Effect of Infill Walls on the Seismic Performance of an Old Building

Uğur Albayrak, Eşref Ünlüoğlu, and Mizam Doğan

Abstract—**The effect of the existence of infill walls on the earthquake performance of a structure is one of the main research areas in terms of seismic safety. This study present a performance based approach to the entrenched procedures for seismic design of buildings contained in standards as Turkish Seismic Code. In this manner, an existing 55 years old industrial R.C. building in Eskisehir, Turkey was modelled and performance-based seismic design deal with the verification of seismic performance uttered in terms of limit states defined in Turkish Seismic Code. Three dimensional finite elements modelling of the building was created based on the measurements and observations on site. SAP2000 was utilized to make earthquake analysis of the 3-D numerical model of the building. According to the results from the performance evaluation of entire building; the contribution of the infill walls to the strength and stiffness of entire structure can be easily demonstrated. These results from performance based design procedure indicate that infill walls in this building are the main reason why this insufficient building in terms of earthquake resistance has been survived for many years under effective earthquakes.**

Index Terms—**Infill walls, old building, performance based design, Turkish seismic code.**

I. Introduction

Turkey is one of the most earthquakes occurred countries and suffered from some of the worst earthquakes in the world. Especially North-western part of Turkey which is the county's most densely populated region and industrial heartland has been struck by major earthquakes in history. The most powerful earthquake to hit Turkey is Izmit Earthquake, on 17 August 1999 measured 7.4 on the Richter scale and continued 45 seconds, killing around 17,000 people and cause 500,000 people homeless [1].

Most codes require that all new buildings must be able to survive after a major earthquake [2]. The building can crack, tilt and even be declared unfit for future use but it must not totally collapse [3]. In Turkish Seismic Code (TSC), the crucial factor to fix life losses at a certain low level is about the building code itself and its application in casual life [4]. The main aim of the code is to define the minimum requirements for seismic design and construction of reinforced or steel etc... buildings and structures which is subjected to earthquake ground motion. The code requirements must be applicable to newly constructed buildings as well as to existing old buildings [4]. The general principle of the code is to limit the damage in structural elements and to prevent the total collapse of buildings in major earthquakes in order to avoid the loss of life. On the other hand, effect of the infill walls on the earthquake performance of a structure is one of the main research areas in terms of seismic safety [5]. Behavior of a R.C. building with infill walls under seismic loads should be modeled to consider the effect of the infill walls on the seismic performance of the structure [6].

II. Method

Investigation and evaluation of earthquake performance of the existing building are carried out according to the essentials for TSC [4]. The building which is investigated in this paper can be assessed according to the rules that defined in relevant section of the code. After an earthquake causing damage in the building, and then to determine the earthquake performance of the strengthened building, essentials given in the code will be used.

First of all, all necessary data have to be collected from the building by measurements to be achieved on the building such the geometry and details of the R.C. elements and materials to be used in determining the capacities of the elements of the existing building [7]. The collected data from the investigation of the buildings determine the information level of the buildings which can be classified as limited, medium and comprehensive where the limited level is defined as there is not any projects of the structural system about the building. Characteristics of materials and existing materials strengths used in the building were determined by collecting at least two samples of borehole concrete from columns and beams. Reinforcement details can be determined according to the visual examination and performing test on the rebar samples. According to the test results, characteristic yield strength of the rebar can be taken as the existing steel strength which is taken into calculations of the element capacities [8]. Information Level Coefficients to be used in the calculation of element capacities can be obtained from the investigated building so it is taken as 0.75 for limited information level.

Performance-based seismic design deal with the verification of seismic performance uttered in terms of limit states defined below [9]. Three limit conditions of damage were defined for ductile elements as Minimum Damage Limit (MN), Safety Limit (GV) and Collapsing Limit (GC) in Turkish Seismic Code while this classification is not suitable for brittle case. The structural elements that the damages with critical sections do not reach MN are within the Minimum Damage Region, those in-between MN and GV are within

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Marked Damage Region, those in-between GV and GÇ are in Advanced Damage Region, and those going beyond GÇ are within Collapsing Region given in Fig. 1 [4].

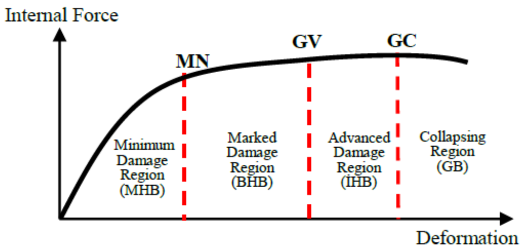

Fig. 1. Sectional damaged areas according to [4].

In order to be able to decide which damage zone the sections are in, it is first necessary to determine internal forces and/or deformations using linear elastic calculation methods or linear non-elastic calculation methods [10]. After this process, the sections are compared with the numerical values with damage limits to define proper damage zone considering the most damaged section.

A. General Principles Related to Earthquake Damage Determination

Elastic acceleration spectrum (non-reduced) will be used for the definition of the earthquake. Building Importance Factor (I =1.0) varied in terms of the purpose of occupancy or building type will not be applied in the seismic calculation. Earthquake forces act on the building in both directions and on both sides separately. The ground parameters such as soil group and local site class were to be considered that will be used in seismic calculations, were have been determined according to Turkish Seismic Code [4]. In the building calculation process, the slabs were assumed as rigid diaphragms on the horizontal axis (z) and any additional eccentricity had not been applied [11]. Information level coefficient of the building was used to identify some deficiencies or uncertainty on the load-bearing systems. All of the columns in the building were defined in the structural model with their free heights. Confinement zones at the beam-column connections can be considered as infinitely rigid end [12]. In reinforced concrete elements under bending effect, Active bending rigidities $(EI)_e$ of the cracked sections should be used. In order to do this, the rigidity values given below should be replaced for active bending rigidities [4]:

- Beams , $(EI)_e = 0.40(EI)_o$
- Columns and frames, $(EI)_e = 0.40(EI)_o$ if $N_D/(A_c f_{cm}) \leq 0.10$
  $(EI)_e = 0.80(EI)_o$ if $N_D/(A_c f_{cm}) \geq 0.40$

For the case of insufficient coupling or splicing length in structural members, yield tensile of the reinforcement should be reduced in proportion as the shortening in length.

III. Case Study

The earthquake performance of a 55 year old reinforced concrete structure was determined by using the Equivalent Seismic Load Method, which is one of the linear elastic calculation methods described before [13]. The examined building with 6-storey which is divided into administrative and production parts was built in 1962 as a flour factory and the production was stopped in 2003 (Fig. 2).

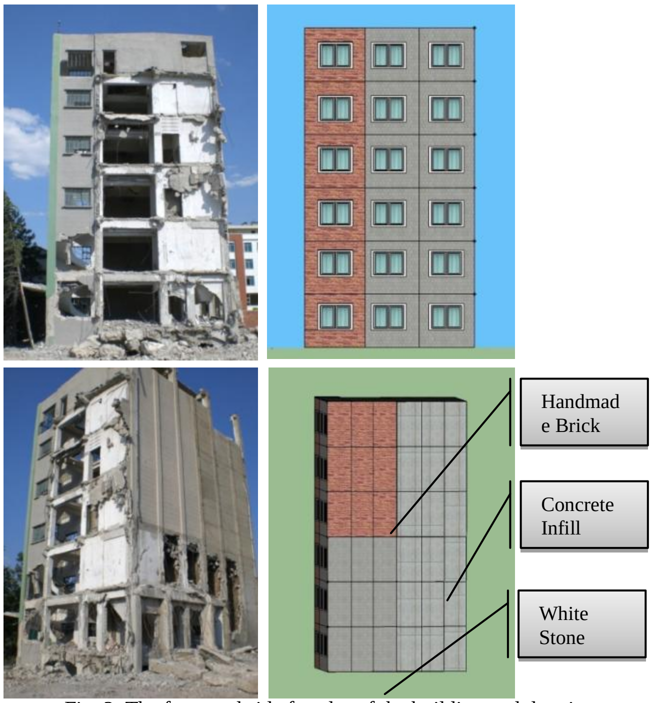

Fig. 2. The front and side facades of the building and drawings.

A. Three Dimensional Finite Element Modelling of the Building

A 6 story R.C. building presented below as a case study to define the effect of the infill walls on the seismic performance of an old building. Three dimensional finite element modelling of the building was created based on the measurements and observations on site. SAP2000 was utilized to make earthquake analysis of the structure [14]. 3-D numerical model of the building was consisting of beams, columns, slabs and walls are shown in Fig. 3. All beams and columns of the building were considered as frame elements whereas slabs were thin plates and walls were shell elements.

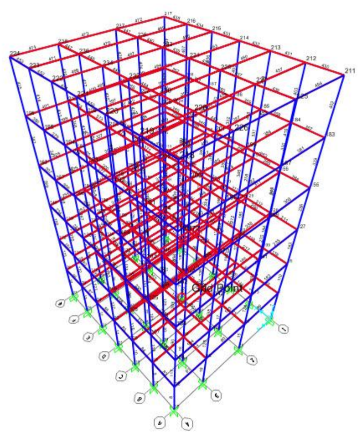
Fig. 3. 3-D finite element model of the load-bearing system.

Process to be used for the seismic analysis of building is equivalent seismic load method and modal analysis to determine earthquake performance of the building with and without infill walls. According to the first results from the observations on site, columns dimensions are not adequate whereas concrete quality was also poor. Adhesion between steel rebar and concrete is not sufficient to prevent pull-out failure. Information about the building is given in Table I.

TABLE I: ALL INFORMATION ABOUT THE BUILDING

Building Properties	Story height	3 m
	Earthquake zone	2
	Local site class	Z4
	Spectrum characteristic periods	0.20 / 0.90
Material Properties	Concrete	C10
	Reinforcement Steel	S220 / Plain
	Modulus of elasticity of concrete	25300 Mpa
	Modulus of elasticity of steel	2×10^5 Mpa
Slab Properties	Reinforced concrete thickness	15 cm
	Leveling concrete thickness	5 cm
	Plaster (lime-cement mixture)	2 cm
	Coating	2 cm

Plan view at the first floor of the models is given in Fig. 4.

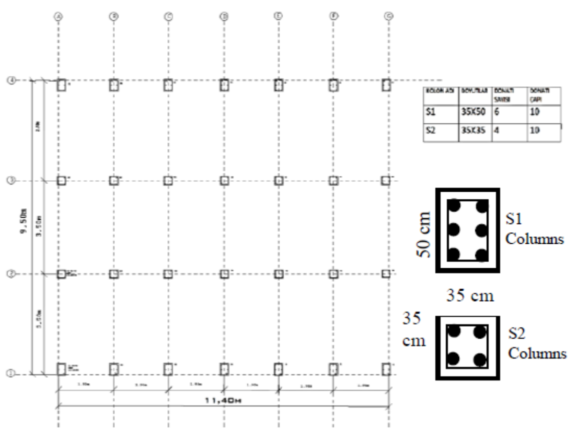

Fig. 4. Plan view of the building.

The structure were analyzed and according to axial load level of the columns *Active bending rigidities* $(EI)_e$ of the cracked sections calculated by interpolation between 0.40 and 0.80 values using the Eq. 1 below and given in the Table II [4]:

$$0.10 \leq N_D/(A_c{*}f_{cm}) \leq 0.40 \tag{1}$$

Effect / capacity ratio of ductile beam, column and walls are calculated by dividing the section moment calculated under seismic load by taking $R_a = 1$ to over moment capacity. On the calculation of effect / capacity direction of the earthquake can be taken into consideration [4].

According to TSC [4], over moment capacities which are the difference between bending moment capacity and moment effect calculated on the section under gravity loads of the all beams in the structure were calculated. All of the beams are the same dimensions and tensile and compressive reinforcement are the same thence moment capacity at point i is equal but negative sign to point j and it can be calculated with Eq. 2 and 3 given in below [4]:

$$M_r = 0.85\, f_{cd}\, b_w\, k_1\, c\, [d-(k_1\, c)/2] + A'_s\, \sigma'_s\, (d-d') \tag{2}$$

$$c = (A_s - A_s')f_{yd}/(0.85 f_{cd} b_w k_1) \tag{3}$$

Over moment capacities (M_R) and redundant moment capacities (M_D) of the sections on the direction X and Y can be calculated and only X direction beams are given in Table III.

TABLE III: MOMENT CAPACITIES OF THE X DIRECTION BEAMS AT FIRST FLOOR

Beam No	M_{Ri} (kNm)	M_{Rj} (kNm)	M_{Di} (kNm)	M_{Dj} (kNm)	M_{Ai} (kNm)	M_{Aj} (kNm)
B1	21.27	-21.17	-1.757	0.8894	22.927	20.2806
B2	21.27	-21.17	-0.489	-0.1692	21.659	21.3392
B3	21.27	-21.17	-0.6414	-0.5983	21.8114	21.7683
B4	21.27	-21.17	-0.5983	-0.6414	21.7683	21.8114
B5	21.27	-21.17	-0.1692	-0.489	21.3392	21.659
B6	21.27	-21.17	0.8894	-1.757	20.2806	22.927
B7	21.27	-21.17	-2.5479	2.0667	23.7179	19.1033
B8	21.27	-21.17	0.1195	0.2183	21.0505	20.9517
B9	21.27	-21.17	-0.4469	-0.462	21.6169	21.632
B10	21.27	-21.17	-0.462	-0.4469	21.632	21.6169
B11	21.27	-21.17	0.2183	0.1195	20.9517	21.0505
B12	21.27	-21.17	2.0667	-2.5479	19.1033	23.7179
B13	21.27	-21.17	-2.2032	1.6753	23.3732	19.4947
B14	21.27	-21.17	0.0048	0.0519	21.1652	21.1181
B15	21.27	-21.17	-0.5042	-0.5231	21.6742	21.6931
B16	21.27	-21.17	-0.5231	-0.5042	21.6931	21.6742
B17	21.27	-21.17	0.0519	0.0048	21.1181	21.1652
B18	21.27	-21.17	1.6753	-2.2032	19.4947	23.3732
B19	21.27	-21.17	-1.5474	0.6482	22.7174	20.5218
B20	21.27	-21.17	-0.5835	-0.2169	21.7535	21.3869
B21	21.27	-21.17	-0.6602	-0.6091	21.8302	21.7791
B22	21.27	-21.17	-0.6091	-0.6602	21.7791	21.8302
B23	21.27	-21.17	-0.2169	-0.5835	21.3869	21.7535
B24	21.27	-21.17	0.6482	-1.5474	20.5218	22.7174

Redundant moment capacity M_A and corresponding axial force N_A defined as follows [4]:

$$M_A = M_K - M_D \tag{4}$$

$$N_A = N_K - N_D$$

And effect / capacity ratios (r) of columns and walls may be defined as follows [4]:

$$r = M_E/M_A = N_E/N_A \leq r_s \tag{5}$$

where M_D or N_D is known from gravity load design and M_E or N_E is known from seismic design.

Moment and axial force capacities of the column sections (M_K, N_K) as the coordinates of K intersection point in Fig. 5 is obtained from geometrically or using the equations [4].

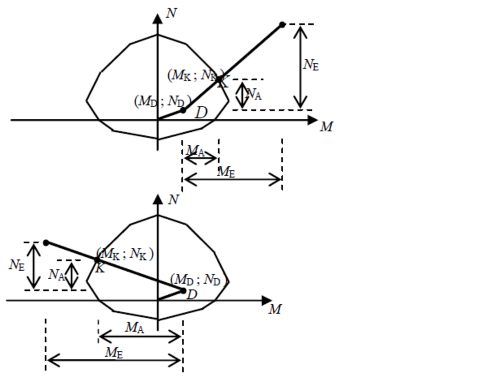

Fig. 5. Moment and axial force capacities of the column sections (M_K, N_K) [4].

TABLE II: ACTIVE BENDING RIGIDITIES OF CRACKED SECTIONS AT FIRST FLOOR

Element No	N_d (kN)	A_c (mm^2)	$N_D/(A_c*f_{cm})$	$(EI)_e$
1	281.705	175000	0.134145	0.57886
2	335.995	175000	0.159998	0.61333
3	359.123	175000	0.171011	0.628015
4	364.24	175000	0.173448	0.631263
5	359.123	175000	0.171011	0.628015
6	335.995	175000	0.159998	0.61333
7	281.705	175000	0.134145	0.57886
8	250.214	122500	0.170214	0.626951
9	298.092	122500	0.202784	0.670378
10	319.003	122500	0.217009	0.689345
11	323.799	122500	0.220271	0.693695
12	319.003	122500	0.217009	0.689345
13	298.092	122500	0.202784	0.670378
14	250.214	122500	0.170214	0.626951
15	319.227	122500	0.217161	0.689548
16	278.886	122500	0.189718	0.652958
17	388.583	122500	0.264342	0.752456
18	338.96	122500	0.230585	0.707447
19	418.031	122500	0.284375	0.779166
20	363.906	122500	0.247555	0.730073
21	424.216	122500	0.288582	0.784776
22	369.086	175000	0.175755	0.63434
23	418.031	175000	0.199062	0.665417
24	363.906	175000	0.173289	0.631051
25	388.583	175000	0.18504	0.646719
26	338.96	175000	0.16141	0.615213
27	319.227	175000	0.152013	0.602684
28	278.886	175000	0.132803	0.57707

B. Determination of the Normal Force and Bending Moment Capacities of Columns

The calculation of the normal force and bending moment capacities of columns for the direction *X* and *Y* were achieved according to [4]. In the performance calculation, the ground and first floor connections were checked in this way only the normal force and bending moment capacities of these joints were calculated in Table IV. Because two different column dimension also two different column capacity exist and calculated as N_{r1}= 545.70 kN and N_{r2}=577.20 kN.

TABLE IV: MOMENT CAPACITIES OF THE COLUMNS AT FIRST FLOOR

Column No	End	N_k (kN)	N_D (kN)	N_A (kN)	Column No	End	N_k (kN)	N_D (kN)	N_A (kN)
1	i	577.2	281.705	295.495	15	i	545.7	319.227	226.473
	j	577.2	268.58	308.62		j	545.7	310.039	235.661
2	i	577.2	335.995	241.205	16	i	545.7	278.886	266.814
	j	577.2	322.87	254.33		j	545.7	269.699	276.001
3	i	577.2	359.123	218.077	17	i	545.7	388.583	157.117
	j	577.2	345.998	231.202		j	545.7	379.396	166.304
4	i	577.2	364.24	212.96	18	i	545.7	338.96	206.74
	j	577.2	351.115	226.085		j	545.7	329.773	215.927
5	i	577.2	359.123	218.077	19	i	545.7	418.031	127.669
	j	577.2	345.998	231.202		j	545.7	408.843	136.857
6	i	577.2	335.995	241.205	20	i	545.7	363.906	181.794
	j	577.2	322.87	254.33		j	545.7	354.719	190.981
7	i	577.2	281.705	295.495	21	i	545.7	424.216	121.484
	j	577.2	268.58	308.62		j	545.7	415.029	130.671
8	i	577.2	250.214	326.986	22	i	545.7	369.086	176.614
	j	577.2	237.089	340.111		j	545.7	359.899	185.801
9	i	577.2	298.092	279.108	23	i	545.7	418.031	127.669
	j	577.2	284.967	292.233		j	545.7	408.843	136.857
10	i	577.2	319.003	258.197	24	i	545.7	363.906	181.794
	j	577.2	305.878	271.322		j	545.7	354.719	190.981
11	i	577.2	323.799	253.401	25	i	545.7	388.583	157.117
	j	577.2	310.674	266.526		j	545.7	379.396	166.304
12	i	577.2	319.003	258.197	26	i	545.7	338.96	206.74
	j	577.2	305.878	271.322		j	545.7	329.773	215.927
13	i	577.2	298.092	279.108	27	i	545.7	319.227	226.473
	j	577.2	284.967	292.233		j	545.7	310.039	235.661
14	i	577.2	250.214	326.986	28	i	545.7	278.886	266.814
	j	577.2	237.089	340.111		j	545.7	269.699	276.001

Shear force control were performed in accordance with the bending capacity at critical sections of structural system components. Columns, beams and walls are to be assumed ductile, V_e shear force have to be calculated in accordance with the bending capacity in the critical sections with respect to bending capacity of the critical sections which calculated shear capacity V_r will not exceed. Shear force controls compatible with bending capacity of columns were achieved for *X* and *Y* directions. Similarly shear force controls compatible with bending capacity of beams were achieved for both directions. All calculated V_r values are more than all V_e than all of the beams in the structure are ductile.

C. Performance Evaluation for Structural Elements

As a result of analysis made on model created in Sap2000, M_E values were obtained and "effect / capacity ratios" (*r*) of beams were calculated using redundant moment capacity M_A. Performance evaluation of beams have been performed using M_E and M_A defined in Eq. 5 and effect / capacity ratios (*r*) which defines the boundary of the damage for concrete beams were calculated. Similarly, effect / capacity ratio of infilled walls are the shearing force strength of shearing force calculated under the effect of earthquake.

Using the calculated normal force (N_K) and moment (M_K) capacities of the columns, effect / capacity ratios (**r**) of the columns were calculated and the boundaries of the damage for columns were determined [4].

$$r = N_E/(N_K-N_D) \tag{6}$$

$$r = M_E/(M_K-M_D) \tag{7}$$

The most damaged section is taken into account while the boundary of the damage of any column is determined. MN, GV and GC intermediate values were found by interpolation.

As a result of the performance analysis with linear elastic calculation methods of the existing infilled building, the percentages of damages of beams and columns calculated under the effect of earthquakes in both directions are given in Table V. Especially at the ground floor of the building, it is a big problem that some of the structural elements of the system are in the Collapsing Region.

TABLE V: DAMAGE PERCENTAGES AND SECTIONAL DAMAGED STATES FOR EXISTING INFILLED BUILDING

Ground Floor	+X Direction				+Y Direction			
	MHB %	BHB %	IHB %	GB %	MHB %	BHB %	IHB %	GB %
Columns		71.4	25	3.6		39	61	
Beams			40	60			33	67

IV. CONCLUSIONS

The following concluding remarks were obtained as a result of Performance based design evaluation performed on a 55 years old building:

The existing old industrial R.C. building was modelled 3-D and analyzed by SAP2000 to make earthquake analysis of the structure. All of the reinforcements are rusty plain bars affected by corrosion while yield strength of steel reinforcement bars is almost 160 MPa. Meanwhile, average compressive strength of the concrete samples taken from the building is almost 8 ~ 10 MPa. In fact, earthquake safety of a building depends on using material quality beside structural factors.

Performance evaluation of the building was achieved and the effect of infill walls on the seismic performance of the structure has been investigated. According to the results from the performance evaluation of entire building; 71.4% of the columns are in Marked Damage Region (BHB) for X direction and 39.0% for Y direction while 3.6% of the columns are in Collapsing Region (IHB) for X direction and 0% for Y direction. On the other hand, the same model without any infill wall (Brick, white stone or concrete) was analyzed then all of the columns and beams would have been in Advanced Damage Region or Collapsing Region for both X and Y directions. As a result, the contribution of the infill walls to the strength and stiffness of entire structure can be easily demonstrated. These results from performance based design procedure indicate that infill walls in the building are the main reason why this insufficient building in terms of earthquake resistance has been survived since 55 years.

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