Engineering Failure Analysis 55 (2015) 157–168

Contents lists available at ScienceDirect

Engineering Failure Analysis

journal homepage: www.elsevier.com/locate/engfailanal

Performance assessment of irregular RC buildings with shear walls after Earthquake

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ARTICLE INFO

Article history:
Received 5 September 2014
Received in revised form 21 May 2015
Accepted 25 May 2015
Available online 3 June 2015

Keywords:
Nonlinear static procedures
Pushover
Time history
Shear wall
Irregular RC building

ABSTRACT

This paper discusses the behavior of frame-wall irregularity on established existing reinforced concrete (RC) structures that were subjected to the 1999 Kocaeli Earthquake in Turkey. In particular, reference is made to the nonlinear static analysis and nonlinear dynamic analysis of reinforced concrete (RC) structures containing shear walls. The layered shell model has been chosen due to its advantages. The damage observed after reconnaissance studies has been captured using the 3D model. The imperfections that the irregularity effects caused are discussed throughout the paper.

1. Introduction

It is a well-known fact that buildings where the primary or only lateral force-resisting mechanism consists of walls are frequently called "*shear wall*" buildings. They are known to be one of the most efficient structural systems that resist lateral loads. However, as mentioned in a report by Sullivan et al. [1] there are several challenges facing the development of design methodology for frame-wall structures. It has been shown by the researchers just mentioned that the interaction between frame and wall members is not well accounted for in current design practices. Among the significant amount of research to correctly compute the behavior of frame-wall systems, the number of proposals that are able to consider higher mode effects is rare.

Nonlinear static procedures (NSP) have started to be used widely on the seismic assessment of structures in recent years. However, many scientists have established the limitations of these types of conventional procedures [2–12]. Conventional NSP's are mostly applicable for regular structures. In the case where irregularities are dominant, those procedures are flawed, since they do not have a robust background. To overcome those deficiencies scientists developed 3D pushover techniques to take into account the torsional effects [13–25]. In their study Fajfar et al. [16] stated that with an increase of plastic deformations the torsional effects tend to decrease. To consider the torsional effects, Fajfar and Gašperšič [12] improved their N2 method which is also proposed for inclusion in Eurocode [13–15]. For plan-asymmetric building structures Fajfar and Gašperšič [12] define torsional correction factors. These factors are in fact a ratio of the normalized roof displacements of the elastic response spectrum analysis to the pushover analysis. As can be inferred, the extended method uses both nonlinear

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http://dx.doi.org/10.1016/j.engfailanal.2015.05.016
1350-6307/

static pushover and elastic dynamic analysis. According to Kreslin et al. [13–15] the period ratios for both directions should be smaller than 1 for the structure to be considered as torsionally flexible. If this ratio exceeds 1 then the structure is considered to be torsionally stiff. These period ratios can be derived by dividing the uncoupled translational period by the uncoupled torsional period.

More detailed information on conventional and improved pushover methodologies, in order to consider irregularity effects, can be found in the research report of Adhikari and Pinho [26], which also highlights adaptive procedures.

It is a fact that nonlinear static analysis or in other words conventional pushover procedures are less accurate compared to a fully nonlinear dynamic analysis. However, in the previous studies it has been shown that a nonlinear static analysis can provide valuable information on the structural response.

2. Investigated buildings

To observe the effect of shear wall irregularities, two RC buildings in Kocaeli, Turkey were considered. The extended N2 method [13–15] proposed by Kreslin and Fajfar was applied on the selected structures. The two RC buildings have the same elevation and structural properties and were built in the same year by the same construction company. The only difference between the buildings is that the bottom story of one of the buildings is edged with a continuous shear wall, whereas the other one is not. In the latter building, there is a supermarket in the bottom story. The outer view of these buildings is given in Fig. 1.

The buildings under consideration are 6-storied and the height of each floor is 2.75 m. Concrete bore samples were taken from the buildings and their concrete strength was found to be 10 MPa. From investigations it was found that S220 steel was used as rebar. Six different column sections and one type of beam section were established from the reconnaissance studies. The sizes of the established columns were found to be 25 cm ∗ 35 cm, 25 cm ∗ 40 cm, 25 cm ∗ 50 cm, 25 cm ∗ 60 cm, 30 cm ∗ 60 cm and 60 cm ∗ 25 cm, whereas the dimensions of all the beams were found to be 20 cm ∗ 60 cm. Soil type was selected as Z2 which corresponds to periods of 0.15 s and 0.40 s (periods of corners of design spectrum proposed by TSC 2007) and since the selected buildings are residential ones, the importance factor and the live load attendance were taken as 1.0 and 0.3 respectively.

After the 1999 Earthquake hit the province of Kocaeli, the building with the shear wall irregularities at its bottom story had major damage: one of its columns had shear damage and had buckled severely. The other building with the continuous shear wall had no damage, though. Fig. 2 shows the observed shear hinge damage pattern.

3. Structural assessment

The considered RC buildings were modeled using SAP2000 [27] finite element software. A typical floor plan for these buildings is shown in Fig. 3. Two types of analysis were applied to the selected structures: inelastic static procedures to determine the capacity and nonlinear time history analysis to determine the drifts.

Fig. 1. Observed RC buildings.

Fig. 2. Damage pattern in the column in which shear hinge was observed.

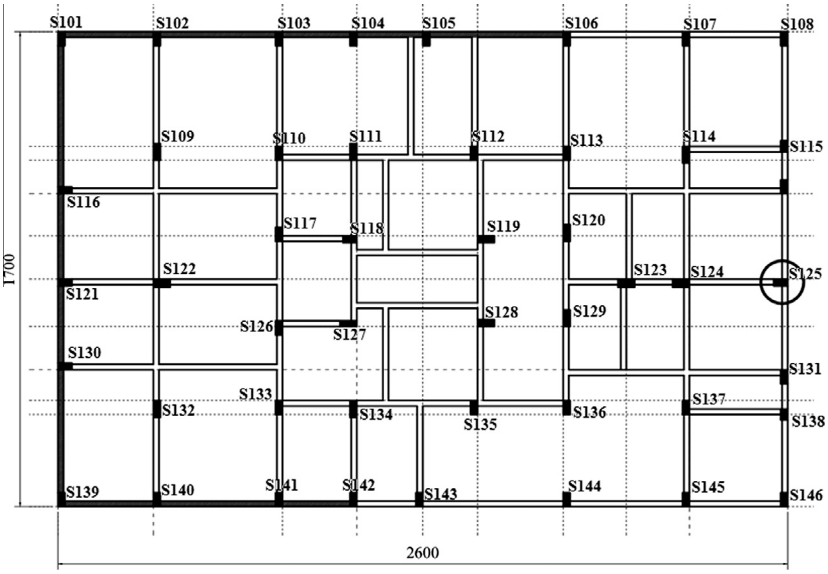

Fig. 3. Typical floor plan of the observed buildings.

3.1. Applied inelastic static procedure

The main objective of the study was to capture the same deformations on the structural model that were established during the field studies. As mentioned before, preceding studies showed that plan-wise irregular buildings require 3-D modeling due to the torsional behavior under seismic excitation [13–25]. Structural schemes of Fig. 3 provide evidence that one of the RC structures is irregular in plan. Irregularity calculations were done by applying the procedures defined in the Turkish Seismic Code (TSC 2007) [28] for this building.

TSC 2007 assessed torsional irregularity by factor η_{bi}, which is defined for either of the two orthogonal earthquake directions as the ratio of the maximum story drift $(\Delta_i)_{max}$ of any story to the average story drift $(\Delta_i)_{ave}$ of the same story in the same direction. η_{bi} is the torsional irregularity factor and if η_{bi} is greater than 1.2, building has torsional irregularity. Determination of the irregularity factor is given through Eqs. (1) and (2). In Eq. (2), $(\Delta_i)_{min}$ is the minimum story drift. Fig. 4 shows the defined parameters in detail.

$$\eta_{bi} = (\Delta_i)_{max}/(\Delta_i)_{ave} \tag{1}$$

$$(\Delta_i)_{ave} = 1/2[(\Delta_i)_{max} + (\Delta_i)_{min}] \tag{2}$$

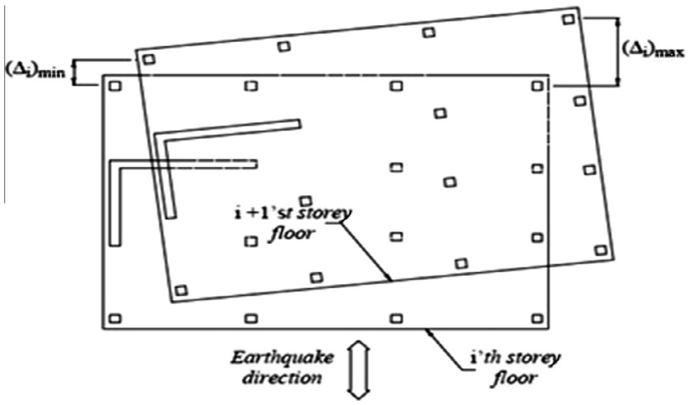

Fig. 4. Torsional irregularity determination (TSC 2007).

Table 1
Irregularity check applying TSC for the building surrounded with shear walls in the *x*-direction.

Story no.	$(\delta_i)_{max}$	$(\delta_i)_{min}$	$(\Delta_i)_{max}$	$(\Delta_i)_{min}$	$(\Delta_i)_{ort}$	$(\eta_b)_i$	Check
6	0.03477	0.02914	0.00456	0.00381	0.00418	1.09	<1.2
5	0.03021	0.02533	0.00715	0.00597	0.00656	1.09	<1.2
4	0.02306	0.01936	0.00834	0.00695	0.00764	1.09	<1.2
3	0.01473	0.01241	0.00872	0.00729	0.00801	1.09	<1.2
2	0.00600	0.00512	0.00586	0.00500	0.00543	1.08	<1.2
1	0.00014	0.00013	0.00014	0.00013	0.00013	1.04	<1.2

Table 2
Irregularity check applying TSC for the building surrounded with shear walls in the *y*-direction.

Story no.	$(\delta_i)_{max}$	$(\delta_i)_{min}$	$(\Delta_i)_{max}$	$(\Delta_i)_{min}$	$(\Delta_i)_{ort}$	$(\eta_b)_i$	Check
6	0.03240	0.02097	0.00438	0.00284	0.00361	1.21	>1.2
5	0.02803	0.01814	0.00692	0.00441	0.00567	1.22	>1.2
4	0.02110	0.01372	0.00777	0.00499	0.00638	1.22	>1.2
3	0.01334	0.00873	0.00799	0.00511	0.00655	1.22	>1.2
2	0.00534	0.00363	0.00509	0.00340	0.00424	1.20	=1.2
1	0.00025	0.00023	0.00025	0.00023	0.00024	1.05	<1.2

Table 3
Irregularity check applying TSC for the building with irregular shear walls in the *x*-direction.

Story no.	$(\delta_i)_{max}$	$(\delta_i)_{min}$	$(\Delta_i)_{max}$	$(\Delta_i)_{min}$	$(\Delta_i)_{ort}$	$(\eta_b)_i$	Check
6	0.03504	0.02951	0.00456	0.00382	0.00419	1.09	<1.2
5	0.03048	0.02569	0.00716	0.00598	0.00657	1.09	<1.2
4	0.02332	0.01972	0.00834	0.00696	0.00765	1.09	<1.2
3	0.01498	0.01276	0.00876	0.00733	0.00804	1.09	<1.2
2	0.00622	0.00543	0.00594	0.00521	0.00557	1.07	<1.2
1	0.00028	0.00022	0.00028	0.00022	0.00025	1.12	<1.2

Table 4
Irregularity check applying TSC for the building with irregular shear walls in the *y*-direction.

Story no.	$(\delta_i)_{max}$	$(\delta_i)_{min}$	$(\Delta_i)_{max}$	$(\Delta_i)_{min}$	$(\Delta_i)_{ort}$	$(\eta_b)_i$	Check
6	0.03528	0.02118	0.00439	0.00284	0.00362	1.21	>1.2
5	0.03089	0.01834	0.00694	0.00441	0.00568	1.22	>1.2
4	0.02394	0.01393	0.00782	0.00500	0.00641	1.22	>1.2
3	0.01612	0.00894	0.00826	0.00513	0.00669	1.23	>1.2
2	0.00787	0.00381	0.00624	0.00341	0.00482	1.29	>1.2
1	0.00163	0.00040	0.00163	0.00040	0.00102	1.61	>1.2

Irregularity assessment of the buildings according to the TSC procedure is shown through Tables 1–4. δ_i in the tables, is defined as the effective story drift of the i'th story of the building.

As well as FEMA regulations [29–31], many studies have shown that conventional pushover procedures are flawed when irregularities arise. This is the main reason why in this study the extended N2 method [13–15] was chosen for the selected RC buildings. The extended N2 method is actually an extension of the N2 method [12] to plan-asymmetric buildings, where torsional effects are important as well as the higher mode effects in elevation irregularities. Some correction factors have to be determined first in order to implement the procedure. A more detailed explanation can be found in [13–15]. Below, a summary of the applied procedure that is proposed by Kreslin and Fajfar et al. [13–15] can be found.

(a) Basic N2 method [12] is applied primarily in each direction.
(b) Standard elastic modal analysis is implemented and the roof displacements are determined. The results are then normalized in such a way that the roof displacement at the center of mass is equal to the target displacement.
(c) The correction factors are calculated: one accounting for torsion and the other for story drift. The correction factor for torsion is defined as the ratio between the normalized roof displacements obtained by elastic modal analysis and by the conventional pushover analysis. The lateral load pattern is chosen to be proportional with the fundamental mode. Lateral load pattern is derived by multiplying the fundamental mode values with the story mass values. The correction factor regarding the story drift (which is due to the higher mode effects) is defined as the ratio between the normalized story drifts obtained by elastic modal analysis and the results obtained by the conventional pushover analysis. The observed damage in the field was in the x-direction of the building with irregular shear walls, for which the calculated correction factors are of interest. The correction factors due to torsion and story drift are found to be 4.8 and 1.42, respectively.
(d) Internal forces, story drifts and all local quantities are multiplied by the aforementioned correction factors.

As shown in Fig. 5, throughout the analysis unconfined and confined Mander models [32] for concrete and the uniaxial steel model with strain-hardening [33] were used. Effective rigidity was taken into account in such a way that the elastic flexural and shear stiffness properties of the cracked sections amounted to one-half of the corresponding stiffness of the uncracked ones. No P-delta effects were taken into account.

Plastic hinges for flexural behavior are defined by a bi-linear moment-rotation relationship. The ultimate moment value was determined for each element using moment–curvature analysis. The axial forces resulting from the vertical loading were considered for the columns.

The bi-linearized moment–curvature relationship graphs, which are used to assign hinges for these sections can be found in Fig. 6. XTRACT [34] was used to derive the moment–curvature relationships. While deriving the moment–curvature relationships, moments about the x and y axes as well as moments about the principal bending axis were taken into account.

The target displacement value was calculated by following the steps that were explained in Fajfar and Gašperšič, Kreslin and Fajfar [12–15]. To capture the observed damage in the columns of the irregular building shear hinges were defined. The pushover results and the number of formed plastic hinges for both the x and y directions can be found in Tables 5 and 6, respectively. The capacity curves regarding both the x and y directions are shown in Fig. 7.

As can be inferred from Fig. 7, the capacity in the weak direction is less than the one in the other direction. This is due to the irregularity effect caused by the shear walls that are on the bottom story. The interesting aspect is that there is little difference between the capacities of the regular and irregular buildings. The reason might be that the applied procedure has not been tested (according to the authors' knowledge) with structures that have shear wall irregularities on the bottom stories. It is believed that the methodology should be further developed to take into account the irregularity effects that arise from the wall-frame elements.

The base shear and displacement values for the irregular building in the x-direction have been calculated as 2614.58 kN and 6.73 cm, respectively. In Fig. 8, performance points which were determined based on TSC 2007 for the regular and irregular RC structures in the damage direction were given. The modal capacity diagram is derived using the capacity curve and

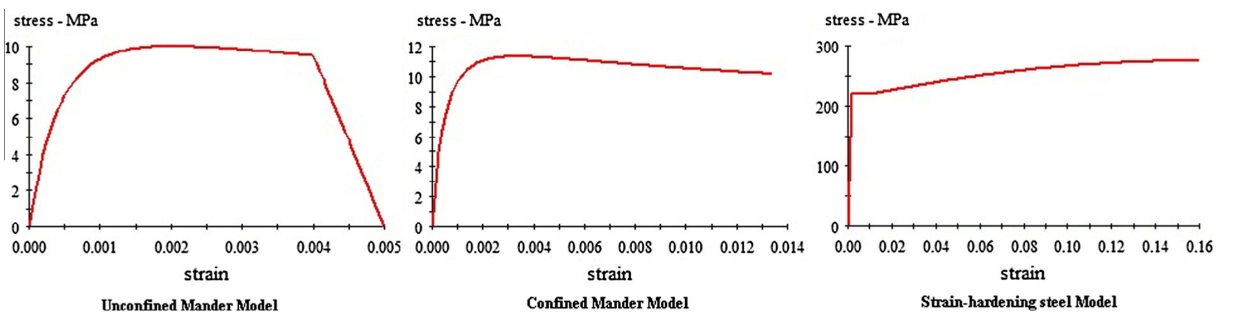

Fig. 5. Material models.

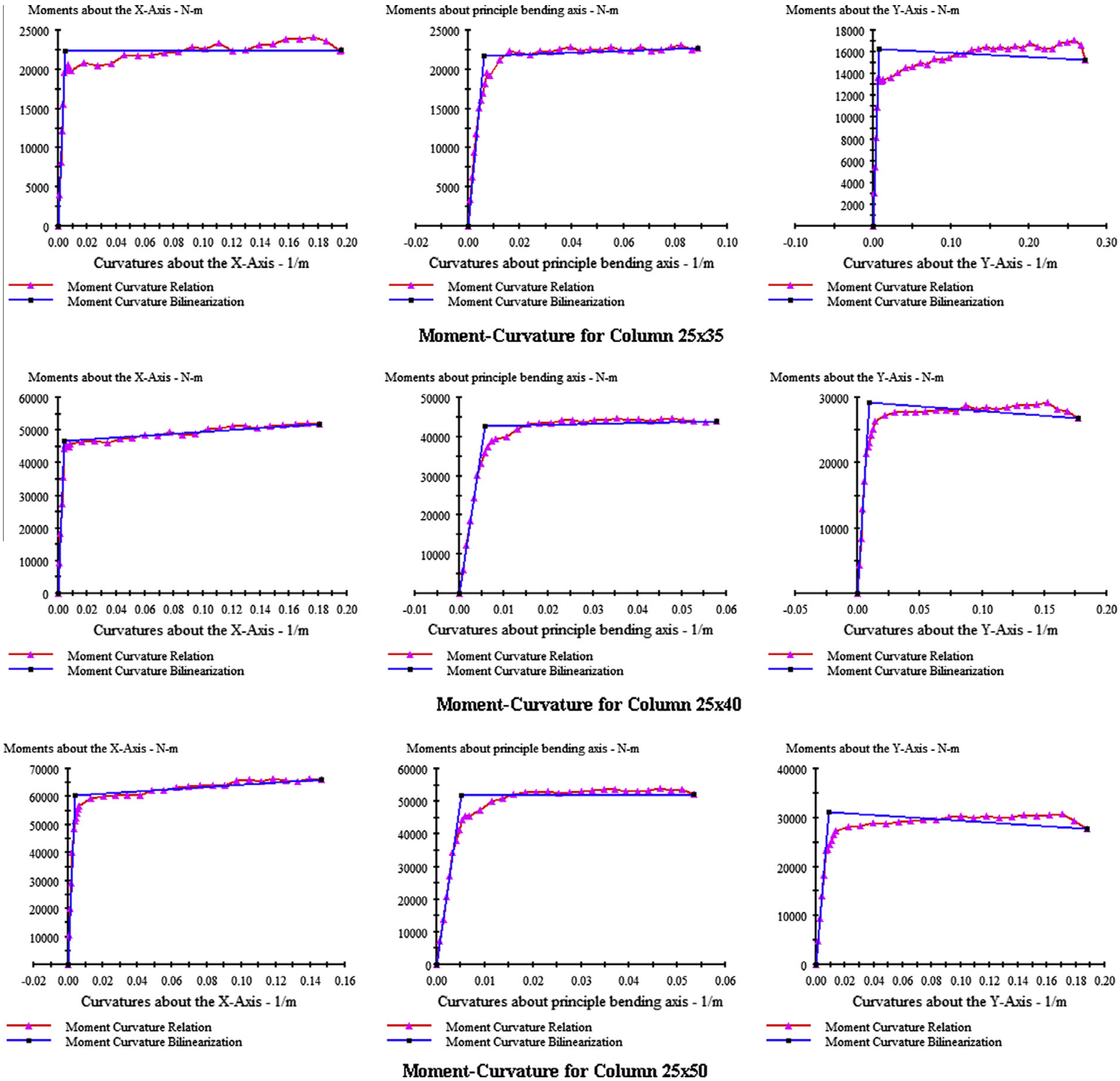

Fig. 6. Moment–curvature relationships for the columns and beam sections.

the spectral displacement–spectral acceleration values. The evaluated equations are given in Eqs. (3) and (4). The difference in the performance points for both the regular and irregular RC structures is obvious.

$$S_{d1} = C_R S_{de} \tag{3}$$

$$u_{xN1} = \varnothing_{XN1} \Gamma_{XN1} d_1 \tag{4}$$

Fig. 9 shows the nonlinear static results that were identified on the selected buildings. Shear hinge formation was established on the marked column for the building with shear wall irregularities. The field observations matched the model with respect to the irregular building. In these observations, 45-degree cracks were established in the abovementioned column. It is clear that the irregularity effects of the bottom story increased the acting shear force and this caused exceedance of the design shear force for that column. Formation of shear hinge for the marked column is to be expected afterwards. This is what damaged the column of the irregular building.

Shear capacity determination for the damaged column of the irregular RC building is shown in Table 7. The design shear force value is found to be approximately 45 kN. As can be seen from the table, the design shear capacity is exceeded for the mentioned column.

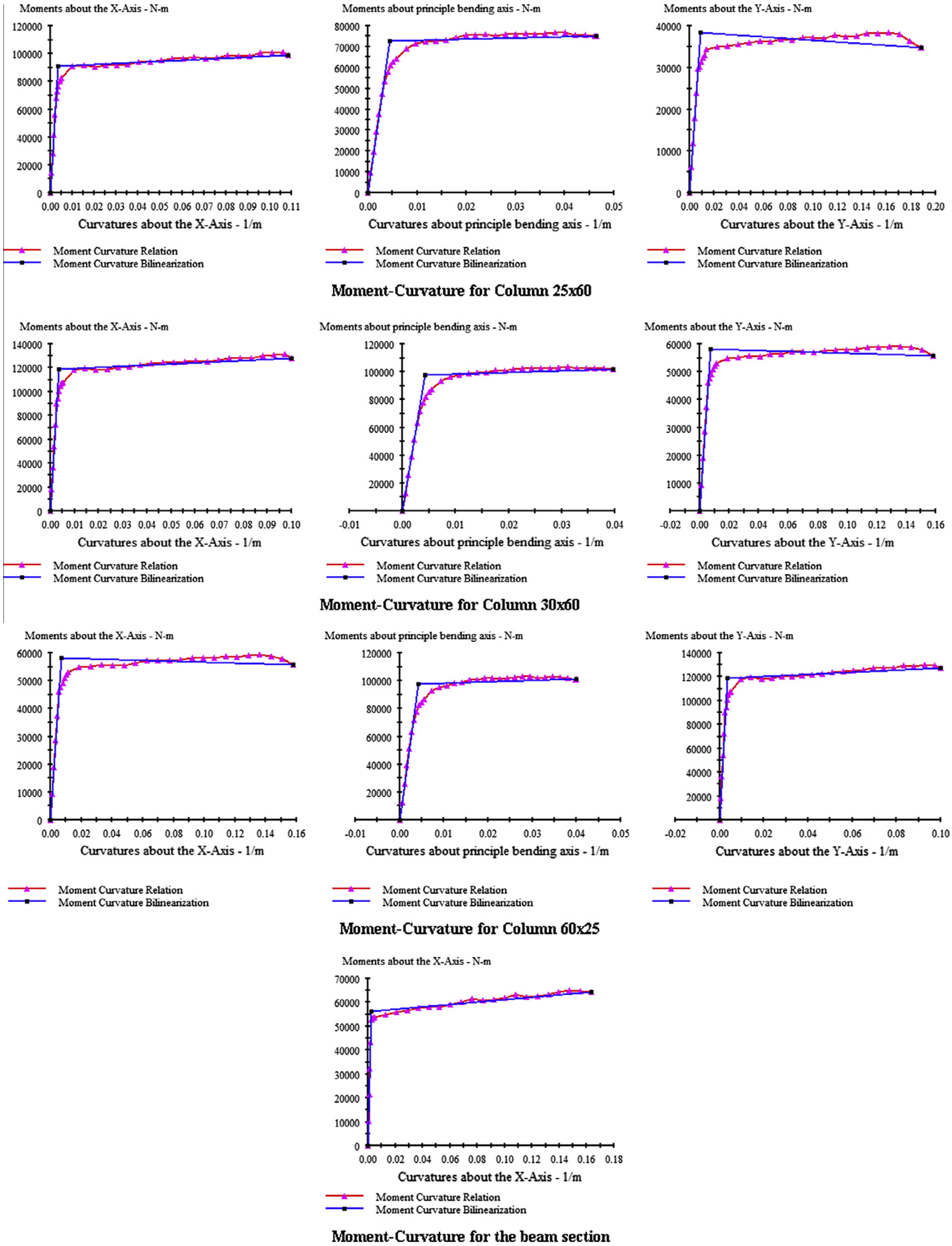

Fig. 6 (*continued*)

Table 5
Pushover results for both the regular and irregular buildings in the *x* direction.

Number of steps	Regular building			Irregular building		
	Displacement (m)	Base shear (kN)	Number of hinges	Displacement (m)	Base shear (kN)	Number of hinges
0	0.000143	0	0	0.000183	0	0
1	0.001121	84.014	0	0.00123	90.881	0
2	0.004958	414.53	7	0.004921	411.787	9
3	0.008508	710.723	9	0.008486	711.978	11
4	0.012241	1021.665	16	0.012866	1080.275	19
5	0.015871	1320.952	53	0.016365	1368.669	48
6	0.019458	1559.908	91	0.019799	1595.958	86
7	0.023149	1749.899	109	0.02339	1782.176	108
8	0.026926	1925.564	124	0.027141	1960.916	129
9	0.030633	2082.868	145	0.030608	2112.557	154
10	0.034989	2237.787	167	0.034104	2238.647	177
11	0.038543	2329.099	186	0.037664	2330.887	208
12	0.042268	2401.323	205	0.04121	2398.564	222
13	0.045863	2454.109	230	0.044736	2451.162	244
14	0.049931	2499.902	254	0.048444	2494.219	268
15	0.054113	2534.999	271	0.052957	2533.661	290
16	0.058463	2564.836	286	0.056392	2559.66	312
17	0.063098	2589.554	298	0.060253	2580.697	322
18	0.067322	2609.518	309	0.063831	2599.246	341
19	0.071143	2623.403	319	0.067318	2614.579	356

Table 6
Pushover results for both the regular and irregular buildings in the *y* direction.

Number of steps	Regular building			Irregular building		
	Displacement (m)	Base shear (kN)	Number of hinges	Displacement (m)	Base shear (kN)	Number of hinges
0	0.000461	0	0	0.000568	0	0
1	0.002173	78.843	0	0.002023	66.489	0
2	0.008923	389.532	1	0.008823	377.044	1
3	0.017359	777.683	2	0.017514	773.799	2
4	0.024394	1092.735	19	0.024592	1088.304	20
5	0.031319	1359.925	39	0.031653	1357.9	39
6	0.038585	1577.9	70	0.038939	1574.319	68
7	0.045962	1740.524	98	0.046014	1730.916	96
8	0.052859	1856.517	118	0.053214	1852.949	120
9	0.060207	1947.96	158	0.060348	1941.992	158
10	0.06796	2019.853	182	0.067278	2009.704	182
11	0.075296	2071.868	211	0.074633	2064.212	206
12	0.082497	2115.774	238	0.081628	2107.749	232
13	0.090939	2161.141	277	0.089224	2150.811	266
14	0.097689	2198.444	303	0.097108	2188.174	295
15	0.104439	2230.62	316	0.105609	2222.031	319
16	0.113193	2265.932	337	0.113185	2250.145	331
17	0.126251	2305.168	352	0.119985	2272.224	338
18	0.133599	2327.261	363	0.127437	2295.586	349
19	0.135461	2331.492	368	0.136568	2321.298	358

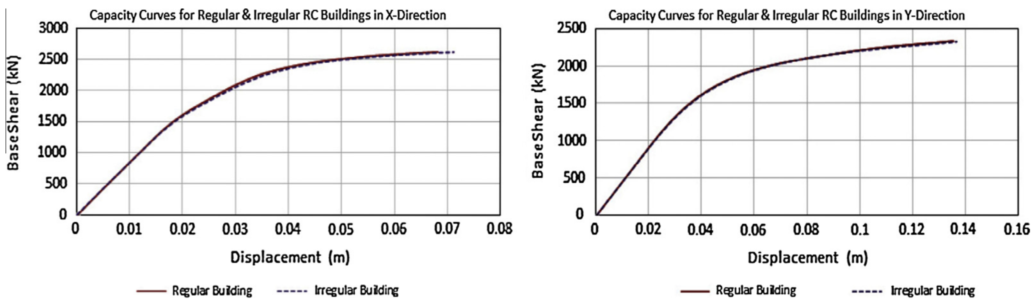

Fig. 7. Capacity assessment of regular and irregular buildings for both *x* and *y* directions.

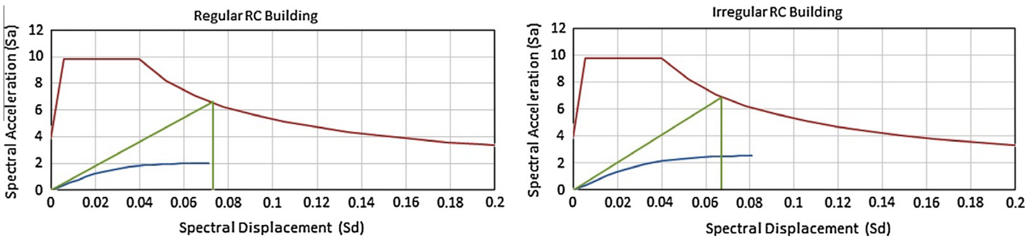

Fig. 8. Performance point determination in the damaged direction using TDC 2007.

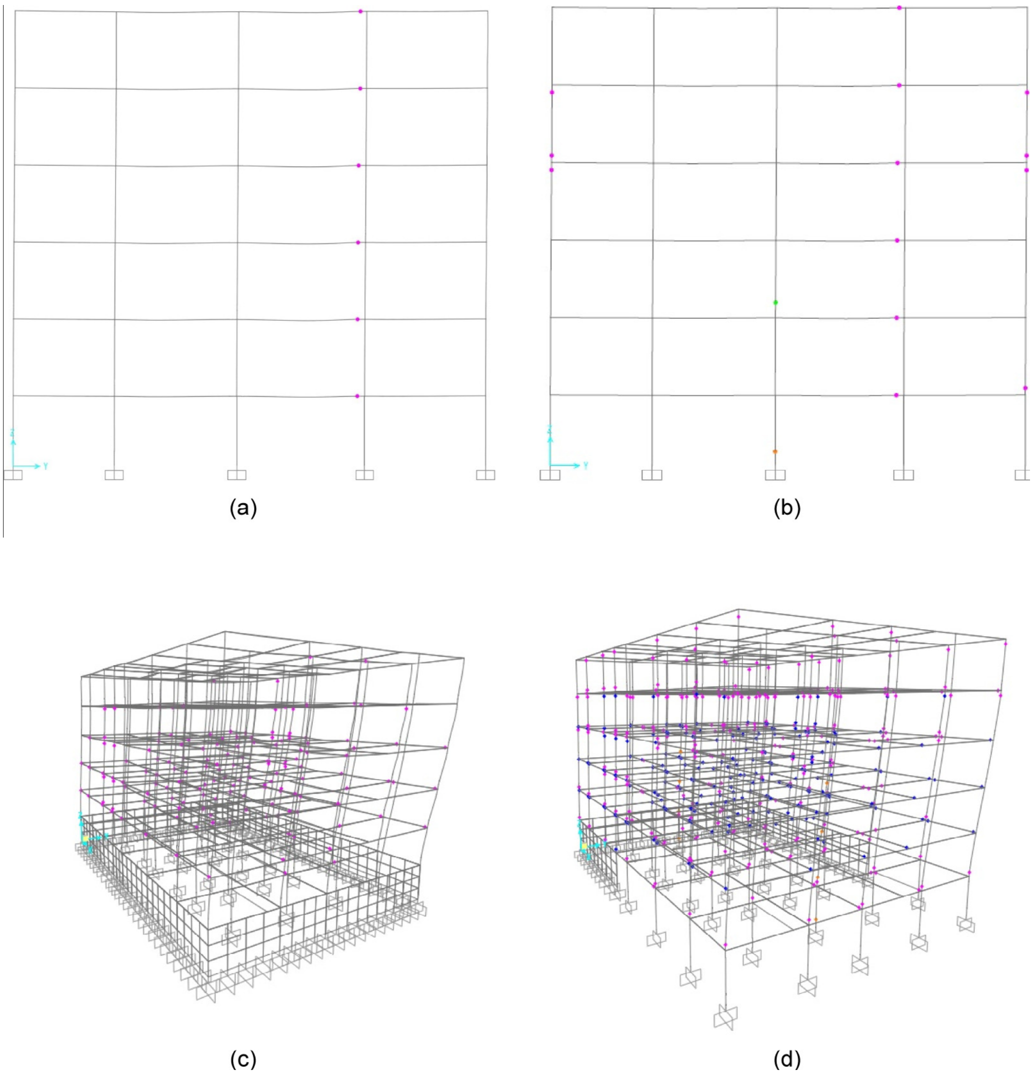

Fig. 9. Hinge formations.

Table 7
Shear capacity determination for the damaged column.

Column no.	Dimensions (cm)	Transverse reinforcement (mm)	Shear capacity (kN)
S125	250 × 500	∅8/500	80.37

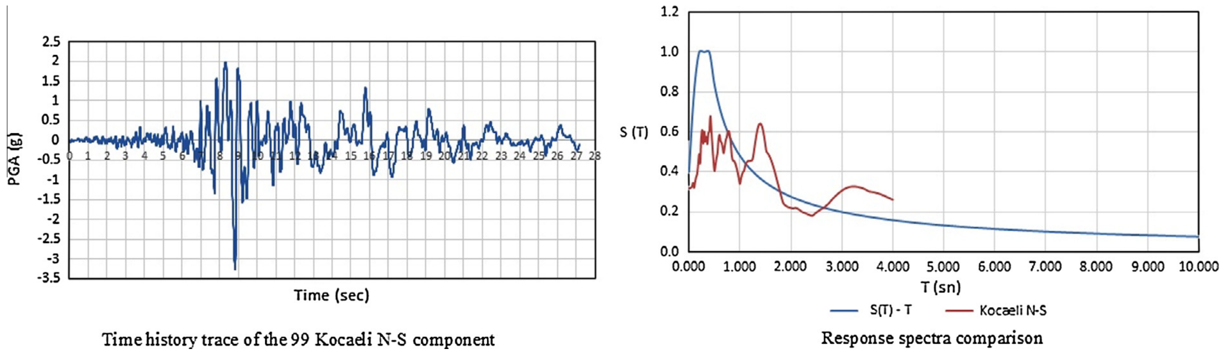

Fig. 10. Time history record and response spectra comparison.

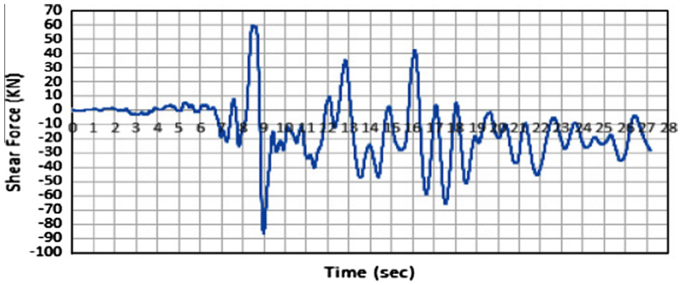

Fig. 11. Shear force–time history trace for the damaged column element.

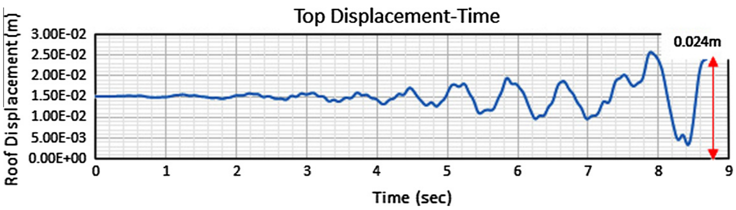

Fig. 12. Top displacement–time graph.

The brittle collapse mechanism of the system due to shear hinge formation is shown in Fig. 9. The shear hinge that occurred in the damaged column of the irregular building is marked with a circle in Fig. 9. The corresponding displacement at this point was found to be 0.019 m and the base shear value was 1742 kN.

3.2. Applied nonlinear time history procedure

As an alternative approach, nonlinear time history analysis was also implemented on the regular and irregular buildings which were subjected to the 1999 Kocaeli Earthquake. Since the PGA value of this earthquake for the N–S direction was greater than the other direction, the time history record of N–S direction was used. Quadratic baseline correction and the Butterworth filter were applied on a selected record using Seismosignal [35]. The time history trace of the selected record as well as a spectral acceleration comparison (using the Turkish design spectra for the considered soil type Z2) are shown in Fig. 10. It can easily be inferred from Fig. 10 that the PGA value of the Kocaeli Earthquake exceeds the design spectra given in TSC 2007.

The shear force–time history trace for the marked column element is given in Fig. 11. The design shear force using Turkish standards was calculated to be approximately 45 kN. As can be seen from Fig. 11, the first shear hinge occurred at $t = 8.7$ sec, when the base shear value was calculated as 86.014 kN. It can be inferred from this figure that the shear force subjected exceeds the design value for the column. This is the main reason for the damage.

Concerning the displacement of the top point at the aforementioned time value, a top displacement–time graph is plotted in Fig. 12. The top displacement at $t = 8.7$ sec was found as 0.024 m, which has good correlation with the nonlinear static pushover analysis results.

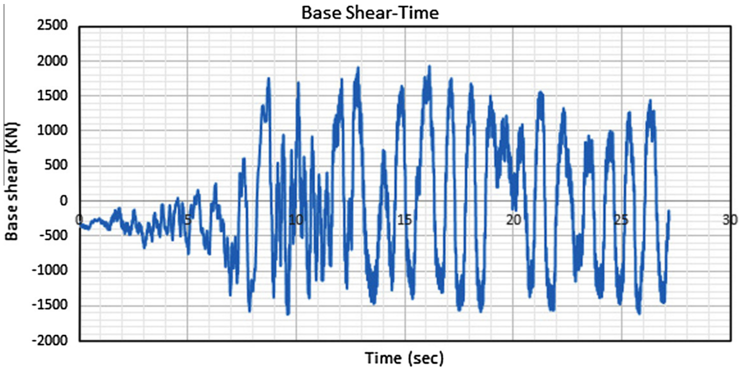

Fig. 13. Base shear–time graph.

Fig. 13 shows the base shear–time graph of the damaged irregular building. The maximum base shear was found to be 1929.46 kN.

4. Conclusion

The damage observed on an irregular building after the 1999 Kocaeli Earthquake is discussed in this study. First, the key properties of the recorded strong ground motion were evaluated. After that, the damage observed to the irregular concrete structures was evaluated using three dimensional structural models and applying both the extended N2 [13–15] and nonlinear time history methods. Based on the investigated results, the following conclusions can be drawn:

(a) Shear hinge formation was observed at $t = 8.7$ sec. The top displacement value at this time was found to be 0.024 m from implementation of the nonlinear time history analysis. The same value was found to be 0.019 m from application of the extended N2 method to the irregular structure. The results of the methods are in good correlation.
(b) The calculated base shear values using the extended N2 method and the nonlinear time history method were found to be 1537 kN and 1929.46 kN, respectively which are also in good correlation.
(c) As a result of the analysis conducted, it can be inferred that the irregularities due to the basement shear walls decrease the capacity of the building appreciably.
(d) Previous studies comparing the extended N2 method and the dynamic time history results in terms of displacements and story drifts conclude that the extended N2 method overestimates the story displacements especially for the upper stories, as well as the story drifts.
(e) It should be noted that the quality of the concrete might also affect exceedance of the design shear force. In Turkey, ready-mixed concrete started to be used just after the 1999 Earthquake hit the Kocaeli Province. This could be another reason for the observed damage.

The authors believe that irregularity checks should be intensified in the Turkish Seismic Code. At the very least, the criteria should be synchronized with European standards.

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