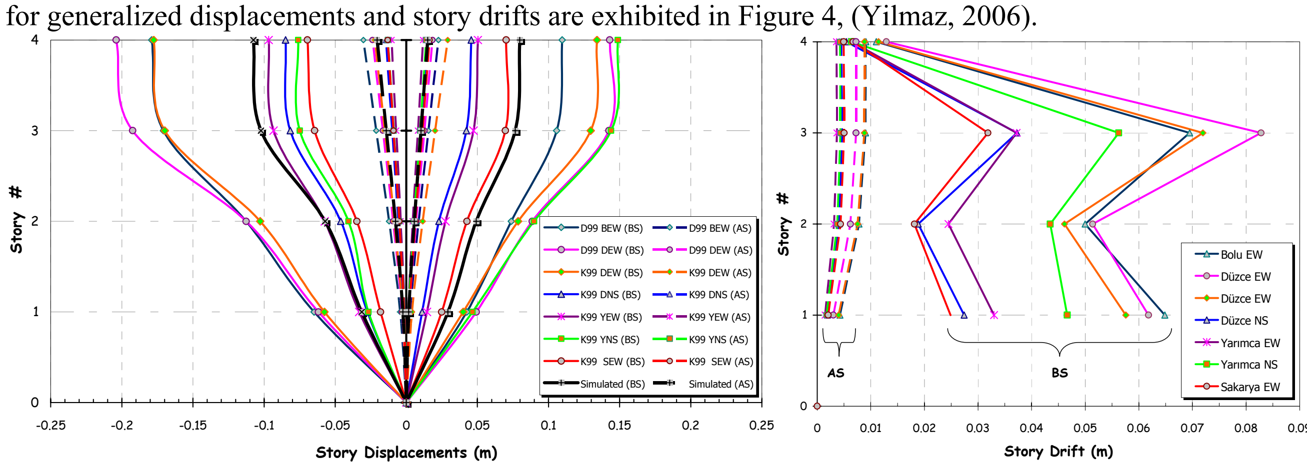

Figure 4 Story displacements and drifts BS and AS states for (w/o walls).

In the generalized displacement envelopes, it is observed that the lateral displacement amount is decreased six times when compared with the (AS) state. Relative displacements on the first and second floor levels are more than the upper two stories for the (BS) state, depending on the degradation in the stiffnesses of structural members. Hence, displacement in the third story is increased compared to the second story, however, decreased in the roof because of the lower values of shear forces. After introducing shear walls into the system, generalized displacements also decreased in lower floors, as expected. The maximum values of top story displacements, $U_{y\text{-}y}$, base shear forces, V_b, and overturning moments, M_0, are listed in Table 4.1 for the two states of the structure. Mean values of these demands for the earthquake ensemble are estimated as 0.137m, 2074kN and 17063kNm for the (BS) state and 0.020m, 6850kN and 57102kNm for the (AS) state, respectively.

Table 4.1 Mean Structural Demands for (BS) and (AS) States of the Building

		Kocaeli EQ					Düzce EQ		Simulated EQ
		Sakarya EW	Yarimca EW	Yarimca NS	Düzce EW	Düzce NS	Bolu EW	Düzce EW	
$U_{y\text{-}y}$ (m)	BS	0.070	0.096	0.149	0.178	0.085	0.179	0.204	0.107
	AS	0.016	0.012	0.014	0.029	0.015	0.030	0.024	0.021
V_b (kN)	BS	1734	1850	1936	2250	1769	2482	2497	2099
	AS	6344	4375	4945	8557	5435	9768	8525	8459
M_o (kNm)	BS	14488	14257	15474	18828	14702	21472	20223	17135
	AS	49439	39146	43448	72204	44248	78410	72817	72451

The following Figure 5 illustrates a comparison of the mean structural demands and the capacity of the system calculated considering the TERDC-2007 limits.

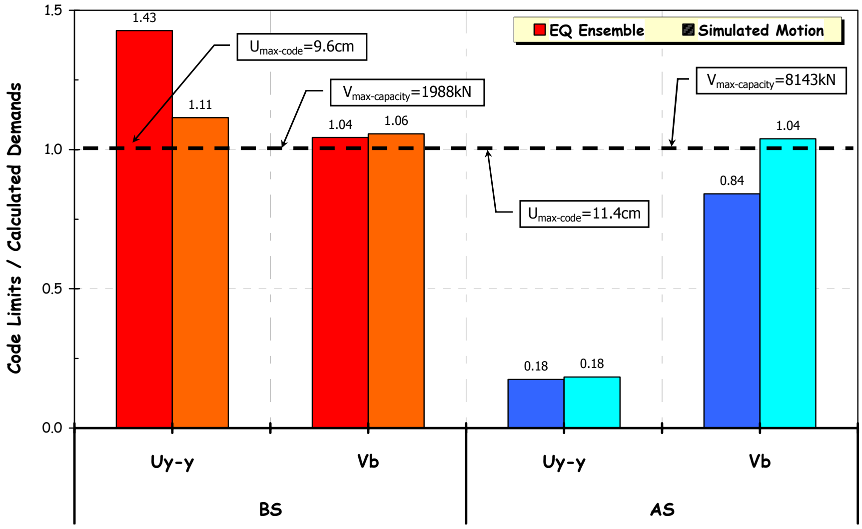

Figure 5 Comparison of the structural capacity and code limits *vs.* the mean demands.

4.2. NL Dynamic Analysis with Infill Walls

When the existence of the infill walls are taken into account, natural vibration periods decreased as expected, and are calculated to be T_1=0.469s, T_2=0.167s, T_3=0.106s and T_4=0.082s before strengthening and T_1=0.244s, T_2=0.060s, T_3=0.026s and T_4=0.016s for the strengthened case, respectively, (Taskin, 2007).

Nonlinear dynamic analyses are performed for the infilled structure under the effect of the same strong motion ensemble. As seen from Figure 6, which shows the V_b base shear demand variation for the as-built structure with (w/) and without (w/o) infill walls calculated for the simulated motion, the peak mean base shear demand is calculated to be 2099.1 kN, while it increases to 2172.1 kN when the infill walls are considered. In the same figure, the top-story $U_{y\text{-}y}$ and base shear V_b relations are obtained for the (BS) and (AS) states of the structure are also shown. It is observed that inclusion of infill walls significantly increases the amount of inelastic displacements as well as the excursions at the beginning of the cyclic loading. Table 4.2 summarizes the structural responses and the demands for (BS) and (AS) states when infill walls are considered.

Occurrence and locations of plastic hinges in the structure are also investigated. When infill walls are ignored, all of the structural members in the first two stories of each frame exceeds yield levels, therefore experiences plastic hinges before strengthening. Averagely, third and fourth story beams seem to remain elastic while plastic hinges occur in columns. This has a strong consistency with the weak column-strong beam design applied to the structural system. With the participation of shear walls after strengthening, 29% of the entire columns in the ground story yield in both ends. This percentage decreases to 20% in the second story, but tends to increase in the third and fourth stories as a result of the increment in the participation of the frame behavior, as well as the decrease in the cross-section areas of columns.

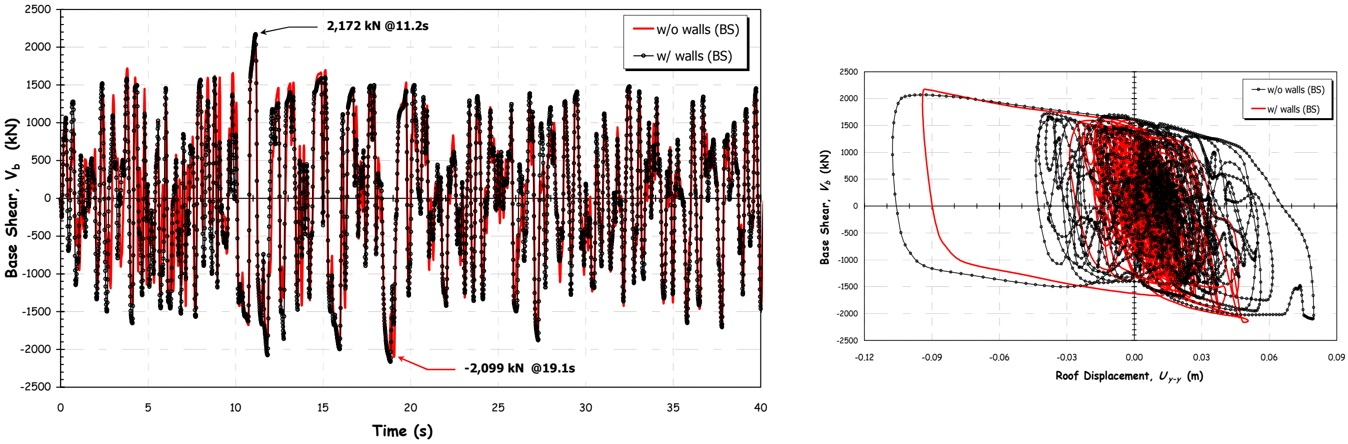

Figure 6 Base shear demand variation and hysteresis curves under the effect of simulated motion

Table 4.2 Comparison of the Mean Demands for the Two States

		Kocaeli EQ					Düzce EQ		Simulated EQ
		Sakarya EW	Yarimca EW	Yarimca NS	Düzce EW	Düzce NS	Bolu EW	Düzce EW	
$U_{y\text{-}y}$ (m)	BS	0.038	0.044	0.068	0.127	0.055	0.157	0.172	0.094
	AS	0.014	0.011	0.012	0.025	0.013	0.026	0.021	0.019
V_b (kN)	BS	1750	1990	2370	2640	2240	2808	2686	2172
	AS	8299	5632	6550	9960	7319	11034	9848	8749
M_o (kNm)	BS	21700	22300	21800	24100	22600	25269	26610	21809
	AS	64007	49236	53633	89992	59254	86345	79397	73402

When infill walls are considered for the as-built structure, it is observed that infill walls in each frame crush first, postponing the occurrence of plastic hinges in the columns. However, depending on the duration of the strong motions, 85% of the entire columns experience plastic hinges, while third and fourth story beams remain elastic. For the strengthened system, 23% and 17% of the entire columns in the ground and second stories yield in both ends, respectively. Similarly there is a tendency to increase in the third and fourth stories, while all beams stay in the elastic range, (Toker, 2006).

4.3. NL Static Analyses for w/o and w/ Infill Walls

Nonlinear static procedures based on pushover analysis are widely accepted and enforced evaluation methods since they practically let engineers to gain insight to nonlinear seismic behavior of structures. Inelastic variation of the base shear with respect to the top-story displacement, in other words the *pushover-curve*, is obtained considering monotonic increments in adaptive load patterns, such as the equivalent seismic loads, first mode shape, *etc*. Then, inelastic demand spectrum for the structure and the capacity spectrum, which is transformed from the pushover curve, are compared and inelastic demands are obtained from intersection of the two curves.

The example building in this paper is also evaluated considering the incremental equivalent seismic load methodology in the TERDC-2007. According to the procedure, capacity curve is established from the pushover curve by transforming the coordinates into modal displacement d_1 and modal acceleration a_1 as:

$$d_1^{(i)} = \frac{U_{yN1}^{(i)}}{\Phi_{yN1}\Gamma_{y1}} \qquad a_1^{(i)} = \frac{V_{y1}^{(i)}}{M_{y1}} \tag{4.2}$$

Here, i is the pushing step; 1 represents the first mode of the structure; y is the direction of loading; N is the symbol of the top-story; Φ_{yN1} is the modal displacement in the top-story; M_{y1} is the effective modal mass and Γ_{y1} denotes the instantaneous participation factor for an earthquake in y direction. Elastic design spectrum having the axes S_{ae1} and S_{de1} is transformed into inelastic demand spectrum as follows:

$$S_{di1} = C_{R1} S_{de1} = C_{R1} \frac{S_{ae1}}{\left(\omega_1^{(1)}\right)^2} \tag{4.3}$$

C_{R1} in Eqn. 4.4 is the spectral displacement ratio and can be calculated depending on the initial vibration period, $T_1^{(1)}=2\pi/\omega_1^{(1)}$. Finally, inelastic displacement demand of the structure is obtained by Eqn. 4.5:

$$U_{yN1}^{(p)} = \Phi_{yN1} \Gamma_{y1} d_1 \tag{4.4}$$

The procedure is applied for the building before and after strengthening states. Furthermore, existence of infill walls is also taken into account as two alternative cases. According to the capacity and demand curves, which are shown in Figure 7, the inelastic displacement demands for the as-built structure are calculated as 0.126 m and 0.079 m when the infill walls are ignored or included, respectively. These values seem to be sufficiently consistent with the nonlinear analysis results above. After strengthening takes place, the building seems to gain the required resisting capacity.

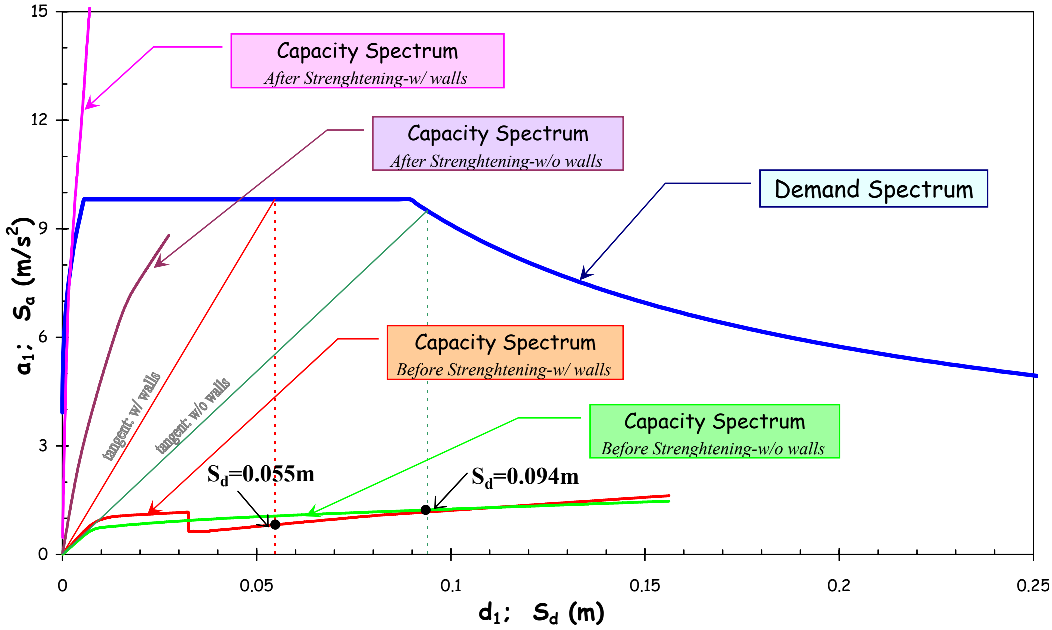

Figure 7 Determination of the inelastic displacement demands

5. CONCLUSIONS

An existing four-story RC frame building moderately damaged during the 1999 Kocaeli earthquake in Yalova province and strengthened afterwards, is investigated in details by performing nonlinear dynamic analysis and applying nonlinear static procedures for each state of the structure. Existence of infill walls is ignored as the first case of evaluation and then included in the later case. A strong motion ensemble is established using 7 earthquake records from August 17, 1999 Kocaeli and November 12, 1999 Düzce earthquakes, exhibiting different characteristics of destructive seismic motions. Besides, a spectrum compatible synthetic strong ground motion is generated and applied to the structure during the non-linear dynamic analysis and successfully compared with the mean structural demands computed subjected to the earthquake ensemble. Assuming a structural damping ratio of 5%, bilinear hysteretic behavior for structural members and an equivalent seismic load distribution during the pushover analysis, this investigation has led to the following results:

- Nonlinear dynamic analysis results indicate mean inelastic top-story displacement demands of 0.137m and 0.094m for the as-built structure, when the infill walls are ignored or included, respectively. These values are calculated as 0.107m and 0.094m when a single simulated ground motion compatible with the design spectrum is applied to the building.
- When the nonlinear static analysis is carried out for the same state of the structure, these demands are computed to be 0.126m and 0.079m, respectively. Consequently, it should be emphasized that nonlinear static procedure yields to satisfying results during the estimation of seismic behavior of structures.
- After the building is strengthened, the mean inelastic top-story displacement demands are calculated as 0.020m and 0.017m when the infill walls are ignored or included, respectively. These values are also consistent with the nonlinear static analysis results.
- Inclusion of the infill walls increases the stiffness of the structure, hence the displacement demands decrease. However, the mean base-shear and overturning moment demands for the as-built structure increase 14% and 37%, respectively. These percentages are computed as 22% and 20% for the strengthened state, respectively.
- Inclusion of infill walls led more realistic results during the estimation of the occurrence of plastic hinges within the structure, since site inspections after the earthquake display no damage in the third and the fourth stories of the building.
- Different types of hysteretic relationships, which might represent RC structures better should be investigated as a future work. Also, more earthquake motions representing various soil properties, near-field effects, *etc.* should also be included in the future research by means of investigating their effects on the elastic deformation demands of structures.

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