STRUCTURAL ENGINEERING ASPECTS OF THE 1967 ADAPAZARI, TURKEY, EARTHQUAKE.

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SYNOPSIS

The authors were members of a team appointed by the Turkish Association of Professional Engineers, to investigate the structural engineering aspects of the Adapazari Earthquake on July 22, 1967. The observations and conclusions arrived at by the authors are summarized here with special emphasis on the causes of the damage, the faults in the design and the inadequacies of the requirement of the Building Code.

A study of three building frames, which totally collapsed, is also included. The lateral load requirements of these buildings according to the Turkish, Chilean, U.S.A., and the Canadian Codes are compared with the results of the dynamic analysis.

1. INTRODUCTION.

At 5.02 p.m. on July 22, 1967 a severe ground motion with an intensity of approximately VI on the modified Mercalli scale, shook the Northwestern Province of Sakarya, Turkey, claiming one hundred lives and destroying numerous houses and buildings. Major structural damage occurred mainly in the capital city of Adapazari. (Fig.1). A team of three investigators, consisting of the authors and Dipl. Eng. Erol Unal, was immediately appointed by the Turkish Association of Professional Engineers to investigate the structural engineering aspects of the earthquake.

It was observed that the damage was very selective. Apart from the collapse of an old high bearing wall in a railroad car factory and of the illegally built penthouse columns of a liquor storage building, only seven appartment buildings in the whole city suffered a total collapse, while several hundred of other reinforced concrete public and private buildings escaped only with a few plaster cracks or even with no damage of any kind. The possible reasons of such a pocket damage are explained in the following paragraphs.

2. LESSONS OF THE DAMAGE.

The following section presents an itemized account of the structures of the building frames closely related to the suffered damage and the significant aspects of the code requirements pertaining to seismic design:

(1) As may be seen from the list of the damaged buildings in Fig.2, six of the seven totally collapsed appartment buildings were designed by the same engineer

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in the same manner. For architectural or other reasons, in order to achieve a flat ceiling free from projecting beams, the floor slabs were built as one-way joists with hollow clay filler tiles. The joists spanned between shallow, but very wide beams, supported by slender narrow columns. It was common experience to see a beam some 80 or 100 cm. wide resting on a column of a widt of only 23 cm. The beam column dimensions were not well proportioned to allow for an adequate connection.

(2) These poor, inadeuqately proportioned assemblies of narrow columns and wide beams were the only structural elements to carry the lateral loads. Ther were no shear walls in any of the buildings. For reasons of space or architecture, the columns were not only offset by large amounts from their genera centerlines but their axes of the weak and strong moments of inertia were ofte changed from one direction to another. The effect of overloading of some columns due to this irregularity was not taken into account in design.

(3) Without exception the collapse in all buildings resulted from inadequate flexural strength of the columns. Whole slabs fell on top of each other like playing cards. The columns were torn from the beams along their planes of contact, while the wide shallow beams suffered little or no damage. Plain reinforcing bars generally did not break, and were still connecting the column to the beams in the collapsed structures. This was an obvious indication that the columns, and not the beams, were unable to carry the seismic moments and that the joints were very poorly designed and built. Dowel and splice lengths of column reinforcing bars and the hook sizes were inadequate.

(4) In general, the quality of the concrete seemed to be very poor; the material contained large size gravel and visible honeycombs. The laboratory tests however, indicated that the compressive strength of the five samples tak from Bldg. No.1, ranged from 165 to 275 kg/cm^2, i.e. above the specified strength of 160 kg/cm^2. It is also a fact that the strength of one or two yea old concrete made of ordinary cement is approximately double the 28 days' strength.

(5) The seismic shear of a column was assumed by the designer simply as 6% of the total dead load plus one half of the live load of that column. The floor shear, however, must have been distributed to the columns in proportion to the relative stiffnesses.

(6) The additional shears due to torsion resulting from the eccentricity betw the centre of rigidity and the centre of mass were completely ignored. Large torsional displacements were clearly observed in Bldg. Nos. 2 and 9.

(7) It was common in Adapazari to use red clay tiles as filler walls. These walls contribute to rigidity, but at the same time they increase significantly the dead weight. Furthermore, because of their brittle character, the ductili of the structure is substantially reduced.

(8) Finally, when determining the lateral seismic design loads in accordance with the Turkish Earthquake Design Regulations, no allowance is made for the difference in stiffnesses and natural periods of buildings.

(9) Adapazari is situated on a very deep, soft alluvial soil with a high water table. A mat foundation is often necessary even under a 4-storey building. (Fig.2). In order to increase the gap between the natural periods of the ground and of the building, relatively stiffer designs of frames would be advisable. The correctness of this conclusion is confirmed by the fact that while hundreds of reinforced concrete buildings with conventional rigid column-beam frames survived the earthquake with virtually no damage, the only six buildings in the whole city, with the clay tile filled joist slabs and slender columns collapsed completely.

3. COMPARATIVE SEISMIC STUDIES OF BUILDING NOS. 1,2 & 3.

In order to compare the lateral load requirements of the Turkish Code with the others[1,2] and also with the results of the dynamic analysis, the structural frames of the Buildings Nos. 1, 2 and 3 have been chosen (Figs.3,4 and 5). Furthermore, the ultimate capacities of some of the most highly stressed columns of these frames were calculated in accordance with the formulae of 1963 Code of the Am. Conc. Inst., and the columns were inadequate to carry the induced seismic loads. The results are summarised in Tables 1, 2 and 3. The maximum probable shears obtained by taking the root mean square of the first ten modal shears are divided by a factor of $(4/0.44 \times 1.5) = 6.1$ in order to reduce the design shears to the level of the code forces. This reduction is based on a ductility factor $\mu = 4$, an allowable yield stress ratio of $\sigma_{all}/\sigma_{yield} = 0.44$, and an overstress factor of 1.5, as permitted by the Turkish Code.

The results lead to the following conclusions:

(i) Consistently, the Chilean Code gives the highest and the U.S.A. the lowest lateral load requirements. The factor of 0.67 in the U.S.A. Code specified for the space frames capable of carrying 100% of the lateral loads seems to be relatively very low.

(ii) It is noted as a possible weakness of the U.S.A. Code, that unlike the other codes, it contains no provision for an increase in the lateral design loads on account of poor soil conditions.

(iii) The distribution of shears along the height of the building in the Turkish Code is made proportional only to the relative weights of the respective floor levels. The Chilean, U.S.A. and the Canadian Codes require, however, greater lateral loads at higher elevations as indicated by the dynamic analysis.

(iv) In the case of dynamic analysis of buildings, the shear rod idealization assuming infinite bending stiffness of the floor elements results in higher and therefore, safer lateral loads than the more realistic idealization of the building into a plane frame, as shown in the last two columns of Table 1, 2 and 3.

(v) Building No.2 was under construction with only the first four storeys completed at the time of the earthquake. The column rigidities were substantially higher than those which would normally be required at this stage of the dead load. The high column rigidity attracted excessive lateral loads. In fact, this is the only reason that the dynamic analysis produced a base shear of 54.60 tons, as against the highest code base shear of 32.12 ton. This difference clearly demonstrates that the seismic lateral loads attracted by a building are greatly influenced by the stiffnesses of the columns.

(vi) As may be seen in Fig.4a, the first two rows of columns, in the front, were rotated 90° and thus resisted the earthquake by bending about their weak axes. Furthermore, a torsional eccentricity of about 2.00 m. induced additional 5 to 6% lateral loads on these columns. This is why only the front two rows of columns collapsed completely, while the rest of the columns survived without damage.

(vii) The ultimate load capacities of various column elements were found to be less than the loads acting on them during the earthquake.

ACKNOWLEDGEMENTS

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(2) Anonymous, "Antiseismic Design of Buildings", New Chilean Provisional Code INDITECNOR 63-9d, 1967.

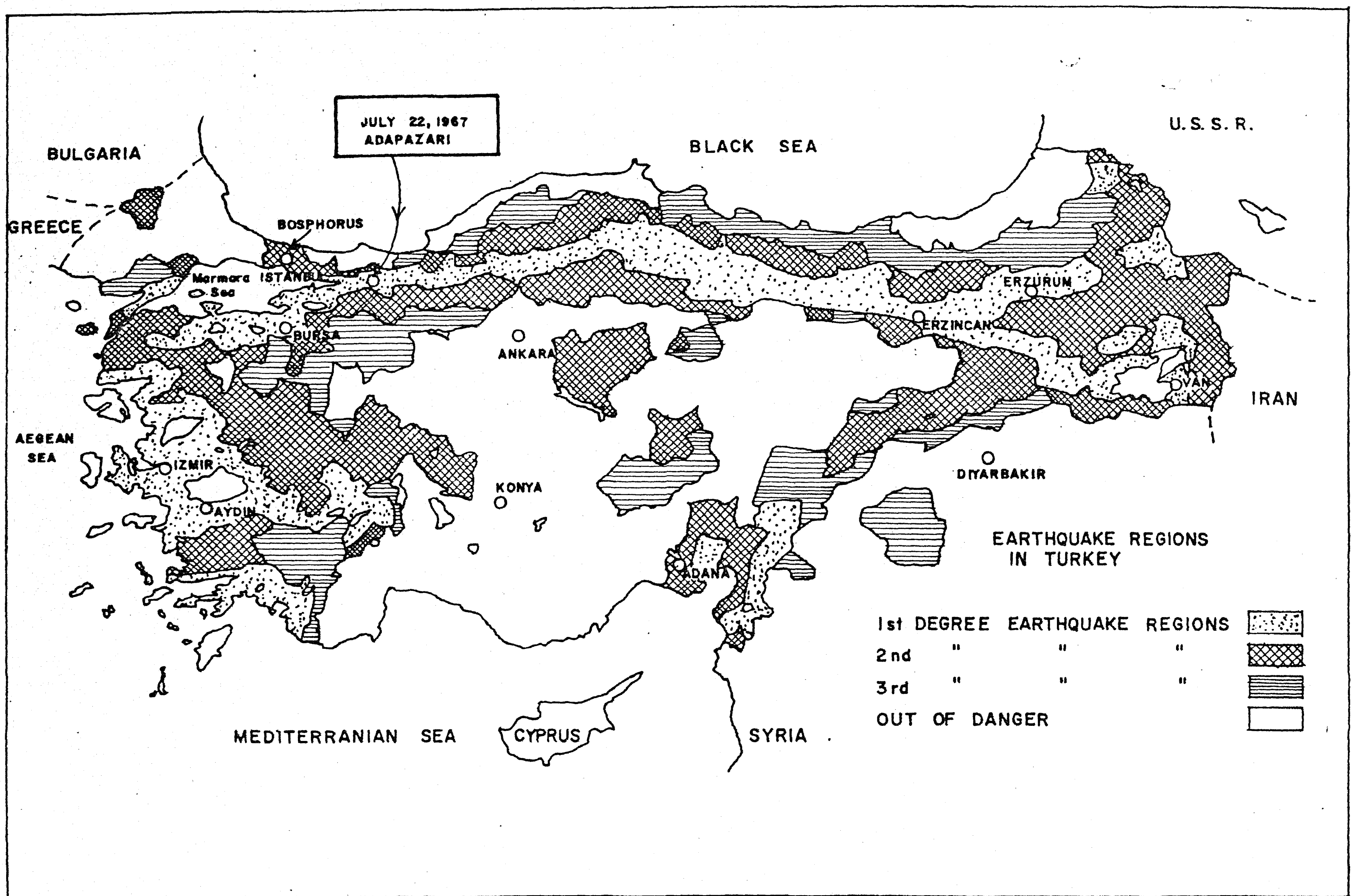
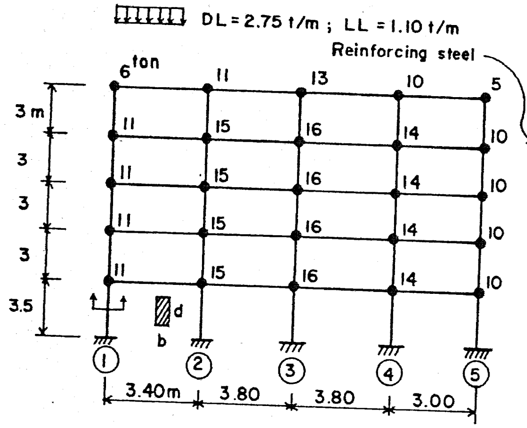


FIG. I — EARTHQUAKE REGIONS IN TURKEY

Name of Street	Plan Dimensions in Meters.*	Type of Damage
1. Sezai Bayraktar Apt. Kavaklar Street	14 by 22.5; Five storeys; Tile filled joists; Mat foundation.	Total Collapse; Moment failure.
2. Orhan Baldogan Apt. İzmit Street	8.35 by 16; Five storeys; Tile filled joists; Fourth storey was on forms; Mat foundation.	Front portion of Top three storeys collapsed; Torsional distortion visible.
3. Ahmet Akkoç Apt. Sakarya Street	20.92 by 22; Five storeys plus penthouse; Tile filled joists; Mat foundation.	Total collapse; Moment failure.
4. Hasan Aydogdu Apt. Subasi Street	Four storeys; Solid slabs; Continuous footings; Red tile filler walls	Total collapse; Moment failure.
5. Bedri Bozkurt Apt. Savaşlar Street	Three storeys; Tile filled joists; Mat foundation.	Total collapse; Moment failure.
6. a) Servet Sezen, and b) Orhan Akkoc Apt. Tar Street	<u>Twin Buildings</u> 9 x 16; Three storeys; Tile filled joists; Continuous footings.	a) Total collapse; b) Top storey collapse Both Moment failure.
7. Ali Türkkan Apt. Sakarya Street	18.60 by 16.85; Three storeys; Tile filled joists; Continuous footings.	Total collapse; Moment failure.
8. City Hall, Sakarya Street.	Six storeys; Beam column rigid frame; Mat foundation	No structural damage; cracks in elevator walls and in one ground floor column.
9. Ismail Çakır (Liquor Storage) Yeni Çarsı St.	Three storeys; Penthouse added later.	Penthouse collapse, only; Torsion failure.

* All are reinforced concrete buildings with material specification B160-StI. Concrete 28 days strength = 160 kg/cm^2, Steel yield stress = 3700 kg/cm^2. All but 2 and 8 are designed by the same engineer.

FIG. 2 - DAMAGED BUILDINGS, ADAPAZARI EARTHQUAKE, TURKEY, JULY 22, 1967.



COLUMNS b/d	
Side cm	Middle cm
20/30 6φ14	20/40 6φ14
20/40 8φ14	20/50 10φ14
20/50 8φ16	25/50 10φ16
20/50 8φ18	25/50 10φ18
30/50 8φ18	30/50 10φ18

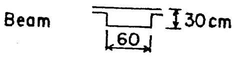


FIG. 3. — SEZAI BAYRAKTAR APT., DD—FRAME (BLDG. NO. 1)

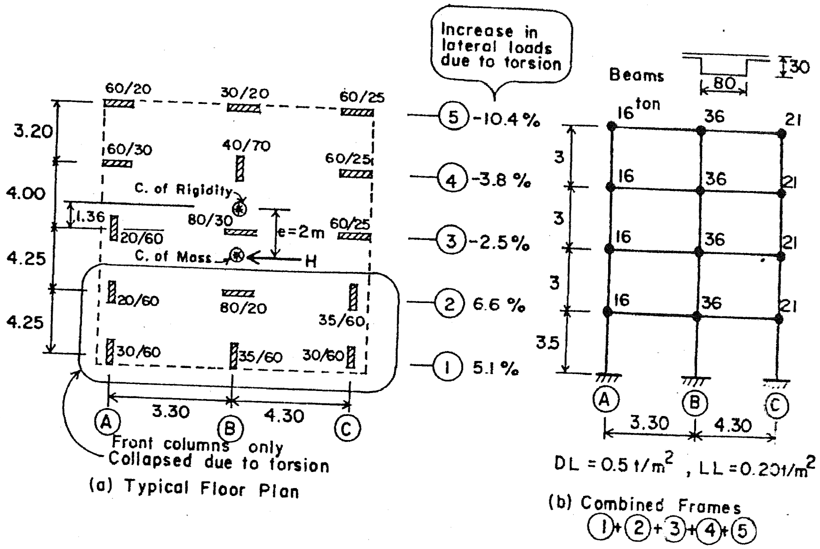


FIG. 4. — ORHAN BAL APT. (BLDG. NO. 2)

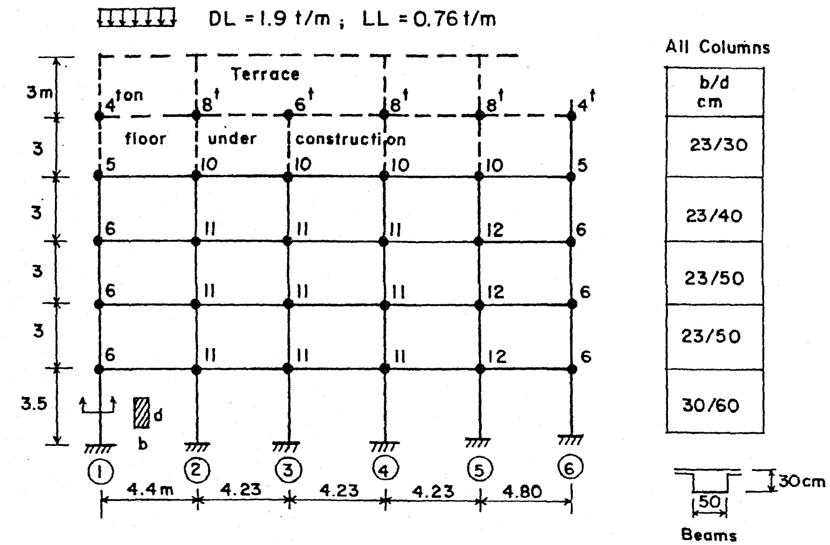


FIG. 5.- AHMET AKKOC APT., EE-FRAME (BLDG. NO. 3)

FLOOR LEVEL	DEAD WEIGHT (ton)	BUILDING CODES: TURKISH *	CHILEAN *	USA (UBC)	CANADIAN (NBC)	0-3% DAMPING MODAL ANALYSIS 1940 El Centro: AS FRAME	AS SHEAR ROD
5^{th}	45	3.24	11.11	3.23	6.17	8.04	8.43
4^{th}	66	4.75	7.15	3.86	7.32	5.79	7.13
3^{rd}	66	4.75	5.06	2.92	5.56	3.42	4.44
2^{nd}	66	4.75	4.62	2.00	3.81	3.42	4.09
1^{st}	66	4.75	4.62	1.09	2.05	2.53	2.51
SUM (ton)	309^{t}	18.24^{t}	32.56^{t}	13.10^{t}	24.91^{t}	23.20^{t}	26.60^{t}
PERIOD (sec)		—	—	0.50	—	1.26	1.09

LATERAL SEISMIC LOADS (tons)

TABLE 1.- SEZAI BAYRAKTAR APT. (BLDG. NO. 1)

* Live Load is assumed to be 40% of Dead Load

FLOOR LEVEL	DEAD WEIGHT (ton)	BUILDING CODES: TURKISH *	CHILEAN *	USA (UBC)	CANADIAN (NBC)	0-3% Damping MODAL ANALYSIS 1940 El Centro: AS FRAME	AS SHEAR ROD
4^{th}	73	5.26	15.80	5.20	9.86	19.25	19.57
3^{rd}	73	5.26	6.40	3.94	7.49	17.65	17.11
2^{nd}	73	5.26	5.10	2.70	5.12	12.30	13.48
1^{st}	73	5.26	4.82	1.46	2.77	5.40	9.07
SUM (ton)	292^{t}	21.04^{t}	32.12^{t}	13.30^{t}	25.24^{t}	54.60^{t}	59.23^{t}
PERIOD (sec)		—	—	0.40	—	0.40	0.23

LATERAL SEISMIC LOADS (ton)

TABLE 2. — ORHAN BAL APT. (BLDG. NO. 2)

FLOOR LEVEL	DEAD WEIGHT (ton)	BUILDING CODES: TURKISH *	CHILEAN *	USA (UBC)	CANADIAN (NBC)	0-3% Damping MODAL ANALYSIS 1940 El Centro: AS FRAME	AS SHEAR ROD
5^{th}	38	2.73	9.68	2.78	5.27	6.71	8.04
4^{th}	50	3.59	5.50	2.96	5.61	5.11	6.96
3^{rd}	57	4.11	4.62	2.57	4.86	4.08	5.88
2^{nd}	57	4.11	3.96	1.75	3.32	3.35	4.45
1^{st}	57	4.11	3.96	0.94	1.79	1.95	2.22
SUM (ton)	259^{t}	18.65^{t}	27.72^{t}	11.00^{t}	20.85^{t}	21.20^{t}	27.55^{t}
PERIOD (sec)		—	—	0.5	—	1.15	0.86

LATERAL SEISMIC LOADS (ton)

TABLE 3. - AHMET AKKOC APT. (BLDG. NO. 3)