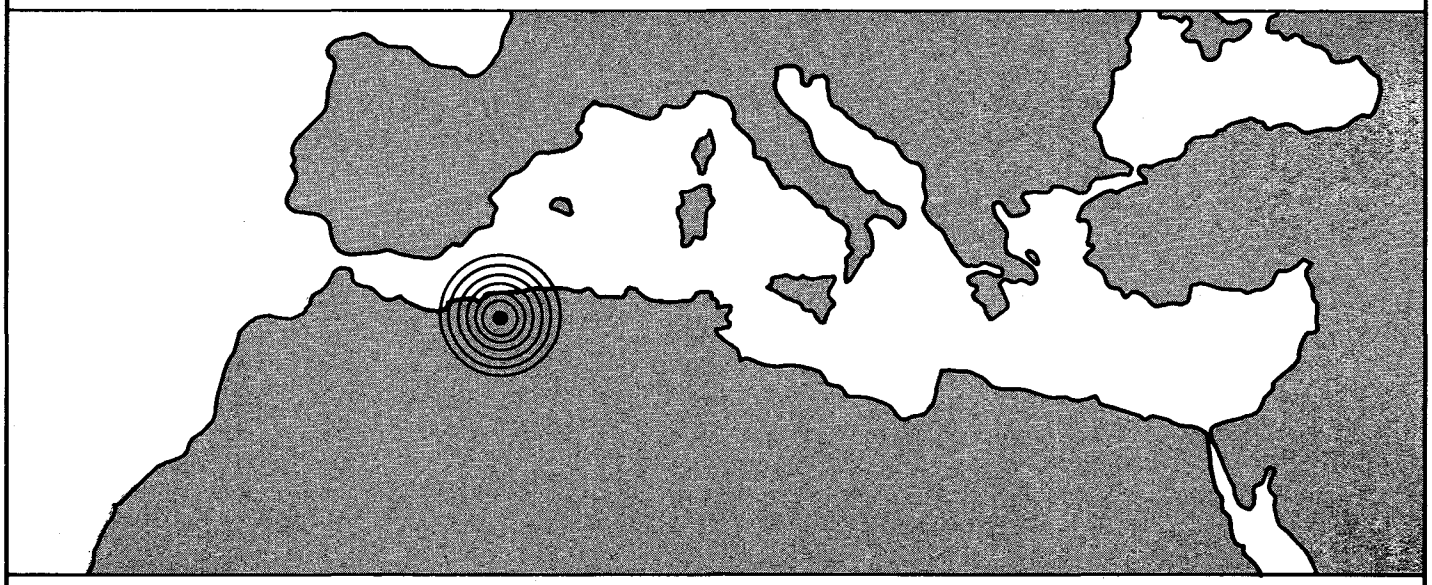
EL-ASNAM, ALGERIA EARTHQUAKE OF OCTOBER 10, 1980

A Reconnaissance and Engineering Report



National Research Council
Committee on Natural Disasters

Earthquake Engineering Research Institute

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EL-ASNAM, ALGERIA EARTHQUAKE
OCTOBER 10, 1980

A Reconnaissance and Engineering Report

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Commission on Engineering and Technical Systems
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The Earthquake Engineering Research Institute was founded in 1949 as an outgrowth of the Advisory Committee on Engineering Seismology of the U.S. Coast and Geodetic Survey. It is a national multidisciplinary society of more than 1000 engineers, architects, geoscientists, planners, and social scientists whose objective is to advance the science and practice of earthquake engineering and the solution of national earthquake engineering problems. The Institute not only investigates and reports on the effects of destructive earthquakes but sponsors conferences, holds seminars, and publishes monographs on earthquake engineering and earthquake hazard reduction.

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CONTENTS

I. INTRODUCTION

1.1 Introductory Remarks

On October 10, 1980 at 13:25:23.7 local time (12.25:23.7 GMT) a destructive earthquake occurred near El-Asnam, Algeria (formerly known as Orléansville). El-Asnam is approximately 170 km (106 mi) west of Algiers (Fig. 1.1). The Richter magnitude, M, of this event was 7.2, which corresponds to a surface wave magnitude, M_S, of 7.3. While initial reports placed the epicenter near the village of Beni Rached, the final location was agreed upon as being at 36.143°N and 1.413°E, 10 km (6 mi) east of El-Asnam. The focal depth of the earthquake was about 10 km (6 mi), and the approximate duration was between 35 and 40 sec. Field estimates place the value of peak ground acceleration at more than 0.40 g. No strong motion records were obtained from the main shock. A major aftershock having a Richter magnitude of 6.0 occurred on the same day at 16:39:09.8 Algerian time. During the period from October 21 to December 7, 1980, numerous aftershocks were recorded having maximum amplitudes of acceleration that ranged between 0.01 and 0.31 g.

Initial news reports were that the earthquake had devastated the city of El-Asnam, population estimated at 125,000, and the nearby towns and villages of Sendjas, Oued Fodda, El-Karimia, El-Abadia, Beni Rached, Zeboudja, and El-Attaf. The large loss of life (reportedly 5000 to 20,000 casualties) and property was attributed to the collapse of buildings.

CTC (L'Organisme Contrôle Technique Construction d'Algérie) of Algiers accepted Haresh Shah's offer to assist with post-earthquake engineering investigations. Shah contacted the Earthquake Engineering Research Institute (EERI), and through the efforts of John Blume, Roger Scholl, and Henry Degenkolb, mobilized the reconnaissance team. This five-man team, led by Shah, included Nicholas Forell, Christian Mortgat, Henry Taylor, and Thomas Wosser; it reached Algiers October 15, five days after the earthquake, and spent nearly a week in the stricken area. Subsequently, the Committee on Natural Disasters (CND) of the National Research Council, with the cooperation of EERI, organized a second team to visit Algeria. Referred to as the investigating team, the second group had five members: Vitelmo Bertero (team leader), Peter Gergely, Max Irvine, Thomas Saarinen, and Marcy Wang.

The investigating team was briefed by Shah before departing for Algeria October 22, 1980 to determine what could be learned about ground motion and surface faulting; effects of the earthquake on the performance of structures, utilities, and transportation systems; and how building codes and design standards could be improved to minimize future damage. The investigating team spent a week in Algeria inspecting the effects of this earthquake.

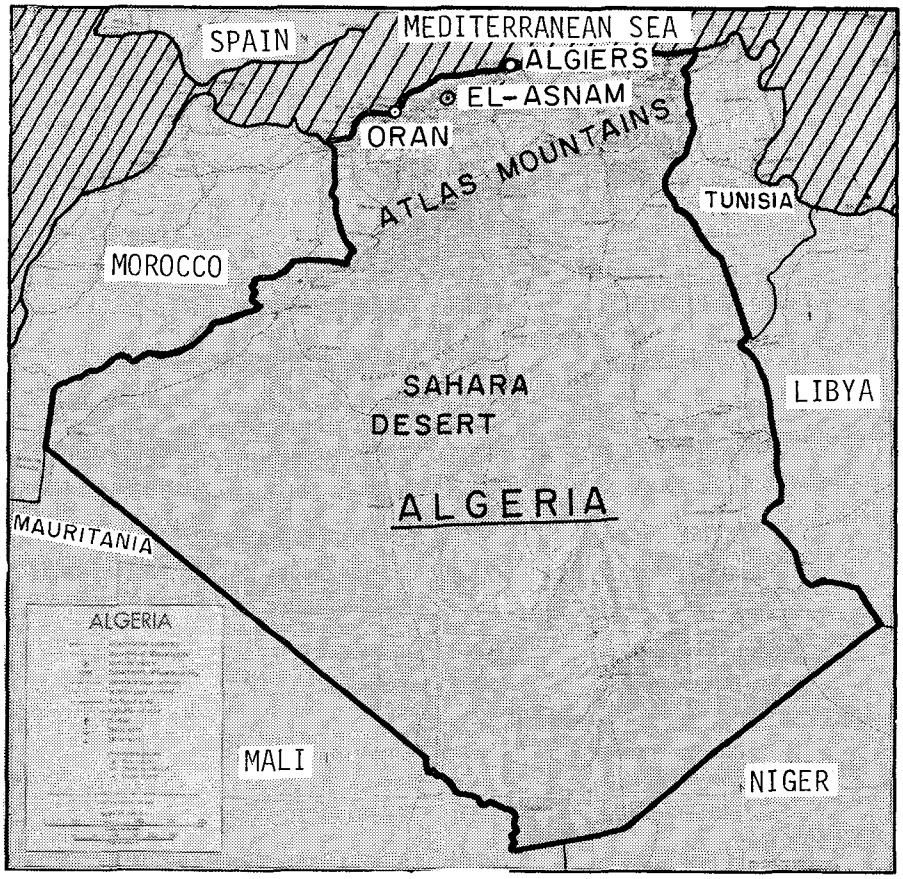

Fig. 1.1 Map of Algeria

A preliminary report by Shah and Bertero, "El-Asnam, Algeria Earthquake of October 10, 1980," combining information gathered by both teams, was published in the January 1981 *EERI Newsletter* (Shah and Bertero, 1981). The present report updates that preliminary report with detailed studies and analysis of the collected data, as well as data obtained subsequently, particularly: 1) geological information supplied by Lloyd Cluff and Frank Swan, who inspected the earthquake area from November 1-10, 1980, and 2) liquefaction and landsliding observations by Wayne Clough (Clough et al, 1981).

This report is organized into six chapters. Chapter I briefly reviews the geography and history of Algeria, before discussing the geologic and seismologic settings of the country. General features of the earthquake are given, and Algerian seismic-resistant design practice and building construction in El-Asnam are also reviewed.

Chapter II discusses the seismologic and geologic characteristics of the earthquake. Surface faulting, aftershock, ground motion record, and site soil condition information is presented; information on the 1954 Orléansville earthquake is presented for comparison.

Earthquake effects are discussed in Chapter III, beginning with a general description of damage. The causes of damage are discussed under four main headings: surface faulting, tectonic subsidence and tilting, ground failure, and strong ground motion (shaking). The program of action by CTC is also reviewed in this chapter.

Chapter IV evaluates building damage and by studying particular buildings (case studies) analyzes the implications of this damage. Detailed discussions of the performance of two buildings are presented, as are probable causes for failure of some major structures, as hypothesized by CTC.

Socio-economic aspects of the earthquake, as well as rescue and relief operations, are briefly reviewed in Chapter V.

The final chapter (VI) describes the lessons learned (and relearned) from investigation of this earthquake and presents conclusions based on analysis of the earthquake effects and the geologic and tectonic viewpoints. The performance of structures, utilities, and transportation systems is analyzed with emphasis on building performance in regard to improvements needed in design, construction, and maintenance of engineered and nonengineered buildings. The problems of planning the reconstruction of El-Asnam are discussed. Finally, recommendations are formulated to mitigate destruction from future earthquakes.

1.2 Geography and History

El-Asnam is 170 km (106 mi) west of the Algerian capital of Algiers, in the Mediterranean zone of North Africa (Fig. 1.1). It lies in a zone of narrow valleys separated by two parallel ranges of the Atlas Mountains. Ninety-five percent of the Algerian people live along this zone on 12 percent of the country's land area. The climate is Mediterranean, and the agricultural products are those typically produced in such a climate: vines, citrus fruits, olives, and grains. The southern (drier) Sahara Desert area of Algeria is sparsely populated but important as a source of oil and natural gas.

Historically, Algeria has been vulnerable to foreign powers because of its strategic Mediterranean coast location. For a millenium, the indigenous Berber tribes were dominated and influenced by outside cultures — Phoenician, Roman, Vandal, Arab, Byzantine, Turkish, and French. Today's Algerians reflect these diverse cultures, particularly the influence of the Arabs and of the French. French and Arab influences are apparent in food, clothing, and architectural styles; and both Arabic and French are spoken. The current government stresses the teaching of Arabic, but French and Berber remain well established languages.

The former French colonial regime established a road and railroad system and a thriving agricultural economy. This farming legacy is conspicuous in El-Asnam, which constitutes one of the most important granaries in Algeria and is an agricultural *wilaya.* Algeria is divided into 31 *wilayas,* regional administrative entities similar to provinces. The *wilaya* of El-Asnam extends 8676 km^2 (3350 sq mi), has 885,200 habitants (last census January 1978), and is divided into six *dairas*. The El-Asnam *daira* is composed of three districts: El-Asnam, Sendjas, and Ouled Fares, with a population of 155,800.

After an eight-year conflict, Algeria achieved independence from France in 1962. The principal domestic objectives of the Algerian government are to achieve economic development through industrialization and to raise the standard of living. Algeria is fortunate to have substantial petroleum resources to aid in this process. The country faces a myriad of problems in attempting to modernize a traditional society and to raise the living standard in the face of rapid population growth. High rates of unemployment and underemployment; a lack of well-trained higher and middle-level cadres; a lagging agricultural sector; and difficulties in providing health, education, and other services are still major problems.

One area of substantial government investment is housing; new apartment complexes may be seen throughout the nation. El-Asnam has had its rapid growth in the years since independence. In 1972 President Houari Boumedienne instituted a special development program of 184 million dinars (approximately $US 46 million) for El-Asnam, which included 1000 urban dwellings, 2000 suburban dwellings, and 5000 rural lodgings. The performance of these buildings in the October 10, 1980 earthquake is discussed later.

1.3 Geologic and Tectonic Setting

Algeria is situated within the northern half of the African continent, and is bordered to the west, south, and east by other African nations, and bounded on the north by the Mediterranean Sea. The northernmost portion of Algeria has historically experienced a moderate amount of shallow (less than 70 km (40 mi) deep) seismic activity. In light of modern plate tectonics, this activity is thought to be associated with plate motions and interactions at plate boundaries.

Nearly all of the African continent lies on the African plate. To the north, the Eurasian plate is thought to be colliding with, and being thrust over, the African plate, with some plate consumption taking place (see Fig. 1.2). The types of features normally associated with a subducting plate boundary are not observed because of the behavior of the continental lithosphere with respect to plate subduction. Rather than the formation of an arc-trench complex, a wide belt of folded mountains is produced because the continental material is too light to sink into the earth's mantle. This collision belt makes up the Atlas Mountains of North Africa (Morocco, Algeria, and Tunisia), a broad zone of crustal shortening up to 400 km (250 mi) wide which has been extensively folded and thrust faulted. The geologic structures within the Atlas Mountains trend generally east-west to east-northeast, parallel to the plate boundary and normal to the direction of plate convergence (see Figs. 1.2 and 1.3).

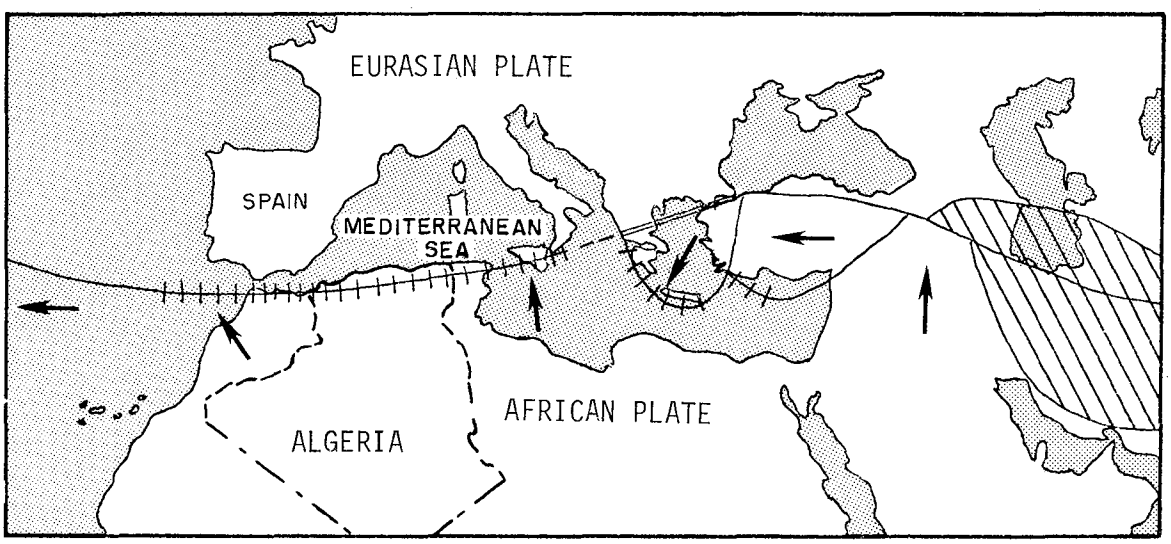

Fig. 1.2 Approximate Positions of Active Plate Boundaries in Mediterranean Region. Arrows indicate direction of motion relative to Eurasian plate. Boundaries creating lithosphere (spreading centers) are shown with double lines, boundaries consuming plates are shown with short lines normal to the line. (McKenzie, 1970)

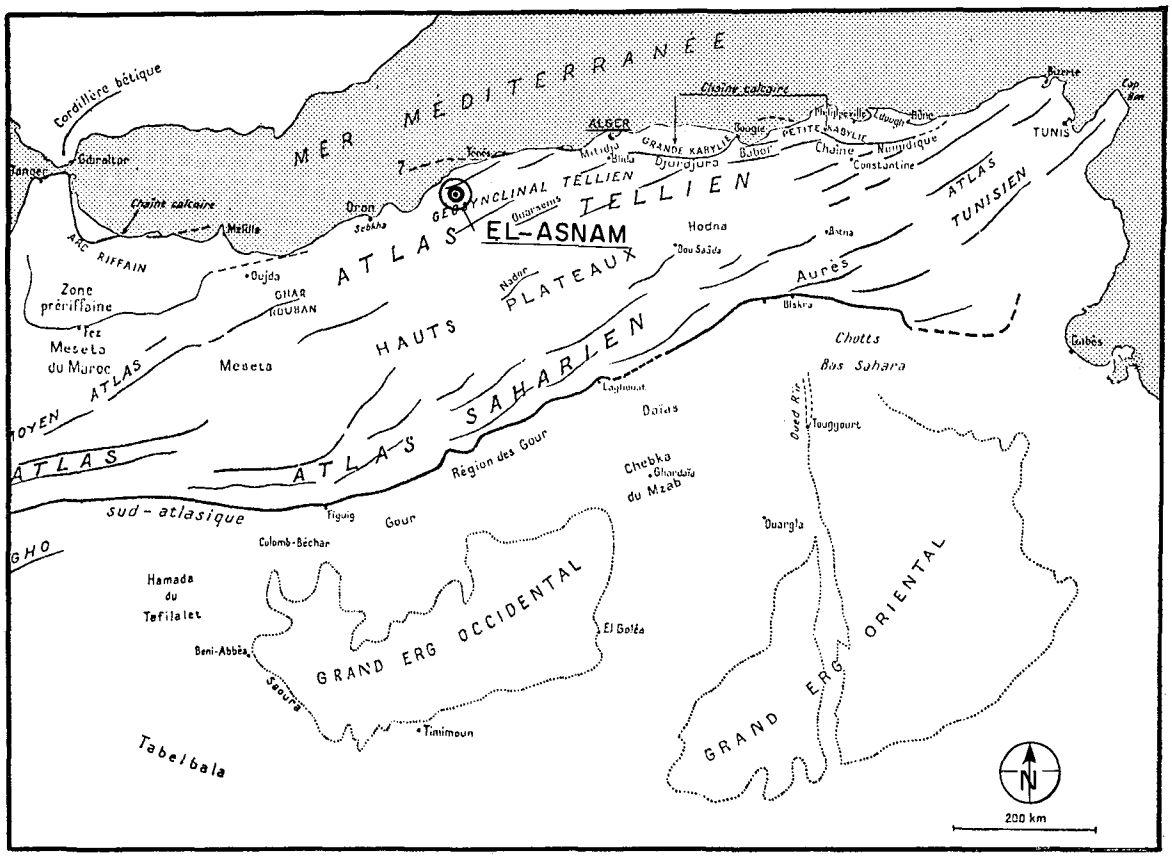

Fig. 1.3 Atlas Mountains and Northern Sahara, Showing General Structural Grain (Deleau, 1952)

The plate boundary west of Spain changes from one of overthrusting to one characterized by right lateral strike-slip faulting (predominantly horizontal motion with the opposite side moving to the right) as determined from earthquake focal mechanisms (McKenzie, 1970 and 1972).

Although there is much uncertainty and controversy regarding the exact configuration and character of this complex plate boundary, it is generally believed that the boundary begins as a transform fault at the Mid-Atlantic Ridge, extends eastward as a subduction zone through the Mediterranean Sea, then connects to the Arabian plate boundary. The slow descent of the African plate under the Eurasian plate [about 3 cm (1 in.) per year as compared to the fast subduction rate of 10 cm (4 in.) per year of the Cocos plate under the North American plate] constitutes one of the major subduction zones of the earth. Because of the slow rate of the African slab's subduction, its lithosphere is absorbed into the mantle before it can reach a considerable depth; thus, the associated earthquakes are not generally deep. It appears that the rate of subduction is impeded by the rigidity and buoyancy of both the African and the European plates.

The wide and diffuse band of seismic activity that extends into both continents and the Atlas and Alpine mountain belts are clearly the result of compressional forces that originate from the difficulty of consumption of continental lithosphere in a continental collision.

Contrary to what is stated above, Ritsema (1969) claims that in North Africa most earthquakes are transcurrent east-west, oriented with right lateral motion.

1.4 Seismologic Setting

There is a band of shallow seismic activity along the North African part of the plate margin. Only a few intermediate to deep focus earthquakes have been recorded along this zone, and no well-defined Benioff Zone has been recognized.

In his study of European seismicity, Kárník (1971) describes the Algerian region as follows:

> The seismic activity is concentrated in the coastal areas and the epicentres are associated with the structural features of the Atlas mountains extending from Agadir (Morocco) to the Gulf of Gabes (Tunisia) . . . There are about three zones of destructive shocks in Algeria. The first can be delineated by the towns Oran-Mascara-Relizane, the second extends from the Massif de Dahra to the Mts. of Hadna and Aures, and the third corresponds to the line Kerrata-Constantine-Guelma. All three zones were shaken by destructive earthquakes, the heaviest ones being on June 24, 1910, M = 6.6, I_O = X (Bibans), and on September 9, 1954, M = 6.5, I_O = X (Orléansville).*

El-Asnam and the surrounding region has experienced moderate to large earthquakes at least a dozen times (including major aftershocks) during the past 250 years. Table 1.1 (after Khemici, 1980) is a list of earthquakes in Algeria between 1716 and 1980. Fig. 1.4 is an epicentral map of Algeria (Mortgat and Shah, 1978). Figs. 1.5 and 1.6 give some idea of the seismic hazard in Algeria (Mortgat and Shah, 1978). Fig. 1-7 shows the major past events in the vicinity of El-Asnam. Of particular interest are the 1934 El-Abadia and the 1954 El-Asnam (Orleansville) events. These two earthquakes were caused by activity of the same fault system as the October 1980 event. The

*I_O = maximum observed intensity (Mercalli-Sieberg Scale).

TABLE 1.1 EARTHQUAKES HAVING INTENSITY > VIII OR MAGNITUDE > 4.9 IN ALGERIA (after Khemici, 1980)

Date	Location	Intensity**	Magnitude	Remarks
Feb. 3, 1716	Algiers	X		Numerous casualties
Oct. 9, 1790	Oran	X		3000 victims
Mar., 1819	Mascara	X		Numerous victims
Mar. 2, 1825	Blida	X		7000 dead
Feb. 9, 1850	Zamora			
	El Guenzet	VIII		
Nov. 22, 1851	Mascara	VII-VIII		
Aug. 22, 1856	Jijel	IX		
	Bejaia	VIII		
Jan. 2, 1867	Mouzaia	X-XI		About 100 dead
Nov. 16, 1968	Biskra	IX		
Jan. 19, 1885	N'Gaous	VIII		
Jan. 8, 1887	Mansoura	VIII		
Nov. 29, 1887	Kala	IX-X		20 dead
Jan. 6, 1888	Mouzaia	VIII		
Jan. 15, 1891	Gouraya	X		
Mar. 11, 1908	Blida	VII-VIII		
Aug. 4, 1908	Constantine	VIII	5.1	
Jun. 24, 1910	Masqueray	X	6.4	
Aug. 6, 1912	Oued Marsa	VI	5.3	
*Aug. 25, 1922	Bordj Abou Hassan	X		
Mar. 16, 1924	Batna	IX	5.6	Several dead
Nov. 5, 1924	Near Algiers	VIII	5.0	
Jun. 10, 1925	Near Boghar	VIII		
Aug. 24, 1928	Oued Rhiou	VIII	5.4	4 dead
Aug. 15, 1931	Djebel Dira	VIII	4.9	
*Sep. 7, 1934	El-Abadia	IX	5.0	
Sep. 19, 1935	Near Chetaibi		5.1	
Feb. 10, 1937	Near Guelma	VIII	5.4	
Apr. 16, 1943	Near Mansoura	IX	4.0	
Feb. 12, 1946	Hodna Mtns	VIII-IX	5.6	246 dead
Aug. 6, 1947	Oued-Hama Mine	VIII-IX	5.3	Many victims
Mar. 13, 1948	Asla	VIII	4.9	1 dead
Feb. 17, 1949	Near Kerrata	VIII	4.9	
Apr. 20, 1950	Near Aflou	VI-VII	5.1	
Jul. 5, 1953	Near Ain Bessam	VIII		
Aug. 29, 1953	Hodna Mtns	VIII-IX		1 dead
*Sep. 9, 1954	Orléansville (El-Asnam)	X	6.7	1243 dead, 20,000 homes destroyed

Table 1.1 (continued)

Date	Location	Intensity**	Magnitude	Remarks
*Sep. 10, 1954	Orléansville	IX	6.2	Aftershock
*Feb. 4, 1955	Orleánsville	VIII		Aftershock
May 8, 1955	Beni Haoua	VIII		
*Jun. 5, 1955	El-Asnam Beni Rached	VIII	5.7	
*Feb. 14, 1956	Bordj Bou Hassan	VI-VII	5.9	
Jun. 28, 1957	Sendjas	VII	5.0	
May 24, 1959	Zamora El Guenzet	VII-VIII	5.5	
Nov. 7, 1959	Bou Medfa	VIII	5.5	
Feb. 21, 1960	Melouza	VIII	5.6	47 dead, 88 injured
Dec. 2, 1961	Annaba (at sea)		5.5	
Sep. 4, 1963	Near Setif		5.7	1 dead, 100 injured
Jan. 1, 1965	M'Sila			5 dead, 24 injured, 1304 homes destroyed
Jul. 13, 1967	Near Sig	VII	5.1	10 dead, 15 injured
Feb. 28, 1968	El Alen	VIII	4.9	1 dead, 4 injured
Feb. 5, 1971	Ames		5.9	
*Feb. 23, 1971	Rouina	VIII	4.9	
Feb. 25, 1971	Asla		5.4	
*Mar. 11, 1973	Near Ténès (off coast)		5.7	
Nov. 24, 1973	B.B. Arrerridj	VII	5.1	4 dead
Nov. 25, 1973	Guenzet	VII	4.9	
Jul. 28, 1974	Setif	VII	5.0	
Nov. 9, 1974	South of Bejaia	VIII	4.1	
Jul. 11, 1975	Setif	VIII	5.0	1 dead, 18 injured
*Oct. 10, 1980	El-Asnam	X to XI	7.2	
*Oct. 10, 1980	El-Asnam		6.0	
*Nov. 8, 1980	El-Asnam		5.6	

*Moderate to large earthquake.
**Modified Mercalli Intensity Scale.

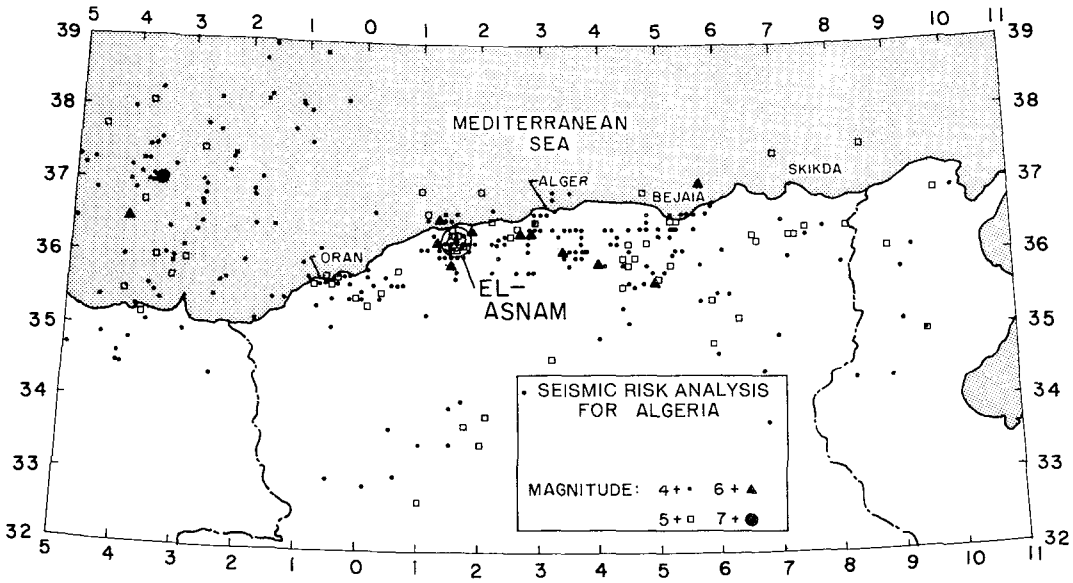

Fig. 1.4 Epicentral Map of Algeria (1819-1965) (Mortgat and Shah, 1978)

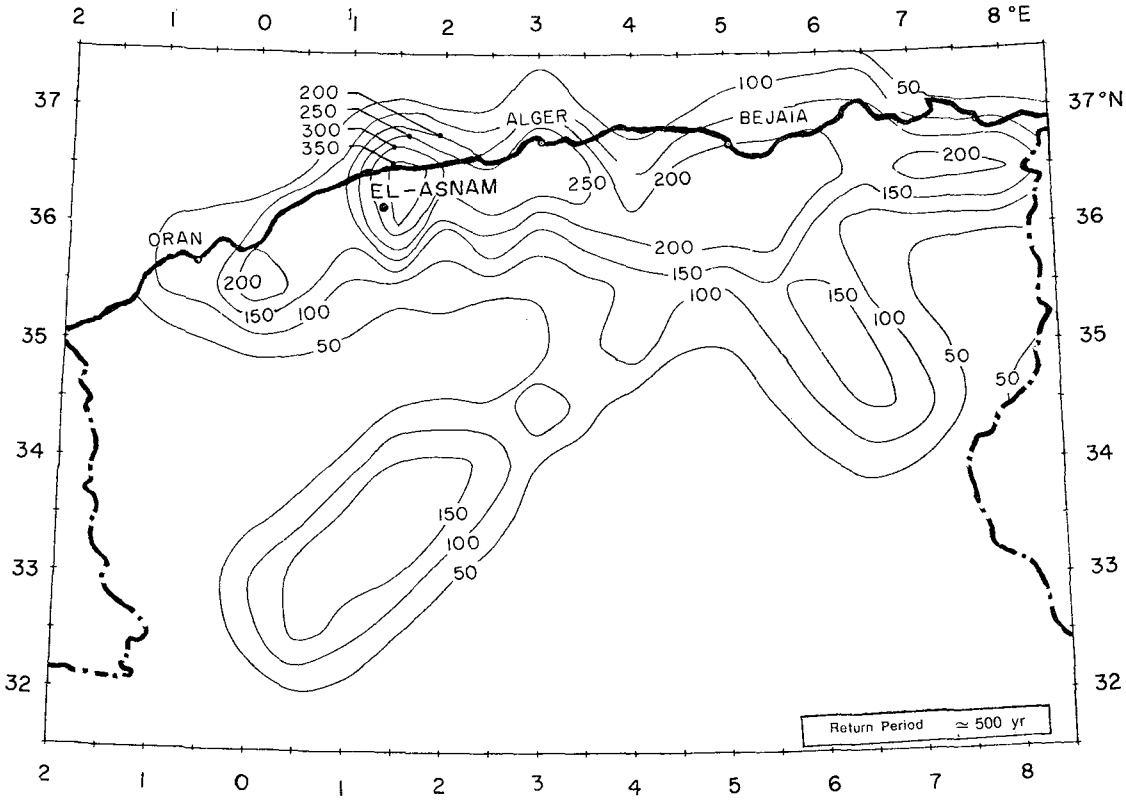

Fig. 1.5 Seismic Risk Analysis for Algeria, Contoured as Peak Ground Acceleration, cm/sec² (Mortgat and Shah, 1978)

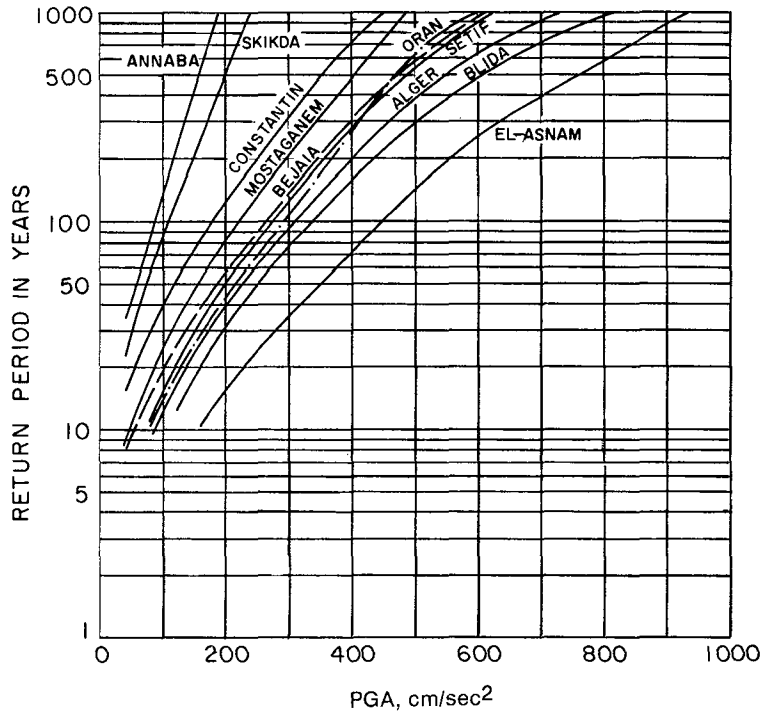

Fig. 1.6 Predicted Peak Ground Acceleration for Major Cities in Algeria (Mortgat and Shah, 1978)

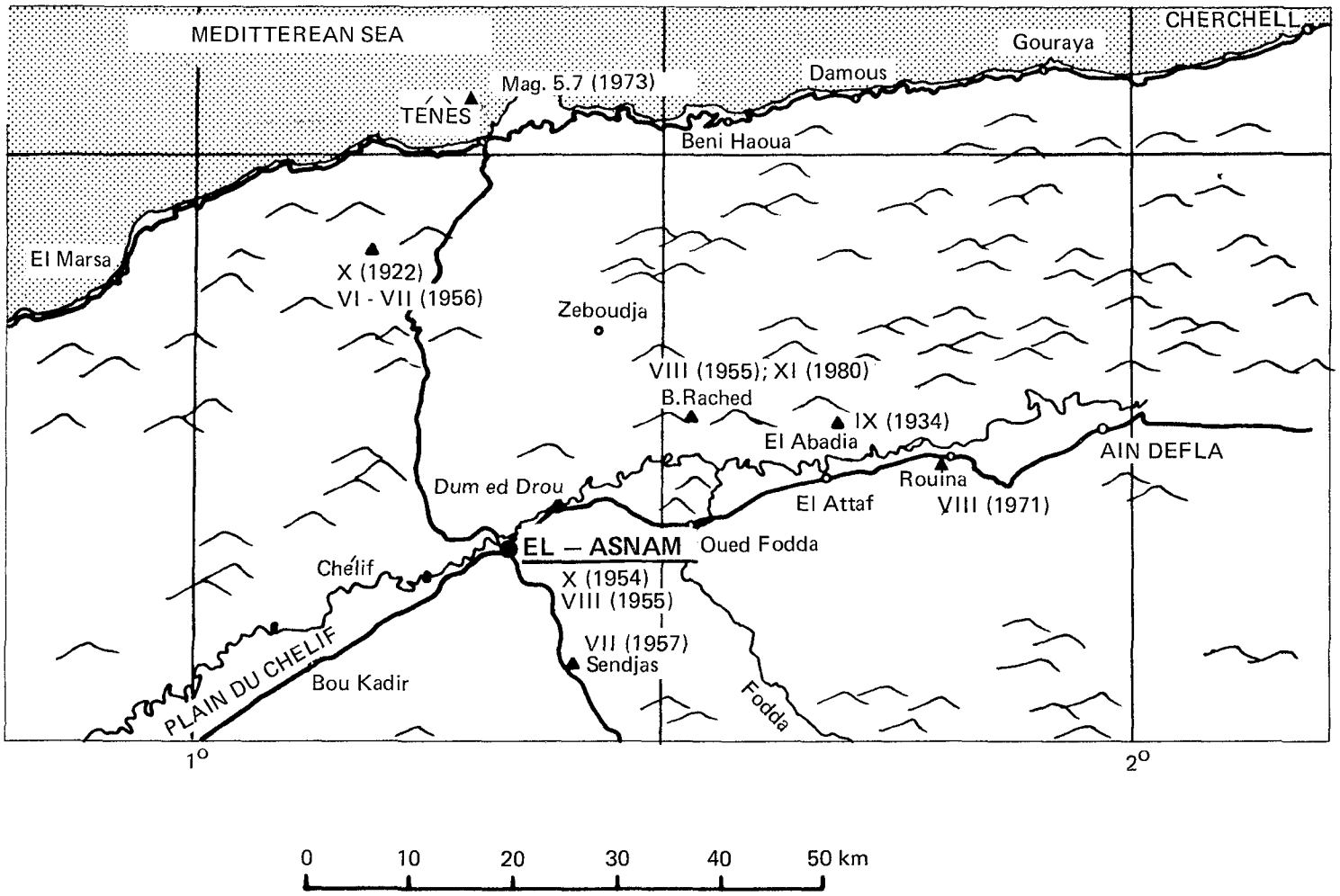

Fig. 1.7 Recent Past Seismic History Around El-Asnam

September 9, 1954 Orléansville earthquake had a Richter magnitude of 6.7, killed about 1500 people, and destroyed 20,000 homes. Most of the buildings destroyed October 10, 1980 were built after the 1954 earthquake.

Fig. 1.8 (Thévenin, 1955) shows the 1954 earthquake epicenter and the damage in the vicinity of Orléansville.

1.5 Seismic-Resistant Design Practice

Prior to 1954 there were no seismic-resistant design provisions in Algeria. For reinforced concrete (RC) and steel construction, the French codes for normal types of loading were generally used. These codes were used officially until Algerian independence in 1962 and used unofficially thereafter. For RC construction, French codes established by the Ministry of Housing and Reconstruction used were BA45 until 1960, then BA60 until 1968. Presently the BA68, modified in 1970, is used. The following steel codes were used: CM46 until 1966, CM66 after 1966 until the present. For wind design, specifications of the NV46 were used until 1965, followed by NV65, still in use.

In the aftermath of the devastating 1954 Orléansville earthquake, the necessity for a seismic-resistant building code was recognized. Consequently, within a month the French had developed provisional recommendations AS 1955; also, see International Association for Earthquake Engineering, "Earthquake Resistant Regulations," A World List, 1980.

AS 1955 (see Appendix A of this report) establishes two seismic zones, with an accompanying map of areas of low and high seismicity. This is followed by general recommendations on concept, foundations, and superstructure. Then the computational rules are given for earthquake forces, and allowable unit stresses. Tables give seismic coefficients for static computations for varying heights with different soil conditions. [A. Brenier, a member of the commission which prepared AS 1955, presented a paper (in English) on the regulations at the Second World Conference of Earthquake Engineering in Tokyo, Japan (Brenier, 1960).]

To take seismic forces into account AS 1955 recommends that the simultaneous or successive effects of a horizontal and of a vertical component be considered. The effect of the horizontal component, H, should be considered as a horizontal force applied to the center of gravity of the element or structure under study, and is given by

$$H = \sigma P \tag{1}$$

where σ is a seismic coefficient $= \sigma_1 \sigma_2 \sigma_3$

σ_1 = seismic zone coefficient for El-Asnam zone [for buildings up to 10 meters (33 ft) high (h) above ground, $\sigma_1 = 0.07$. For $h \geqslant 10$ meters (33 ft),

$$\sigma_1 = 0.07 + 0.02\,(h - 10 \text{ meters})]$$

σ_2 = soil coefficient $0.75 \leqslant \sigma_2 \leqslant 1.25$

σ_3 = foundation depth coefficient $1.00 \leqslant \sigma_3 \leqslant 1.25$

and P is the weight of the active mass which is the dead load, W, for apartment building.

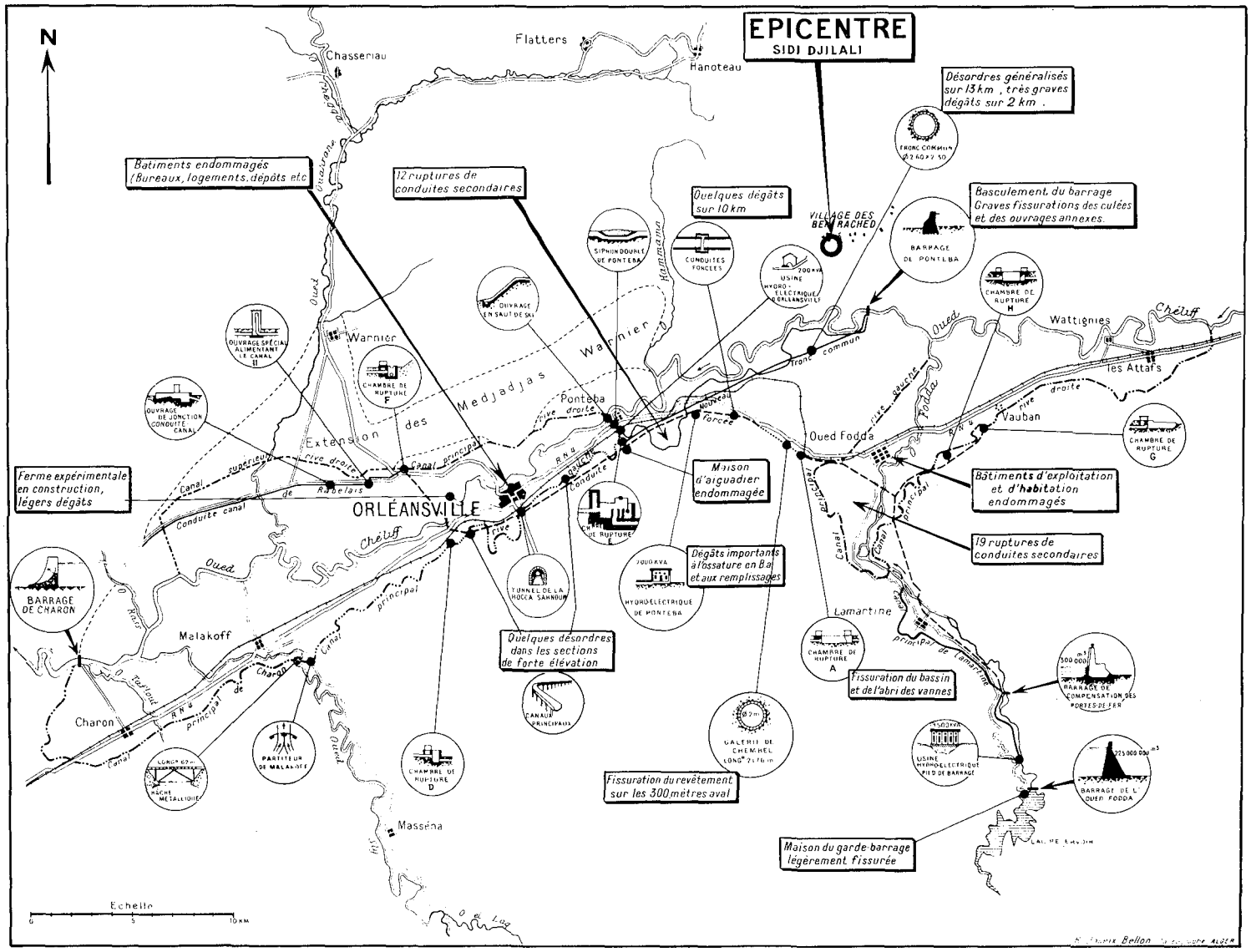

Fig. 1.8 Damage to Main Engineered Construction in September 9, 1954 Earthquake (Thévinin, 1955)

Thus,

$$\text{for } h \leqslant 10 \text{ m (33 ft)}, 0.053\,W \leqslant H \leqslant 0.094\,W \tag{2}$$

$$\text{The vertical component} = \pm\, 2\sigma P \tag{3}$$

The structure is required to respond to these seismic loads in combination with the effects of gravity loads (dead and live) at a rupture (ultimate) limit state of the critical sections of each structural member. However, if a method for computing such ultimate strength has not been codified, use of the classic elastic method, considering as limit stresses the elastic limit of the steel and 0.8 of the 90-day cube compressive strength of concrete, is allowed.

When the above regulations are compared with present U.S. seismic-resistant specifications established by the Structural Engineers Association of California (SEAOC) in 1980, it becomes clear that for short buildings [$h < 10$ meters (33 ft)] AS 1955 when applied to El-Asnam leads to smaller lateral load requirements than specified in the United States for a zone of similar seismic risk.

The AS 1955 recommendations in use in El-Asnam were elaborated and revised into a more substantial form in 1962 as "Règles Parasismiques 62," or PS62. These rules were applicable in France as well as in Algeria; however, Algeria was then independent and for nearly a decade the rules were not applied. PS62 was modified in France in 1969 and published as PS69, but these new provisions were not enforced in Algeria. Hence, it is not known to what extent they were used in practice.

In 1973 the Algerian Ministry of Public Works and of Construction issued seismic recommendations in pamphlet form based mainly on the French PS69. The 1973 recommendations contained an appendix which indicated the seismicity in different areas of the country. Again, there was no enforcement of the provisions.

CTC was established in Algiers in 1971, and its El-Asnam regional office opened in 1975. The main responsibilities of CTC are as follows:

> **To check compliance** with code of practice of all building construction where public funds are involved, including industrial construction, silos, and reservoirs. (Scope of activity does not include major civil engineering construction such as bridges, dams, etc. . .)
>
> **Plan review:** Critical review of plans and shop drawings. Check stability of structures for all loadings in code. Check detailing.
>
> **Inspection:** Perform number of site visits. Make sure construction is built according to plans. Take and test samples of materials.

A table of seismic coefficient, σ, for each *wilaya* to be used in conjunction with PS69 was prepared by CTC in 1976. The Minister of Public Works circulated a declaration to builders that it was compulsory to use earthquake-resistant design, but it was not explicitly stated that French rules PS69 should be used. There were vigorous objections by builders to incorporating seismic-resistant design rules in their design. Consequently, in April 1976 the declaration was modified;

the rules were no longer compulsory but buildings could be checked for seismic risk during a transition period. Compulsory use of the existing seismic-resistant design code (unofficially PS69) was decided upon in November 1979 by the Minister of Urbanism, Construction, and Housing.

Since 1976 CTC has been actively attempting to improve existing seismic-resistant code recommendations and to enforce applications of code, but without much success. The heavy demands placed on CTC for development preclude all buildings being checked or supervised during the construction phase. Currently, 170 engineers work at CTC and there are approximately 3500 projects needing review and inspection. Junior engineers are responsible for as many as 30 projects at a time; so, clearly, it is not possible to adequately analyze and supervise under these conditions. In addition, CTC does not have the power to halt construction of a project. CTC can recommend that insurance coverage not be provided for buildings that have not been properly designed, but this recommendation has rarely prevented the construction of a structure.

In 1976 CTC reached an agreement with Stanford University requesting, first, development of a seismic zoning map for Algeria (Mortgat and Shah, 1978), and second, formulation of a seismic-resistant code for buildings (Zsutty and Shah, 1979). Most of this work was completed in August 1979. The Stanford recommendations were put into practical code format by the Directoire de la Règlementation et de l'Information Technique du CTC. The official code has been adopted and was published in July 1981. It is called "Règles Parasismiques Algériennes 1981 - RPA 1981," and is published by the Ministère de l'Habitat et de l'Urbanisme.

As in many seismically active countries, Algerian engineers receive only limited education in their universities in seismic-resistant construction. A few engineers have acquired sufficient knowledge in this field, but they are too few for the present building demands. The consequences of the inability to enforce code standards and to adequately educate engineers in seismic-resistant design and construction are evident in the inadequate performance of engineered structures during this earthquake.

Regarding the adequacy of existing seismic-resistant design provisions, it should be remembered that the seismic provisions of PS69 were intended to provide complete protection from an earthquake of Intensity VIII on the Modified Mercalli Scale, and partial protection for an Intensity IX event. The intensity of the 1980 El-Asnam earthquake was between IX and X. Therefore, modifications of PS69 are due in view of the intensity of this earthquake, and the seismic-resistant design rules for building structures prepared by Zsutty and Shah in 1979 should be reviewed.

In April 1981, Zsutty (Zsutty, 1981) helped CTC formulate the National Seismic-Resistant Building Regulation which applies to the reconstruction activity in El-Asnam and to all other seismic regions of Algeria. Through the efforts of CTC, Algeria was able to achieve a workable seismic code by early 1982, which is a worthy complement to the great post-earthquake rescue and care effort by that government in El-Asnam. This achievement is a major step toward minimizing death and destruction in future earthquakes.

The new seismic-resistant design regulations are based on the estimation of a base shear as proposed by Zsutty and Shah in 1979:

$$V = ADBQW \tag{4}$$

where

A = effective zero period acceleration for the four seismic zones

D = spectral shape or dynamic amplification factor (DAF) for the two general site soil conditions

B = structural system reduction factor (like 1/R in ATC-3)*

Q = structural quality factor that increases significantly when the structural system lacks redundancy or has irregular or nonsymmetric features

W = weight of the structure

The resulting seismic design load levels in the El-Asnam region are about three-fourths to full UBC Zone 4 values, depending on the Q factor for the structure. In the coastal regions containing the major population centers, such as Algiers, Oran, and Constantine, loads are at about one-half UBC Zone 4 values.

The Algerian regulations contain most of the basic items found in the SEAOC Recommendations (1980 Blue Book); however, some modifications were necessary to meet the prevailing structural design methodology, and the type of quality of construction used in Algeria.

Designers in Algeria use French norms for reinforced concrete which are straight-line stress theory and working stress design. Details necessary to achieve sufficient ductility of frame and wall elements have to be specified explicitly since any references to strength capacity, ultimate strain, or yield hinges are not in the average designer's terminology.

High seismic load values were kept at a moderate level, and restrictions were placed on allowable types of concrete construction. The failure of typical short column foundation supports in the crawl space at the base of the buildings, called the *vide sanitaire*, and the lethal behavior of thin column, thick beam frames, both with and without brittle tile infill material, provided impetus in the El-Asnam region to require that the design and construction of RC seismic-resistant frame systems be braced by continuous 100 percent seismic load-resisting shear walls. Confined edge members are required for important walls. The seismic-resistant structural system of the buildings cannot be based on ductile moment-resisting space frames alone. This restriction is based on the belief that design practice, material quality, construction methods, workmanship, and inspection in Algeria cannot provide the required performance of ductile frame to be effectively seismic resistant in regions of high seismic risk, such as El-Asnam.

In less seismic regions, concrete frames (without shear walls) are permitted but must contain continuous steel, sufficient stirrups and joint ties, and must have relative beam-column sizes to provide reasonably ductile performance. For the common practice of tile or block infill, frames must have extra stiffness and shear resistance to control damage due to interaction with the infilled panels. The short columns used to support the *vide sanitaire* are replaced by continuous perimeter walls with access openings.

1.6 Building Construction

1.6.1 Architectural Influences in Algeria. Despite the seismicity of northern Algeria, where most of the people live, a style of architecture has not emerged that specifically reflects this environmental hazard through the choice of configuration, materials, or architectural elements. (This is not surprising because there are only a few regions in the world where this has occurred.)

*See "References" Applied Technology Council.

Rather, the architectural influences on Algerian buildings are more of an eclectic result of the region's history, which includes occupation or invasion by such diverse groups as the Romans, Arabs, Byzantines, Turks, and French. Buildings constructed throughout the years of French colonization (1848-1962) are still quite prevalent in Algeria.

During much of the French colonial era, seismic resistance was not foremost on the list of problems that the French found most worrisome about building in Algeria and the other North African territories. A March 1936 issue of *L'Architecture d'Aujourd'Hui* devoted to French architecture in North Africa summarized the concerns of Europeans. The major problems cited were the differences in cultural, aesthetic, and sanitation standards that Europeans and Algerians were accustomed to in housing. The threat of earthquakes was scarcely mentioned and never stressed as a major construction problem.

The same issue of *L'Architecture d'Aujourd'Hui* contained proposed town plans prepared by Le Corbusier for Algiers and Nemours. The proposals must have appeared quite radical and futuristic at the time (1936), and so those specific city plans were never implemented. On the other hand, sketches of buildings Le Corbusier envisioned are startlingly similar to buildings that the French were to erect in Algiers two decades later.* An illustration from that publication (Fig. 1.9) shows Le Corbusier's design for an enormous RC housing and transportation structure with columns (*pilotis*) at both ground and occasional intermediate levels. Le Corbusier's *Oeuvres Complètes 1910-1965* (1967) contains a 1933 proposal for prototypical Algiers apartment buildings (Fig. 1.10) which he describes in this way:

> The building is located on a site characteristic of this hillside city. A primary proposal: there should be a municipal regulation obliging all buildings along the boulevard paralleling the bay to be constructed on columns, thus leaving the ground floor entirely free so as to allow the inhabitants of Algiers an unobstructed view of the sea.

These proposals, which were published by *L'Architecture d'Aujourd'Hui* and by Le Corbusier in his books, *La Cité Radieuse (The Radiant City)* (1964) and *Oeuvres Complètes, 1910-1965* (1967), are interesting for their prophetic accuracy. The high-rise apartment shown in Fig. 1.11 is part of a five-building complex constructed by the French in Algiers in 1952. Another building erected that same year on the outskirts of the city is pictured in Fig. 1.12a. Figure 1.12b shows a modern hotel recently built in Algiers. Such RC buildings where the short ground story columns and intermediate floor columns create structural discontinuities are common in Algiers.

1.6.2 Building Construction in El-Asnam. The destructive earthquake of September 9, 1954 which devastated Orléansville (now El-Asnam) abruptly awakened the French to the enormous earthquake hazard that existed; and they immediately began reconstruction of Orléansville (on the same site) with intentions of making it much more earthquake resistant.

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*Le Corbusier believed that his plan was rejected because of "the weakness of the authorities." In 1934, he wrote, "Algiers drops out of sight like a magnificent body, but covered by the sickening scabs of a skin disease. A body which could be revealed in all its magnificence through the judicious influence of form . . . But I have been expelled, the doors have been shut in my face. I am leaving and deeply I feel: I am right, I am right, I am right . . ." (from *The Radiant City,* 1964, p. 260).

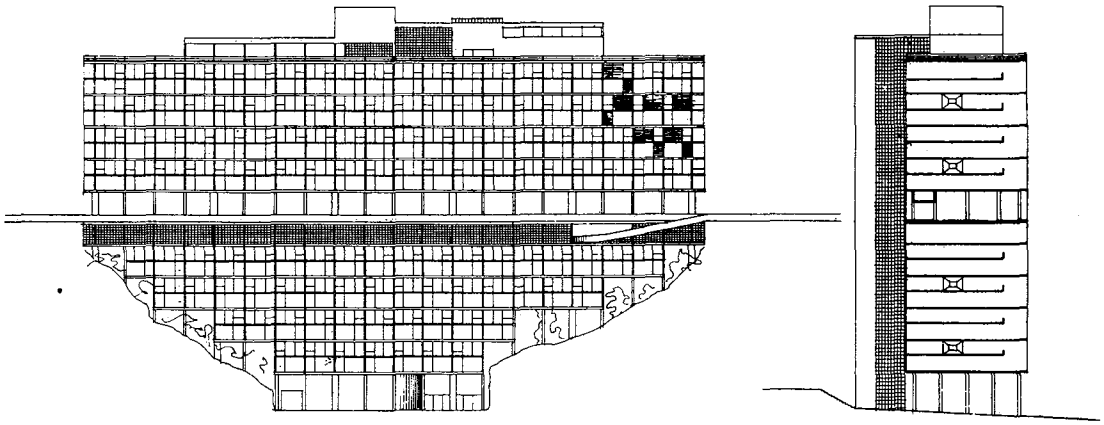
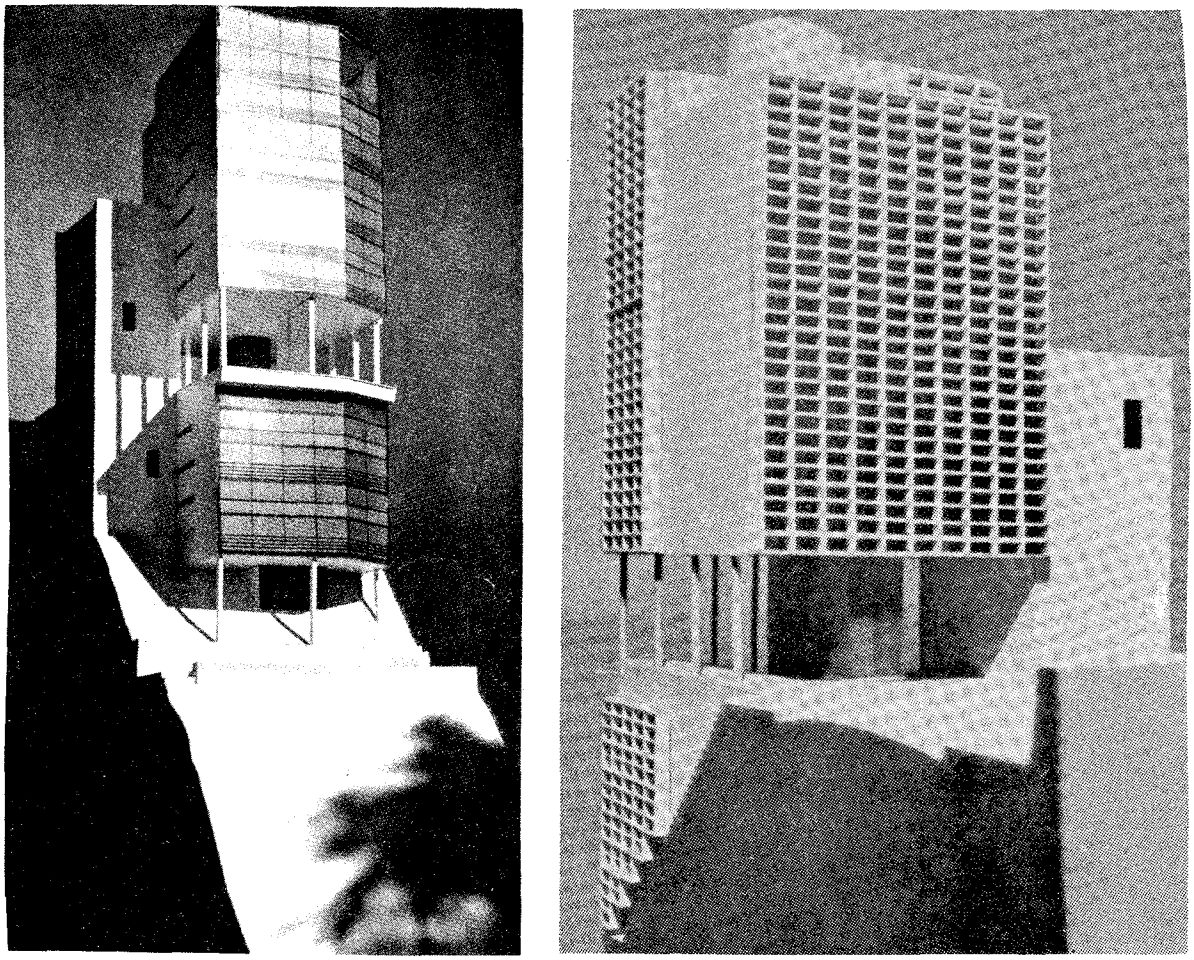


Fig. 1.9 Le Corbusier's Scheme for Large Transportation-Building Structure for Algiers (Never Built) (*L'Architecture d'Aujourd'Hui,* March 1936)

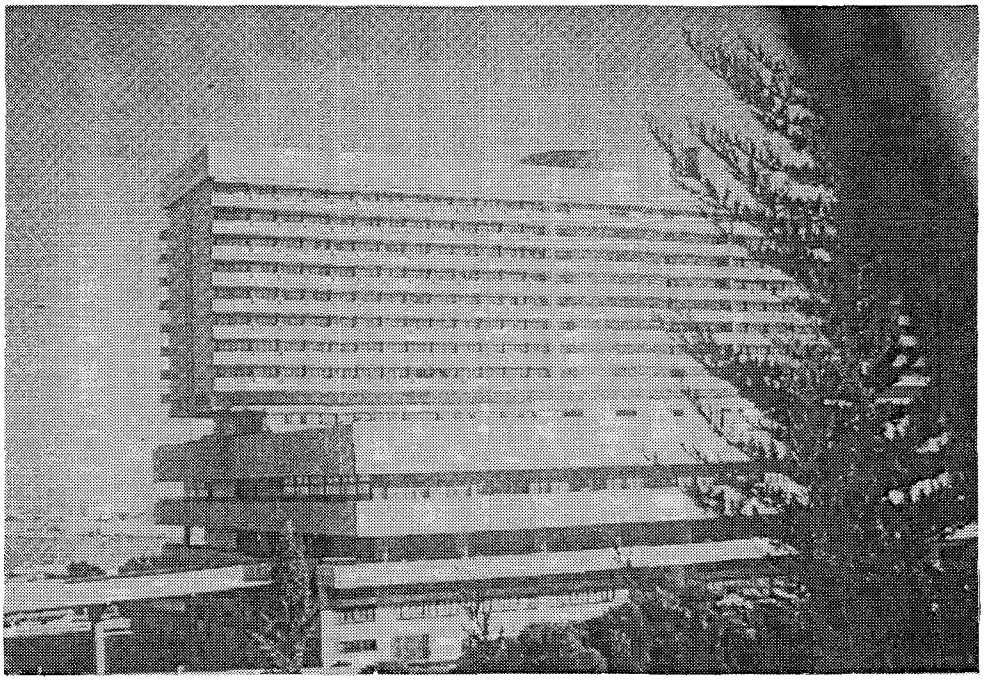


(a) (b)

Fig. 1.10 Le Corbusier's Proposed Prototype Apartment Buildings in Algiers, with Columns at Ground Floor to Give View of City and Sea (Le Corbusier, 1967)

Fig. 1.11 High-Rise Apartment Building in Algiers Built by French in 1952 *(L'Architecture d'Aujourd'Hui,* June 1955)

(a) Building Constructed by French in 1952



(b) Modern Hotel

Fig 1.12 High-Rise Buildings in Algiers (*L'Architecture d'Aujourd'Hui,* June 1955)

The June 1955 issue of *L'Architecture d'Aujourd'Hui* published extensive documentation of the destruction, ambitious planning schemes for the new city, and several RC building projects that were already under construction. The cause of the great devastation in the 1954 earthquake was attributed to the poor quality of construction in old, traditional masonry architecture that was generally neither reinforced nor tied. Steel frame construction was encouraged for this area; however, the expense of steel in this region precluded its widespread use even after the devastating earthquake. Traditional masonry buildings were largely replaced by concrete frame structures, infilled with unreinforced masonry.

Typical apartment building construction from the French period after the 1954 earthquake consisted of two-way RC frames having 3 meter (about 10 ft) modules. The floors were hollow precast concrete elements having a 4 to 5 cm thick topping of unreinforced concrete. Interior, as well as exterior, walls were usually hollow precast concrete infill. Such construction was generally used for two, three, and four-story buildings in El-Asnam; and usually the building was elevated from the ground.

The buildings present in El-Asnam when the 1980 quake occurred can be roughly divided into three categories:

1. Buildings that survived the 1954 earthquake, some of which were reinforced and strengthened after the quake
2. Buildings constructed after 1954 under French rule
3. Buildings constructed since the independence of Algeria in 1962

Buildings in category 1 were generally unreinforced masonry having different types of roofs, sometimes tile but more commonly heavy RC slabs with a thick layer of granular material (loose aggregate or crushed bricks), presumably for thermal protection. Some old buildings had composite construction of brick and mortar bearing walls with light steel beams providing floor support. The floor and roof were generally quite heavy. In most of these buildings, long masonry walls were inadequately braced by cross walls. Thick masonry walls, floors, and roofs gave the buildings a solid appearance; but, because of the general inadequacy of tieing them laterally, they exacerbated the earthquake problem, as will be discussed later. Fig. 1.13 is a dramatic example of the resulting failure. Several of these masonry buildings that were reinforced with external RC frame (columns and beams) after the 1954 earthquake behaved quite well (Fig. 1.14).

Buildings in category 2 (those built during the French period after the 1954 earthquake) were predominantly RC frame buildings, having 3 to 4 meter (10 to 13 ft) modules (Fig. 1.15) with heavy unreinforced masonry infill. Major official buildings and some housing were usually elevated from the ground story on supporting columns known as *pilotis*, whereas apartment buildings were commonly built atop a short crawl space — *vide sanitaire* (literally: sanitary void) — supported by stubby columns. Floor and roof systems consisted of RC slabs, usually one-way joist slab although waffle slab was also used. The roof was generally very heavy; and, again, its thickness and weight were increased by use of a layer of loose granular material for thermal insulation. Fig. 1.16 illustrates this roof system. Most of the main buildings in the downtown area of El-Asnam, including schools, hospitals, and hotels (see Fig. 1.17), were in category 2. These buildings performed poorly, with many suffering total collapse. A detailed discussion of reasons for this adverse behavior is presented later.

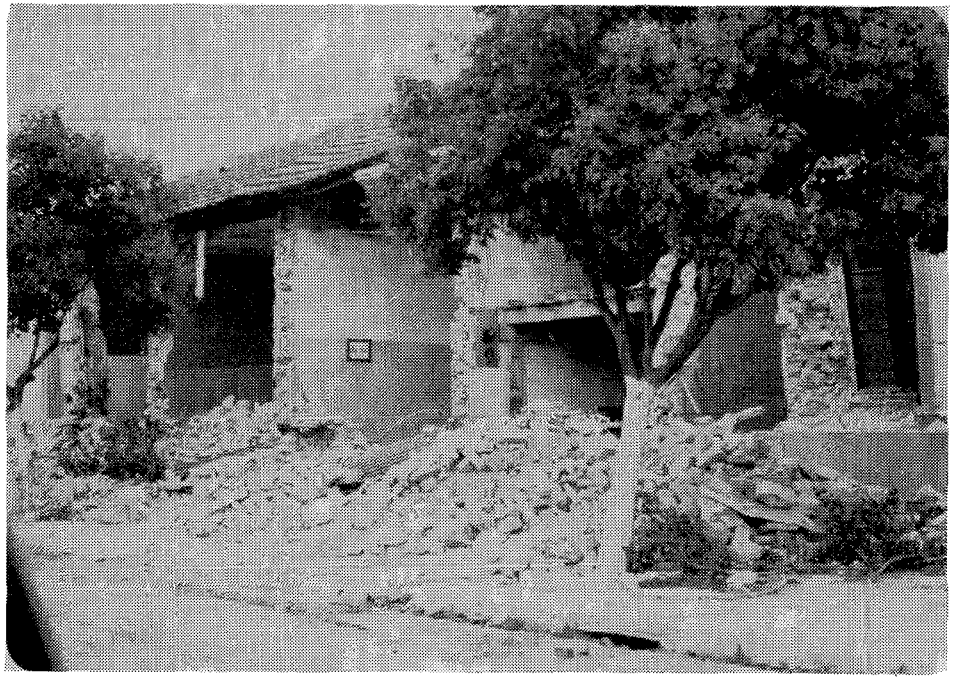


Fig. 1.13 Collapsed One-Story Masonry Structures in El-Asnam

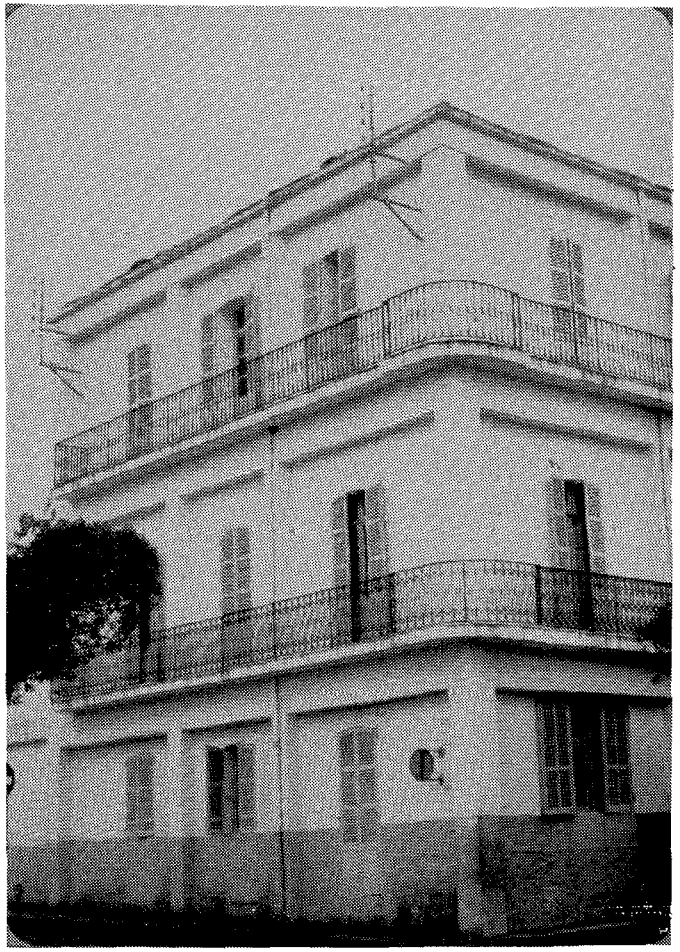


Fig. 1.14 Structure in El-Asnam Strengthened with RC Frame After 1954 Earthquake. It sustained only minor damage in October 10, 1980 earthquake.

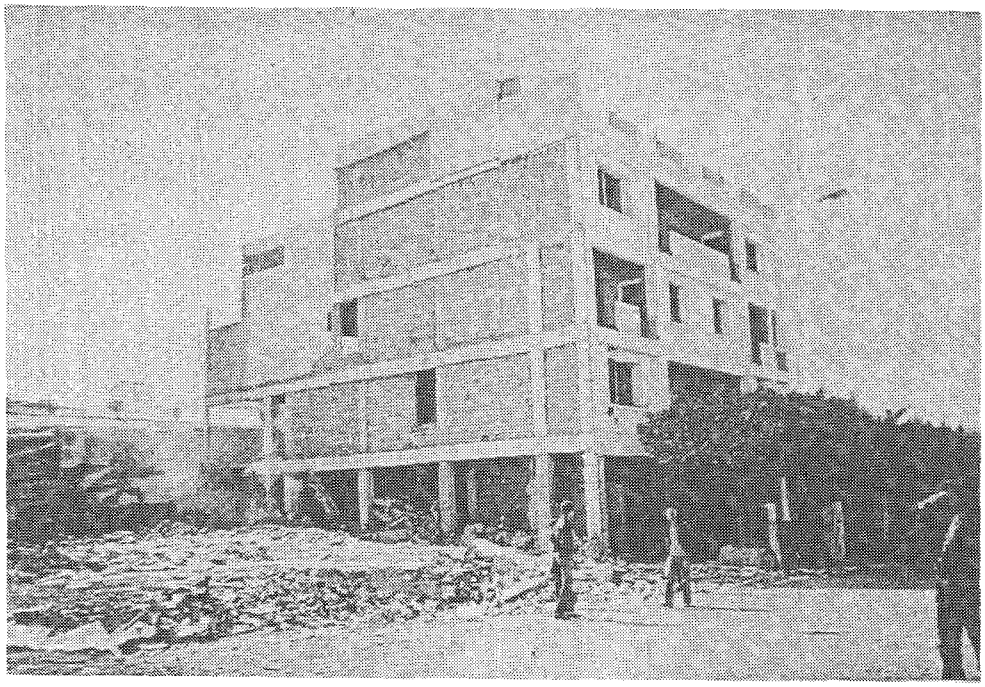


Fig. 1.15 Typical Construction in El-Asnam (Concrete Frame with Brick Infill)

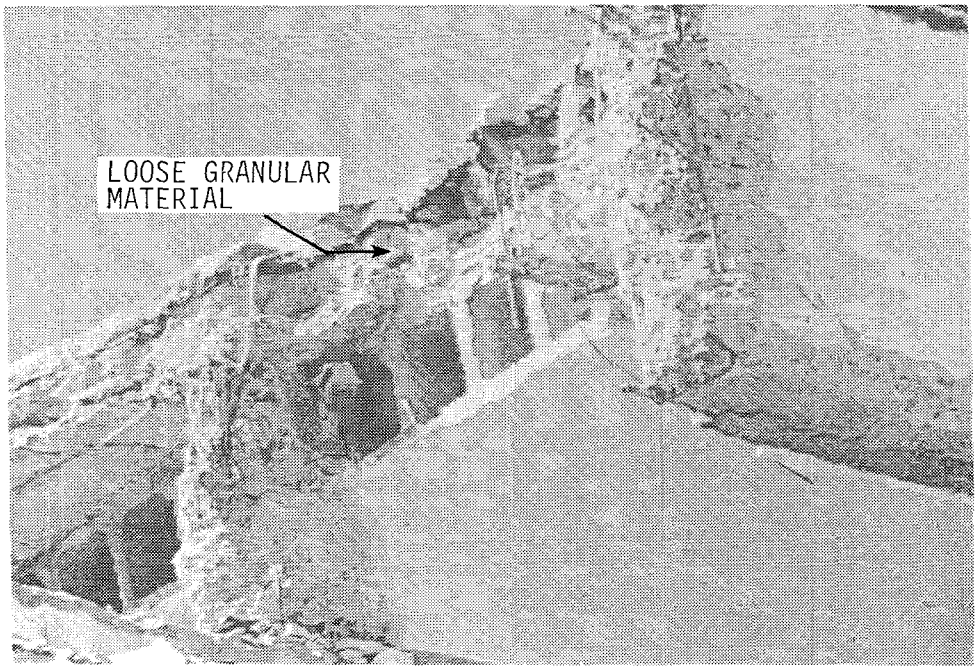


Fig. 1.16. Typical Roof System. Note use of layer of loose granular material for thermal insulation.

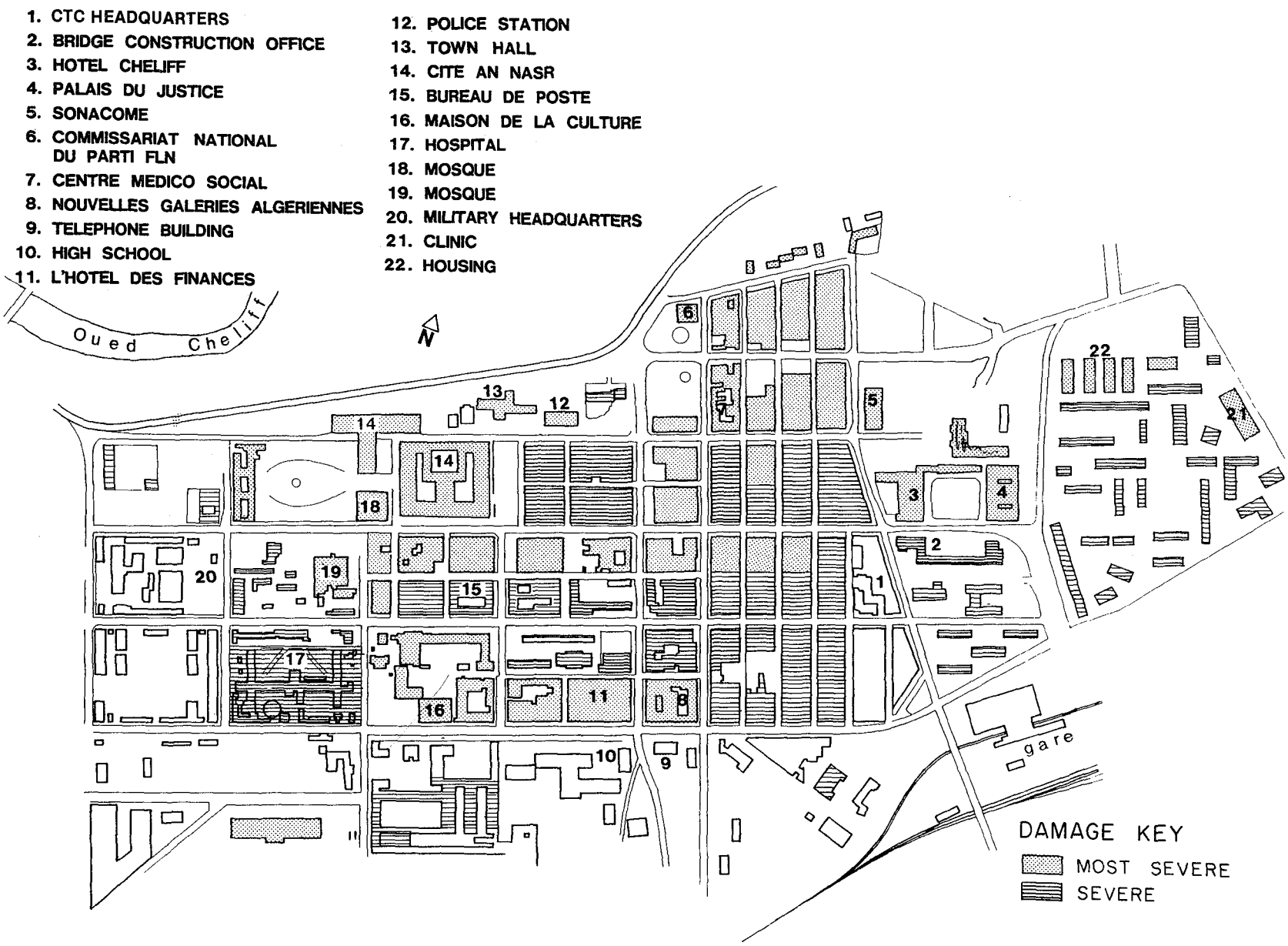


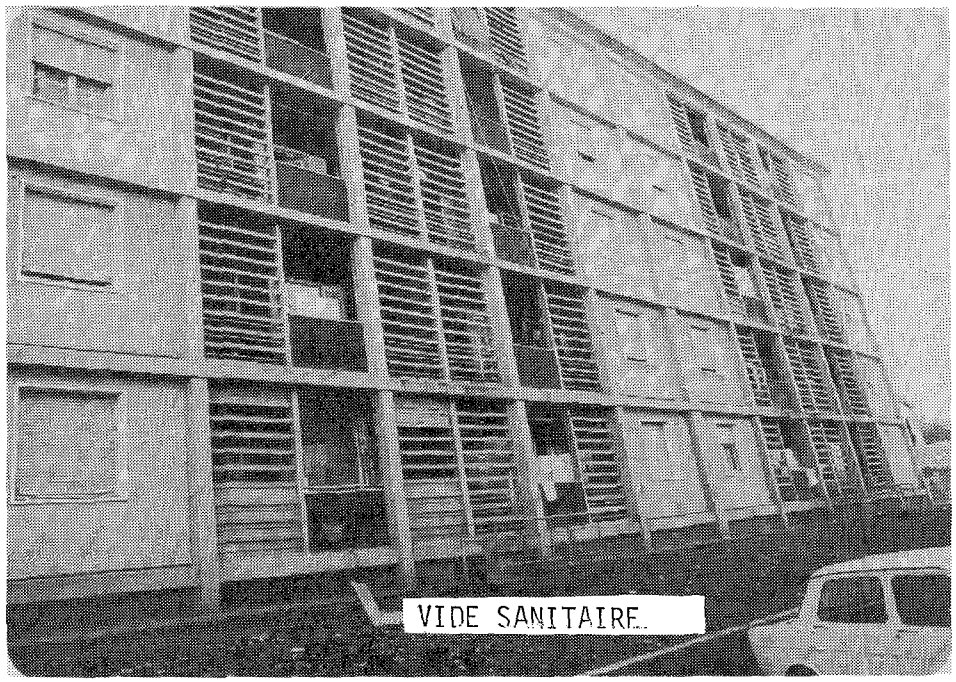
Fig. 1.17 Downtown El-Asnam

Buildings in category 3 (recently constructed) are of special interest. Following independence from France in 1962, the Algerian government assumed the task of solving its enormous housing problem. The pressure of this task led the government to concentrate on quantity, rather than careful analysis of the most effective type of building construction for a high seismic risk region. Consequently, the new buildings were structurally a continuation of the French type of construction. Buildings observed in El-Asnam and El-Attaf were generally three or four stories tall, although one and two-story developments were also being constructed.

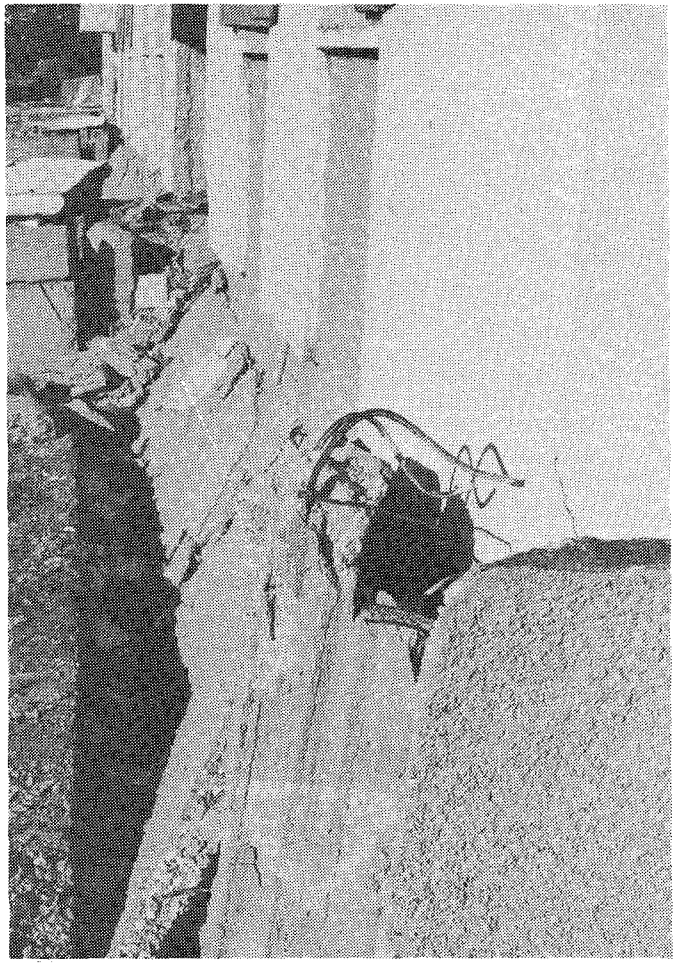
The typical three or four-story apartment buildings exemplify the type of construction dating back to the French period. Typical construction consists of a two-way concrete frame with a 3 meter (10 ft) module. A typical building may be three bays wide (9 meters) by ten bays long (30 meters). Cast-in-place beams approximately 75 cm on center span between the frame beams (frequently pinwheeled). These subframing beams support hollow precast concrete elements approximately 30 cm wide, installed flush top and bottom with the supporting beams. The floor assembly is topped with an unreinforced slab 4 to 5 cm thick. Exterior and interior walls are hollow masonry infill. The entire structure has at its base the *vide sanitaire* which is generally 1 meter (39 in.) high. This *vide sanitaire* provides space for plumbing and ventilation under the first-floor slab and serves as a barrier against transmission of humidity from the ground to the first floor. Infill between the perimeter stub columns is nonstructural, either masonry or a minimal unreinforced concrete wall. On a typical building, as described above, the entire lateral load is therefore transmitted by the first floor slab acting as a diaphragm to 4 x 11 = 44 one meter high stub columns, generally 25 x 25 cm or 20 x 40 cm in cross section (Figs. 1.18 and 1.19).

This type of construction was widely used in the past, and, unfortunately, still continues. CTC's authority is apparently limited, for it has not succeeded in eliminating or drastically improving this type of construction.

Throughout the outskirts of El-Asnam, large housing projects had been completed or were under construction when the 1980 earthquake occurred. Building types varied from one-story single family units to four-story apartment buildings. Generally, the structural system was similar to that of the typical construction just described, i.e., it consisted of concrete columns and beams, concrete or composite slabs, and masonry exterior infill and interior partitions. Some use of concrete shear walls was noted in recent construction under the control of CTC.

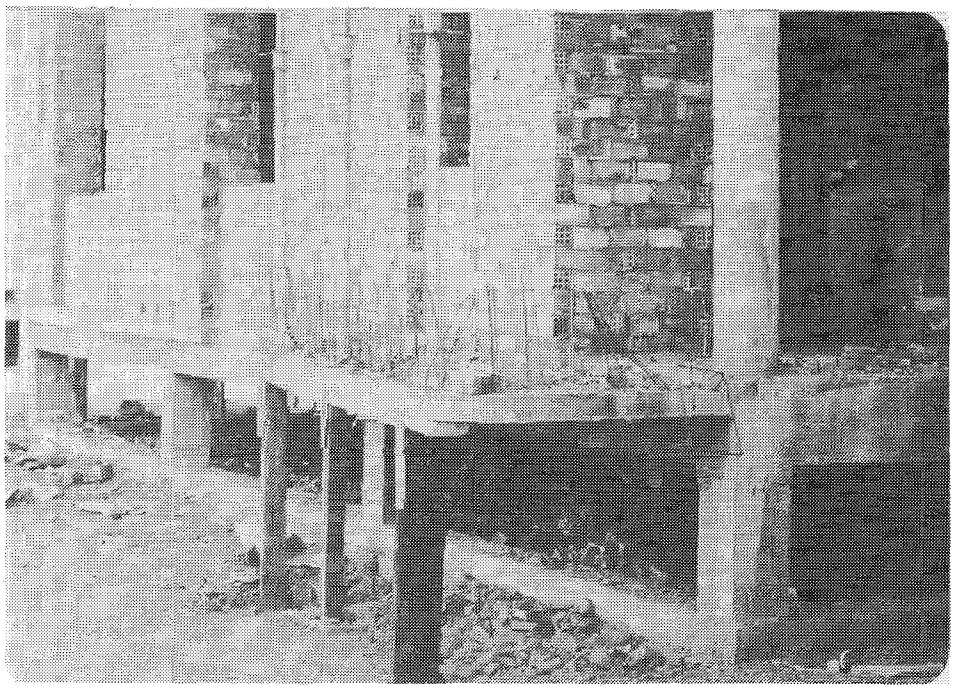


(a) Building in Algiers. Most buildings of this type in El-Asnam lost their 1 meter high columns.

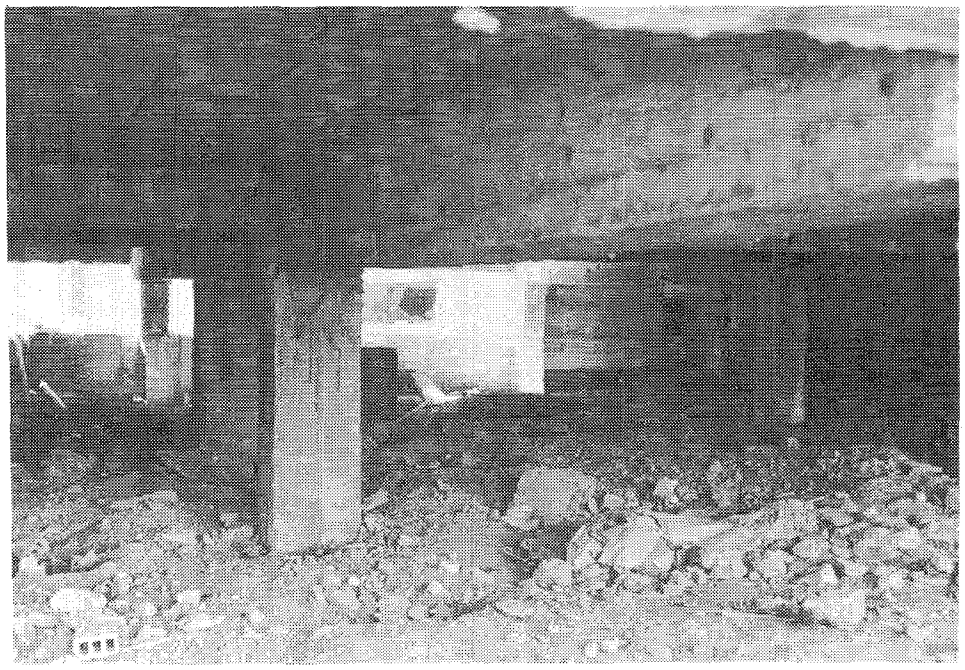


(b) Failure of *Vide Sanitaire*

Fig. 1.18 Typical Frame Construction with *Vide Sanitaire* (Crawl Space). Note short columns.



(a) Perimeter



(b) Interior Columns and Beams, First Floor

Fig. 1.19 Building Under Construction with *Vide Sanitaire*

II. SEISMOLOGIC AND GEOLOGIC FEATURES

2.1 Location, Magnitude, Intensity, and Pattern of Shaking

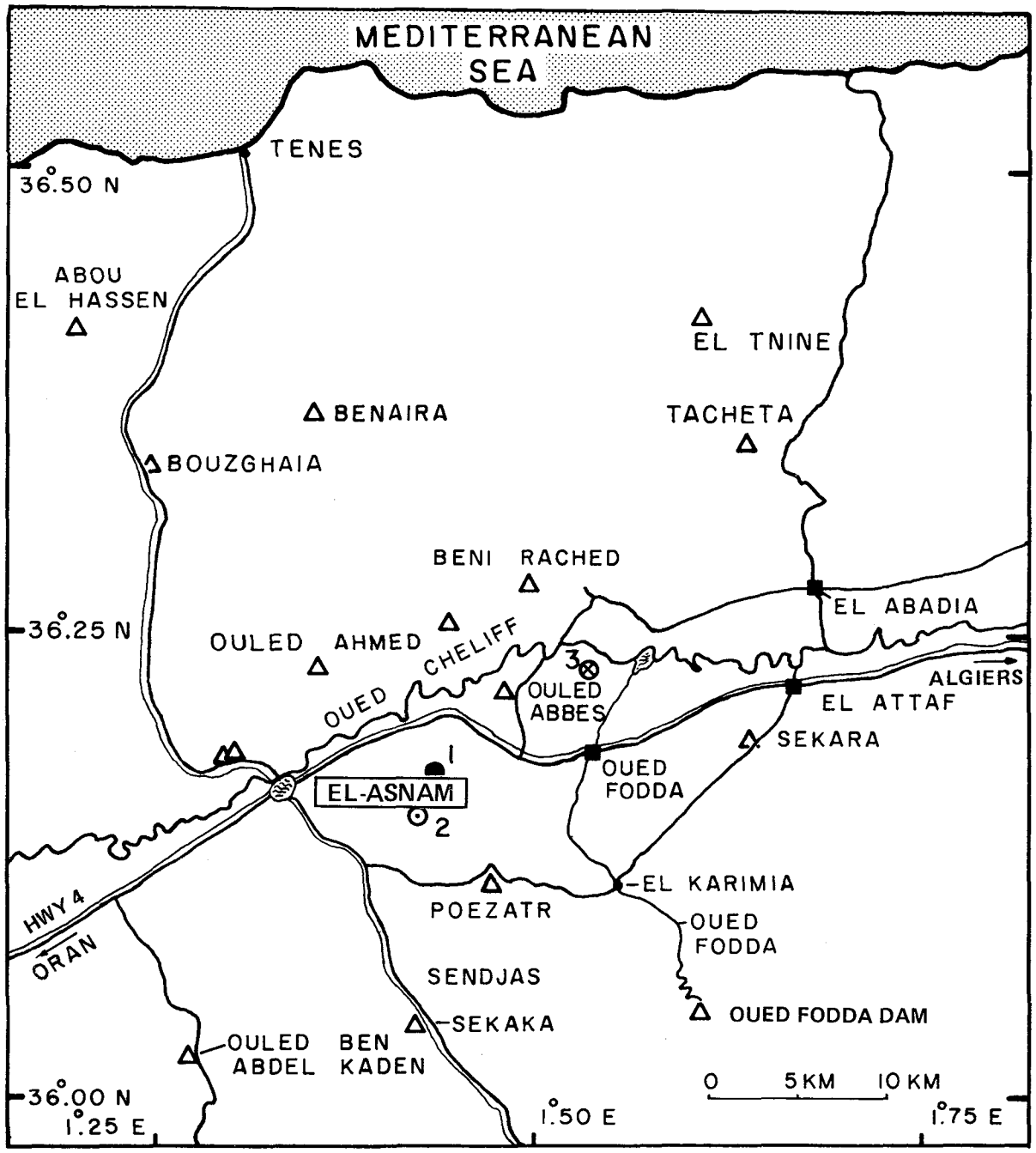
The area around El-Asnam and the valley of Oued Cheliff is shown in Fig. 2.1. This east-west valley lies between the two ranges of the Atlas Mountains and is crossed by the national railroad, the east-west highway, underground as well as elevated aqueducts, gas pipelines, and power lines. Fig. 2.1 pinpoints the epicenters of the main shock of the October 10, 1980 earthquake located by the International Seismological Center (ISC) and by the U.S. Geological Survey (USGS).

The main shock occurred at 13:25:23.7 Algerian time (12.25.23.7 GMT). The surface wave magnitude of the event, M_S, was assigned to be 7.3. According to calculations by Papastamatiou (1980), the seismic moment of the event was 6×10^{26} dyne-cm (assuming source rigidity of 3×10^{11} dyne/cm^2, fault rupture area of 40×25 km^2, and an average fault dislocation of 2 meters). A stress drop of 55 bars was calculated.

Fig. 2.2 is a preliminary isoseismal map of the main shock. It can be seen that the highest intensity (Modified Mercalli Scale, MMI) El-Asnam experienced was between IX and X. At first sight the extensive damage to buildings in El-Asnam appeared to justify a X or even XI. However, if it is considered that the extensive collapse of buildings was due to poor seismic-resistant design and construction practice, an MMI Intensity IX seems more justified and to correlate better with other estimated parameters of the earthquake.

The most severely affected communities were El-Asnam (IX to X), El-Abadia (IX), Beni Rached (X), Sendjas (IX), Oued Fodda (IX), El-Attaf (VIII), and El-Karimia (VIII). From the isoseismal map (Fig. 2.2) it is apparent that the earthquake was felt over a wide region of northern Algeria from Oran (IV), some 200 km west of El-Asnam, to Algiers (IV), approximately 200 km to the east. Damage, however, was much more local.

The area of strongest shaking and therefore of major damage was skewed to the east of the epicentral region. From Algiers westward, the first town with serious damage was Khemis Miliana, about 60 km east of El-Asnam. Further west of Bou Kadir, about 20 km from El-Asnam, there was no serious damage. The southern limit of damage was at the villages of Sendjas and El-Karimia. To the north, damage was essentially confined within a radius of 15 to 20 km from El-Asnam. In the northern coastal town of Tènés there was little evidence of damage although Tenes was no further from the epicentral region than other towns where heavy damage occurred.



△ Instruments

Note: Double lines are national highways, single lines are improved roads

● 1 – Epicenter located by International Seismological Center

⊙ 2 – Epicenter located by USGS

⊗ 3 – Aftershock located by USGS

Fig. 2.1 Epicenters of October 10, 1980 Earthquake (Rothé, December 1980)

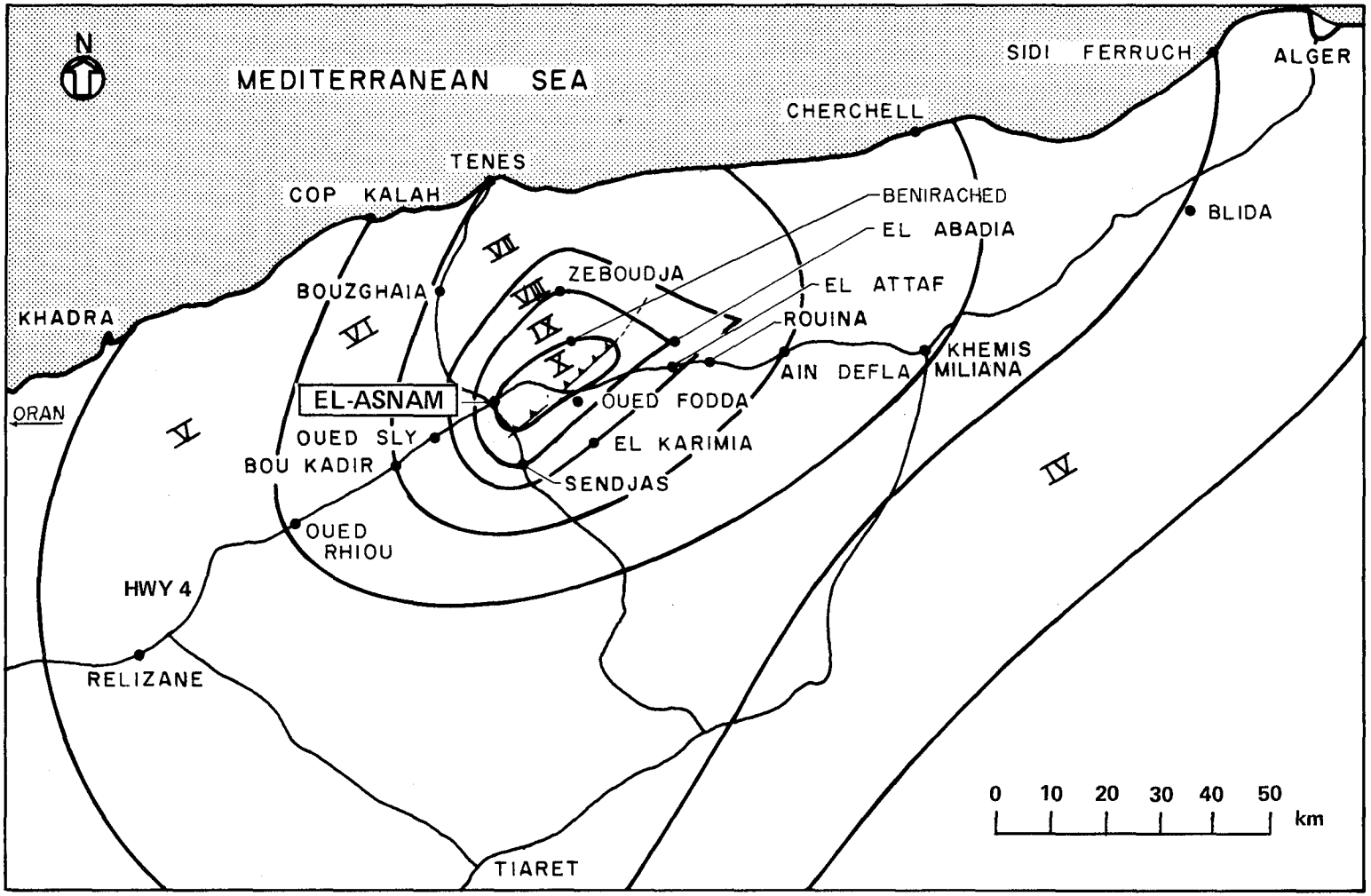


Fig. 2.2 Isoseismal Map (Preliminary) of October 10, 1980 El-Asnam Main Shock (CTC, 1981, based on MMI Intensities)

2.2 Surface Faulting

Fig. 2.3 shows the main faults that ruptured during the earthquake. The main shock was produced by displacement on a northeast-trending thrust fault that dips northwestward; this fault has subsequently been named the Oued Fodda fault after the closest principal city along its surface trace. Numerous secondary fissures and normal faults occur on the upthrown block of the main thrust within a zone that extends to 2 km from the main trace. Surface faulting also occurred along the Beni Rached fault, a normal fault that may also be a secondary fault.

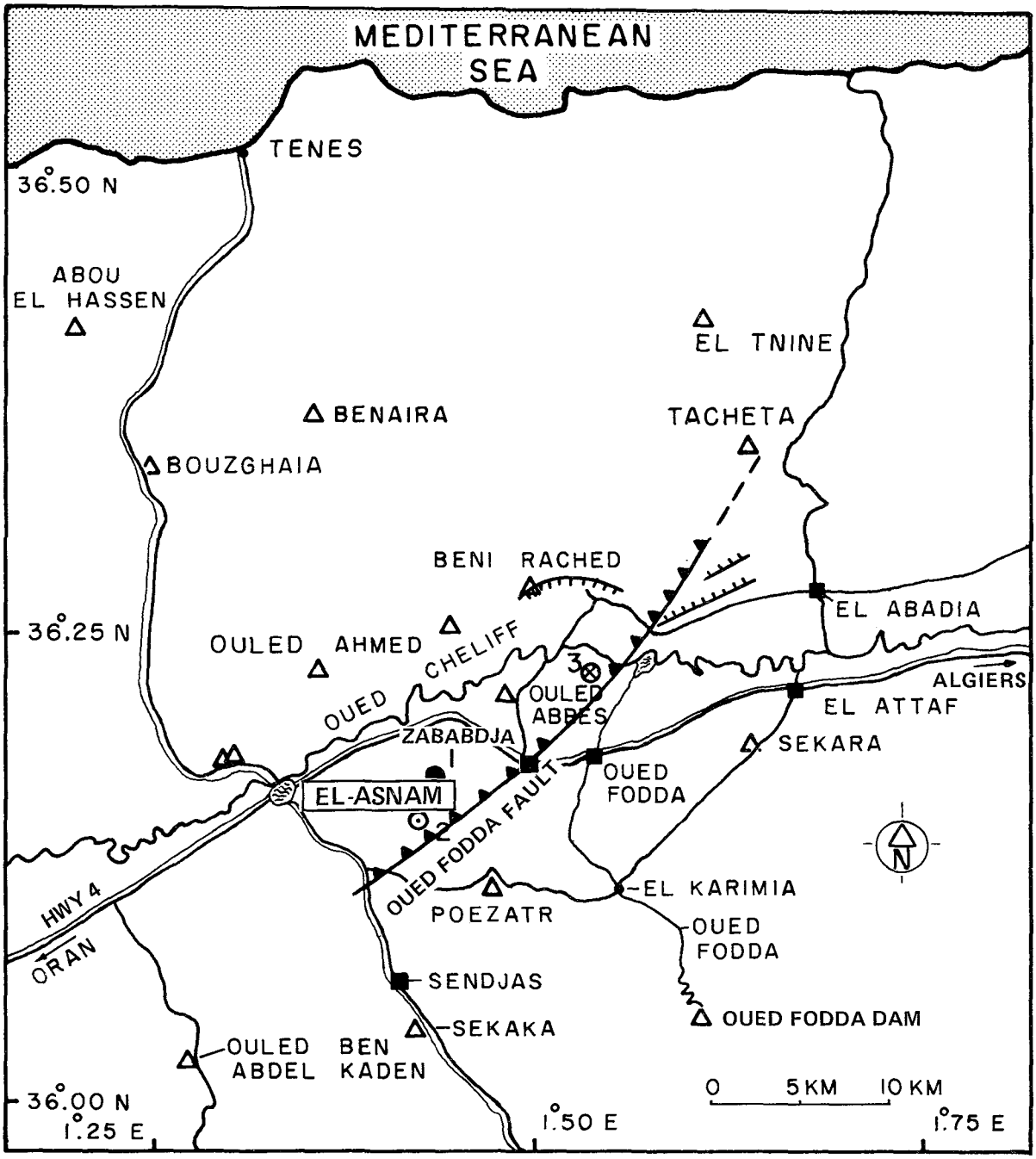
Surface faulting occurred along the Oued Fodda fault, which is located south and east of El-Asnam; the closest distance of the surface trace to El-Asnam is about 7 km. The surface faulting occurred along a zone that extends at least 30 km from a point 5 km north of Sendjas northeastward through the village of Zababdja, along the northwest side of Oued Fodda valley to a point about 4 km west of El-Abadia. Secondary normal faulting and ground cracking suggests that the primary thrust fault rupture may extend an additional 2 km to the southwest and 4 km further eastward to El-Abadia, suggesting an overall rupture length of 35 km. In most places, the primary fault is a low angle thrust that dips 10° to 20° northwestward. Locally, the dip of the primary fault steepens to form a reverse fault that dips as steeply as 55°.

The primary fault is typically expressed as a series of low scarps and compression ridges (Fig. 2.4) that have various surface patterns. They generally occur as either subparallel en echelon breaks, or in an anastomosing pattern where the scarps and pressure ridges branch and reconverge. Both left and right stepping fracture patterns were observed, and apparent left and right lateral displacements were measured. In a few places the primary thrust occurs as a single fault scarp. Typically, the vertical displacement on individual scarps is a meter or less, and the cumulative verical displacement across the zone of primary faulting is generally not more than 1.5 to 2 meters. Estimates of the net slip vary from less than 1 meter to about 6 meters. The average net slip appears to be between 3 and 4 meters.

Maximum vertical displacement measured along the primary fault trace was 2.6 meters; at this location the fault intersected a sloping surface and produced a scarp 4.2 meters high (Figs. 2.5 and 2.6). The dip of the fault could not be measured at this location; however, the morphology of the scarp is similar to other locations where the fault dips steeply (45° to 55°). Assuming a 50° dip, the dip slip displacement would be 3.4 meters.

Extensive fissures and normal faulting occurred on the upthrown block of the primary thrust (Figs. 2.5 and 2.7). These fissures generally were within 2 km of the trace of the primary thrust. Both down-to-the-southeast and down-to-the-northwest displacement occurred. In most places, the displacement on the secondary high angle normal faults produced scarps that are more pronounced than those along the primary thrust fault. Vertical displacements of about a meter are common on these normal faults, and in some localities produced scarps 2 to 4 meters high. Displacement on the Oued Fodda fault was the principal source of the seismic energy released during the October 10, 1980 earthquake.

The Beni Rached fault is a high angle normal fault that appears to be a secondary splay of the Oued Fodda fault (Fig. 2.3). In most places, the surface faulting along the Beni Rached fault is expressed as a single well-defined fault scarp (Figs. 2.8 and 2.9). Vertical displacement on the fault is typically about 1 meter. Numerous small graben and extensional cracks occur on the downthrown side along the base of fault scarp. Apparent right and left lateral offsets of approximately 0.5 to 1 meter were measured locally along the fault.



△ Instruments

Principal fault (Thrust fault)

Secondary fault (Normal fault)

● 1 – Epicenter located by International Seismological Center

⊙ 2 – Epicenter located by USGS

⊗ 3 – Aftershock located by USGS

Fig. 2.3 Principal Faults — October 10, 1980 Main Shock (Rothé, December 1980)

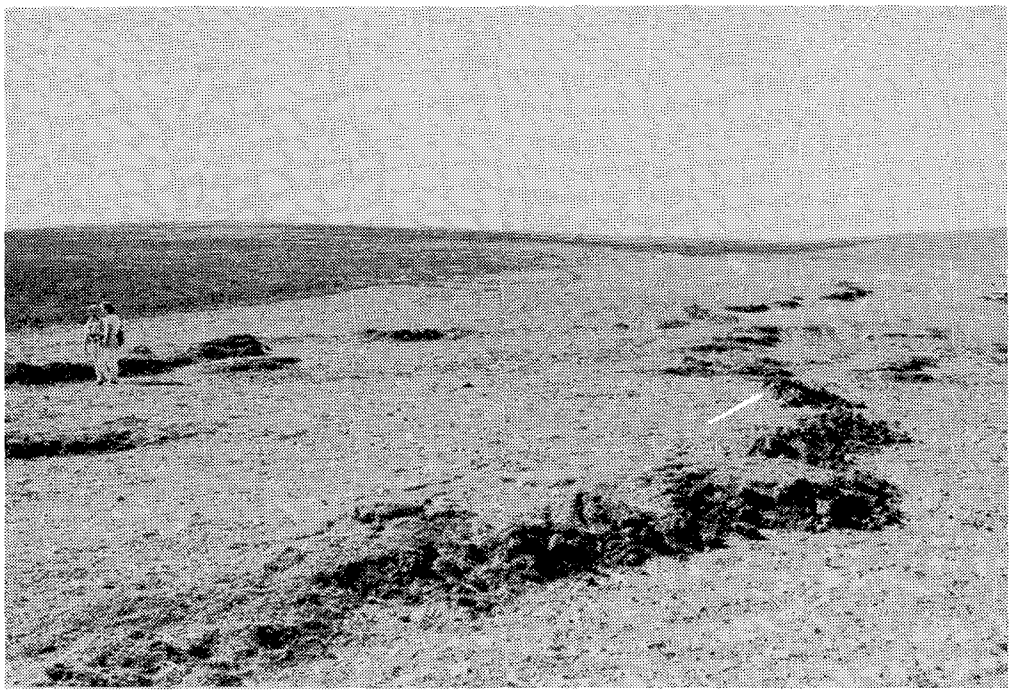


Fig. 2.4 Compression Ridges Along Surface Trace of Oued Fodda Fault Southwest of Zababdja. At this location the main fault is a low angle thrust dipping 10° to 20° northwest. View is northeast.

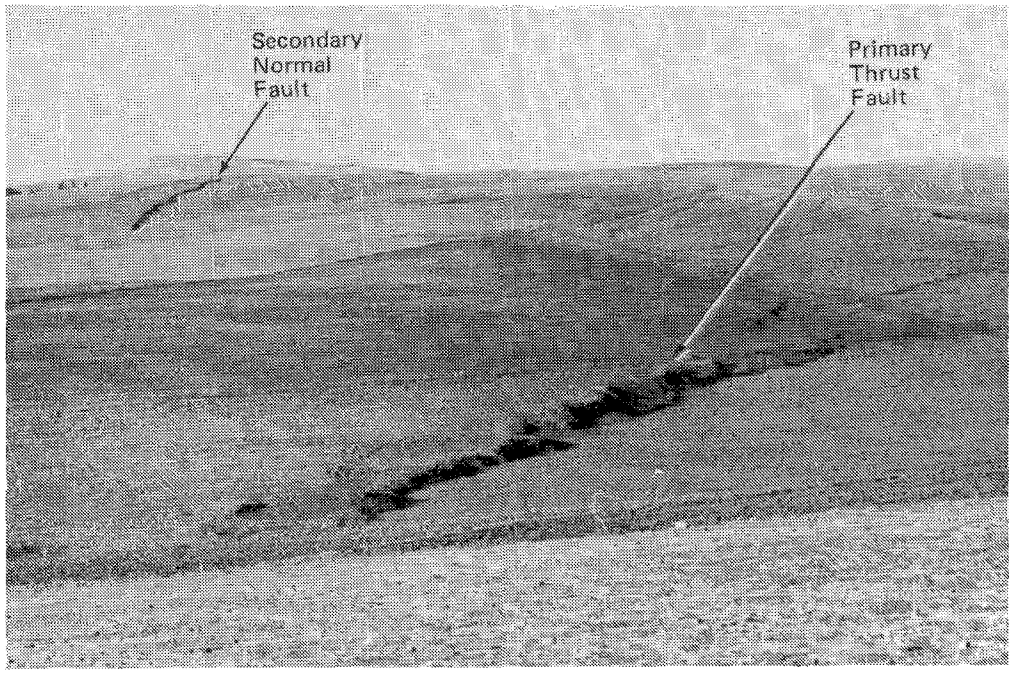


Fig. 2.5 Oued Fodda Fault North of Sendjas. View is northward.

Fig. 2.6 Fault Scarp Along Oued Fodda Fault North of Sendjas. The 4.2 meter high scarp was produced by 2.6 meters vertical displacement on a reverse fault dipping approximately 50° northwest. View is southwest.

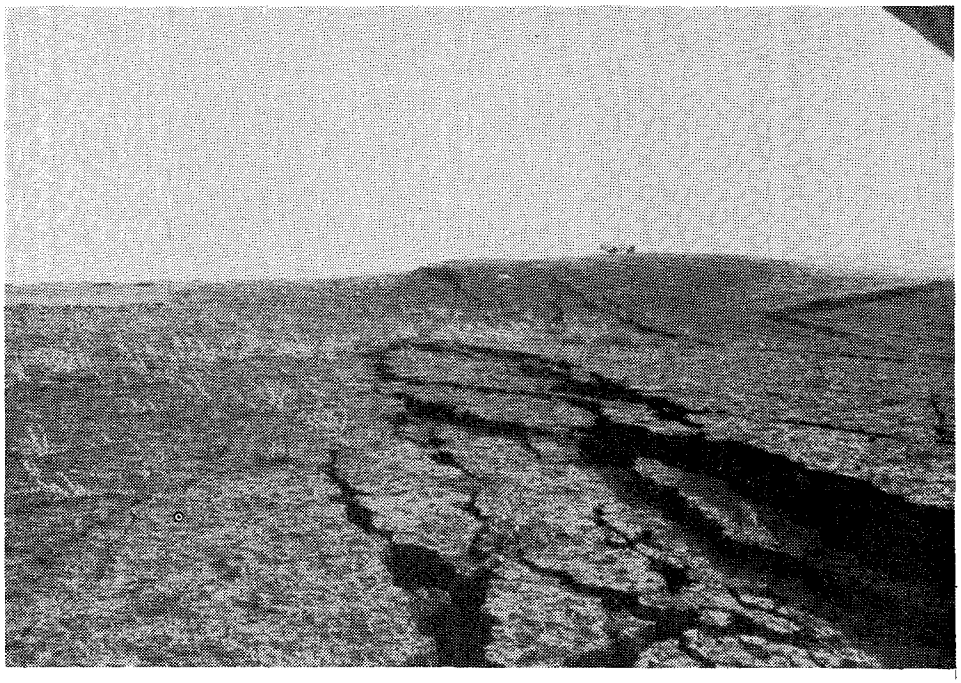


Fig. 2.7 Secondary Normal Faults on Upthrown (Northwest) Block of Oued Fodda Fault South of Oued Cheliff. View is southwest.

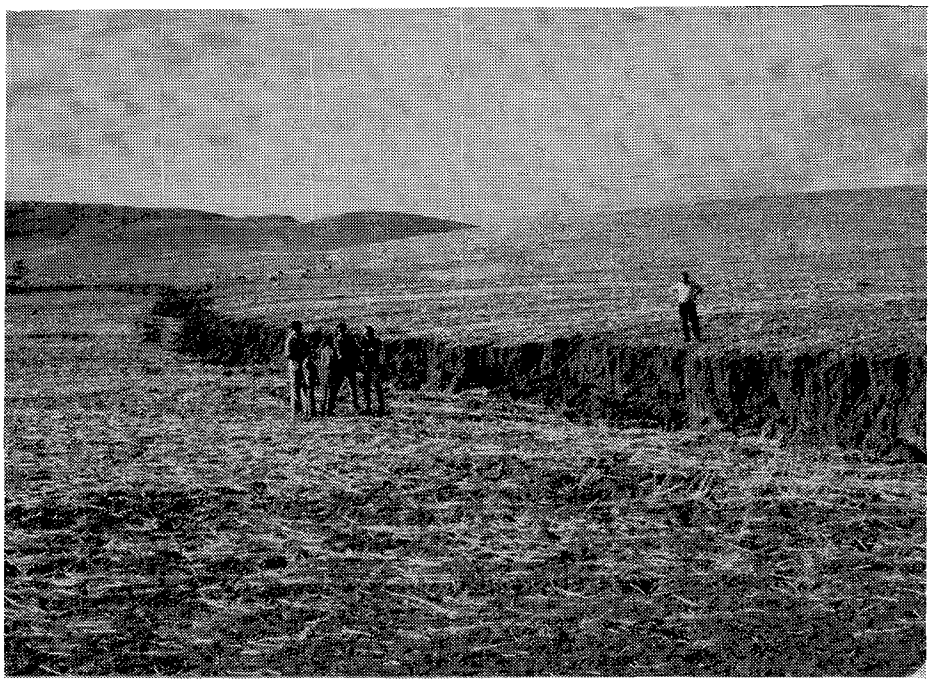


Fig. 2.8 Normal Fault Scarp Near Beni Rached

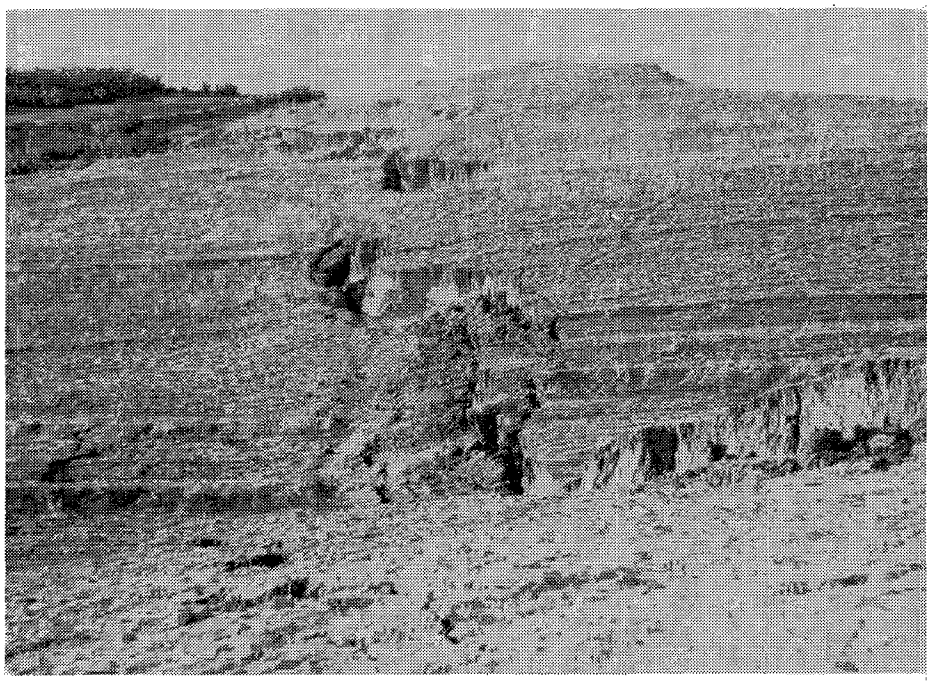


Fig. 2.9 Break Along Beni Rached Fault

There is abundant geomorphic and geologic evidence that the zone of surface along the Oued Fodda fault occurred along a pre-existing fault zone that has slipped repeatedly during the Holocene period (approximately the past 10,000 years). However, because of the irregular pattern and the subdued nature of scarps produced by thrust faults, and the relatively rapid rates of erosion of these scarps, the specific location of surface fault rupture would have been difficult to predict before the earthquake.

Accounts from villagers living in the vicinity of Beni Rached at the time of the September 9, 1954 Magnitude 6.7 earthquake, as well as a report by Rothé (1955), indicate that surface faulting took place along parts of the Beni Rached fault during the 1954 earthquake. Although there is no evidence that there was surface faulting along the Oued Fodda fault during the 1954 earthquake, this earthquake was probably produced by slippage along the Oued Fodda fault at depth. For further details refer to *Geosciences* (1981).

2.3 Aftershocks

Table 2.1 shows the magnitude, date, and local time of aftershocks of the 1980 earthquake greater than surface wave magnitude, $M_S = 4.0$. The biggest aftershock, Magnitude 6.0, occurred October 10, 1980 at 16:39:09.8 Algerian time. The second major aftershock, Magnitude 5.6, took place November 8, 1980; a similar tremor occurred December 7, 1980. Most of the epicenters of these aftershocks were located on the main thrust fault.

TABLE 2.1. PRINCIPAL AFTERSHOCKS

Date	Local Time	Magnitude, M_S
Oct. 10, 1980	16:39	6.0
Oct. 23, 1980	17:23	4.1
Oct. 23, 1980	10:58	4.4
Oct. 24, 1980	13:58	4.4
Oct. 31, 1980	00.38	5.0
Nov. 8, 1980	08:54	5.6
Nov. 10, 1980	01:02	4.9
Dec. 5, 1980	14:32	5.2
Dec. 7, 1980	18:37	5.6

2.4 Ground Motion Records

The strong ground motion of the main event, as well as that of the main aftershock of the October 10, 1980 earthquake, was not recorded by any accelerographs. There were strong motion instruments in the vicinity (four instruments were in place on the Steeg, or Oued Fodda, Dam 30 km southwest of El-Asnam and one on a dam 20 km south of Relizane and 90 km southwest of El-Asnam) (Fig. 2.10). However, none of these instruments functioned during the main event or aftershocks. Therefore, little quantitative information on the acceleration, frequency, and duration of strong motion is available. There are many conflicting reports regarding duration. According to some, the strong motion of the main shock lasted about 15 sec, while the total duration was about 35 to 40 sec.

Kinemetrics, Inc. installed an SMA-1 accelerograph in El-Asnam October 15, five days after the main event. Two days later, five additional instruments were brought in by the International Institute of Seismology and Earthquake Engineering (IZIIS), Skopje, Yugoslavia. Thus, all major aftershocks after October 17 were recorded (IZIIS, 1980). The IZIIS instruments were distributed in the epicentral region: three in El-Asnam, one in Beni Rached, and one in El-Attaf. The most significant aftershock record was obtained in El-Asnam November 8, 1980 from a Magnitude 5.6 event. Fig. 2.11 shows the accelerograms from the Sogedia Food Processing Factory. From these accelerograms, it can be seen that the vertical component of acceleration was the largest. This trend was observed in many other aftershock records. The peak value of spectral acceleration occurred at a period of about 0.1 sec in most of the records.

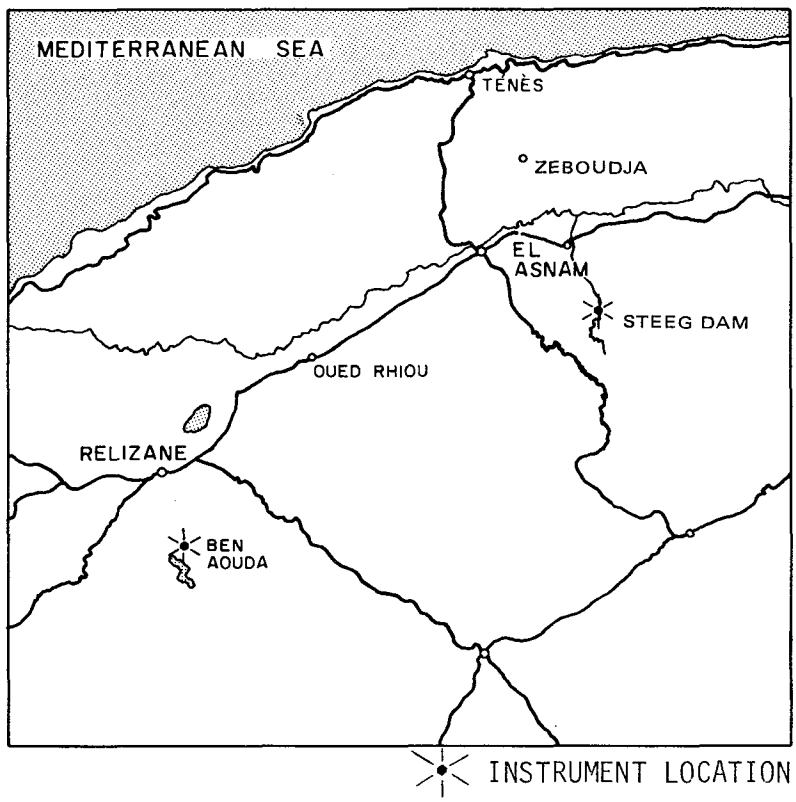


Fig. 2.10 Strong Motion Instrument Locations in October 10, 1980 Epicentral Region

Dimitri Papastamatiou
consulting engineer

2-10a

Ms Arline Leeds
EERI
2620 Telegraph Avenue
Berkeley
California 94704

London, 17 September 1983

Dear Arline,

In the EERI reconnaissance and engineering report on the 1980 El-Asnam earthquake, reference is made to a report I released two months after this earthquake on my field observations regarding the strong motion aspects of this damaging event (1).

In general, quotations from my early El-Asnam report are taken out of context In particular, on page 2-11 of the EERI report it is stated that 'Papastamatiou estimates vertical acceleration in access of 1.g'. This reference to my field measurements is inaccurate and within the wrong context: almost all my field measurements quantified the horizontal component of motion with the only exception of the upthrown stones in the extension zone behind the main thrust break. The latter observation is related to the local feature of tectonic deformation and should not be generalised for the whole epicentral region. In a later communication (2), during the special UN meeting on the El-Asnam earthquake in Algiers in June 1981, I presented my field observations on the ground deformation associated with this earthquake. The combination of field evidence on ground deformation and ground motion enabled me to speculate on the overall magnitude of vertical motion in the epicentral region. The integrated field evidence pointed to a vertical component about equal to the horizontal component of motion.

Unfortunately, the hard evidence on the actual ground motion in the epicentral region of this important earthquake is missing as no strong motion record was obtained from the main event. A strong motion network was deployed soon after the main event by the Earthquake Engineering Institute of Yugoslavia. I understand that the numerous strong motion records triggered by the aftershocks of the October 10 event showed predominance of the vertical component only at stations close to the epicentre of these aftershocks.

Yours sincerely,

Papastamatiou

(EERI member)

references:

1. Papastamatiou D., 1980, El-Asnam, Algeria earthquake of October 10,1980:field evidence of ground motion in the epicentral region, Geognosis report GP05/80.
2. 1981, On the correlation between ground motion and deformation in the epicentral area of the El Asnam 1980 earthquake, Proc Sci Sessions on the El Asnam earthq,Algiers,June 15-17.

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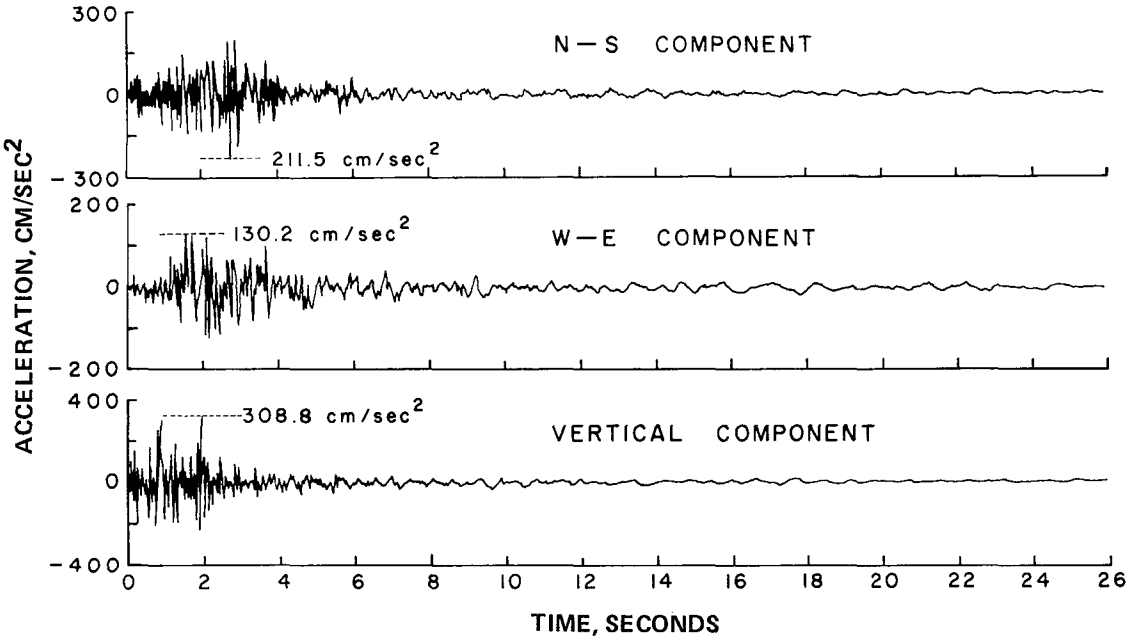


Fig. 2.11 Accelerograms from Sogedia, Food Processing Factory, El-Asnam, from November 8, 1980, 08:54 Aftershock (Magnitude 5.6)

It is interesting to note that for an aftershock of 5.6 magnitude, the largest horizontal acceleration recorded in El-Asnam was about 0.21 g and the largest vertical acceleration 0.31 g. It is estimated that the main event had the largest horizontal acceleration, in excess of 0.40 g, and the largest vertical acceleration, in excess of 0.50 to 0.60 g [Papastamatiou estimates vertical acceleration in excess of 1 g (Papastamatiou, 1980)].

2.5 Site Soil Conditions

El-Asnam, Oued Fodda, and El-Attaf lie in a broad alluvial valley flanked to the north and south by ranges of hills that rise to a height of approximately 1000 meters. The valley is drained by the Cheliff River. Although there was clear evidence of different types of soil failure, most of these failures occurred in a region where no engineered structures existed, thus loss of life and property because of soil failure was minimal.

Settlement of structures may have occurred, particularly in fill areas, and most backfills behind bridge abutments settled. Numerous slope failures were observed in the mountains, some involving the whole side of hills in the region of fault movements. No major slope failures were observed in the city of El-Asnam. Soil liquefaction occurred over widespread areas in the flood plain of Oued Cheliff, particularly in the region of Oued Fodda, El-Abadia, and El-Attaf. Numerous sand boils were visible. Some of these were 4 meters in diameter (Fig. 2.12). Water spouts up to 2 meters high were reported in many of the sand boil areas (Clough et al, 1981). Partially as a result of liquefaction subsidence, a large earthquake lake (Fig. 2.12) formed southeast of the canyon mouth where the Oued Fodda and Oued Cheliff Rivers join and flow northwestward through the uplands on the upthrown block of the Oued Fodda fault.

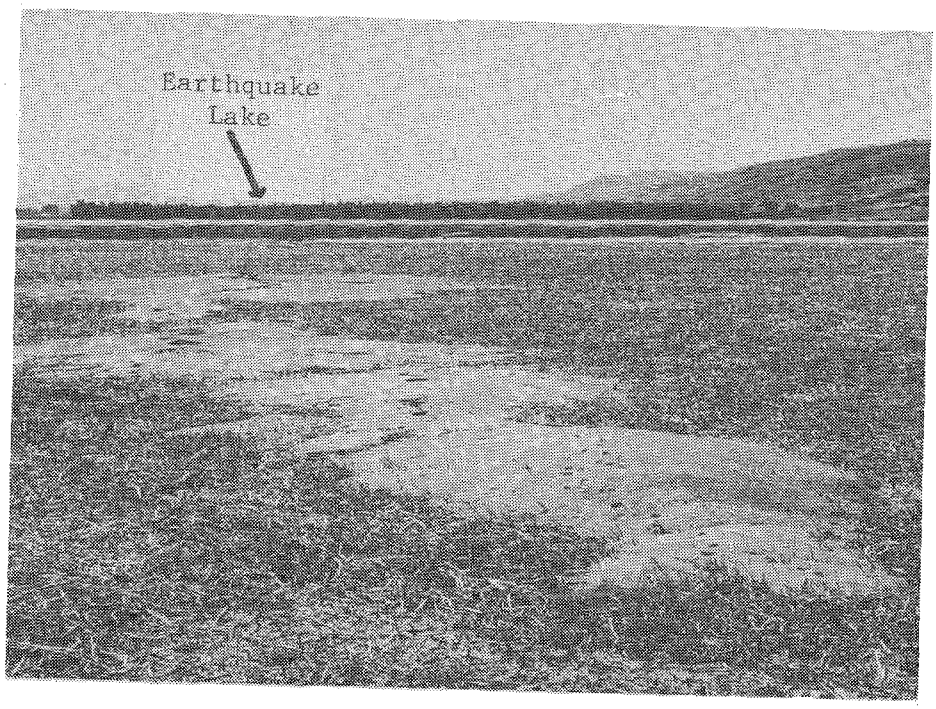


Fig. 2.12 Sand Boils and Earthquake Lake

III. EARTHQUAKE EFFECTS

3.1 General Description of Damage

The locations of the main and aftershock epicenters are shown in Fig. 2.1. As one traveled from Algiers to El-Asnam, the visible damage to engineered structures could be clearly observed from the town of Ain Defla westward. Signs of earthquake damage became increasingly obvious as one approached El-Asnam.

The isoseismal map (Fig. 2.2) indicates that damage caused by the earthquake was most severe in El-Asnam, Sendjas, Oued Fodda, El-Karimia, El-Abadia, Beni Rached, Zeboudja, and El-Attaf. As already noted, the major city in the region affected by the earthquake was El-Asnam, a city of some 15,000 buildings. Many engineered structures collapsed or suffered serious damage. In downtown El-Asnam, an area about six blocks square (Fig. 1.17), at least 20 percent of the buildings collapsed completely during the main shock, another 60 percent were so significantly damaged they had to be demolished, while the remaining 20 percent survived.

CTC conducted a preliminary survey of the entire city of El-Asnam to determine the level of damage to each structure (see section 3.6). All field investigators used the same damage evaluation form (see Appendix B). In mid-November 1980, CTC prepared a table based on survey results which showed the damage pattern to various structures according to their use (Table 3.1). A detailed description of damage is included in CTC's report, *Rapport Général sur le Séisme du 10 October 1980 à Ech-Cheliff,* October 1981.*

Damage was not confined to older structures (category 1, see section 1.6.2). Many modern complexes (categories 2 and 3) also performed poorly. Numerous major buildings in the city collapsed; these included the large multipurpose Cité An Nasr complex, police station, city hall, hall of justice, and main hospital. All lifelines of the city failed.

Modern three to four-story apartment complexes generally fared badly, and several which survived the earthquake required demolition because their structural system was beyond repair. Boucaa Sahnoun, a major residential complex on the southern outskirts of the city which was under construction at the time of the disaster, was almost completely destroyed. The absence of functioning administrative centers (hospitals, police, and city administration), combined with the pressing shortage of housing, meant that the plight of the homeless (estimated at 150,000 in the El-Asnam *wilaya*), was desperate in the first months following the earthquake.

The surface faulting closest to El-Asnam was at first thought to be located about 15 km to the east, near Oued Fodda. Later information located the faulting about 7 km southeast (Fig. 2.3). The city of El-Attaf, the main city in the El-Attaf *daira,* sustained substantial damage to both older and modern construction. However, the city seemed to function with its basic utilities intact.

*El-Asnam has now been renamed Ech-Cheliff.

TABLE 3.1 RESULTS OF PRELIMINARY DAMAGE SURVEY OF ENTIRE CITY OF EL-ASNAM (CTC, mid-November 1980)

Structure Use	Level of Damage, percent		
	No Damage to Light Damage (Green)*	Moderate to Major Damage (Orange)*	Condensed or Collapsed Bldgs (Red)*
Administrative	5	55	30
Multifamily Housing	5	50	45
Single-Family Housing	20	70	10
School	5	25	70
Industrial	80	15	5
Commercial	10	75	15
Hospital	10	60	30
Water Reservoir	50	40	10
Recreation	30	60	10
Socio/Cultural	5	60	35
Overall	22	52	26

*See section 3.6.

El-Abadia, 5 km north of El-Attaf, appeared virtually destroyed; most major buildings were partially or totally collapsed. However, a few older one-story masonry structures, although severely cracked, survived the earthquake. No modern buildings were observed in El-Abadia. It was reported that 600 people were killed by collapsing houses (Fig. 3.1). In the countryside the predominant form of construction is traditional adobe brick with timber rafters and tile or thatch and mud roofing. In some villages near the epicentral region, almost all housing collapsed. In the outlying villages much loss of life occurred. The major cause of death was from falling adobe bricks rather than roofing material (Fig. 3.2).

The village of Beni Rached and farm houses near the epicenter presented a strange phenomenon. The buildings in this region were generally one-story adobe or masonry buildings with tile roofs. Except where ground rupture occurred on the site of the structure, damage was sporadic and not severe. It was not unusual to find one part of a building that had not been destroyed by ground rupture virtually undamaged (Fig. 3.3). Cracks in masonry walls and displaced or fallen roof tiles were common, but total collapse was rare.

It is well known that damage to structures can be caused by different earthquake effects, usually classified as direct or indirect effects. The seismic effect of concern to the structural engineer is the response (vibration) of the structure to ground shaking at its foundation (see section 3.5). In many earthquakes, damage resulting from other earthquake effects, such as faulting, subsidence and tilting, liquefaction, landslides, fire, flooding, etc., exceeds that caused by structural vibration. However, in the 1980 El-Asnam earthquake, nearly all damage observed resulted from the response of structures (particularly buildings) to ground shaking.

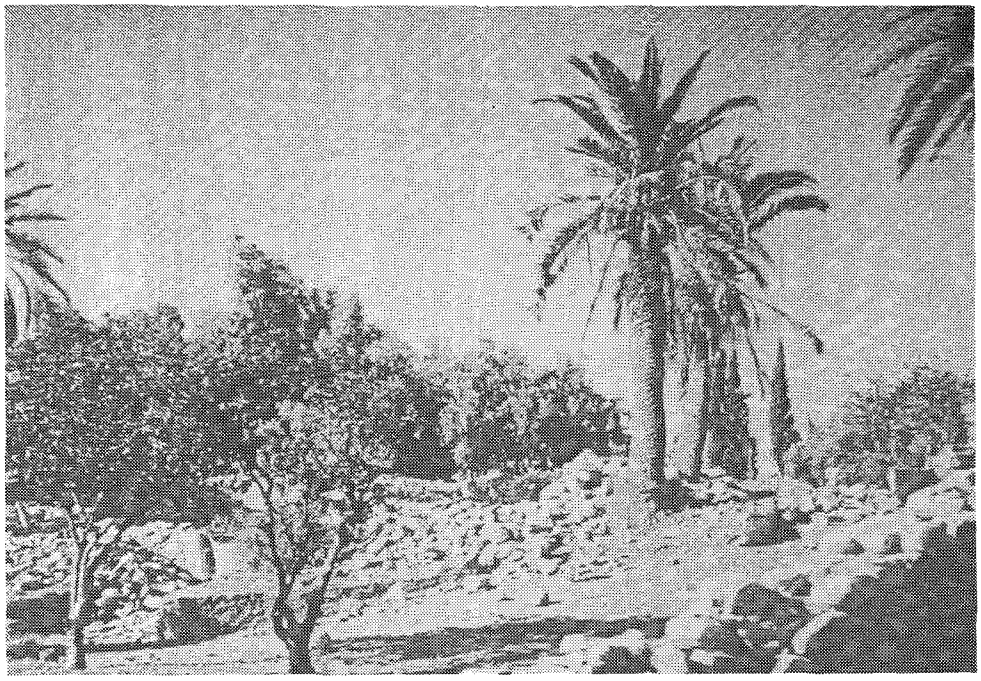


Fig. 3.1 Debris from Collapsed Houses, El-Abadia

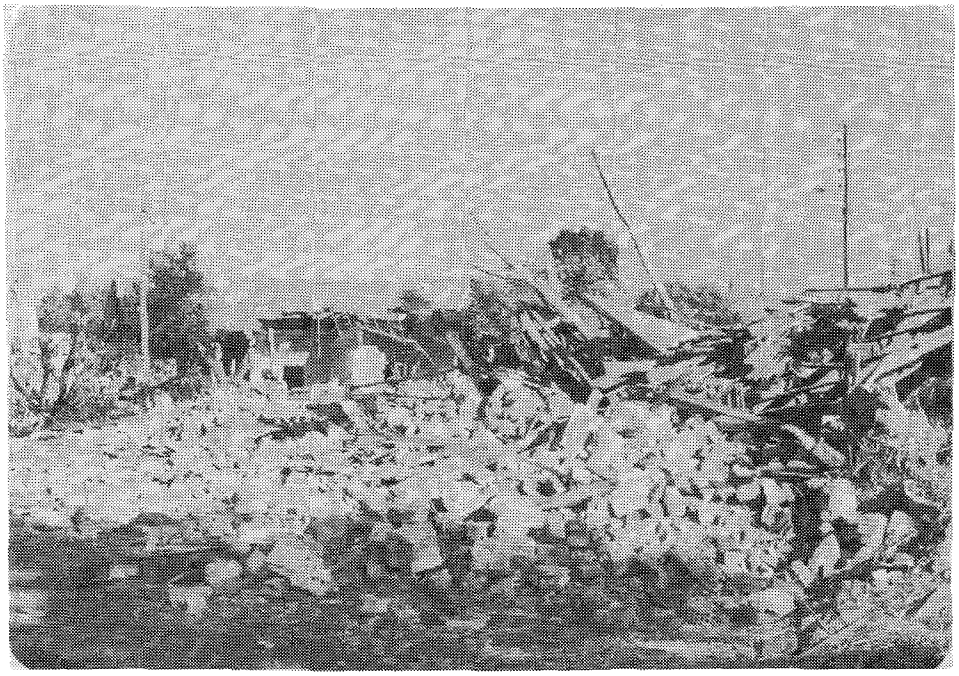


Fig. 3.2 Collapsed Stone and Adobe Huts

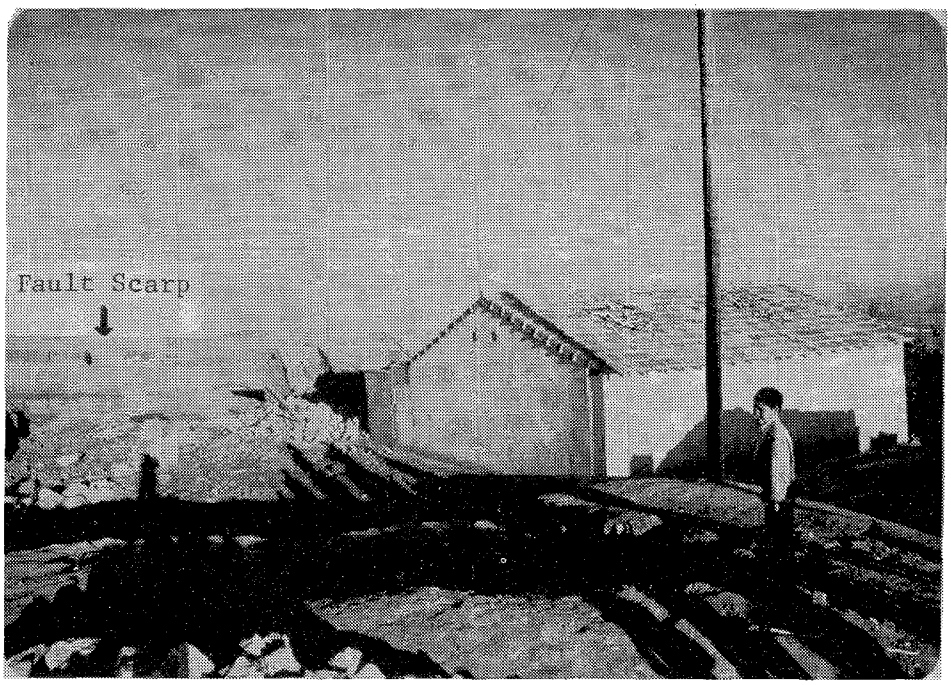


Fig. 3.3 Unreinforced Masonry House with Loose Tiles That Did Not Suffer Major Damage. Note fault rupture in background.

3.2 Surface Faulting

As stated above, the major damage from the El-Asnam earthquake was caused by ground shaking. But some damage to buildings and lifelines in the region because of surface faulting was observed.

3.2.1 Aqueducts and Irrigation Distribution Systems. An irrigation water main, 1 meter in diameter, crosses the Oued Fodda fault at an angle of about 70°. This water main ruptured because of the vertical and lateral fault displacement. Figs. 3.4 through 3.6 show the damage.

The Oued Fodda and Oued Cheliff are the two rivers used extensively for irrigation. After the earthquake, a lake approximately 2 sq km formed southeast of the canyon mouth where the rivers join and flow northwestward through the uplands on the upthrown block of the Oued Fodda fault (Figs. 2.12 and 3.7). This earthquake lake submerged many acres of fertile orchards and farmland and appears to be the result of a combination of factors including: 1) tectonic downwarping of the downthrown block parallel to the fault, 2) uplift of the fault block on the northwest (resulting in tectonic damming of the river), and 3) subsidence due to differential settlement and possible liquefaction.

3.2.2 Buildings. In the village of Beni Rached (Fig. 3.8), a large number of stone houses and adobe huts that were either on the fault or within 50 meters of the fault collapsed. However, a few houses and a mosque, constructed of unreinforced masonry or adobe with loose tiled roofs, sustained surprisingly little or no damage although they were within 50 meters of the fault trace (Figs. 3.3 and 3.9).

Fig. 3.4 Irrigation Water Main Ruptured by Secondary (Normal) Trace of Oued Fodda Fault

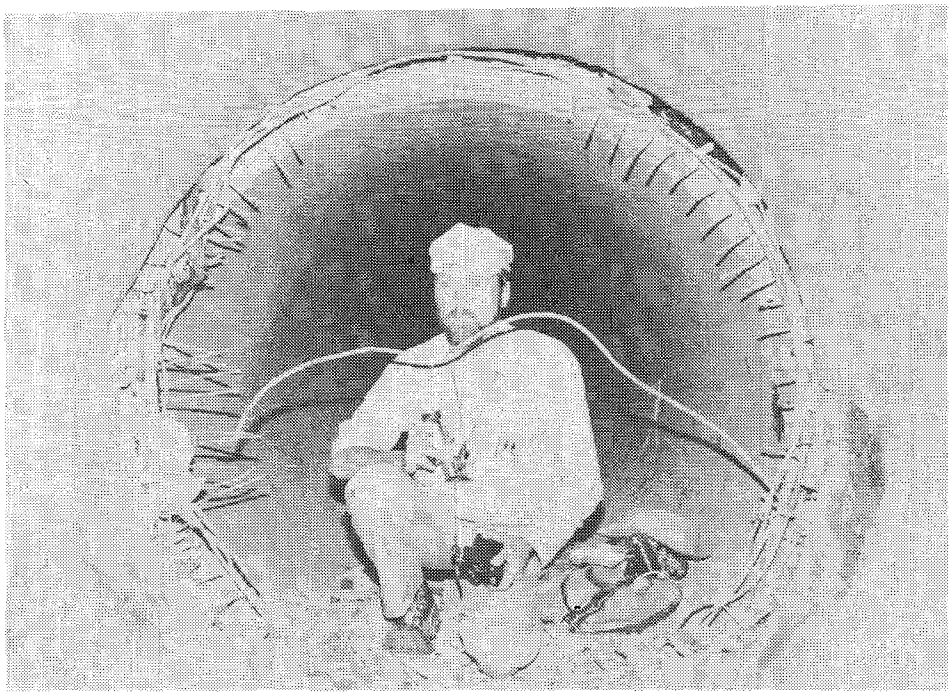


Fig. 3.5 Inside of Irrigation Water Main Where It Crosses Main Trace (Thrust) of Oued Fodda Fault. Compression has shortened steel RC pipe about 0.5 meter where two pipe segments join.

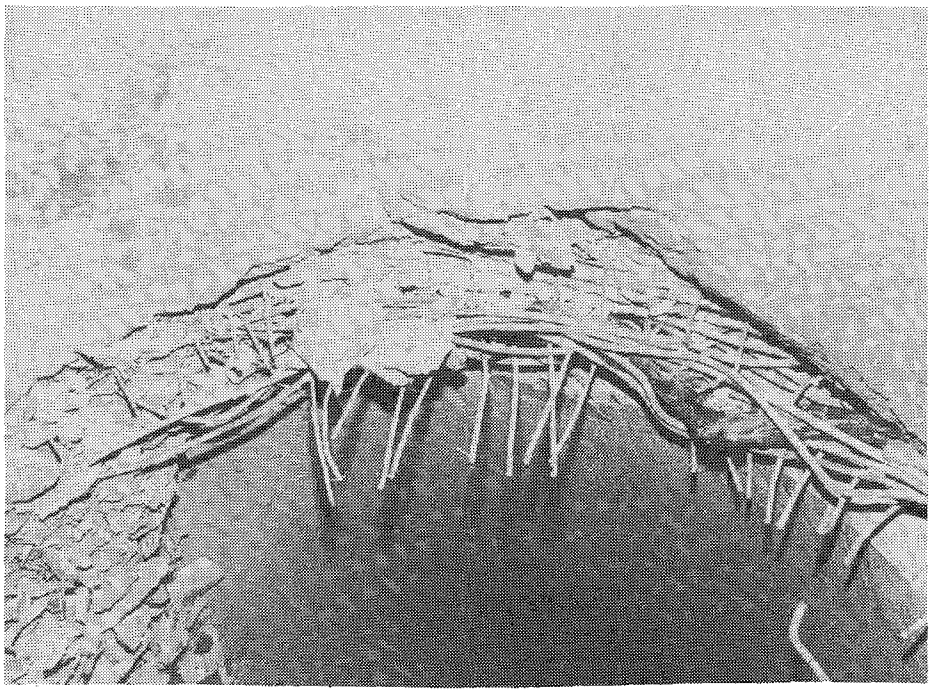


Fig. 3.6 Closeup of Damaged Irrigation Main Shown in Fig. 3.5

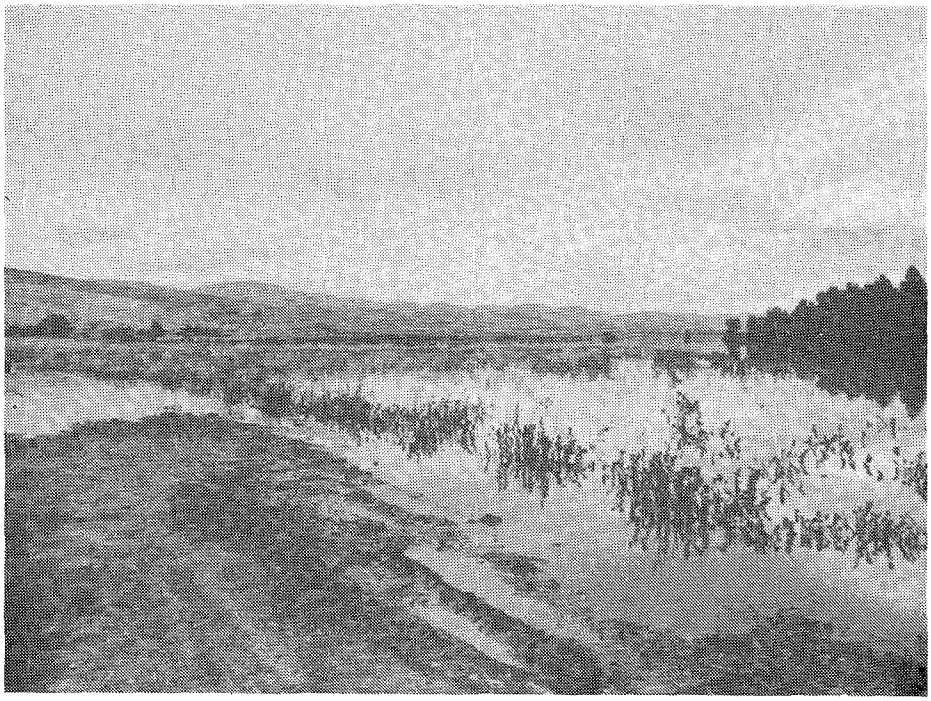


Fig. 3.7 Earthquake Lake

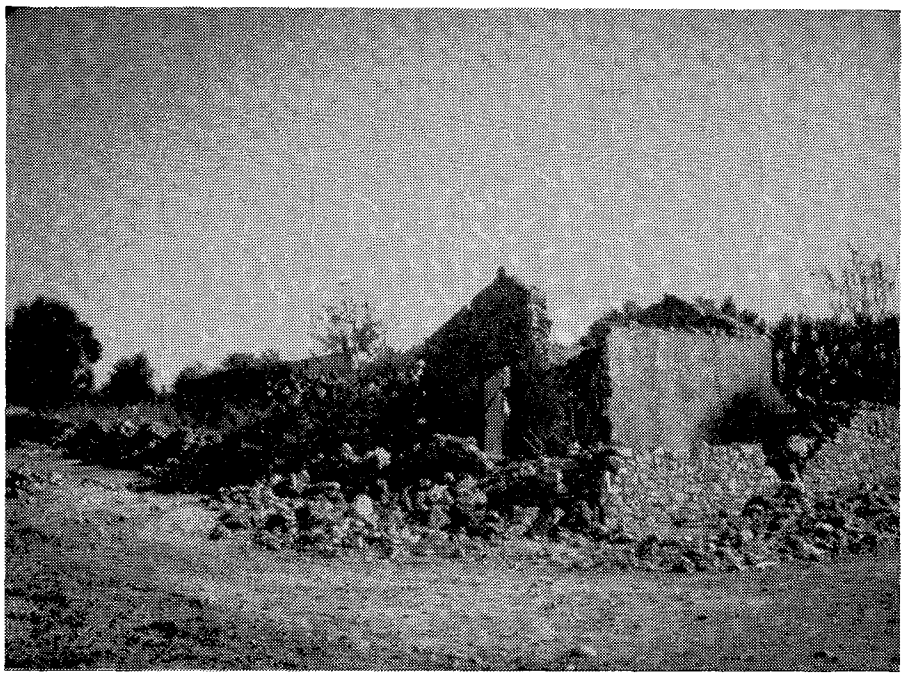


Fig. 3.8 Collapsed Stone Houses and Adobe Huts, Beni Rached

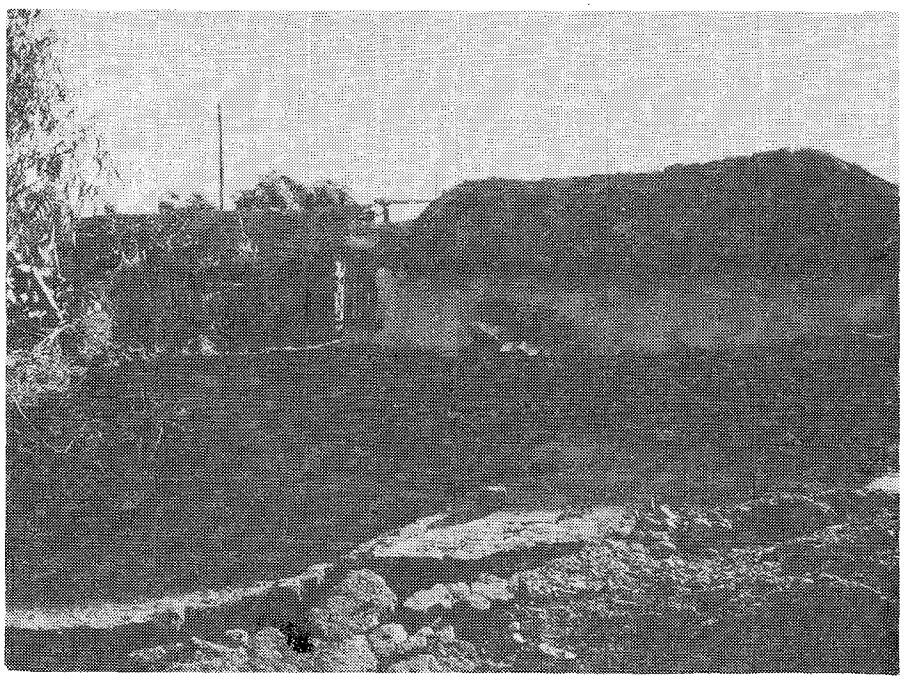


Fig. 3.9 Adobe Hut with Thatched Roof That Did Not Collapse. Note ground failure in foreground. This hut is barely 30 meters from fault break.

3.2.3 Highways and Railroads. The east-west railroad between Algiers and Oran and passing through El-Asnam crosses the main thrust of the Oued Fodda fault about 15 km east of El-Asnam outside the town of Oued Fodda. At the time of the main shock a train going from El-Asnam to Algiers was straddling the fault. The train was completely overturned (Figs. 3.10 and 3.11). It took about seven days before the work crews could clear the wreckage and reopen the railway. At another location, railroad tracks were bent (Fig. 3.12). Although it has been suggested that the rails buckled under seismic stress (Papastamatiou, 1980), it is suspected that a secondary fault may have caused this bending.

The main highway from Algiers to El-Asnam (Highway 4) was extensively cracked and broken where it crossed the region of fault rupture. The width of the zone in which the road was broken was about 0.5 km. Since all emergency and rescue operations from Algiers and other communities east of El-Asnam had to use this highway, the government was forced to construct a temporary road immediately after the earthquake. In general, road traffic was not interrupted although it was slowed because of frequent pockets of heavy damage to the roadway.

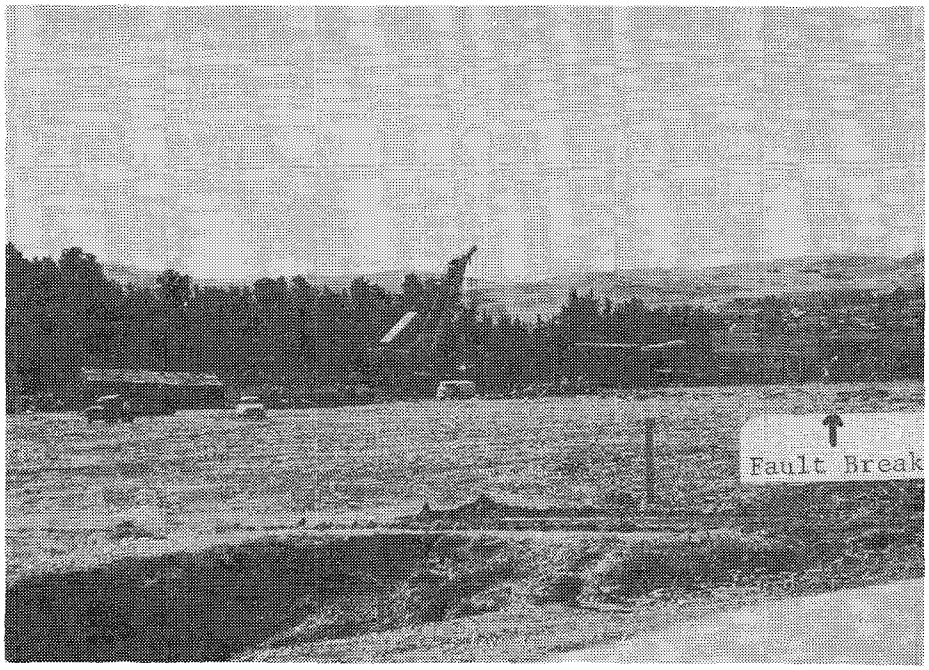


Fig. 3.10 Overturned Train. Note fault break.

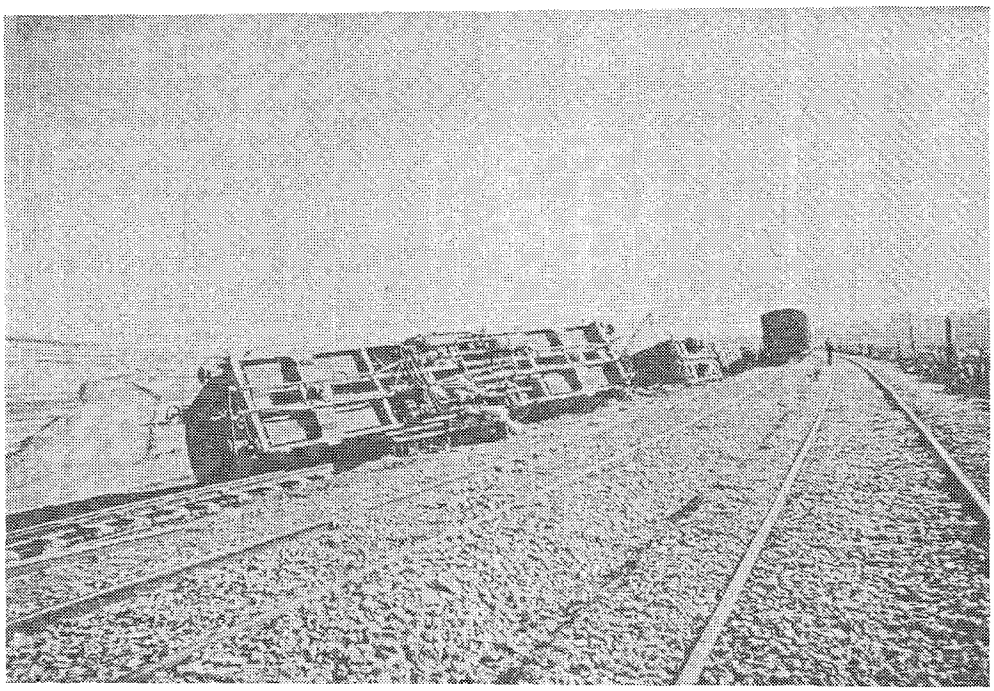


Fig. 3.11 Overturned Train

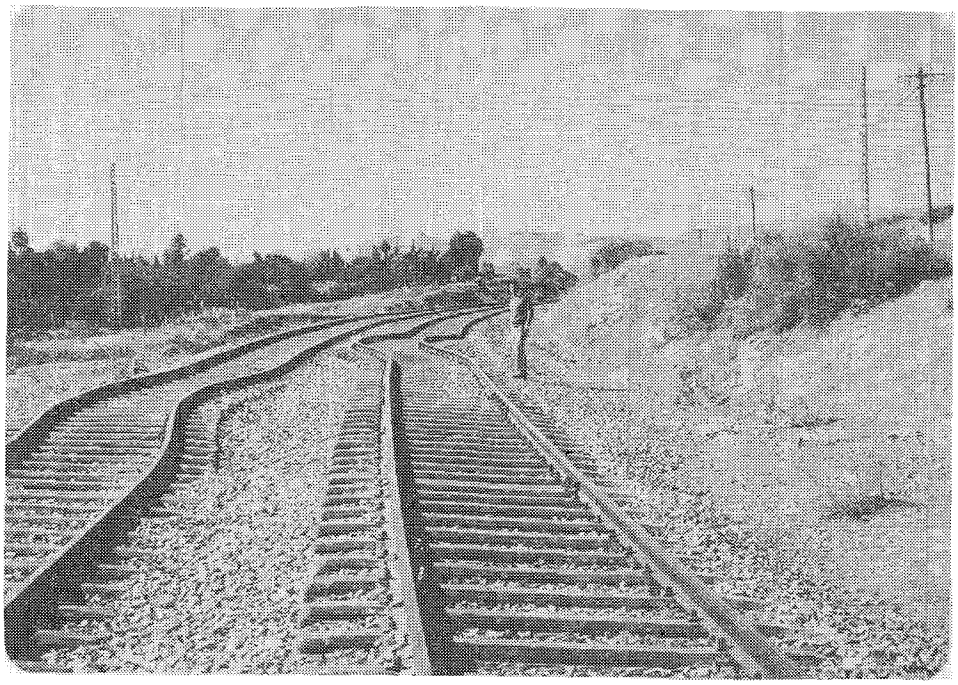


Fig. 3.12 Bent Rails Between El-Asnam and Oued Fodda

3.3 Tectonic Subsidence and Tilting

The only visible effect of tectonic subsidence and tilting was formation of the earthquake lake (see section 3.2.1). This lake (Fig. 3.7), which formed near the road leading from the village of Vauban northwesterly toward the Oued Cheliff River, grew very fast. By December 1980, the area occupied by the lake had grown fivefold since viewed by the reconnaissance team three days after the earthquake, and its width near Vauban was about 5 km. Along the perimeter of the lake near the base of the main east-west railroad embankment in December 1980 it was observed that the lake was growing at the rate of 7 m/hr (21 ft/hr). Field mice and snakes were seen fleeing ahead of the water toward higher ground.

3.4 Ground Failure

Extensive evidence was found of ground disturbance or failures due to liquefaction, landslides, or differential settlement. In El-Asnam no direct problems leading to structural damage could be attributed to liquefaction or landslide phenomena. One possible exception was the road damage north of the Cité An Nasr market (Fig. 1.17). Lurching of the steep embankment around the Oued Cheliff caused pavement and retaining wall damage. It is difficult to estimate the extent to which this damage affected the collapse of the market.

Differential settlement between bridge structures and their abutments was responsible for damaging the approaches to most of the bridges in the El-Asnam and Beni Rached area.

3.4.1 Liquefaction. Most of the liquefaction occurred in the Oued Cheliff and Oued Fodda flood plains. These are essentially rural agricultural areas. Clough et al (1981) describe liquefaction in and around El-Asnam. The liquefaction sites were marked by sand boils, lateral ground displacement, subsidence, and lurching (Figs. 3.13 through 3.15). At one location near Highway 4 near the town of Oued Fodda, sand boils broke through a thick clay overburden, leading to lurching and subsidence of the ground. Within the city of El-Asnam, most of the liquefaction was confined to the Oued Cheliff flood plain. Some unusual phenomena were also observed:

1. Trees sinking on Oued Cheliff flood plain in El-Asnam. Passage of linear sand boils between trees (Fig. 3.16).
2. Sand boils forming through a clay overburden with a thickness of more than 6 meters.
3. Water spouts up to 2 meters occurring in many of the sand boil areas.

Clough et al give results of grain size analysis for samples taken from different locations around El-Asnam that did liquefy. Fig. 3.17 shows locations where soil samples were taken for analysis. Liquefaction was also studied at these sites. Table 3.2 shows the results of grain size analysis.

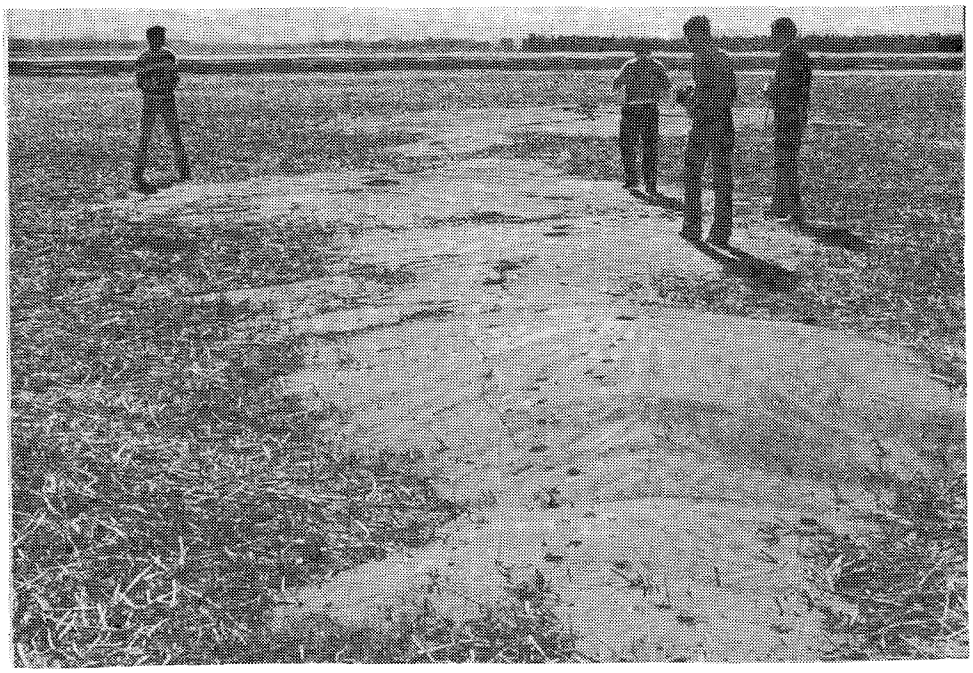


Fig. 3.13. Sand Boils at Site 3 in Vicinity of Earthquake Lake Immediately After Earthquake. Site locations are identified in Fig. 3.17.

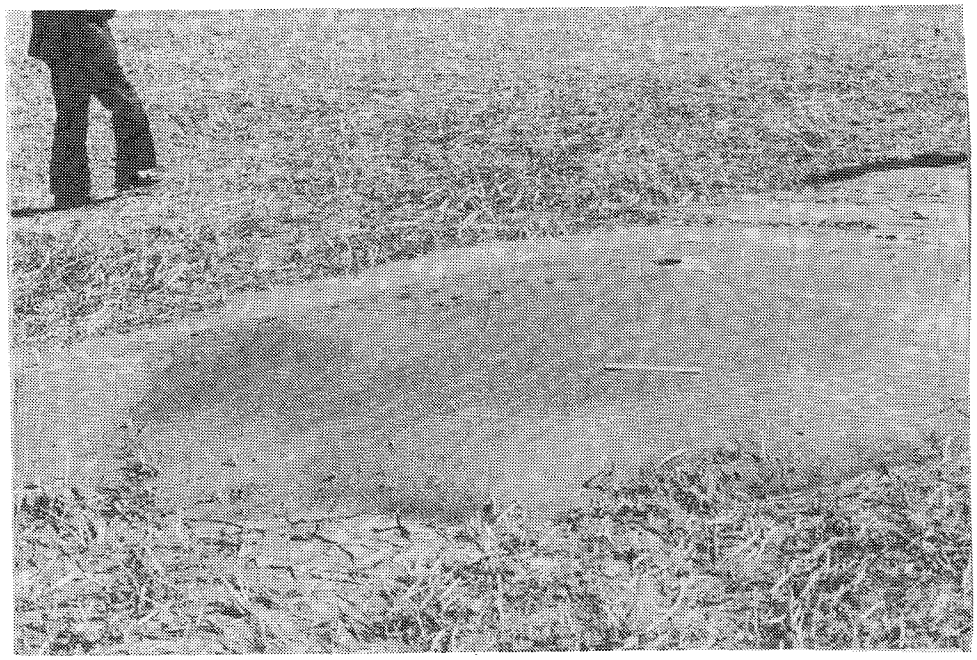


Fig. 3.14 Sand Boil at Site 3 Immediately After Earthquake

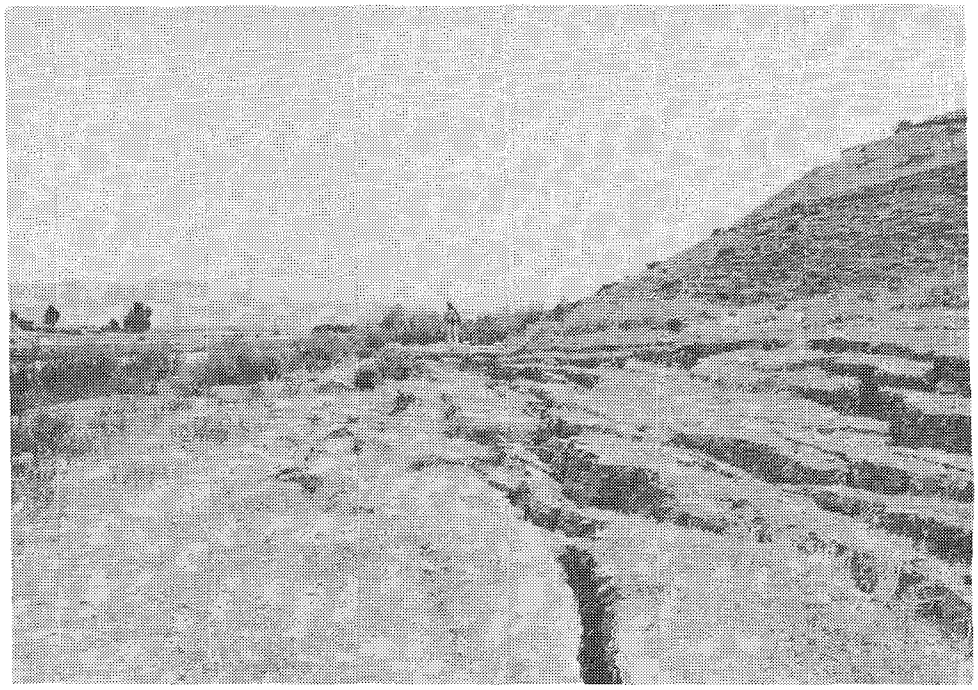


Fig. 3.15 Fissures and Fault-like Scarps Produced by Lateral Spreading of Alluvium Toward Oued Cheliff Channel

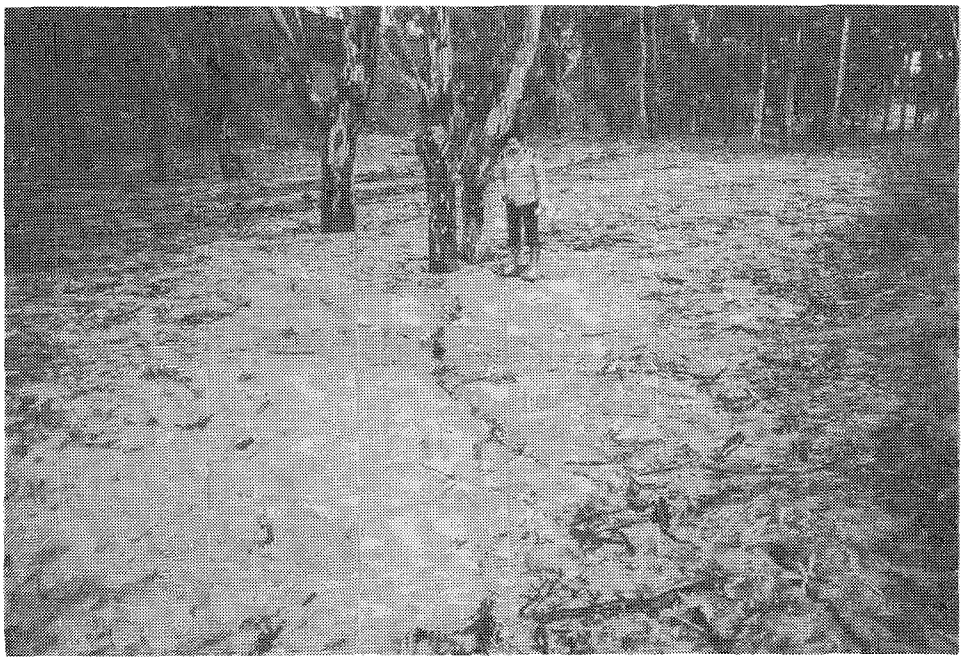


Fig. 3.16 Sand Boil Along Ground Crack at Site 2 Terminating at Tree Trunk. Note depressions around trees.

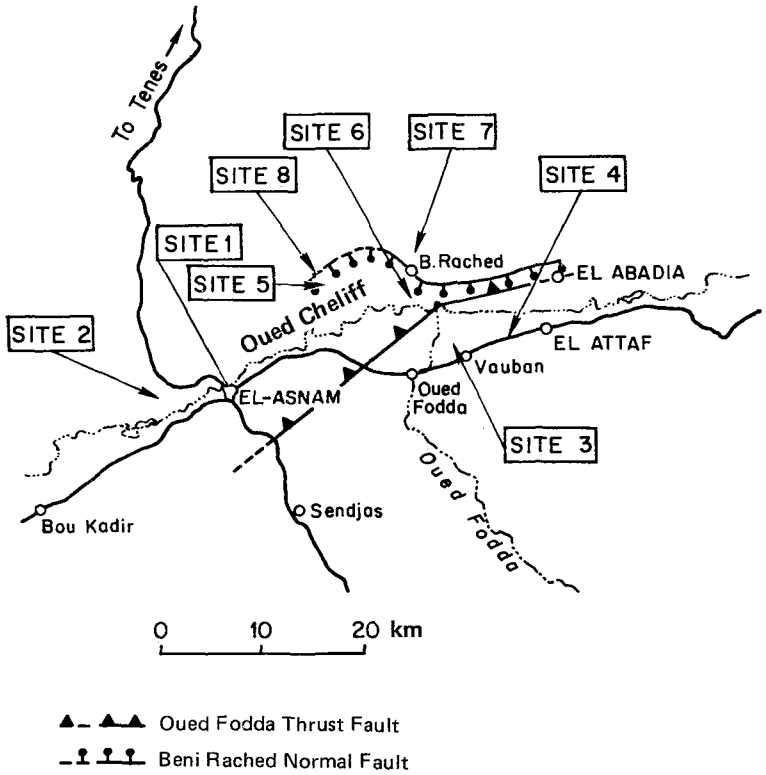


Fig. 3.17 Surface Faults and Sites Where Liquefaction or Landsliding Was Observed

TABLE 3.2 DIGEST OF RESULTS OF GRAIN SIZE ANALYSES OF SAND BOIL MATERIALS

Site No.	Location	D_{50}, mm	Uniformity Coefficient, C_u	% Passing #200 Sieve	Classification*	Comments
1	Bank of Oued Cheliff Inside El-Asnam	0.27	1.8	4	SP	Sample of Sand Boil
2	Bank of Oued Cheliff Village of Boutaiba	0.16	2.2	8	SP-SM	Sample of Sand Boil
3	Earthquake Lake, Vauban	*	*	*		No Sample Could Be Obtained
	Along Main Hwy to Algiers					
4a	(a) Right of Hwy	0.10	2.2	27	SM	Sample of Sand Boil
4b	(b) Left of Hwy	0.15	2.6	13	SM	Sample of Sand Boil
	El Aukayat, 4 km West of Beni-Rached					
5a	(a) Source Matl., Red Sand	0.22	2.4	8	SP-SM	Sample from Subsidence Scarp - Red Sand
5b	(b) Source Matl., Yellow Sand	0.16	2.1	8	SP-SM	Sample from Subsidence Scarp - Yellow Sand
5c	(c) Sand Boil, Red Sand	0.20	2.3	7	SP-SM	Sample of Red Sand Boil
5d	(d) Sand Boil, Yellow Sand	0.15	2.5	12	SP-SM	Sample of Yellow Sand Boil

*United Soil Classification System
SP - Poorly graded clean sands, sand gravel mix
SM - Silty sands, poorly graded sand-silt mix

3.4.2 Landslides. The landslides that occurred during the earthquake were generally within 10 km of the major fault systems. Fortunately, they were also in sparsely populated rural areas. The most startling occurrence was the reactivation of a large old landslide near Beni Rached which led to the generation of a geyser of water. According to several villagers, this geyser spouted 20 to 40 meters into the air. This reactivated landslide is immediately north of the Beni Rached fault, near site 7 (Fig. 3.17). A sketch of the site, designated as site 8, is given in Fig. 3.18.

The old landslide mass occupies an area about 1 km^2 with a 10 to 15 meter scarp at the head of the slide. This scarp has an arcuate shape and a length of nearly 2 km. The earthquake activated the whole slide mass, as evidenced by a fresh 1 meter wide subsidence crack around the entire base of the old scarp (Fig. 3.19).

The landslide mass extended outward for about 150 meters. The surface of this mass was highly irregular and in a number of areas tilted back into the mountain, as shown in Fig. 3.20. At the rear of the back-tilted zones, the soil was wet and soft, with a conspicuous growth of green grasses. Several sinkholes into which water drained were observed. Springs existed at two locations along the downslope flanks of the back-tilted mass.

Several villagers explained that at the time of the earthquake the area shook strongly and shortly thereafter a water geyser appeared from the north side of the landslide mass, shooting 20 to 40 meters into the air. A circular crater having a diameter of 1.5 meters was found from which the water geyser had erupted (Fig. 3.21). The soil in the crater was saturated and depressed below the lip, supporting the story of the water geyser.

Subsequently, a number of debris flows were triggered at the crest of the old landslide mass; these flowed down into a ravine, and the residual failure masses were observed approximately 400 meters downslope. Numerous other grassy hummocks of ground existed in the same vicinity as the fresh debris flows, suggesting that this had occurred before.

A postulated mechanism for the unusual geyser and debris flows is depicted in Fig. 3.20. Before the earthquake, the sinkholes on the back-tilted areas served as funnels for water that seeped below the landslide mass. Under normal circumstances the water flowed out at the flanks of the landslide in the form of springs; however, during the earthquake, the landslide mass as a whole slipped downward, as evidenced by the new subsidence scarp around the head of the landslide. This large-scale movement was likely due, in part, to the lubricating effect of the long-standing seepage under the landslide. The sudden downward thrust of the landslide mass impacted the water-laden soils under it and generated exceedingly high excess pore pressures. Under this unusual head, the water flowed rapidly upward through the overburden and, in one location (probably a cracked or weak zone), burst forth as the geyser. The remainder of the surface soil, now saturated and subjected to a high seepage condition, essentially liquified and moved rapidly downslope in the form of debris flows.

Because general conditions at this site remain unchanged, it is likely that the behavior observed in the 1980 earthquake will be repeated in future seismic events. The villagers noted that a significant landslide had occurred in the same location during the 1954 Orléansville (El-Asnam) earthquake.

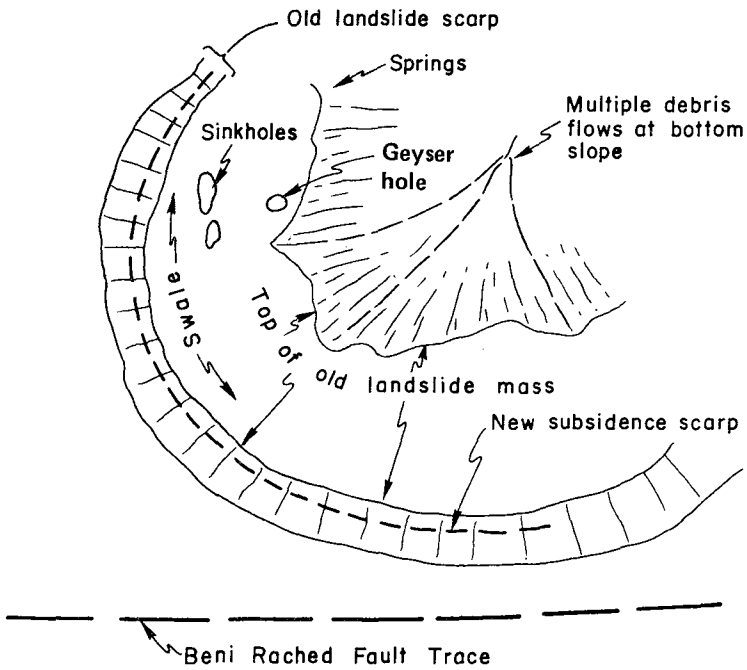


Fig. 3.18 Landslide Area at Site 8 (see Fig. 3.17)

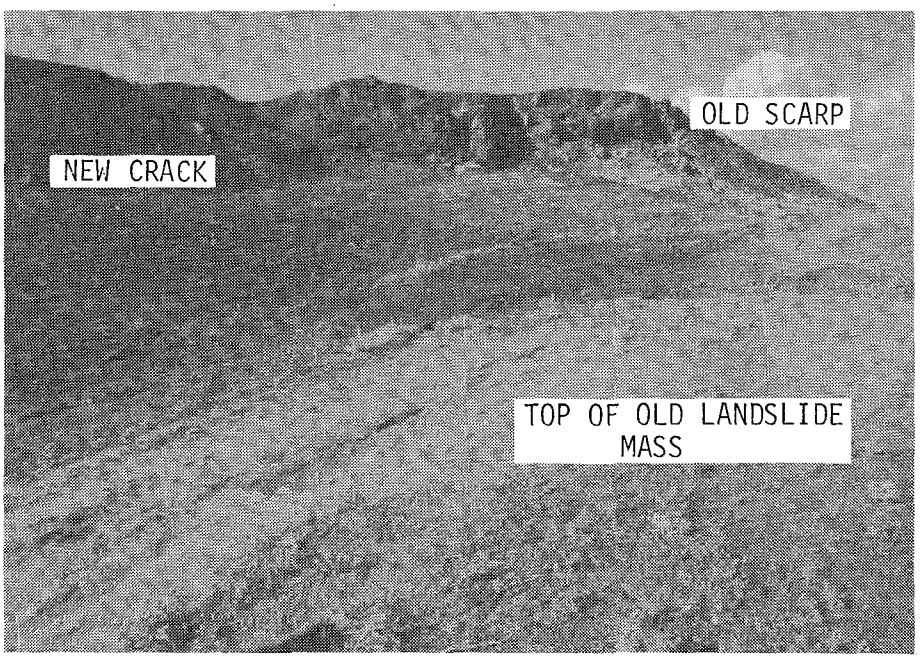


Fig. 3.19 Old Landslide Scarp, New Subsidence Crack, and Top of Old Landslide Mass at Site 8

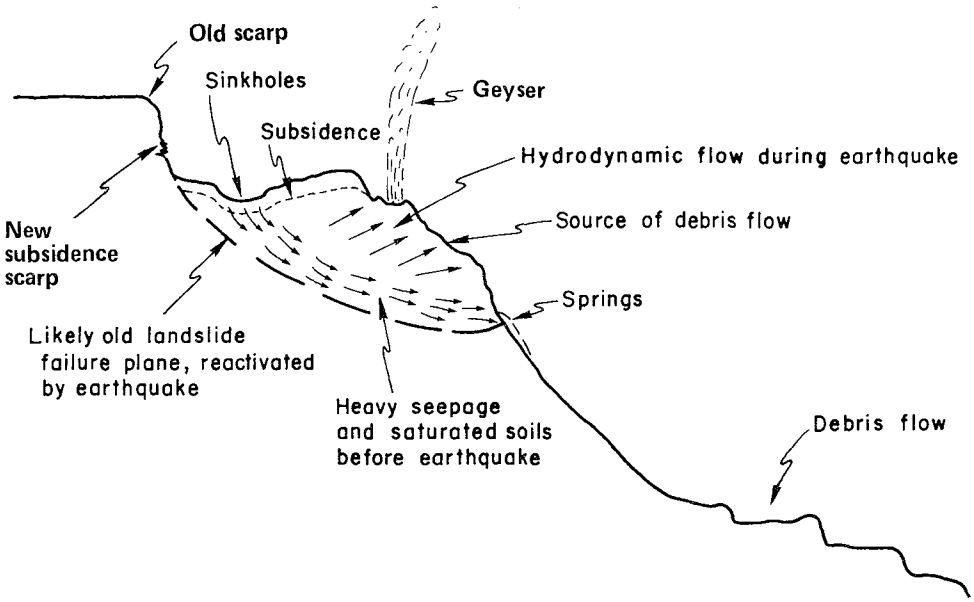


Fig. 3.20 Mechanism for Formation of Geyser and Small Flowslides at Site 8 During Earthquake (see Fig. 3.17)

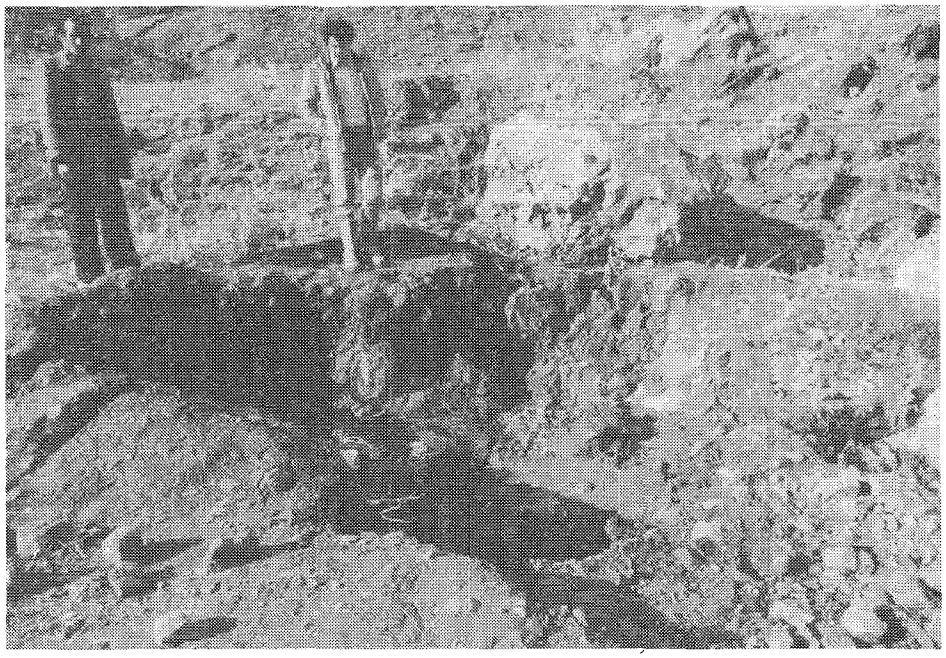
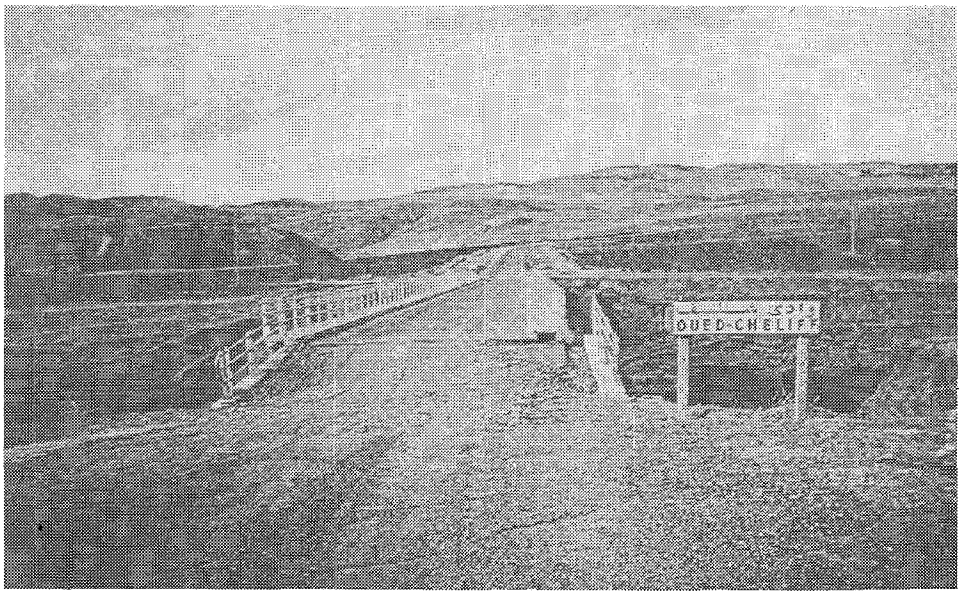
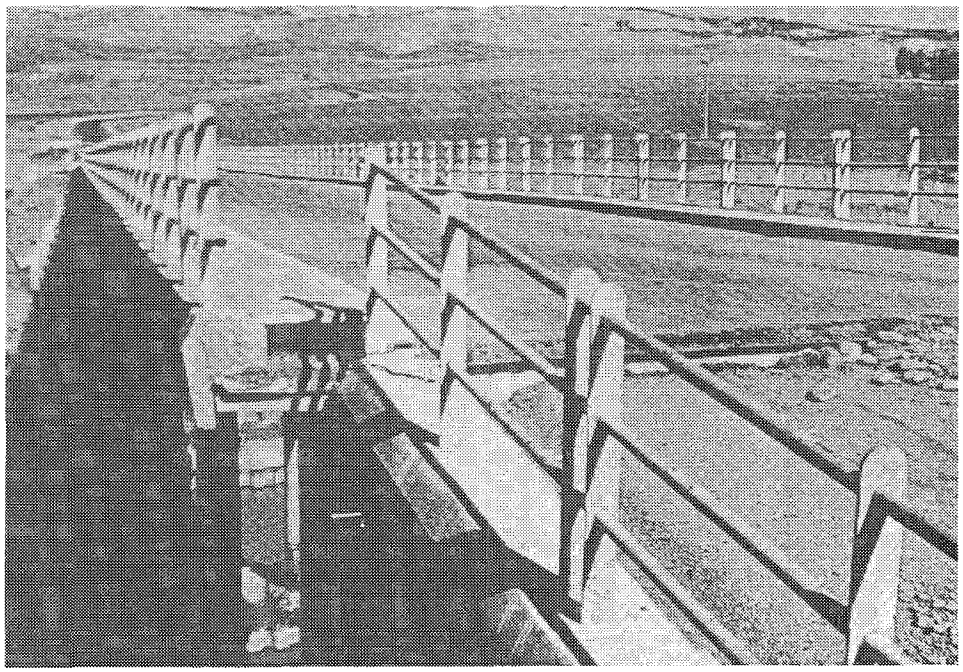


Fig. 3.21 Geyser Crater at Crest of Old Landslide Mass at Site 8

3.4.3 Differential Settlement. Most bridge abutments settled during the earthquake, thereby damaging bridge approaches. The differential settlement was jointly due to lurching, liquefaction, and uneven compaction. Fig. 3.22 shows a bridge over the Oued Cheliff that sustained considerable damage. The abutments of this bridge moved 1 meter horizontally with respect to the bridge superstructure. This resulted in considerable damage to end supports.

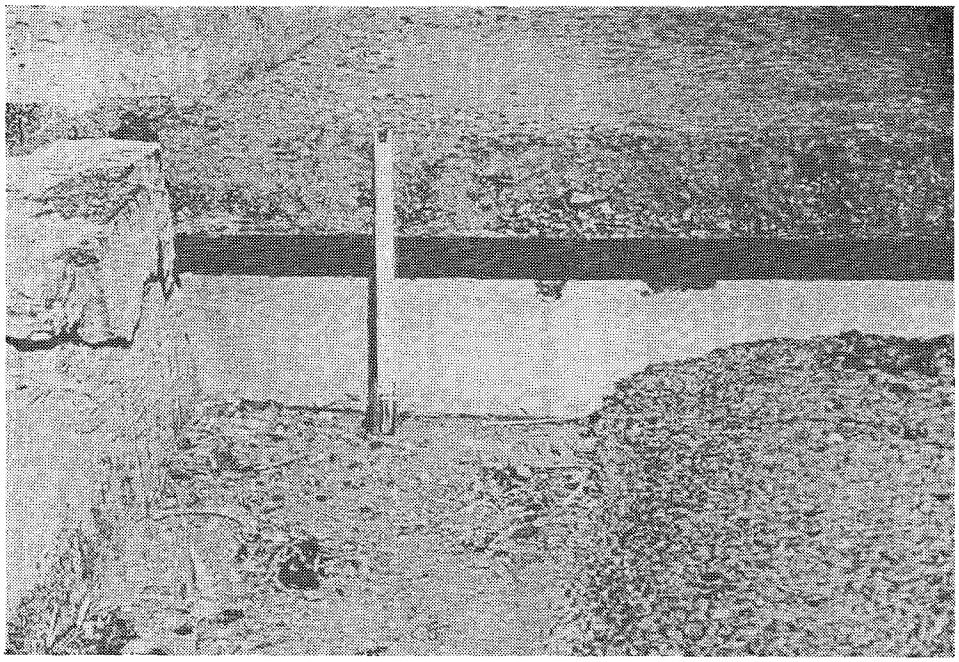


(a) Overall View

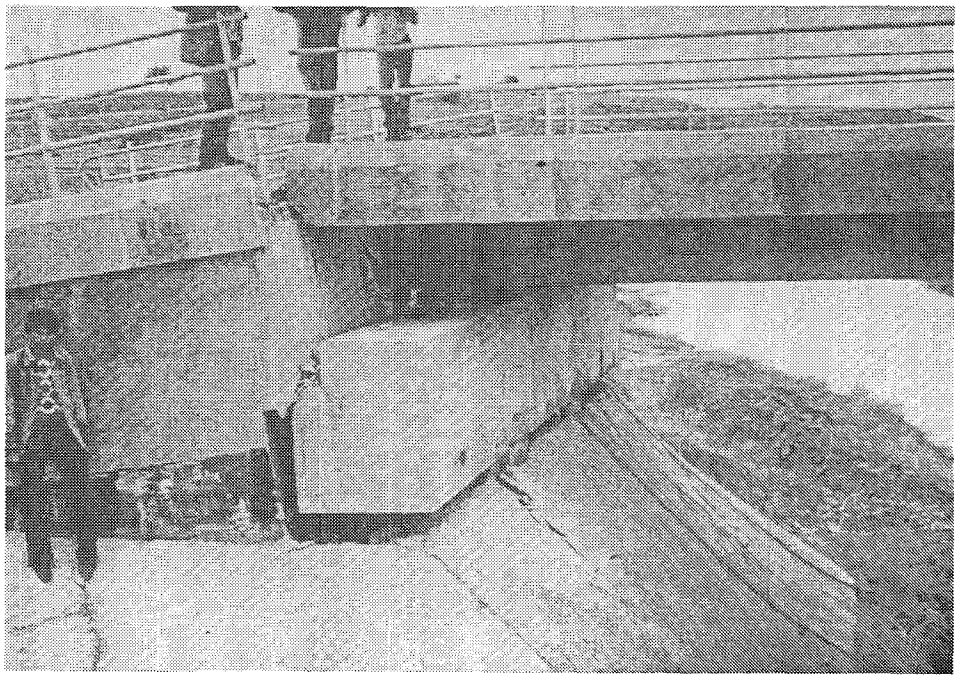


(b) Horizontal Movement

Fig. 3.22 Abutment Settlement of Bridge Across Oued Cheliff



(c) Vertical Movement



(d) Damage at End Support

Fig. 3.22 (cont.)

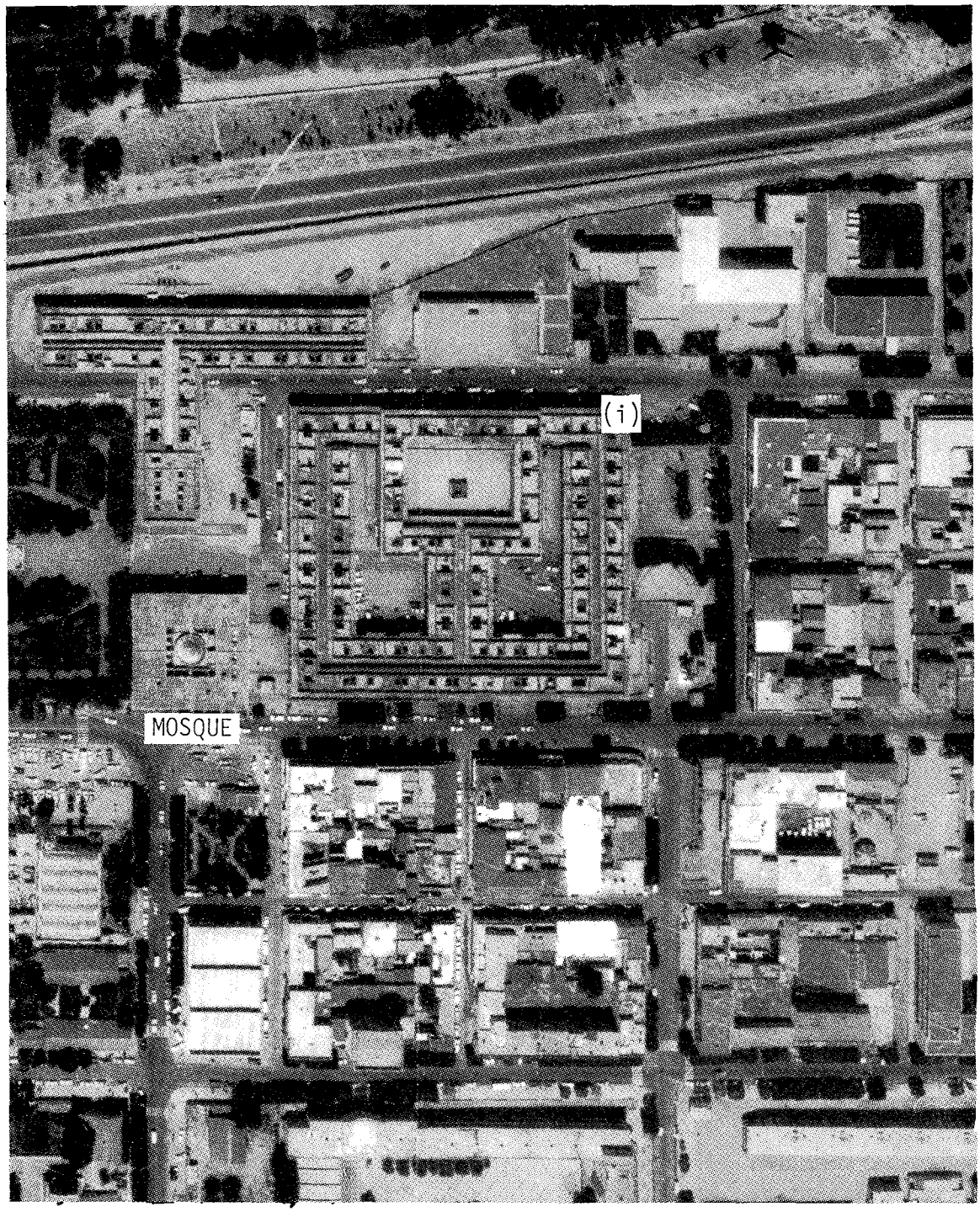
3.5 Strong Ground Motion (Shaking)

To reiterate, most of the damage observed resulted from the response of structures to ground shaking. The most dramatic damage and widespread destruction from this earthquake occurred in the downtown area of El-Asnam (Fig 1.17). El-Asnam was a modern city, as noted in section 1.6, because the majority of buildings were built after the destructive earthquake of 1954. Close to 80 percent of the buildings in this area failed — collapsed or suffered such severe damage that they had to be demolished. There were entire blocks of more than 100 x 100 meters (328 x 328 ft) in which most buildings collapsed and the few that remained standing required demolition.

An example is the Cité An Nasr market (Nos. 14 and 18 in Fig. 1.17), which covered an area of 150 x 150 meters (492 x 492 ft) and was a complex of two to three-story buildings. These collapsed completely (pancaked) except for part of the mosque (Bldg 18 in Fig. 1.17) and one corner building which was barely standing, marked (i) in Fig. 3.23 and illustrated in Fig. 3.24. The collapse of these buildings caused a large number of deaths. Another example is the block where the Hotel du Cheliff and the hall of justice were situated (Nos. 3 and 4 in Fig. 1.17); after the quake only a small portion of the hall of justice remained standing in this block (Fig. 3.25). The main reasons for these failures are discussed later in section 4.3.1.

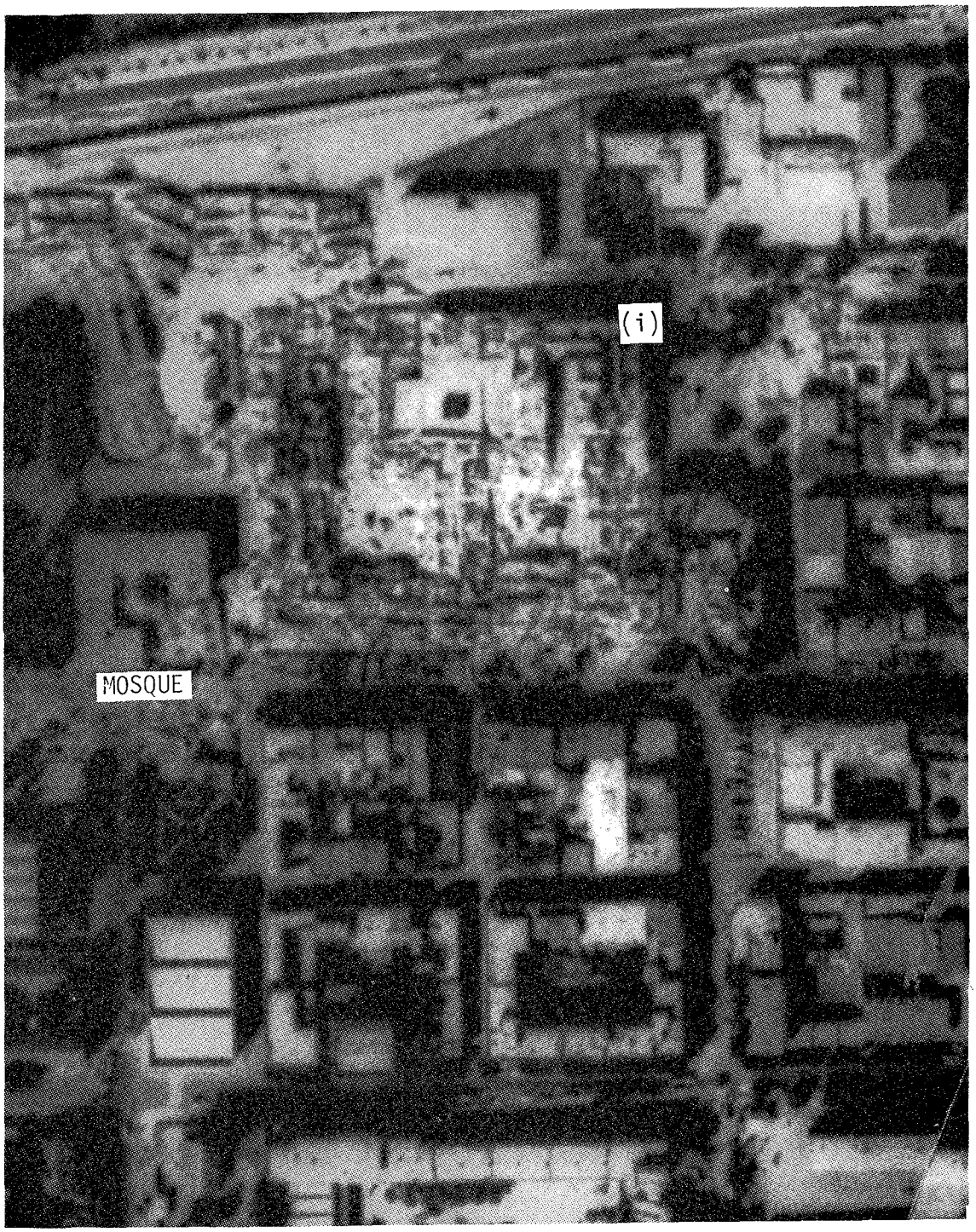
3.5.1 Hospitals, Schools, Fire and Police Stations, Administrative Buildings, and Small Commercial and Residential Buildings. The majority of building units of the city hospital in El-Asnam failed (No. 17 in Fig. 1.17). A new clinic under construction at the time of the earthquake collapsed (Fig. 3.26). Most schools (70 percent, see Table 3.1) collapsed (Fig. 3.27) or suffered sufficient structural damage to require demolition. The disaster planning and fire station was in ruins (Fig. 3.28). Part of the police station collapsed (Fig. 3.29), as did part of the city hall (Fig 3.30). Most hotels failed (Fig. 3.31). The majority of commercial buildings collapsed or suffered so much damage that demolition was required (Figs. 3.32 through 3.35).

Most residential buildings collapsed (Figs. 3.36 and 3.37) or suffered such serious damage that evacuation was required. All city utilities were so severely damaged that three weeks after the earthquake their services had not yet been reestablished. Most of the 125,000 inhabitants of El-Asnam were left without housing and had to live in temporary tents (Fig. 3.38).



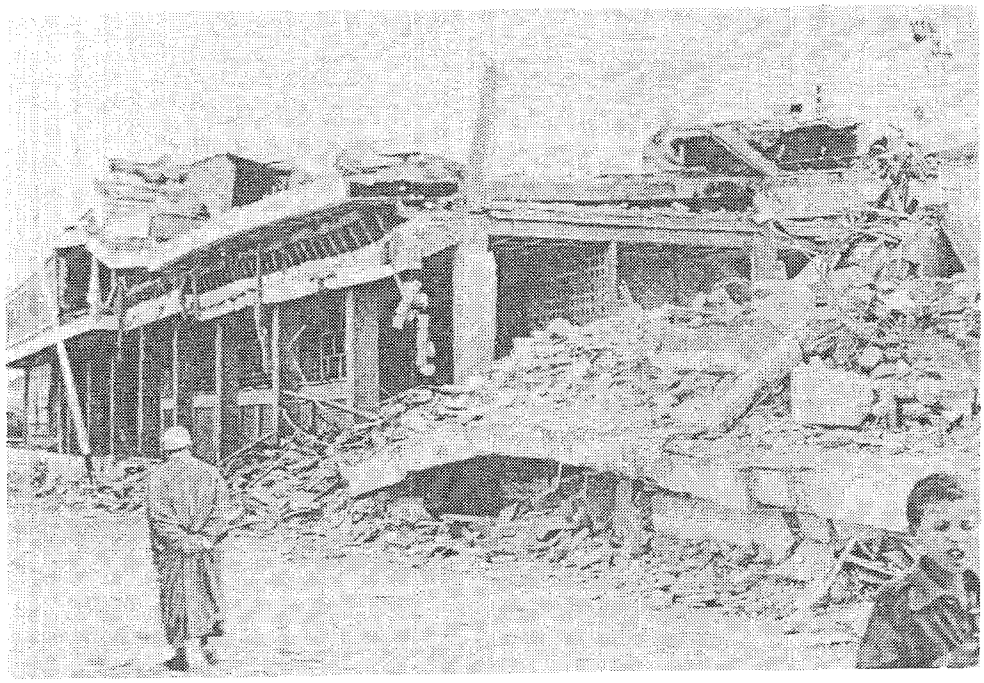
(a) Before Earthquake

Fig. 3.23 Aerial View of Cité An Nasr

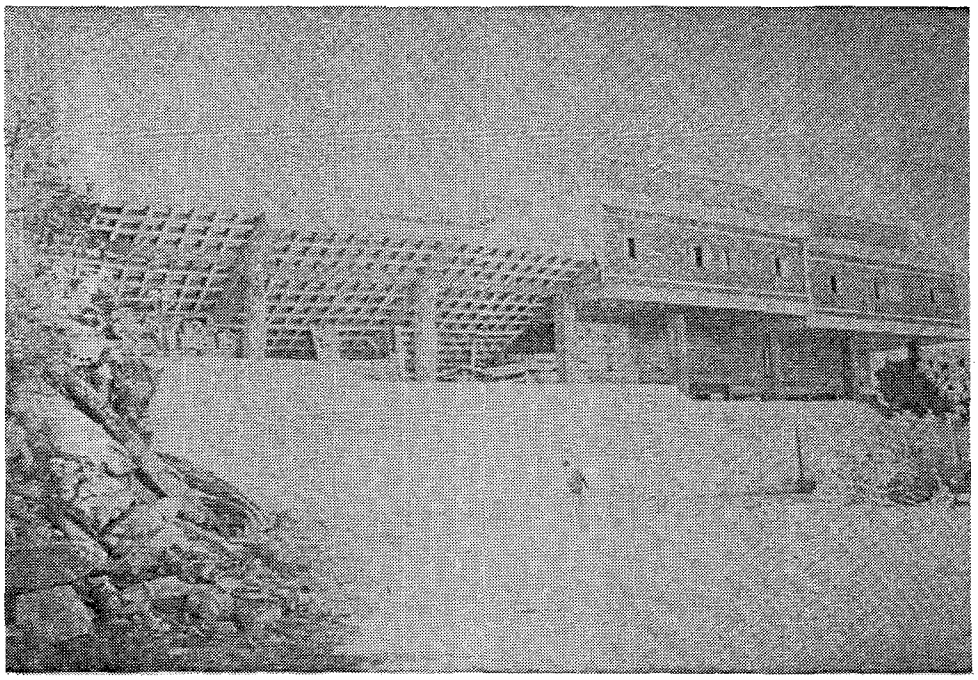


(b) After Earthquake

Fig. 3.23 (cont.)

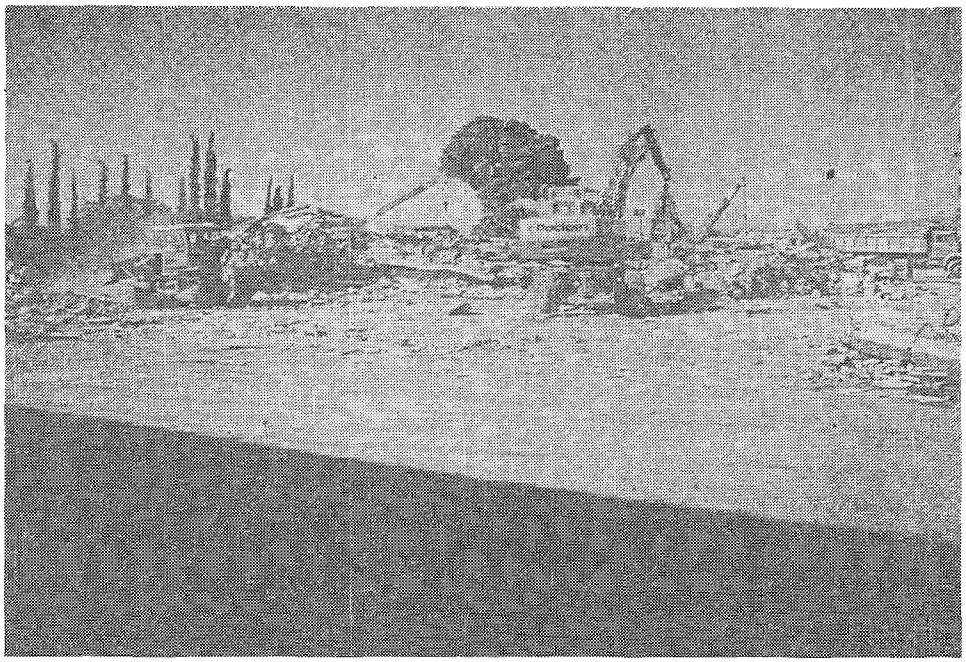


(a) North Side

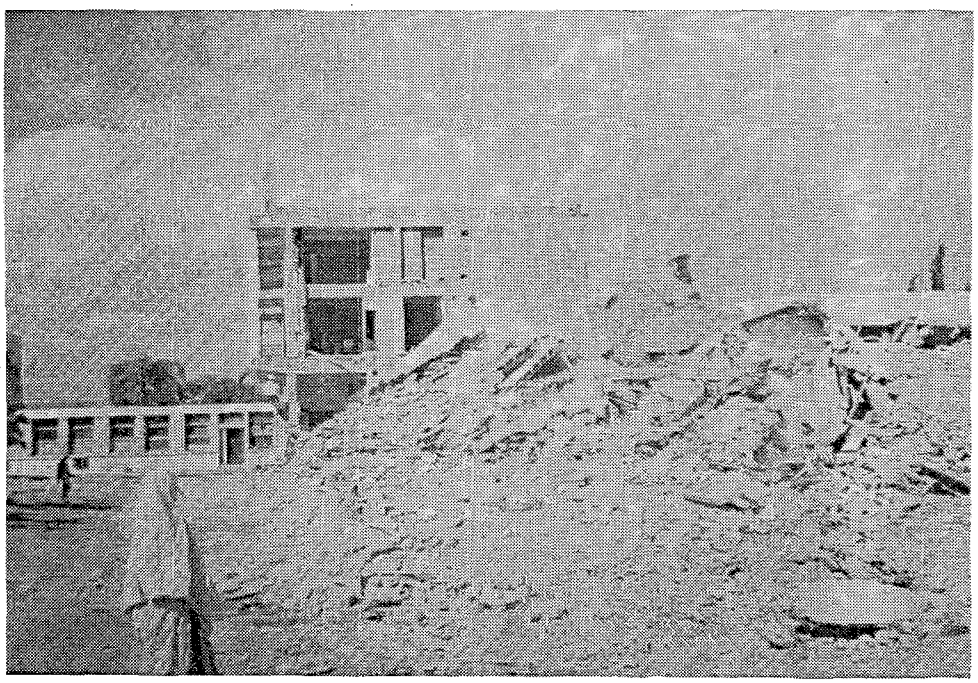


(b) East Side

Fig. 3.24 Corner of Cité An Nasr That Remained Standing



(a) Bulldozers Removing Debris of Hotel du Cheliff



(b) Only a Portion of Hall of Justice Remained Standing

Fig. 3.25 Collapsed Hotel du Cheliff and Hall of Justice

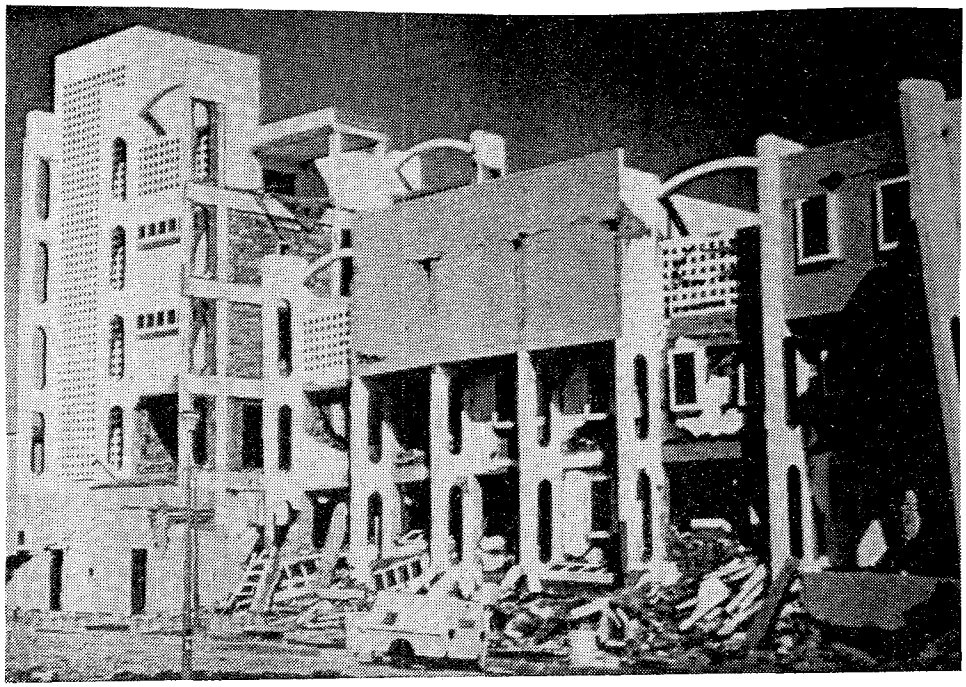


Fig. 3.26 Collapsed Clinic Which Was Nearing Completion. Soft first story failed.

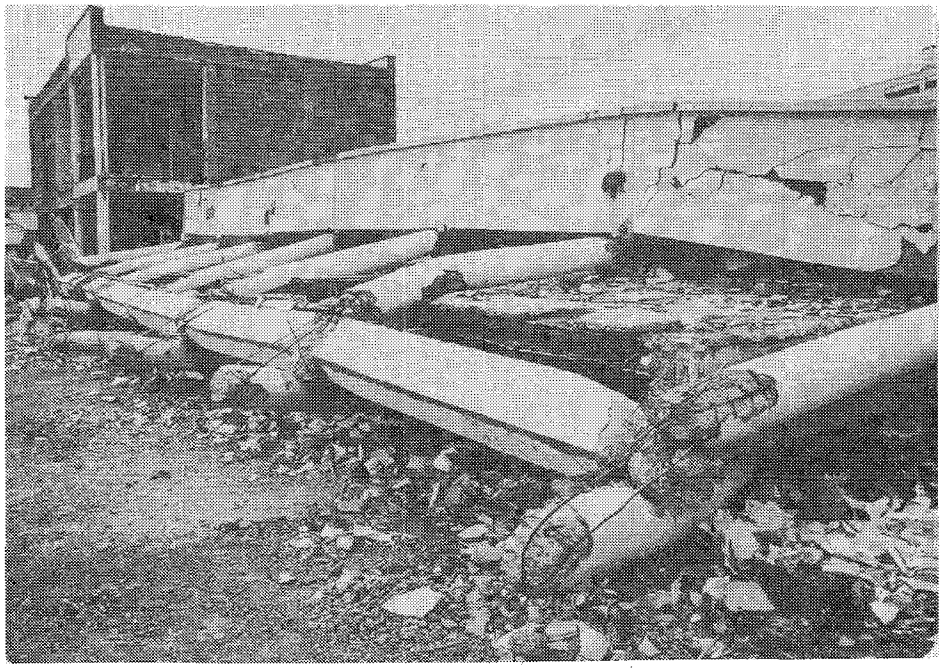


Fig. 3.27 Collapsed High School. Note lack of ties in beam-column connections.

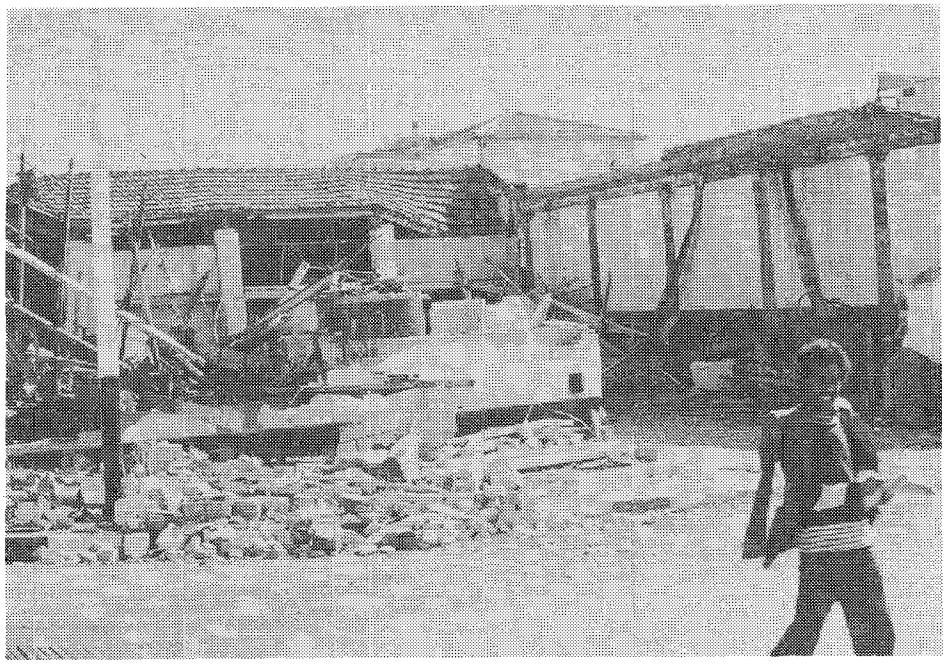


Fig. 3.28 Disaster Planning and Fire Station

Fig. 3.29 Police Headquarters and Mayor's Office

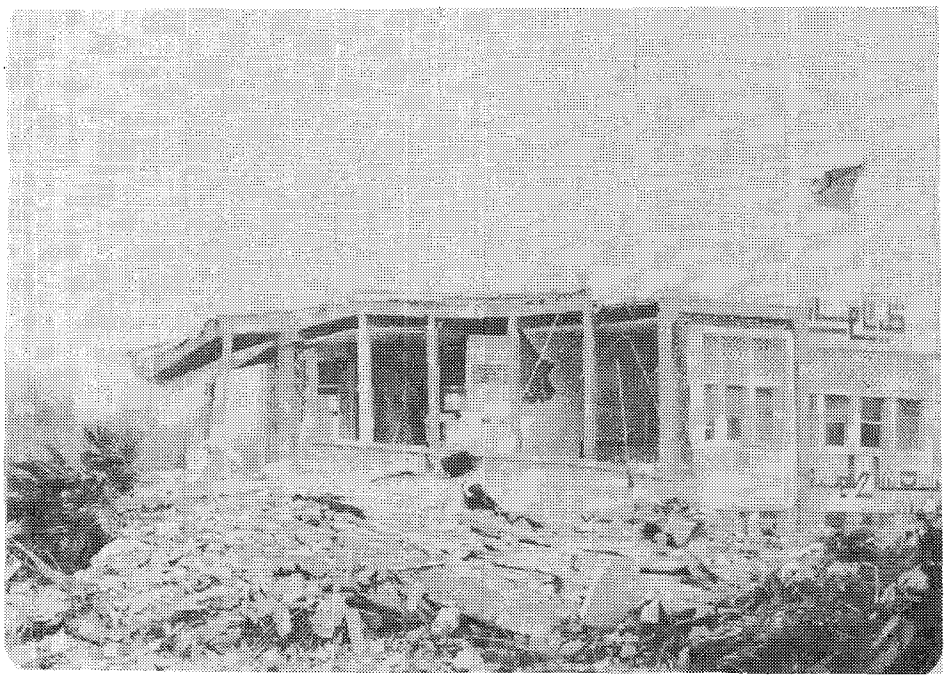


Fig. 3.30 Collapsed Portion of City Hall

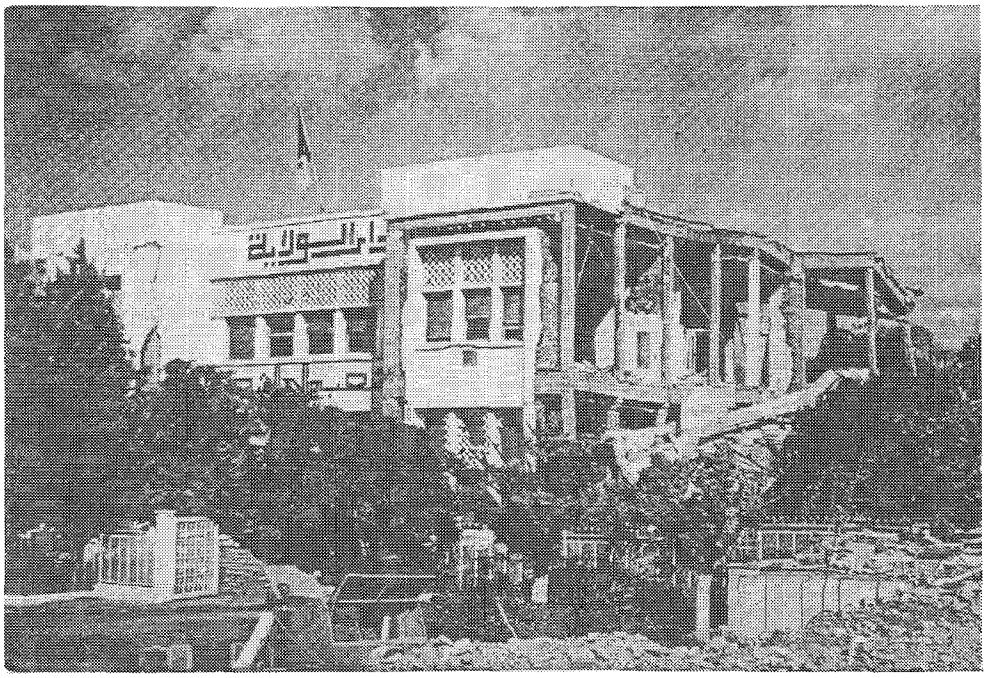


Fig. 3.31 Partially Collapsed Hotel

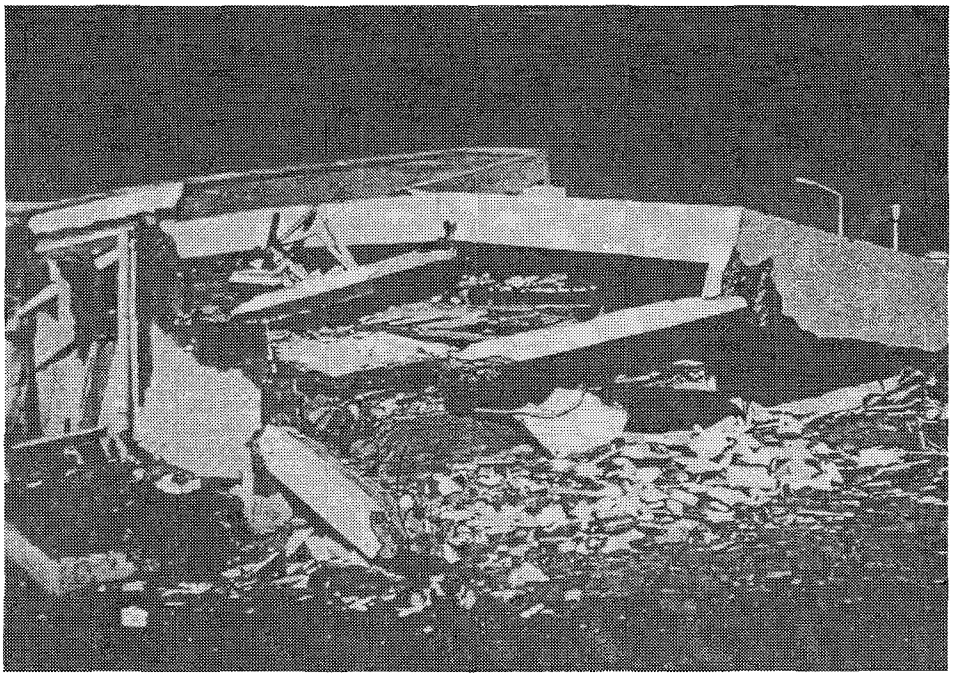


Fig. 3.32 Collapsed Commercial Building

Fig. 3.33 Collapsed Store

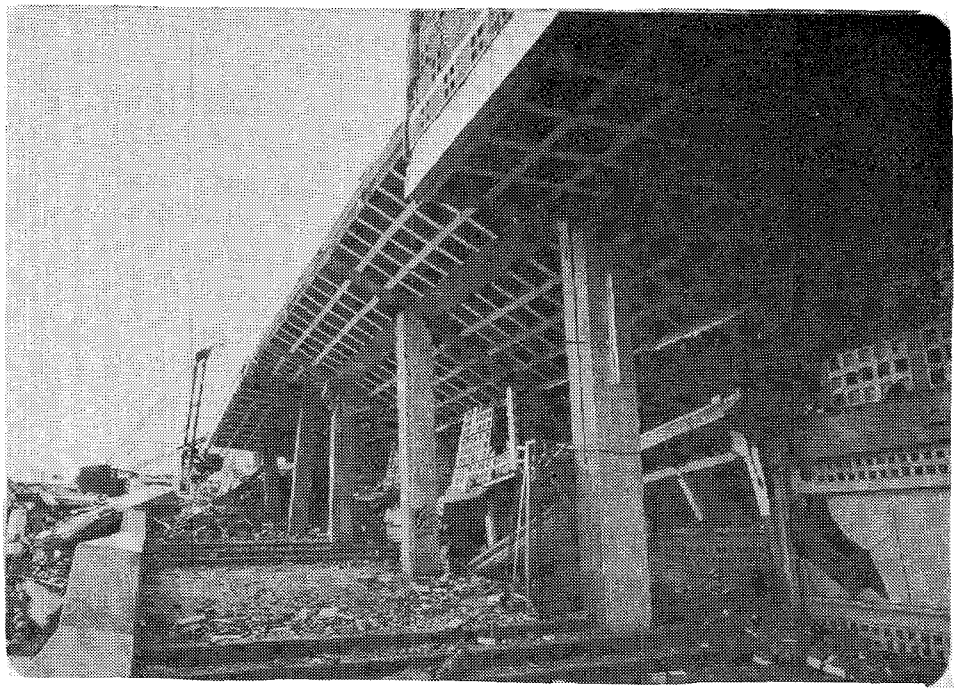


Fig. 3.34 Collapsed Cité An Nasr Market. A huge shopping mall and apartment complex covering one large block collapsed.

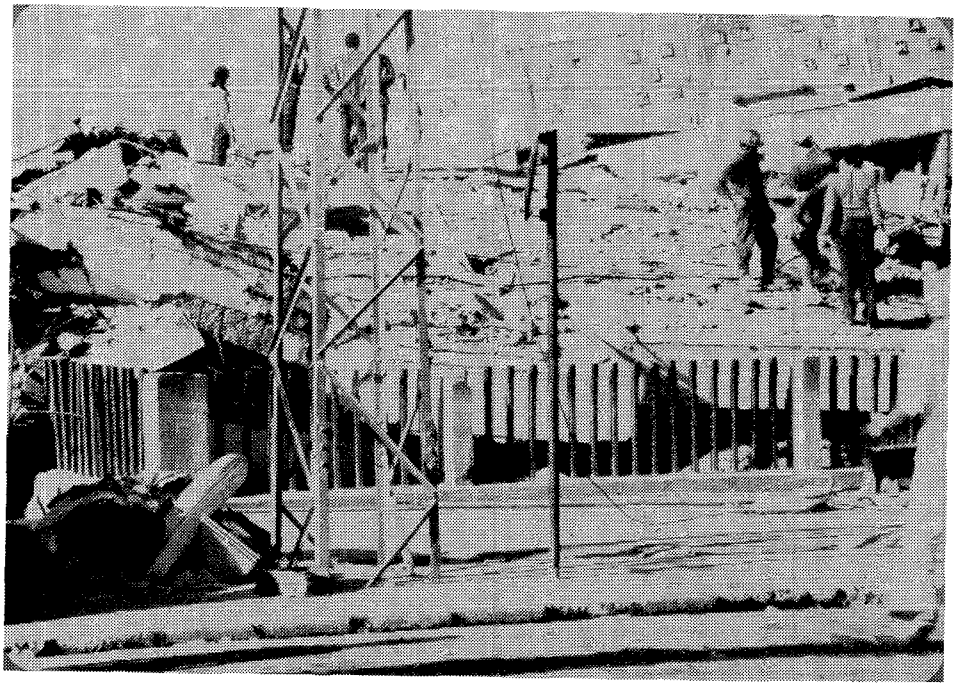


Fig. 3.35 Collapsed Cité An Nasr Market. Some 3000 people were believed dead under this structure. Pancaked floors had to be lifted carefully because of buried bodies.

Fig. 3.36 Collapsed Three-Story RC Apartment House

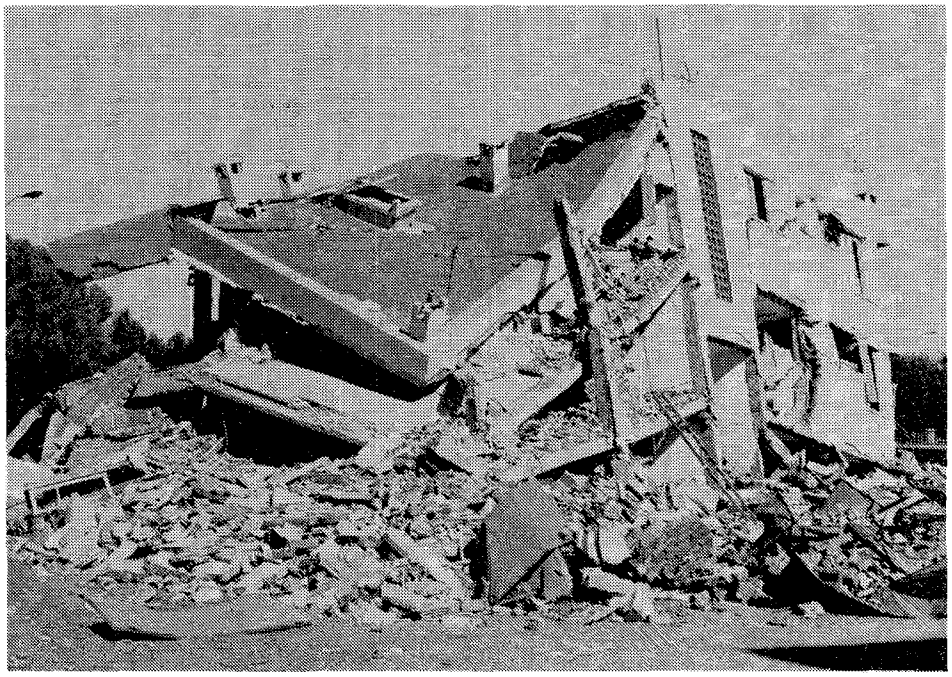


Fig. 3.37 Collapsed Three-Story RC House

Fig. 3.38 Temporary Tents

3.5.2 Large Industrial and Commercial Buildings. There were several low-rise industrial and commercial buildings within El-Asnam and some large industrial buildings in the suburbs and near the city. A complex of one-story industrial and commercial buildings was situated near the downtown area, about 200 meters (700 ft) southeast of the clinic building (see map, Fig. 1.17). All but one of these buildings were constructed of light steel trusses supported on columns. The roofs were very light, consisting of corrugated fiber cement supported on steel purlins. Except for some buckling of the horizontal bracing, no other structural damage was observed in these buildings; in contrast, however, the office building in this complex, which was of reinforced concrete with masonry walls and a heavy roof, collapsed (Fig. 3.39).

Seven km west of El-Asnam there is a cement fabrication plant. This plant has two towers approximately 45 meters (147 ft) high. The towers are constructed of reinforced concrete to about 15 meters above ground and then of structural steel (Fig. 3.40). Each of the four steel columns is anchored by six 6.5 cm bolts. A gap about 2 cm occurred between the nuts and baseplates of all four columns (Fig. 3.41). This could have been caused by three factors: yielding of bolts, pullout of bolts, or crushing of cement pad under baseplates. As can be seen in Fig. 3.41, the pad was severely crushed, which allowed a downward movement of the plates. Perhaps because of poor workmanship or quality control of the material used in the cement or mortar pad, the compression strength of the pad was insufficient even though the quality of construction was generally excellent throughout the plant. In any case, the gap indicated that large forces developed. The only other observed structural damage was failure of connections at two internal vertical steel bracing elements in the top story of the tower.

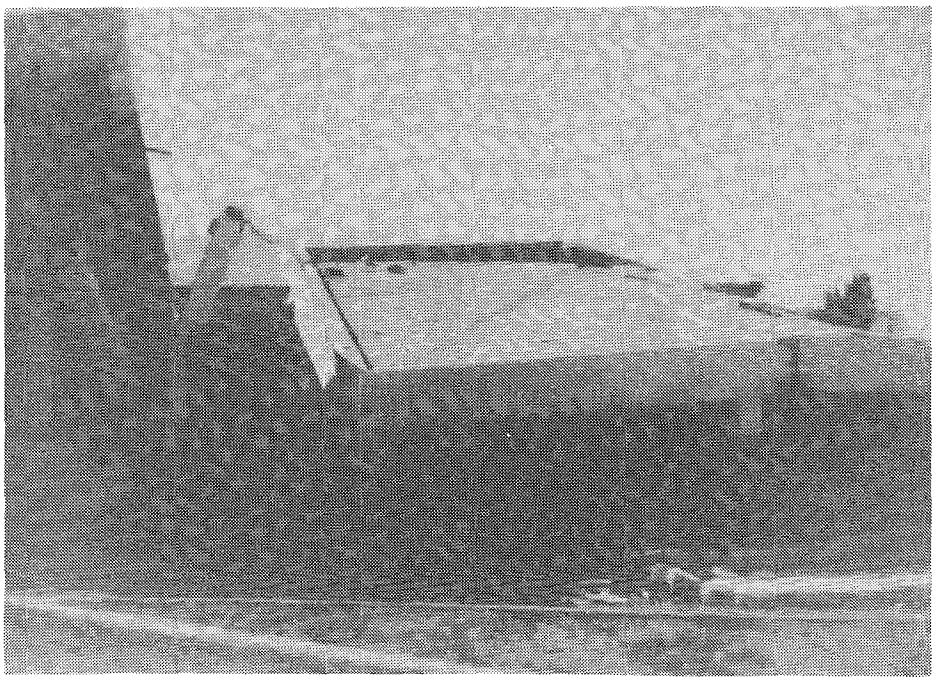
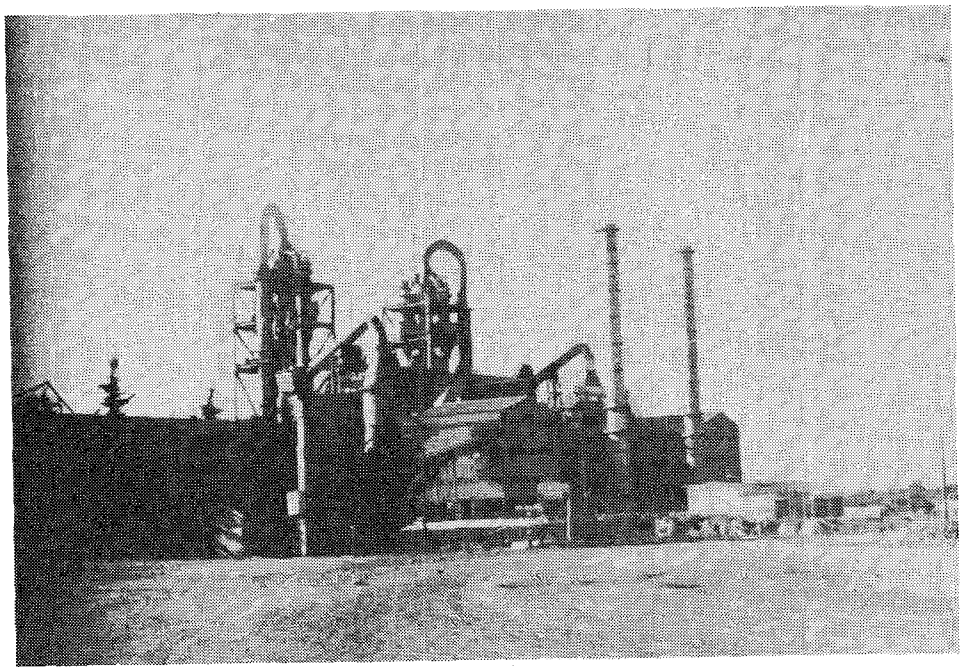
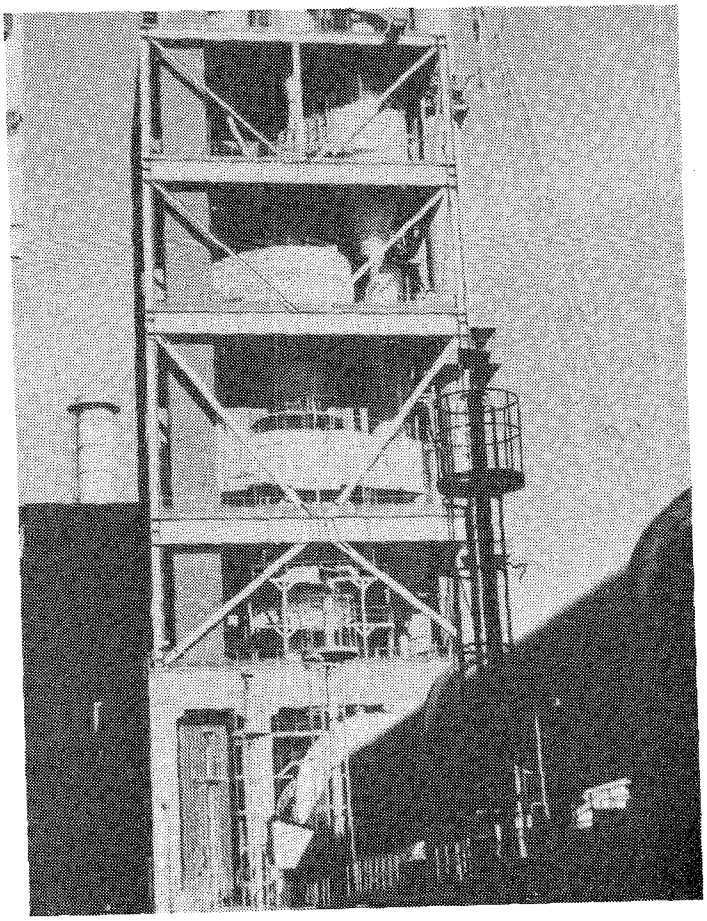


Fig. 3.39 Collapsed RC Industrial Plant Office Building



(a) Overall View



(b) Tower

Fig. 3.40 Cement Plant 7 km West of El-Asnam

Fig. 3.41 Damage at Supports of Steel Frame of Tower Shown in Fig. 3.40

3.5.3 Transportation Facilities. As previously pointed out, El-Asnam is located between the two main cities of Algeria — Algiers and Oran. The main forms of transportation leading to it are a two-lane highway (Highway No. 4) and a railroad. Along the highway, particularly in the region close to Oued Fodda where there was surface faulting, there was evidence of severe damage to the pavement because of soil movement. Although most of this damage was repaired quickly, there was a portion that was not yet repaired by the time of the reconnaissance or investigating teams' visits, and an alternate route had to be used (November 1, 1980). The most serious damage to the highway and other roads was at the bridge approaches, as described in more detail below. The railroad trains near Oued Fodda had been overturned (Fig. 3.11) and the rails bent in many places (Fig. 3.12). Rail communication between Algiers and El-Asnam was out of service for nine days after the October 10, 1980 earthquake.

3.5.4 Bridges. A number of highway and railway bridges were examined. No important bridges in the region collapsed. In the city of El-Asnam there are three major bridges; all have one span and were serviceable after the earthquake with no apparent structural damage. Two of these bridges serving as overpasses to the railroad are of welded steel. The only observed damage was relative displacement (vertical and lateral) of the bridge decks with respect to their approaches. This was because of the relative lateral movement of the bridge at its abutment support and the settlement of the backfill at the abutments, which was on the order of 10 cm (4 in.).

The most serious damage of this type was observed at a two-lane modern prestressed concrete bridge continuous over five spans. This bridge is located on a secondary highway as it crosses the Cheliff River about 15 km (9 mi.) northeast of El-Asnam and about 5 km (3 mi.) southwest of Beni Rached (Fig. 3.22). Intermediate spans are supported on twin piers, the lower ends of which are protected from scour by a steel caisson lining. Except for some cracking of concrete at the foundation in the steel caisson, there was no evidence of structural damage to the spans, piers, or foundation. There was also no evidence of pier settlement. The only faulty detailing was at each end, where the bearing beams that transferred the bridge load to the wing walls had moved (Fig. 3.22d). These bearing beams were keyed into the abutment approach structure but were not tied back; so in the case of the southern abutment, which had undergone a significant rigid body rotation, its bearing beam was almost lost. Relative movement to 1 meter (40 in.) horizontally and 0.30 meter (12 in.) vertically occurred between the approach and the deck (Figs. 3.22b and 3.22c). Along the river in the neighborhood of the bridge, considerable land movement and soil liquefaction were observed.

3.5.5 Utilities (Lifelines) in El-Asnam. The earthquake destroyed water, sewer, electric, gas, and telephone lines in El-Asnam, requiring months before operation could be restored. All water lines (underground pipes) were fractured extensively. Water was brought to the survivors (more than 125,000 were temporarily housed in tents outside El-Asnam) by Army water tank truck convoys. While elevated concrete water tanks did not collapse, they were usually damaged at concrete joints and column bases. A section of an irrigation aqueduct constructed of half-circular precast concrete sections and supported above ground on concrete columns collapsed. There appeared to be no positive connection between the precast concrete sections and the columns (Fig. 3.42). Fortunately, water for fighting fires was not needed because only very small fires broke out during and immediately after the main shock.

Fig. 3.42 Elevated Irrigation Channel

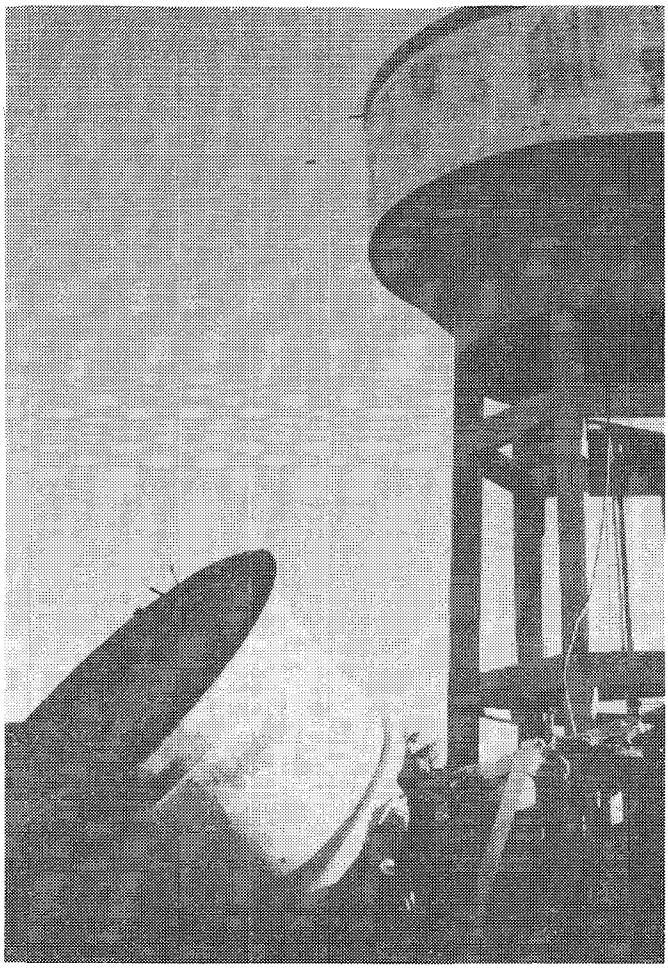
In the town of El-Abadia, about 30 km east of El-Asnam, there were two elevated RC water tanks. The tower of the smaller tank collapsed; the other remained standing although there was some significant damage at the ends of the horizontal beams and at the columns at the beam-column joints. It appears that the smaller tank was empty when the earthquake struck. The main reason for the failure appears to be lack of adequate shear reinforcement in the columns and/or poor anchorage of the column reinforcement to the bottom beam supporting the tank (Fig. 3.43).

The sewer system in El-Asnam was completely out of service after the earthquake, posing a major sanitary and health problem. Accordingly, the population was inoculated against cholera and typhoid. No electricity was available in El-Asnam because transformers were overturned or seriously damaged (Fig. 3.44).

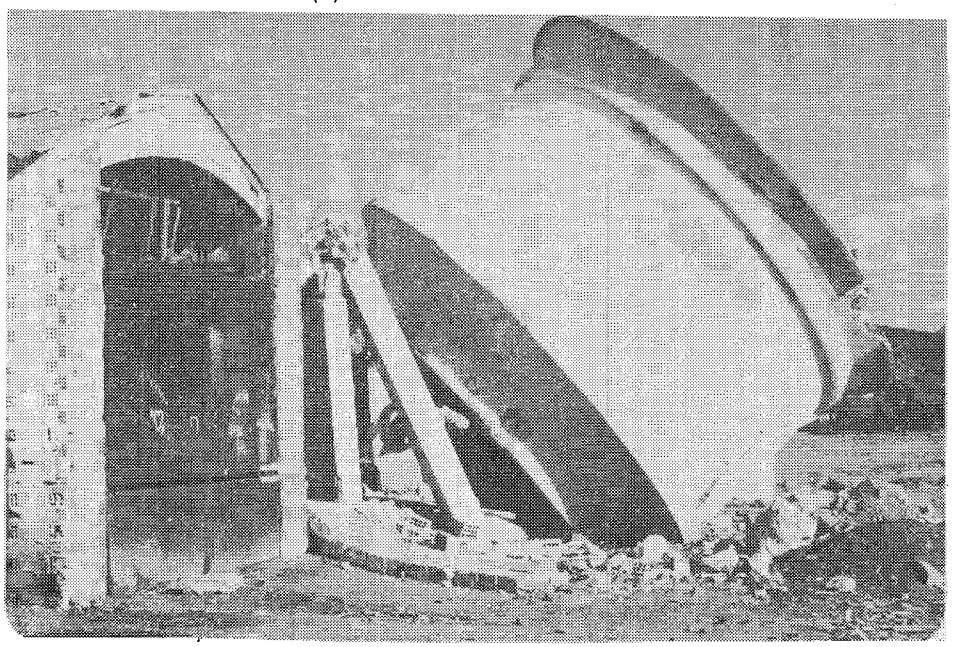
Telephone communications were interrupted because there was such severe damage to the telephone equipment that even after three weeks no attempt had been made to repair it. The building housing this equipment was a modern three-story RC moment-resisting space frame infilled with masonry (Fig. 3.45). Although the structure did not sustain major damage, in constrast to the other modern construction in the city which was destroyed, the nonstructural damage was enormous. The building appears to have undergone considerable deformation resulting in significant damage to the masonry walls and to the telephone equipment (Figs. 3-4a, b, c) . Free-standing relay racks overturned. Fig. 3.45d shows a tall chimney attached to one side of the telephone building that failed near its base, hitting the transverse walls and breaking near the top.

3.5.6 Dams. Two dams are in the heavily shaken region: the Oued Fodda (or Steeg) Dam on the Fodda River some 30 km southeast of El-Asnam, and the Sly Dam on the Sly River southwest of El-Asnam. Details of these dams may be found in the section on "Dams in Algeria" in the register compiled by the International Commission on Large Dams (1973). Neither dam was damaged. Steeg Dam had some superficial cracking. Water levels were low, as was the level of seismic loading.

3.5.7 Indirect Effects. This category of earthquake effects includes damage caused by fires, or floods caused by dam failure or rivers dammed because of landslides. Small fires were reported immediately after the earthquake, the most important occurring at the telephone exchange. Fortunately, no deaths or serious damage because of these indirect effects were reported.



(a) View of Two Tanks



(b) Tank That Failed

Fig. 3.43 Elevated Water Tanks in El-Abadia

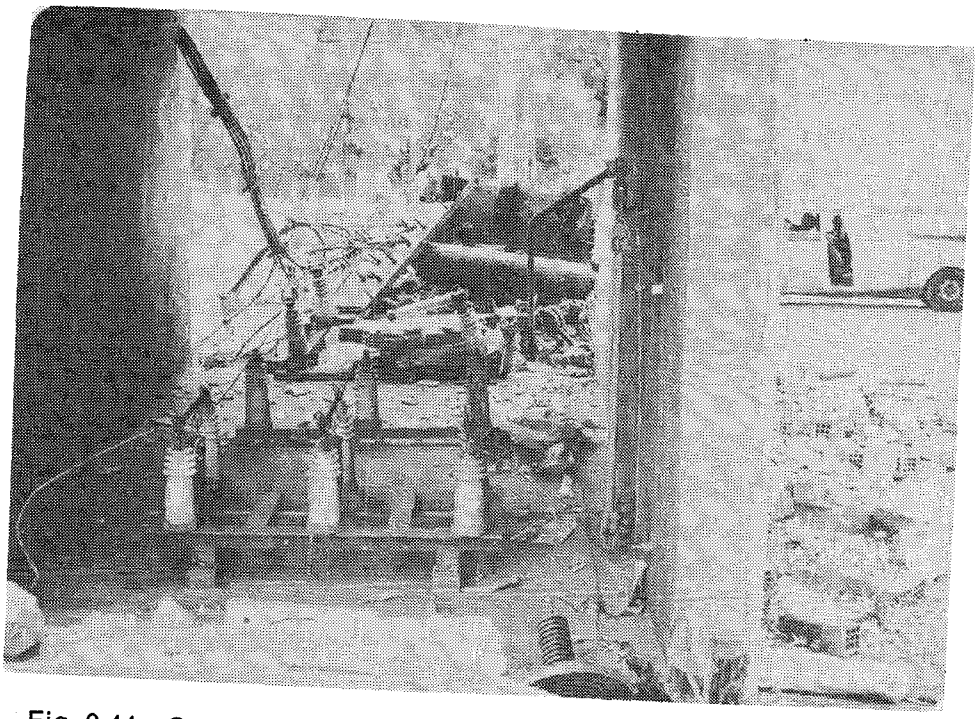
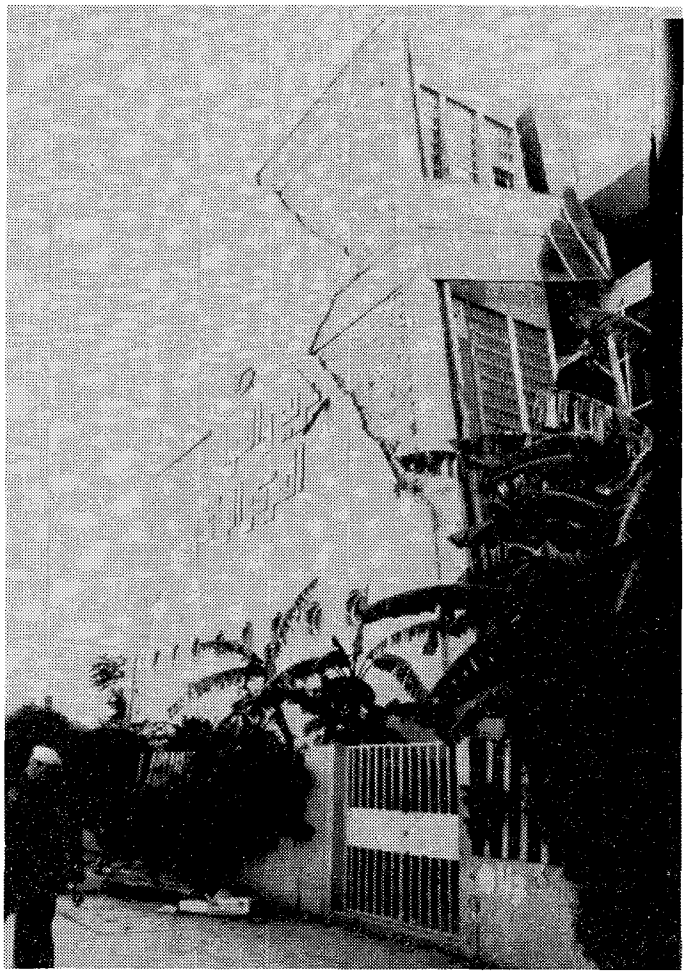
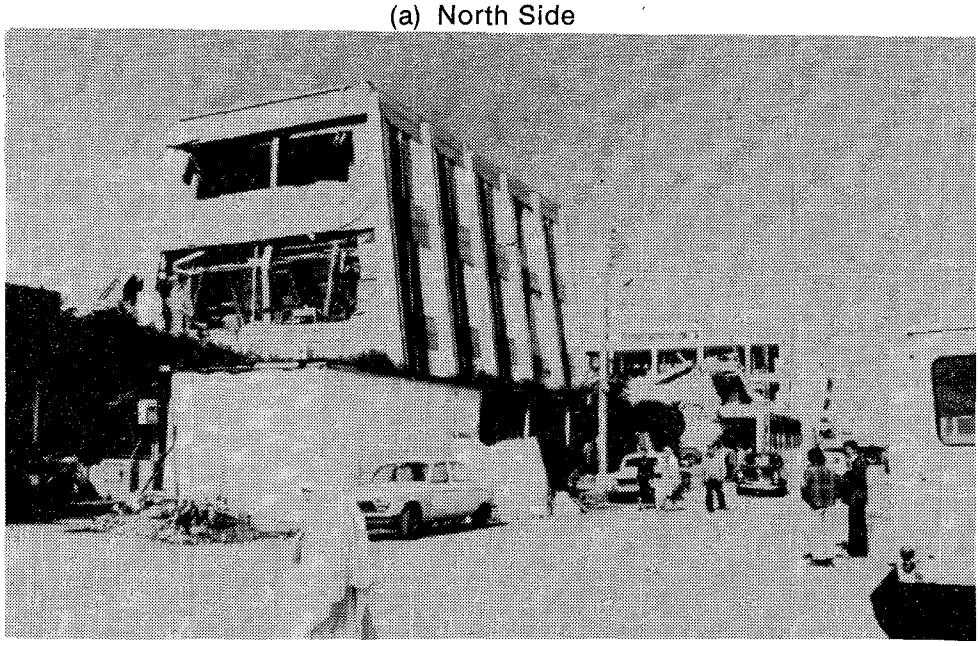


Fig. 3.44 Overturned Transformer and Other Electrical Equipment

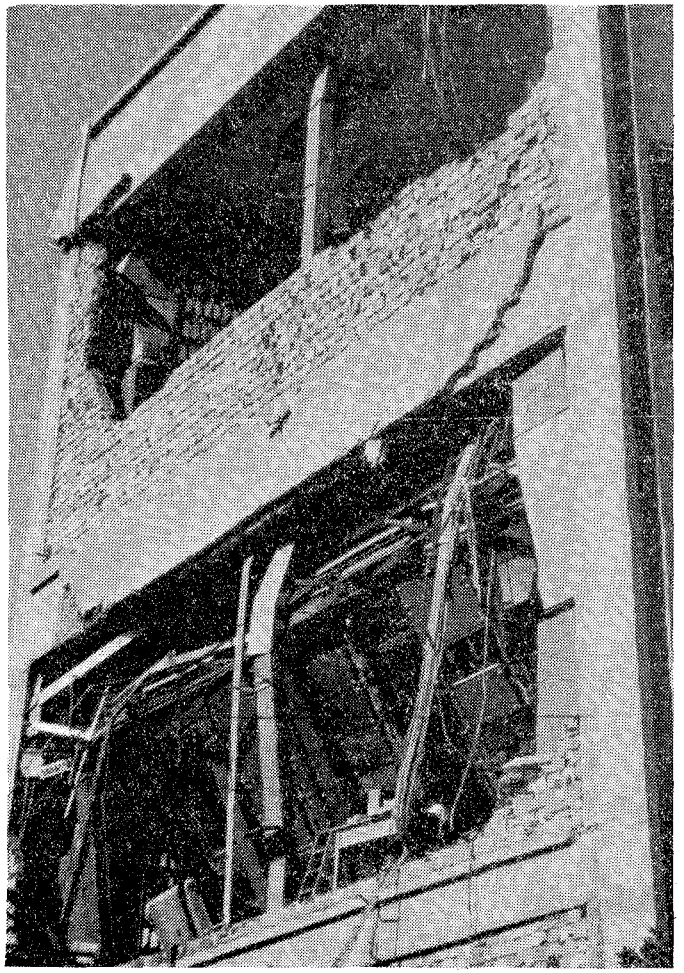


(a) North Side

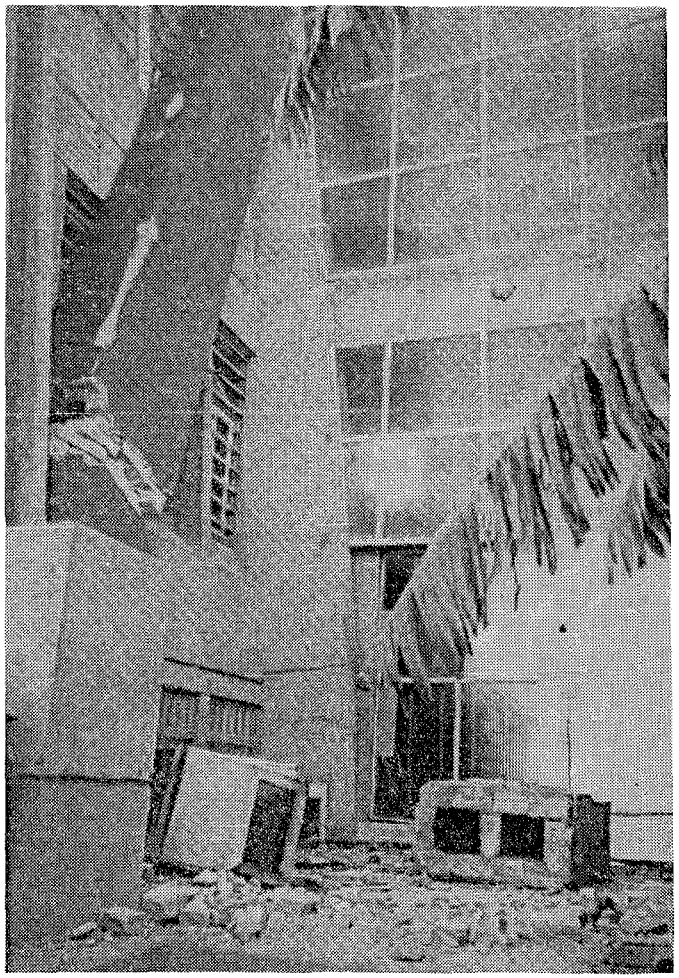


(b) South Side

Fig. 3.45 Damaged Telephone Building



(c) Damaged Masonry and Equipment



(d) Failed Chimney Attached to Building

Fig. 3.45 (cont.)

3.6 CTC Program to Evaluate Damage

Soon after the El-Asnam earthquake, the Algerian Ministry of Housing and Construction directed CTC to investigate the engineering aspects of the earthquake, specifically the behavior of buildings in response to this earthquake and to assess the reasons for the response.

In answer to the above directive, CTC gathered a team of engineers to investigate the behavior of structures. CTC engineers developed an evaluation form with the help of the ATC-3 Report, the EERI documentation on *Learning from Earthquakes,* and suggestions from Shah, who proposed a five-level damage classification. The evaluation form went through three stages of development in response to conditions encountered in the field. The final version of the form is shown in Appendix B. The city of El-Asnam was divided into 10 sectors with each sector further divided into 10 zones (Fig. 3.46). One hundred engineers, most from CTC and the rest from other governmental organizations in Algiers, were assembled to form the field investigative team, and each sector was assigned to a team of 10 engineers; each team had a leader and a deputy leader. The team's task was to investigate every building in El-Asnam and fill out the form (given in Appendix B). The first task of this field investigation was to classify all buildings into one of the following categories:

Green: Very little damage. Can be reoccupied immediately.

Orange: Needs further study before it can be either occupied or condemned.

Red: Condemned and should be demolished.

Results of this preliminary classification are presented in Table 3.3.

Further analysis of the field survey would be essential to understanding the behavior of various types of construction under seismic loading. The method of upgrading or repairing the structures, and the codes and regulations necessary to rebuild El-Asnam, would be dependent on the results of this field survey. A comprehensive analysis has now been completed and reported by CTC in *Rapport Général sur le Séisme du 10 October 1980 à Ech-Cheliff,* October 1981.

Based on the analysis of damage data conducted by CTC on 5131 buildings, Petrovski, Director of IZIIS, has summarized building performance in the 10 sectors of El-Asnam (Fig. 3.46). Table 3.3 shows these results. It can be seen that about 33 percent of the buildings were given a "green" classification, 42 percent an "orange" classification, and 23 percent a "red" classification.

TABLE 3.3 DAMAGE CLASSIFICATION OF EL-ASNAM BUILDINGS (Petrovski, 1981)

Sector	Total Number of Buildings	Damage Classification*							
		Green		Orange		Red		Undefined	
		Number	%	Number	%	Number	%	Number	%
I	566	108	19.08	341	60.24	116	20.49	1	0.17
II	360	112	31.11	164	45.55	80	22.22	4	1.11
III	715	154	21.53	322	45.03	238	33.28	1	0.13
IV	256	97	37.89	98	38.28	61	23.82	0	0.00
V	686	219	31.92	253	36.88	214	31.19	0	0.00
VI	949	429	44.50	389	35.16	185	18.19	11	1.14
VII	343	161	46.93	132	38.48	38	11.07	12	3.49
VIII	367	156	42.50	157	42.77	40	10.89	14	3.81
IX	490	136	27.75	243	49.59	99	20.20	2	2.44
X	384	146	38.02	109	28.38	129	33.59	0	0.00
Totals	5131	1718	33.48	2158	42.06	1200	23.39	55	1.07

*Based on classification of damage by CTC, El Djazair, Algeria.

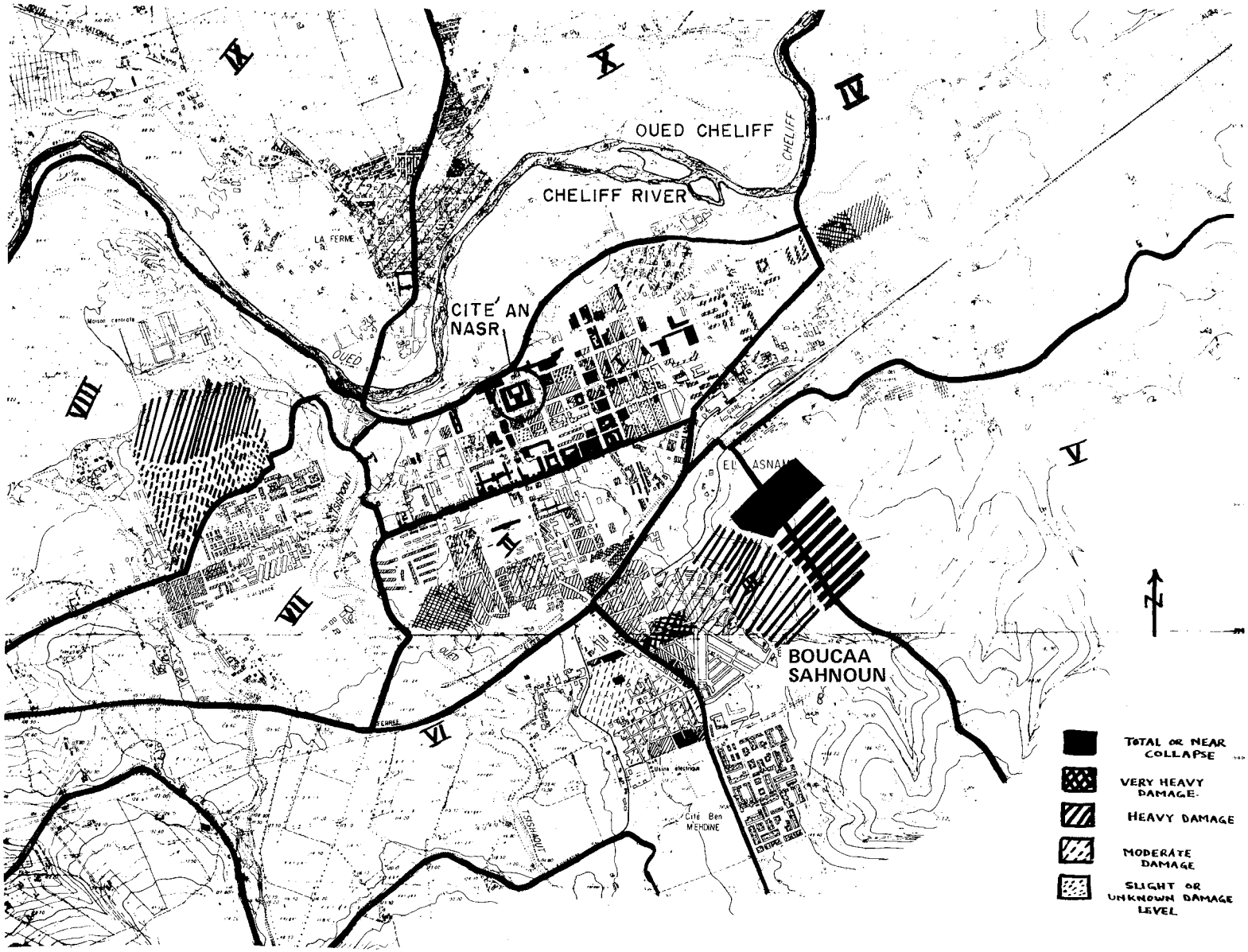


Fig. 3.46 Identification of 10 El-Asnam Sectors for CTC Survey

IV. IMPLICATIONS OF BUILDING DAMAGE: CASE STUDIES

4.1 General Evaluation

As discussed in section 3.5, nearly 80 percent of the buildings in downtown El-Asnam either collapsed or suffered such serious damage from the October 10, 1980 earthquake that demolition was necessary. Let us examine the reasons for such extensive destruction. Was it 1) the severity of ground motion? 2) defects in building design or construction? or 3) a combination of these two factors?

The absence of strong-motion records of the main shock and of the main aftershock that occurred October 10, 1980, as well as the lack of detailed design information (no drawings and computations) and data regarding the mechanical characteristics of the materials, makes it difficult to precisely determine the triggering mechanisms and sequence of failures of many of the buildings in downtown El-Asnam that collapsed completely. Although there is evidence that the ground motion at El-Asnam and surrounding areas was severe [a preliminary evaluation of acceleration and velocity was made by Papastamatiou, 1980 (Fig. 4.1)], inspection of the building construction practice and of the general performance of the buildings (both damaged and undamaged) indicates that the answer for the extensive destruction of buildings can be attributed to reason 2, above. Some of the main defects in the design and construction of the buildings are discussed in the following paragraphs.

4.1.1 Poor Conceptual Design (Building Configuration and Structural Layout) for Seismic-Resistant Buildings. In general, the configuration of the building and its structural layout were far from adequate for seismic-resistant construction. The most notable defects were the following:

1. **Use of the *vide sanitaire,*** a crawl space about 1 meter (3 ft) above ground level (Figs. 1.18, 1.19, 4.2, and 4.3). In most cases, the only structural elements in this space were short columns $\leq$ 1 meter (3 ft) tall, without any walls or partitions except for perimeter masonry walls used to enclose this space. As noted in section 1.6, in some buildings the perimeter walls were replaced by 10 cm (4 in.) concrete walls that were very lightly reinforced. Because the RC framework above this crawl space was infilled with stiff masonry walls, the *vide sanitaire* constituted a soft story with very short columns. Actually, it was not just an issue of being a soft story. As discussed in section 1.6.2, the available shear area of the stubby columns was totally inadequate. These stubby columns were sheared off by the horizontal inertia forces induced by the earthquake ground motions. As a result, the entire building was thrown out-of-plumb (Fig. 4.3). Although several buildings with this crawl space remained standing after the earthquake, they were inclined as much as 20° and dropped up to 1 meter (3 ft), producing damage in the first story, while upper stories sustained little damage to the concrete frame and infills (Fig. 4.3).

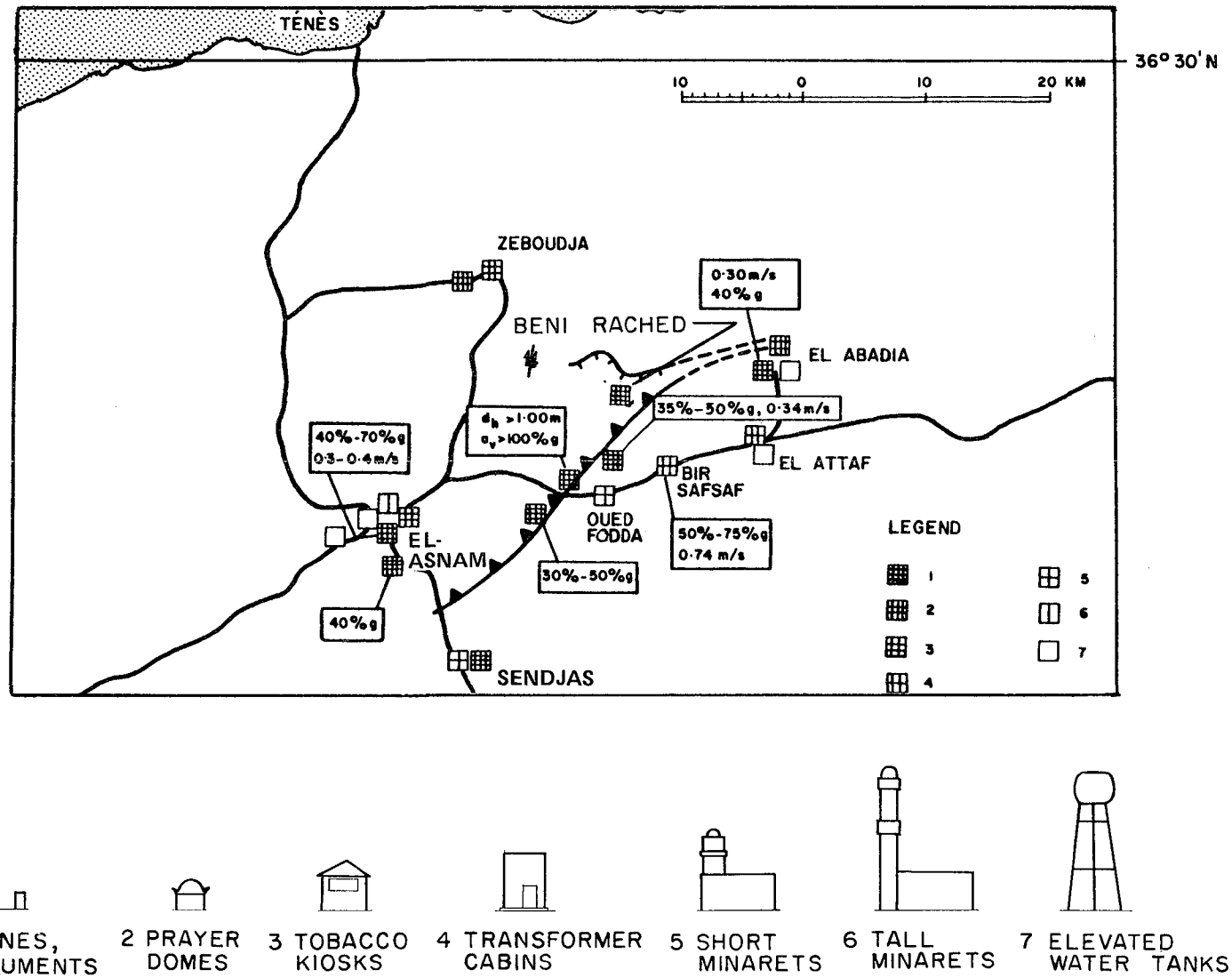
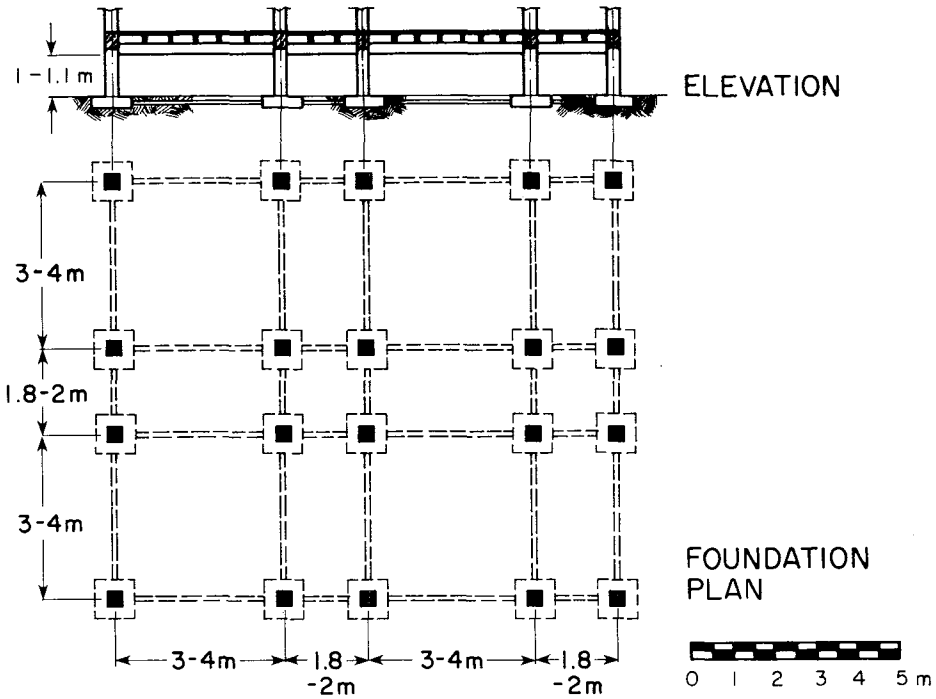
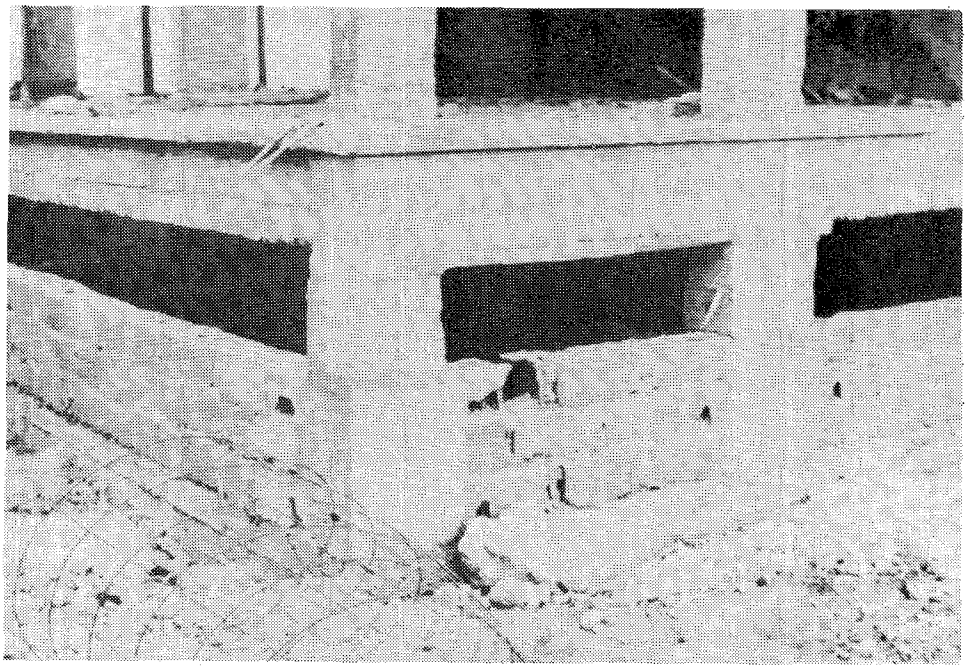


Fig. 4.1 Preliminary Evaluation of Ground Motion in Epicentral Area. Symbols indicate different standard structures or objects damaged or displaced. (Papastamatiou, 1980)

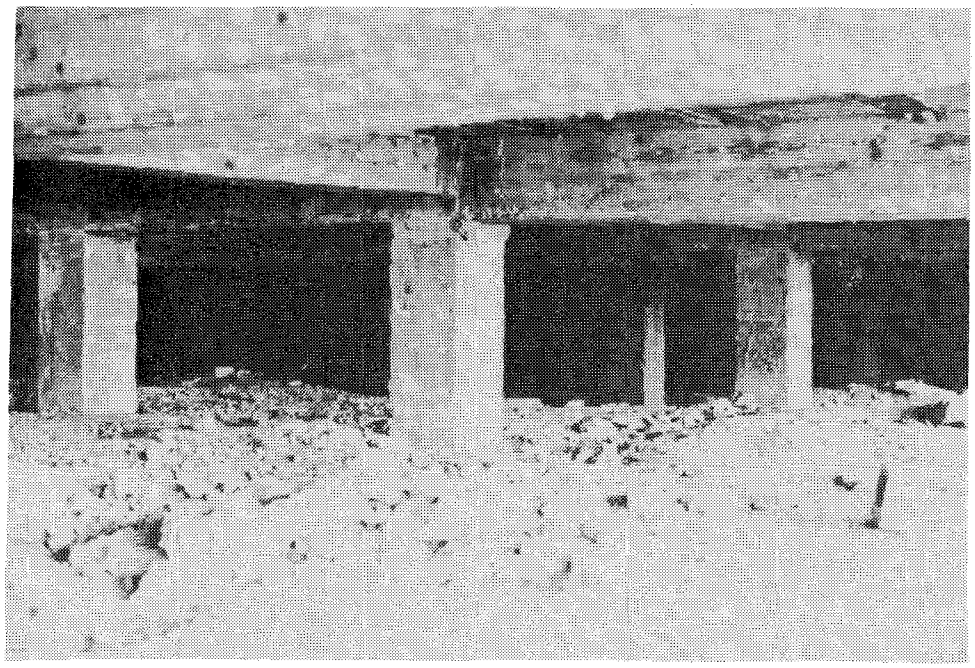


(a) Typical Foundation Plan and Elevation

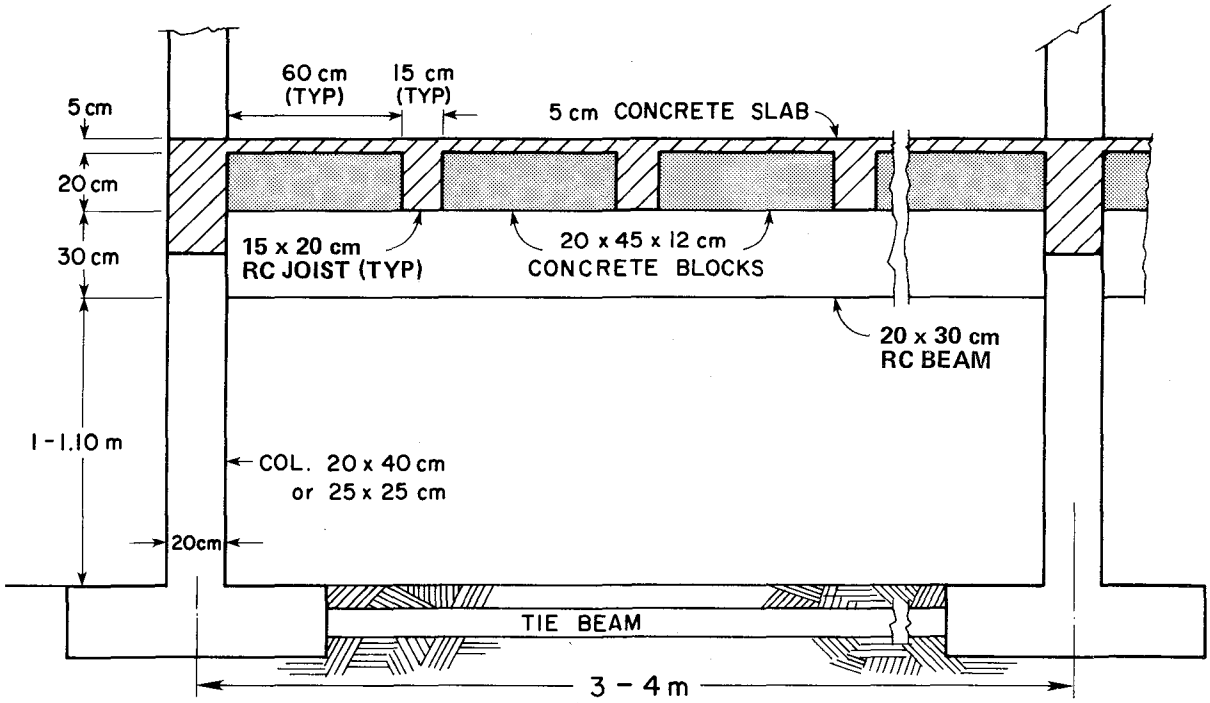


(b) External View (Building Under Construction)

Fig. 4.2 Structural Details and Views of *Vide Sanitaire*

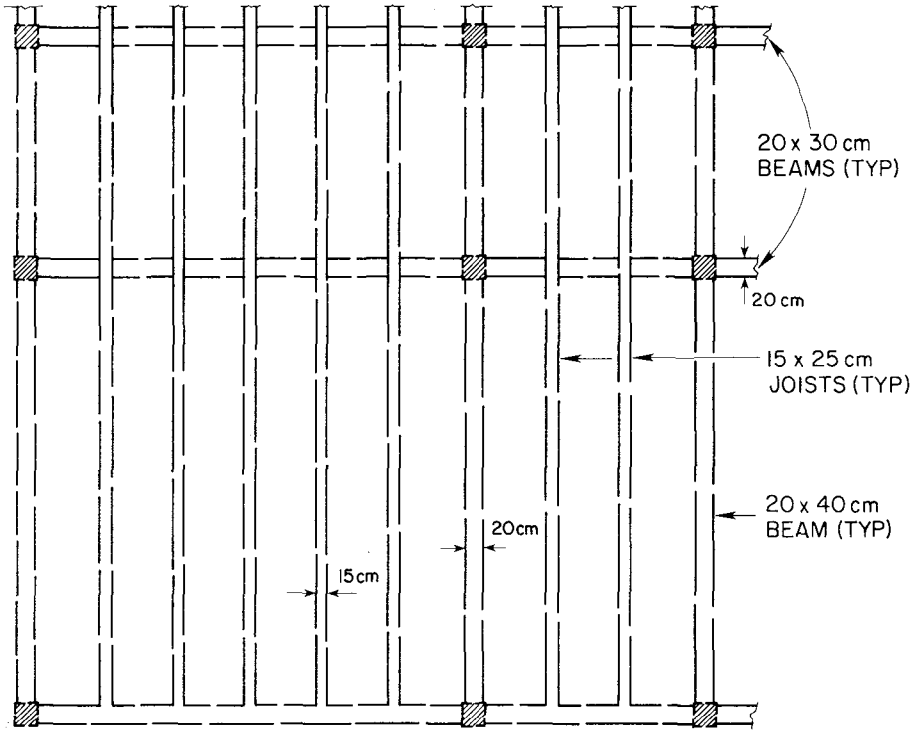


(c) Interior Beams and Columns

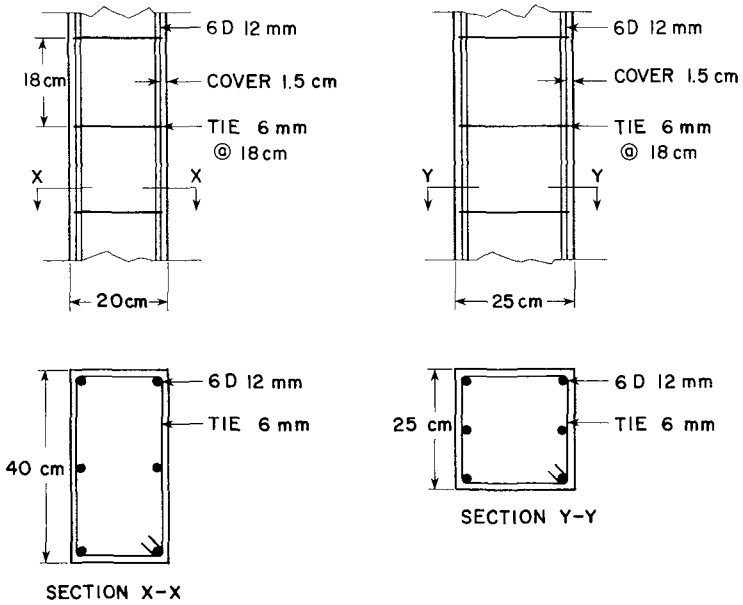


(d) Details: Elevation

Fig. 4.2 (cont.)

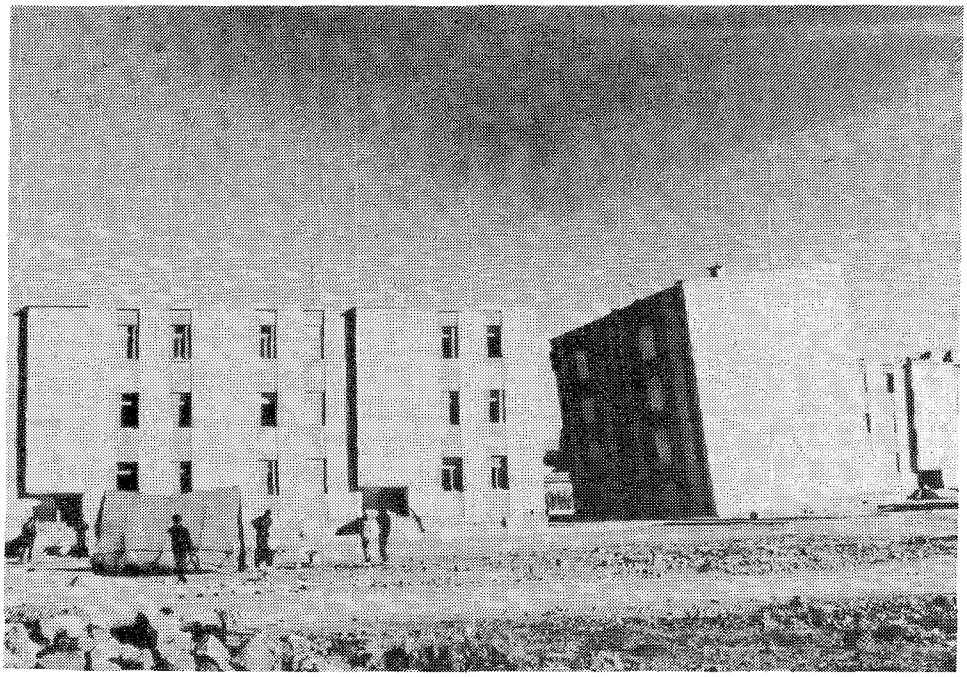


(e) Details: Floor Plan

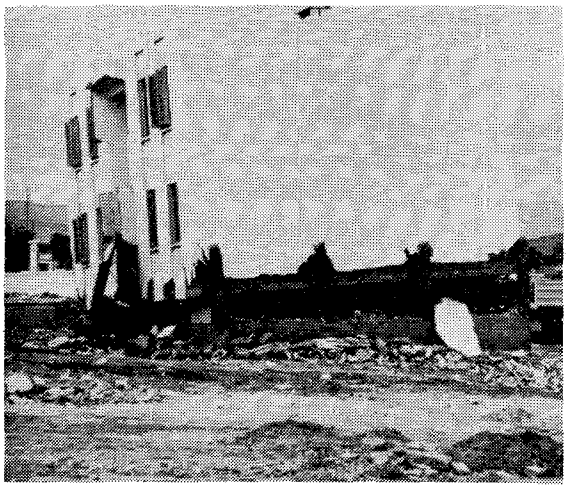


(f) Details: Columns

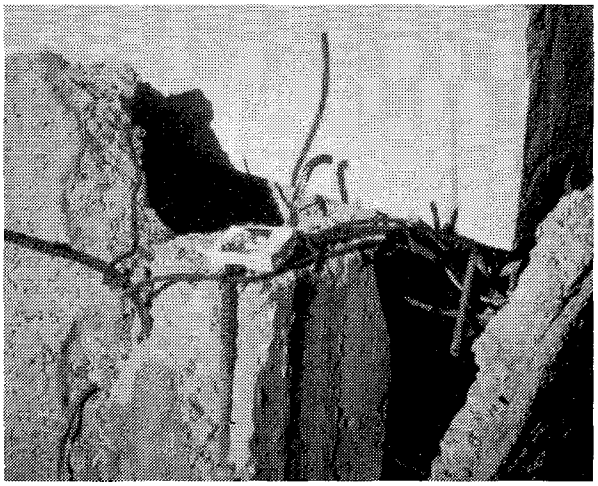
Fig. 4.2 (cont.)



(a) Three-Story Apartment



(b) Two-Story Apartments



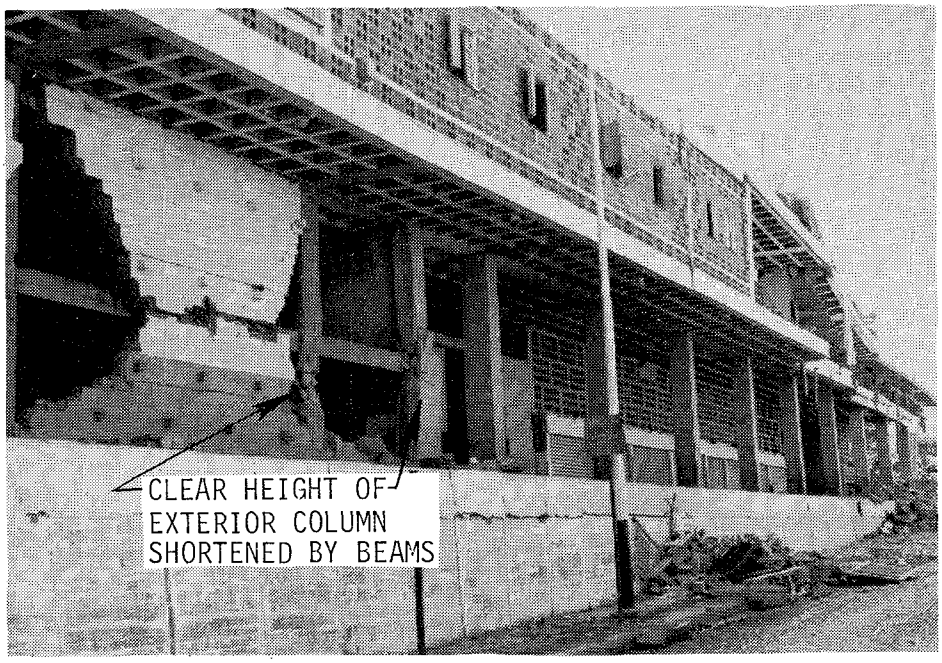
(c) Failed Column

Fig. 4.3 Examples of *Vide Sanitaire* Collapse

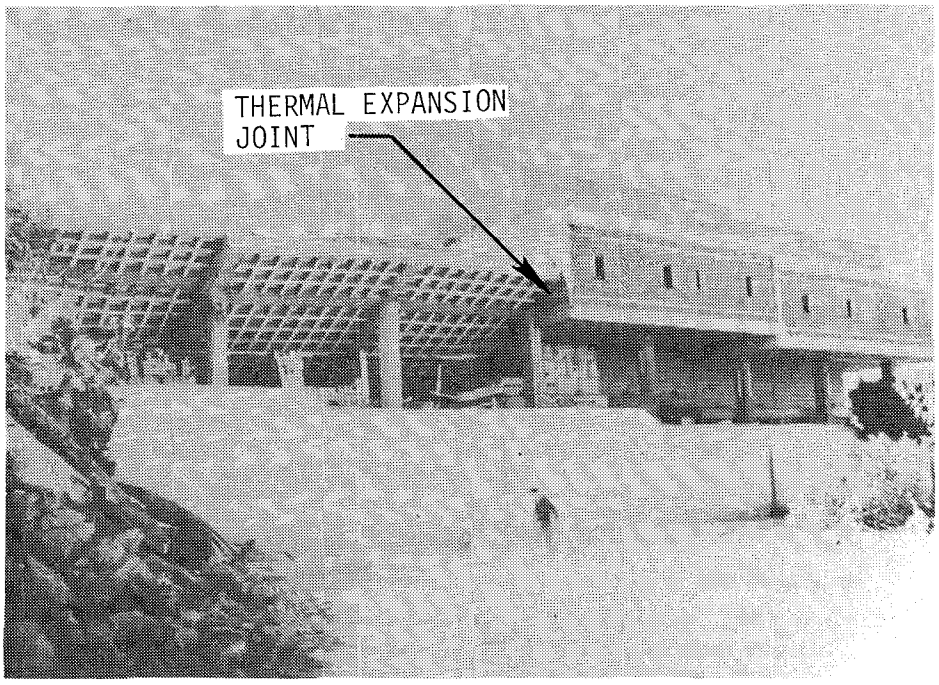
2. **Use of irregular building configurations with severe discontinuities in mass, stiffness, strength, and ductility.** This was the situation at the Cite An Nasr market where, as will be discussed later, in the first story there were exterior columns with a clear height of nearly 4.7 meters (15.4 ft), while the height of the interior columns was 2.3 meters (7.5 ft) or less (Fig. 4.4). Heavy stiff walls were used in the second story where the columns were 3 meters (10 ft) high. Other typical examples were observed in some of the buildings of a new medical clinic, almost completed but not yet occupied (Fig. 4.5). These buildings exemplified modern buildings with Algerian ornamentation, which added significant mass but no extra seismic resistance. The building shown in Fig. 4.5 exhibited severe stiffness and strength discontinuities at the first story and failed. It was elevated on columns (*pilotis*), more for the functional purpose of accommodating a restaurant and garage on the ground story than as a conscious carryover from the modern practice of lifting the building off the ground for visual reasons. Consequently, the building behaved as a soft first-story system with columns not designed to resist seismic loads. Corner location of a stiff stairway and an RC shear wall contributed to the collapse of one of these buildings. However, an adjacent, almost identical twin building (Fig. 4.6), differed only in the omission of free-standing ground story columns and remained standing after the earthquake. The situation is almost laboratory-like in providing a comparison in behavior of two nearly identical buildings in an earthquake, with the sole difference being ground story stiffness. The building with the soft story collapsed; while its twin, without a soft story, not only did not collapse but suffered very little damage.

3. **Use of very heavy roofs.** As discussed in section 1.6, most apartment, office, and commercial buildings had a heavy, one-way slab-on-joist roof system with hollow concrete blocks filling the space between the joists (Fig. 1.16). Above the roof slab there was usually a layer of 5 to 15 cm (2-6 in.) thick of thermal insulation material, covered by a waterproofing membrane, and then the roof finishing. The joists were supported on very stiff, heavy girders supported on relatively weak columns (Fig. 4.7).

4. **Use of too many heavy ornamental elements at building facades** (Figs. 4.5, 4.6, 4.8, and 4.9), **as well as unnecessary parapets on the roofs** (Fig. 4.10). One building under construction which partially collapsed was the new cultural center (Maison de La Culture), constructed of concrete frame and masonry infill. In Fig. 4.8a two of the main buildings of this center are pictured — one collapsed completely, the other remained standing. These two buildings appear to have been identical; therefore, the reasons one collapsed and the other did not were a puzzle. The standing structure, however, provided some clues as to the contributing factors for its twin's failure. The cultural center buildings are of the more ornamental and heavy style of architecture, superimposed upon a regular modern concrete frame. The use of stiff spandrel masonry walls (parapets) resulted in short captive columns that increased the shear demands beyond the shear capacity supplied. The lack of good transverse reinforcement led to total failure of these columns in the case of one building and to a state close to collapse in the standing building (state of the columns is illustrated in Figs. 4.8b and c). Because of the brittle nature of the shear failure in these inadequately reinforced columns, it appears that the building that remained standing was on the borderline of collapse. The long and heavy cantilevered portion of facade (Fig. 4.8d), that seems to be more of a stylistic expression than a functional requirement except for sun shading, also contributed to the damage observed in these structures.

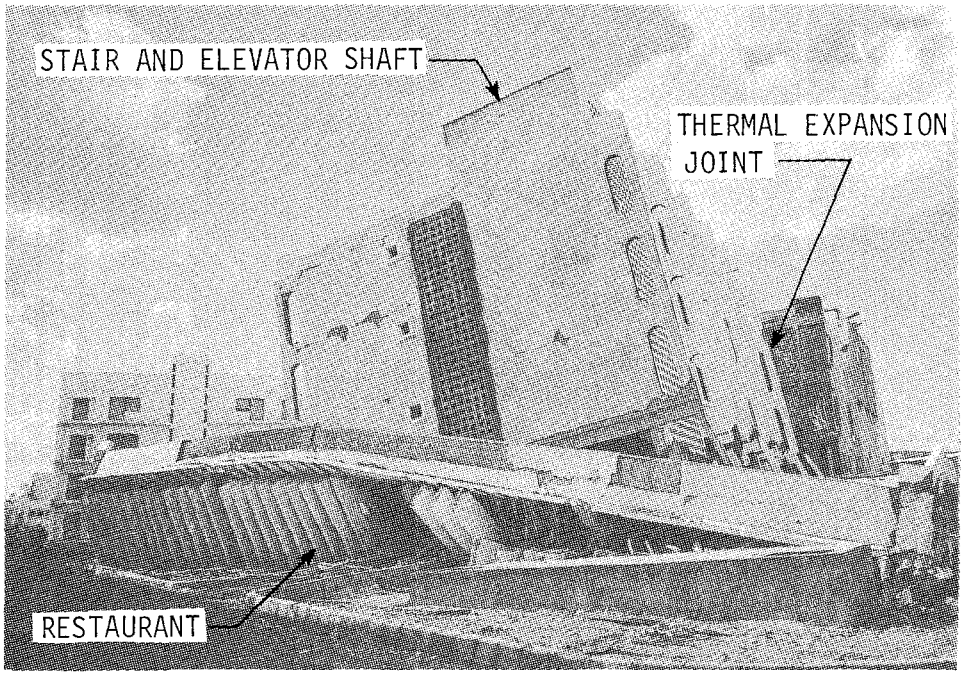


(a) Exterior Columns with Clear Height of 4.7 Meters. Some of this height was shortened to less than 2.3 meters by beams.

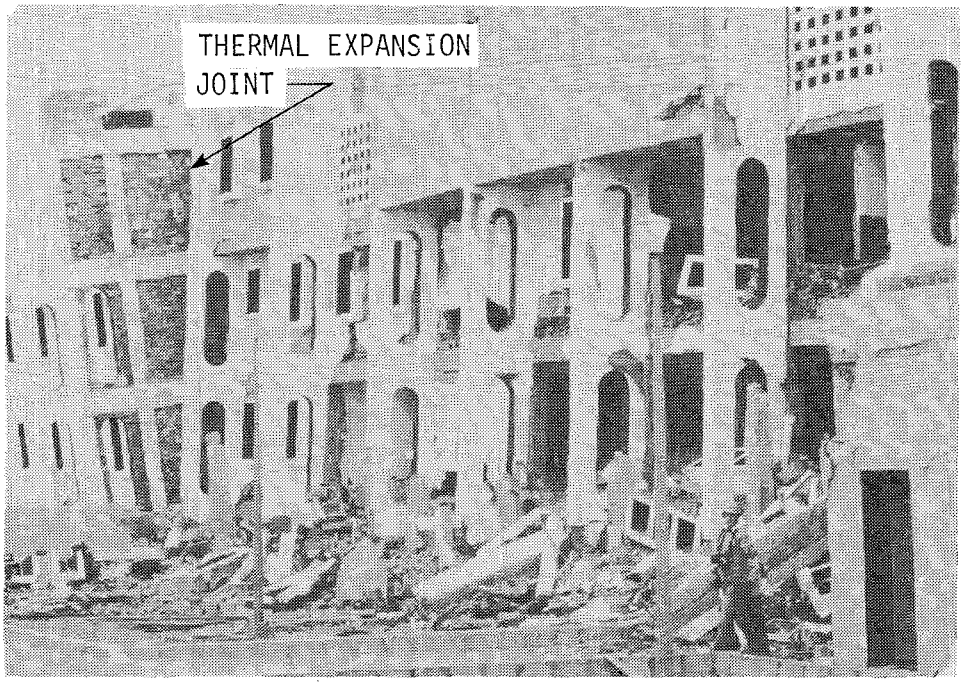


(b) Heavy Stiff Walls at Tip of Long Cantilevers, and Thermal Expansion Joints in Second Story

Fig. 4.4 Cité An Nasr



(a) Collapsed First-Story Restaurant



(b) Collapsed First-Story Garage

Fig. 4.5 New Medical Clinic, El-Asnam

Fig. 4.6 Clinic Building Similar to That Shown in Fig. 4.5, Which Suffered Slight Damage. Note heavy ornamentation.

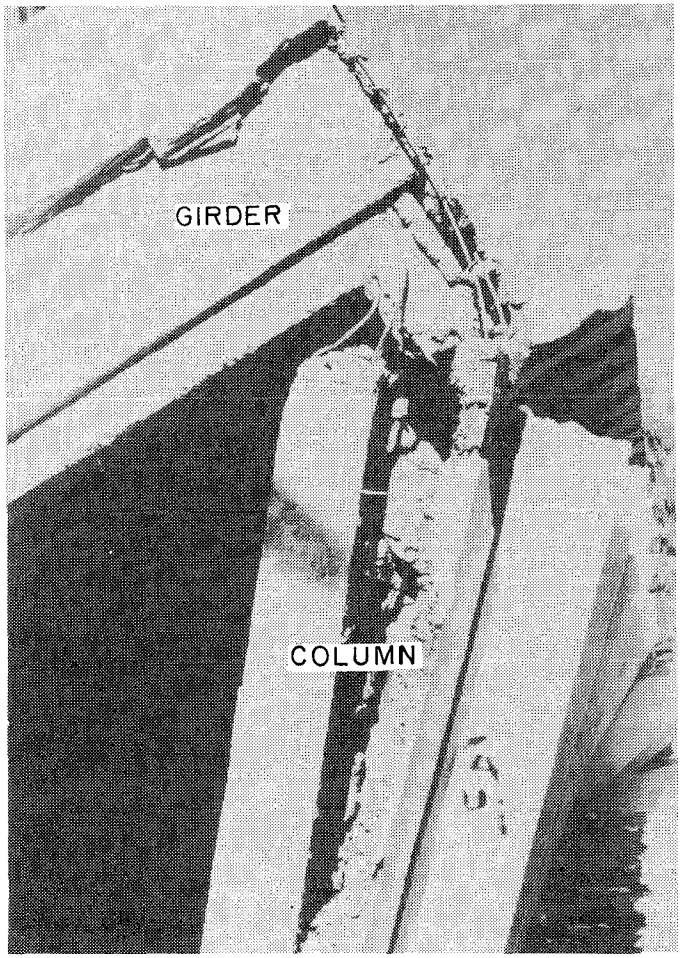
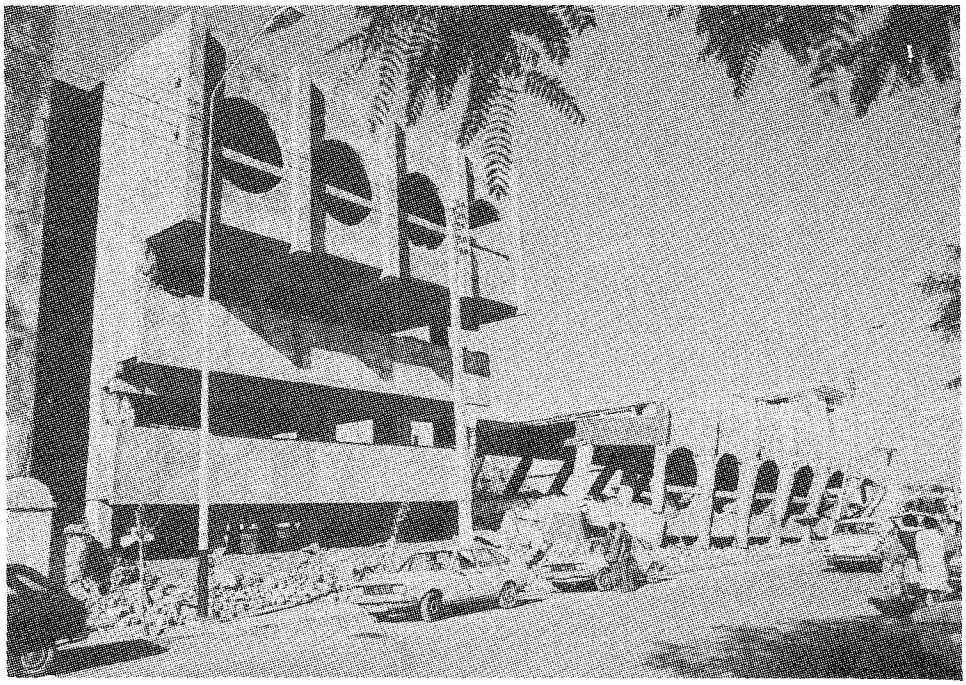
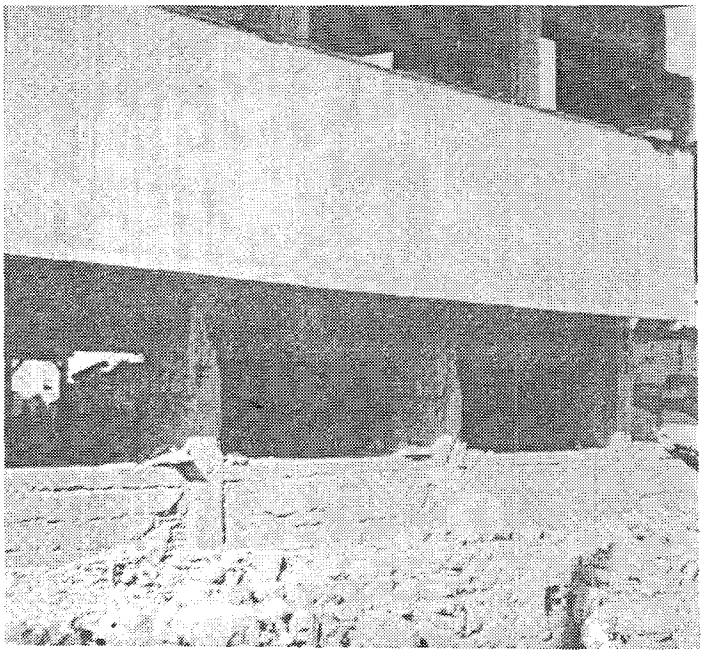


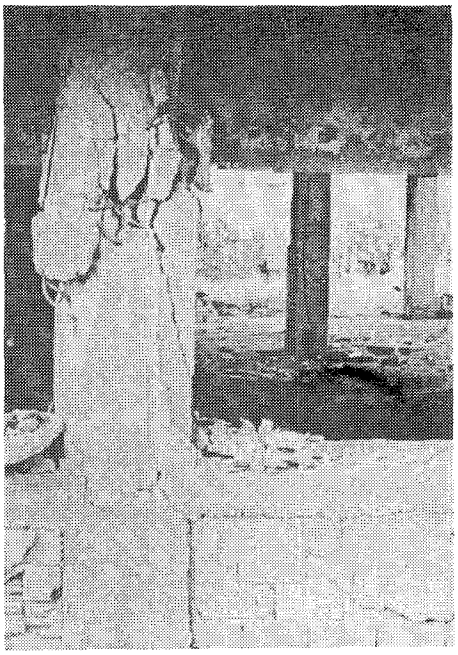
Fig. 4.7 Very Stiff and Heavy Girder Supported on Relatively Weak Columns



(a) Two Cultural Center Buildings. One collapsed completely; the other remained standing with moderate damage.

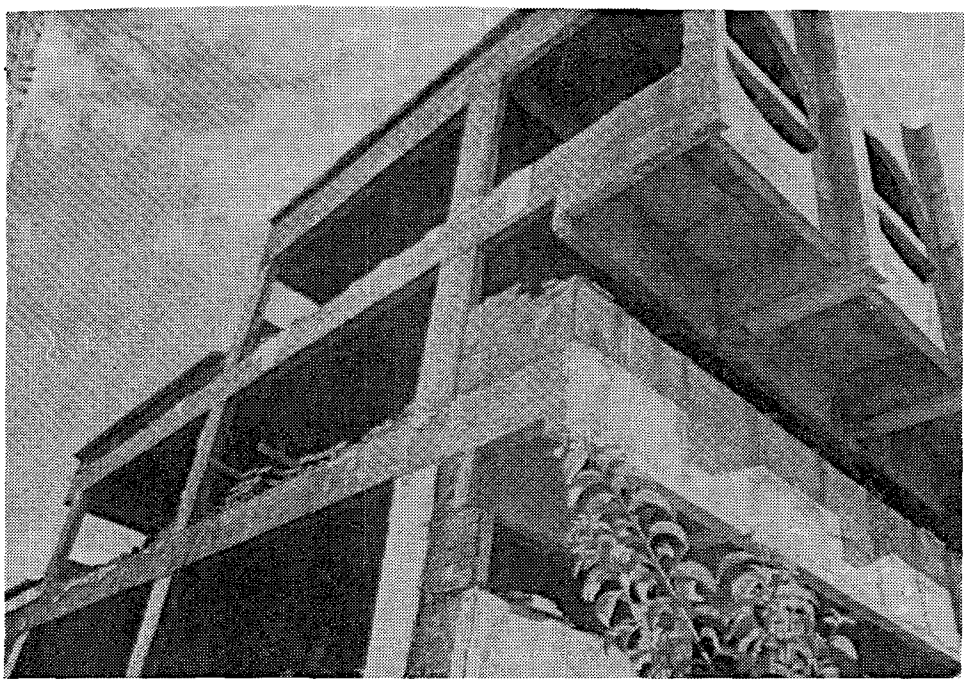


(b) First-Story Short Column Failure



(c) Closeup of Short Column Failure

Fig. 4.8 Damaged Cultural Center Building



(d) Architectural Form Which May Have Aggravated Structural Weaknesses During Earthquake

Fig. 4.8 (cont.)

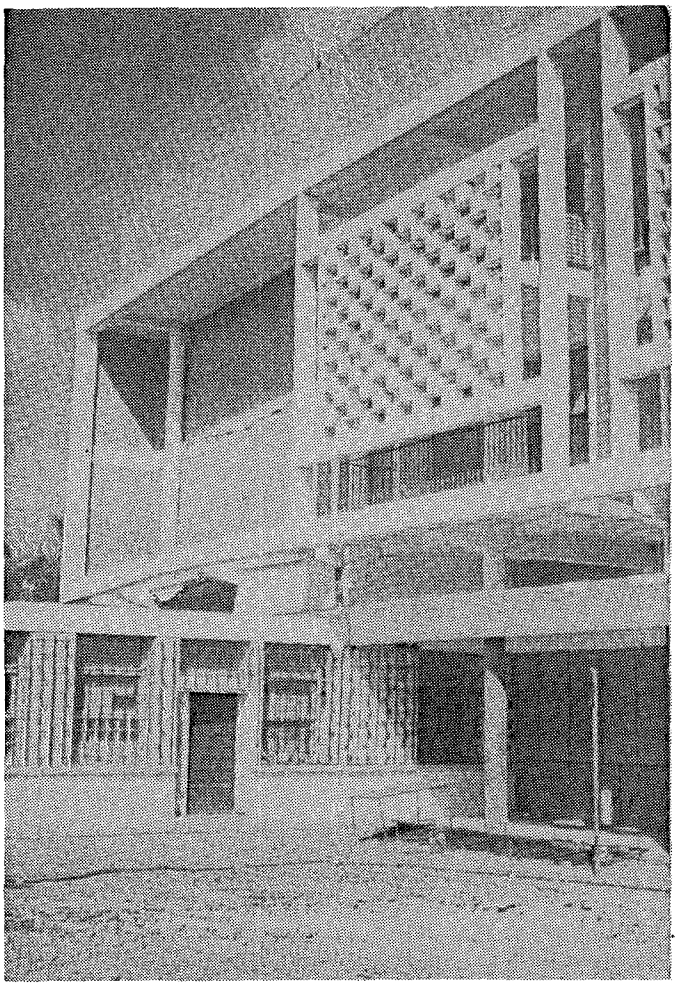


Fig. 4.9 Heavy Ornaments on Facade of Hall of Justice

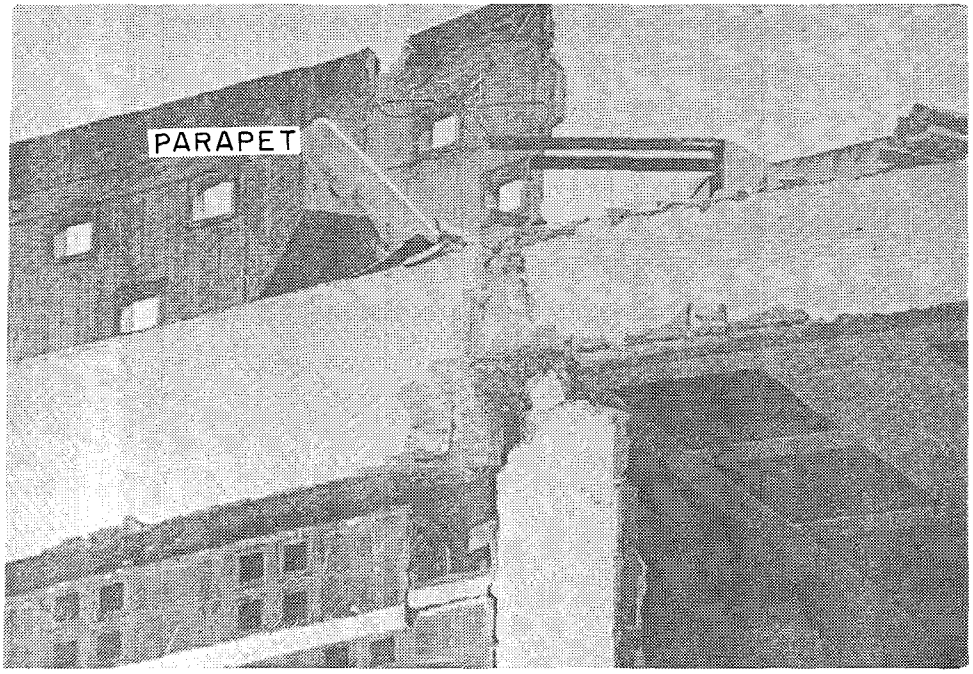


Fig. 4.10 Parapets on Roof of Cité An Nasr

5. **Use of long, heavy cantilevers,** as in the cultural center (Fig. 4.8d) and the An Nasr (Fig. 4.4) buildings. This led to significant vertical deformation and considerable nonstructural damage.

6. **Use of thermal expansion construction joints** about each 20 meters (66 ft) or at the most 30 meters (98 ft), **without proper separation between adjacent parts of the building,** which allowed battering between buildings (Figs. 4.4 and 4.5).

7. **Use of strong girder/weak column, moment-resisting frame structural systems** (Figs. 4.7 and 4.11). Few buildings had RC shear walls. In some new apartment buildings shear walls were used, but most were built in one direction only. Shear walls were also used to construct elevator shafts in multistory buildings. Unfortunately, in some cases, these RC wall shafts were not symmetrically situated structurally and, therefore, induced large torsional moments, as in one of the three blocks of the new medical clinic (Figs. 4.5 and 4.12).

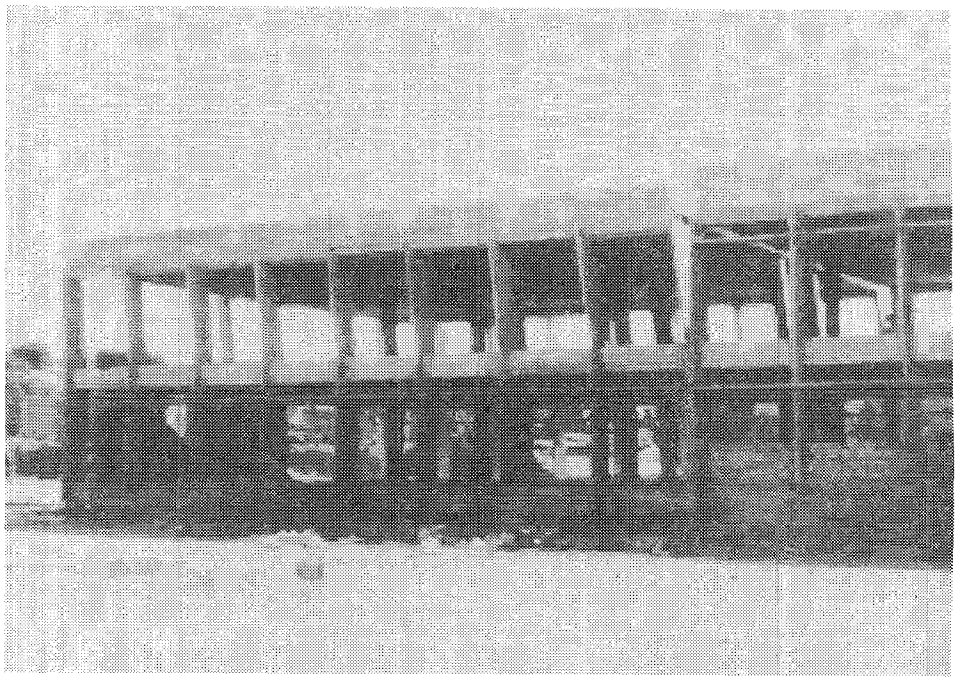
In summary, it is believed that one reason for poor seismic performance was the poor architectural conception of the building and poor structural configuration in relation to seismic-resistant construction. The configurations used usually contained a combination of several of the defects just discussed. For example, several new housing projects under construction, as the one in Boucaa Sahnoun (Fig. 4.13), a district of El-Asnam, and one in El-Attaf (Fig. 4.14), had structural systems consisting of an RC frame similar to that proposed by Le Corbusier (shown schematically in Fig. 4.15). This type of frame using very slender columns, a stiff floor system, and a particularly heavy roof to provide thermal insulation, led to construction of a weak column/strong floor system. This was aggravated by the use of unsymmetrically placed stairs, clearly illustrated in Figs. 4.14 and 4.15.

The defects and weaknesses in the design and construction mentioned above point out the need for a comprehensive approach to building design, an approach that would include designing for all environmental conditions to which the building can be subjected. In the case of buildings in El-Asnam, these include atmospheric conditions *(vide sanitaire* at ground level and thermal insulation on the roof) as well as seismic conditions.

4.1.2 Poor Structural Material. As already indicated, the most prevalent structural material used was reinforced concrete, which generally was unconfined. It is well known that ductility of such a material, unless well detailed, is limited. In most cases, quality and placement of concrete were poor. It was sometimes evident that adequate amounts of cement were lacking, or there was a poor gradation of aggregate with complete lack of fine aggregate, and in some cases, the use of too coarse aggregate. In some buildings under construction, it was observed that the aggregate, particularly the sand, was of poor quality and badly contaminated with dirt (Fig. 4.16). Combinations of these factors were observed in the two-story houses under construction in Boucaa Sahnoun, a district of El-Asnam (Figs. 4.13 and 4.17).

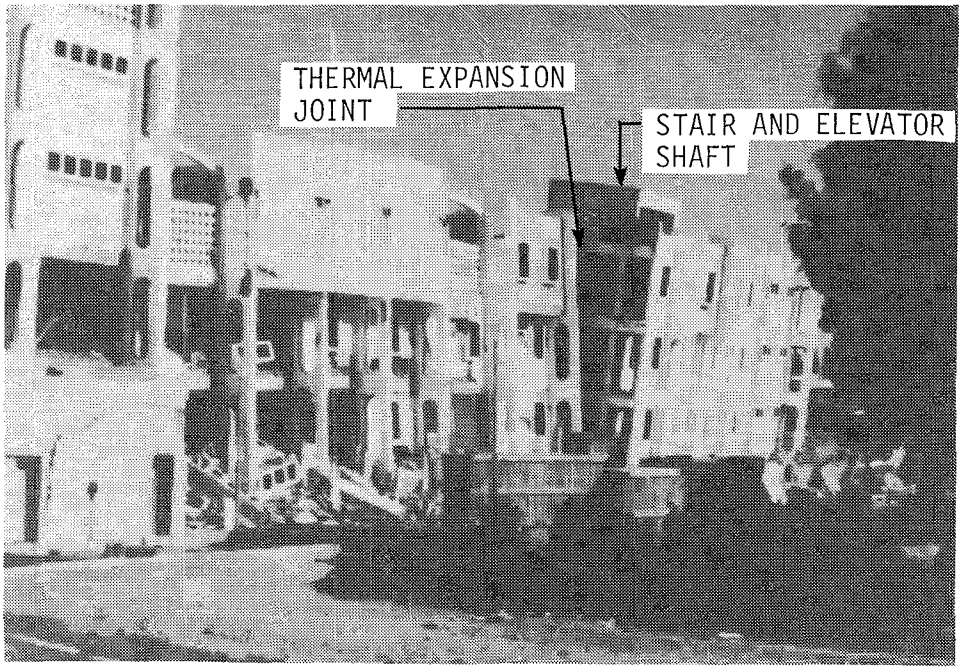
In several instances considerable honeycomb was observed with complete lack of mortar; it was possible to break away large parts of concrete with just a screwdriver or the kick of a boot (Fig. 4.18). Proper attention was not given to concrete mixing, placement, consolidation, or curing. The poor quality of the unconfined concrete was demonstrated by its disintegration in the regions of the members that were overstressed (Fig. 4.19). It was stated that the concrete compression strength in El-Asnam frequently falls below 120 kg/cm^2 (1740 psi) or less than 45 percent of minimum required design strength, which is generally 270 kg/cm^2 (3915 psi). The quantity of

(a) Collapsed Apartment House. Buildings in background had shear walls and performed well.

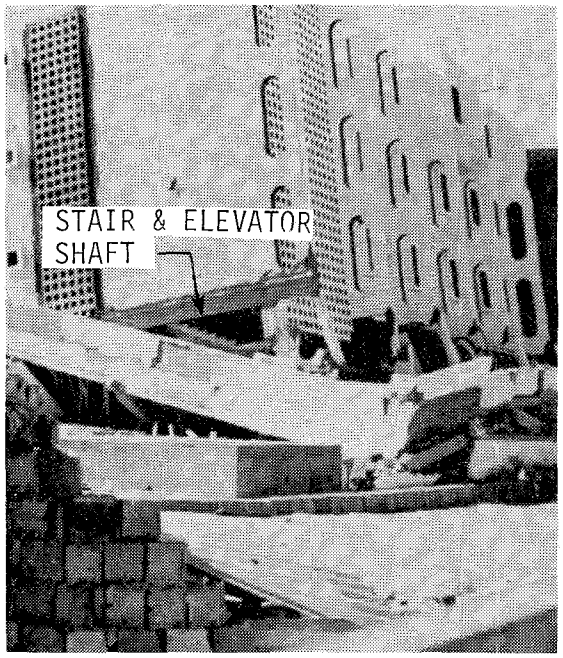


(b) Apartment House Under Construction. Note hinging of second-story weak columns.

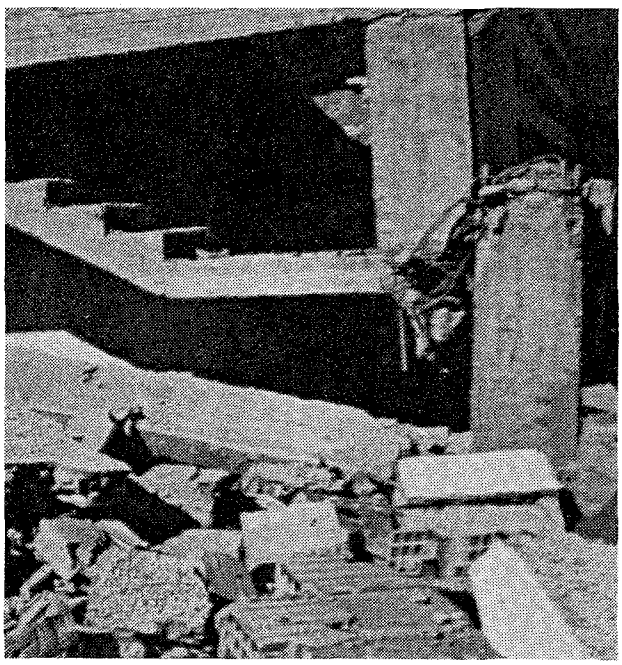
Fig. 4.11 Strong Girder/Weak Column Construction



(a) Overall View of Three Blocks of Clinic



(b) Corner Where Shaft Is Located



(c) Closeup of Column Failure Due to Stairways

Fig. 4.12 New Medical Clinic. Note stair and elevator shaft at corner of building.

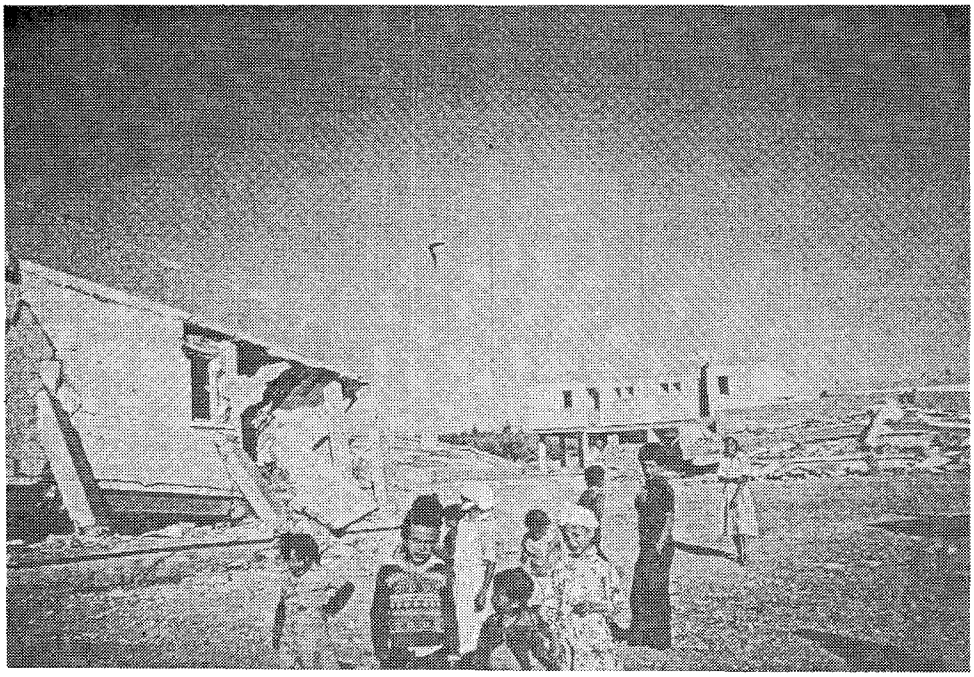


Fig. 4.13 Collapsed Two-Story House Under Construction, Boucaa Sahnoun

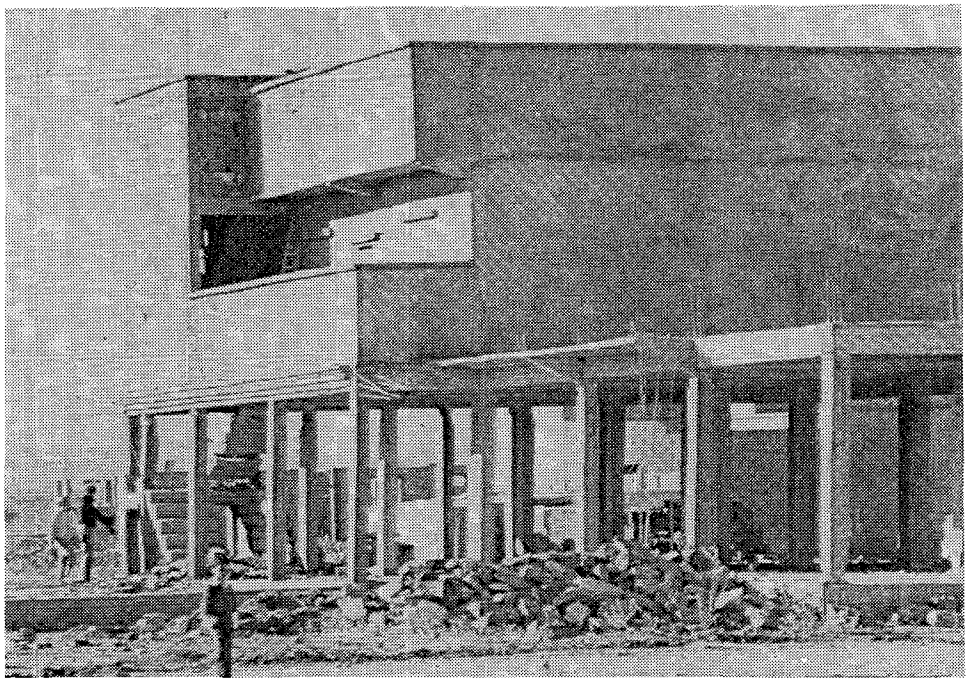
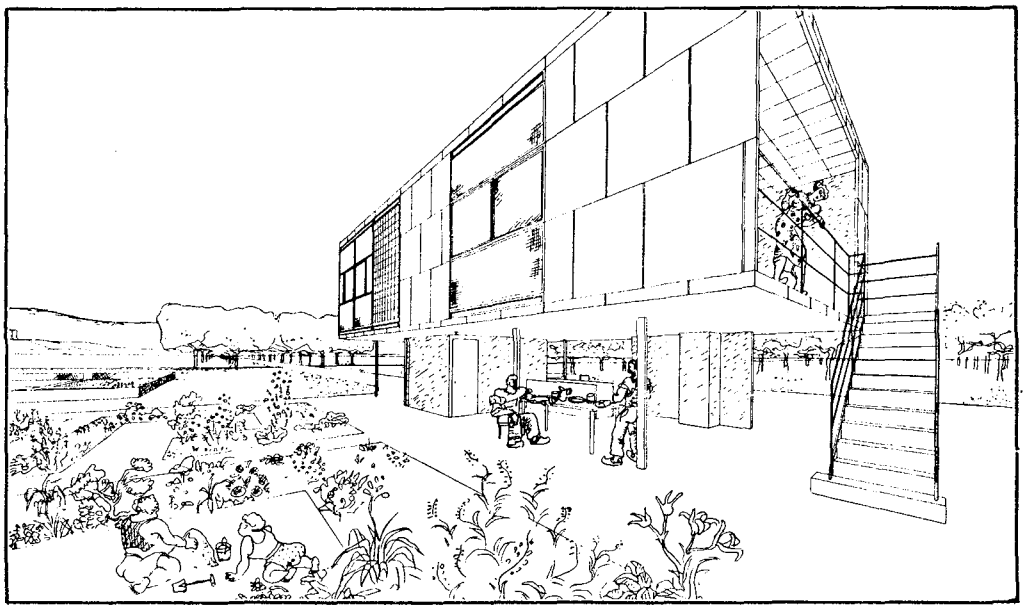
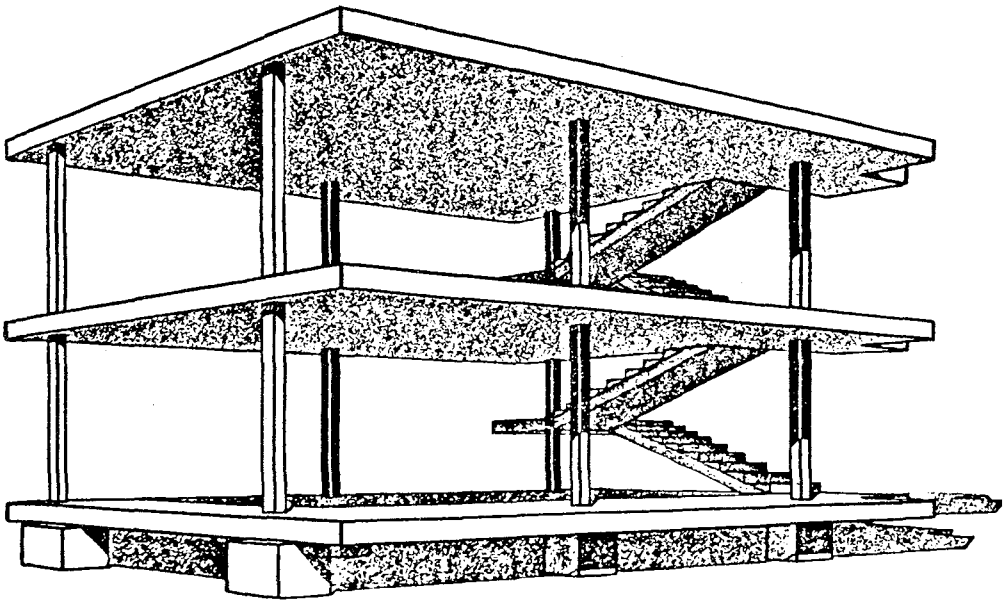


Fig. 4.14 Damaged Three-Story House Under Construction, El-Attaf



(a) Peasant Housing (Raising Dwelling from "Mother Earth")



(b) Famous Drawing of RC Frame

Fig. 4.15 Le Corbusier's Architectural Concept, Recognizable in Common Construction of Most Regions of Modern World, Including Algeria (1929, reprinted 1967)

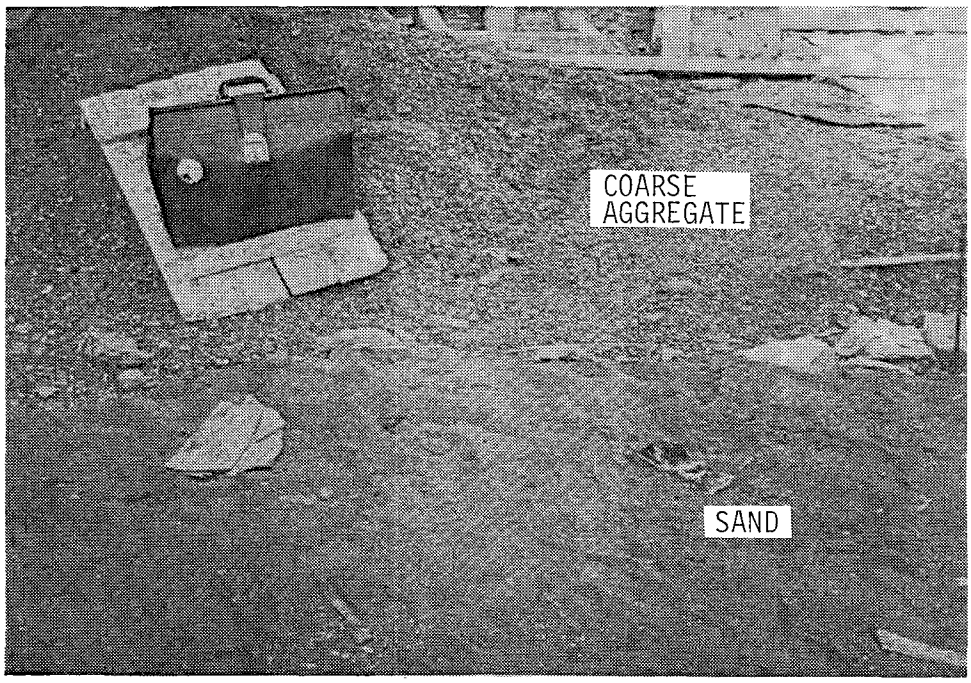
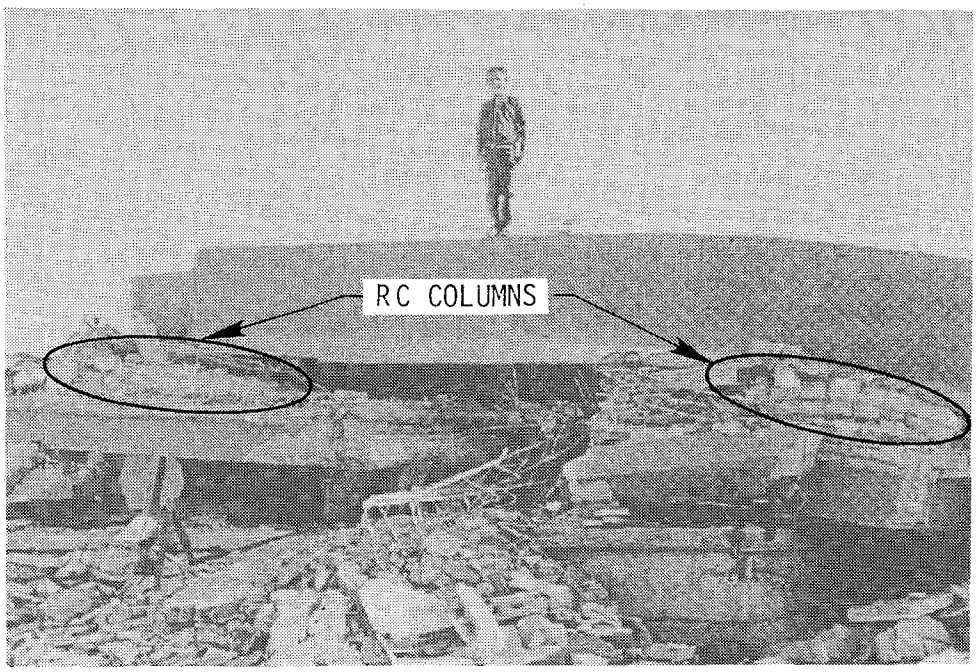
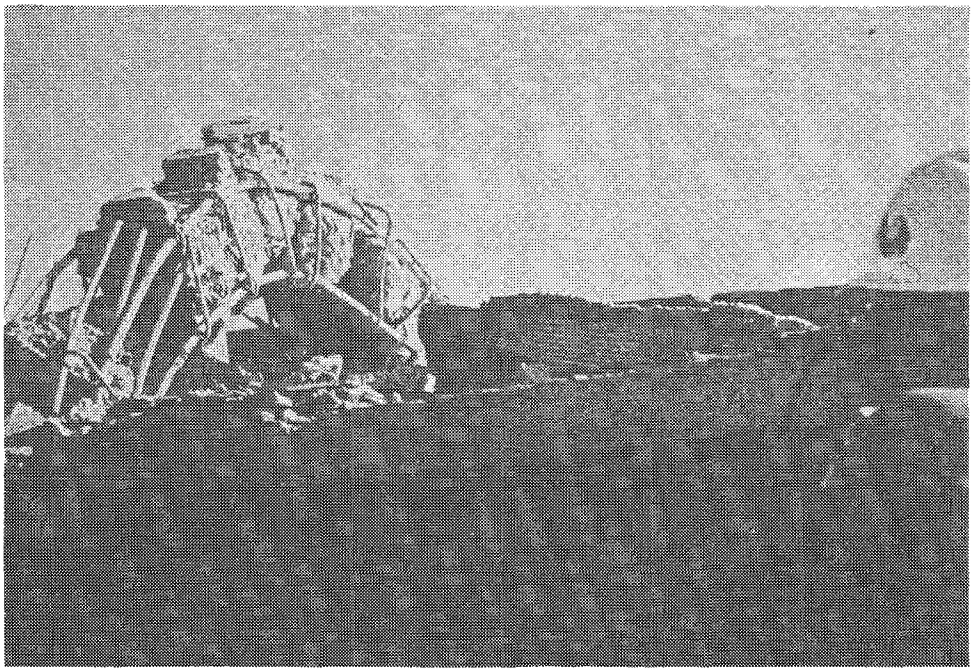


Fig. 4.16 Contaminated Aggregate Used in RC Construction

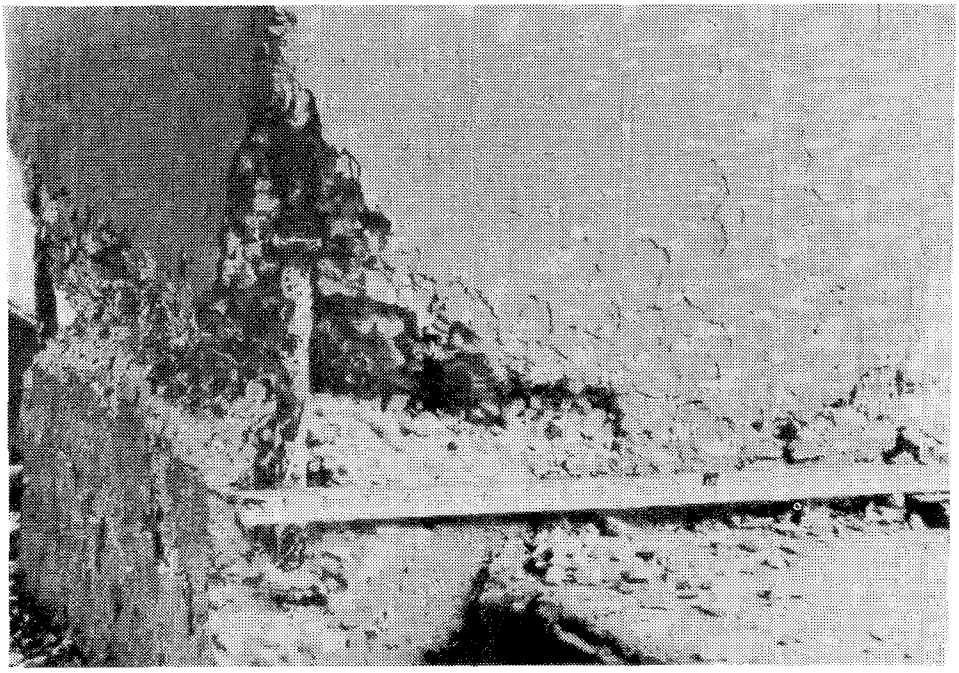


(a) Disintegrated Concrete in Columns

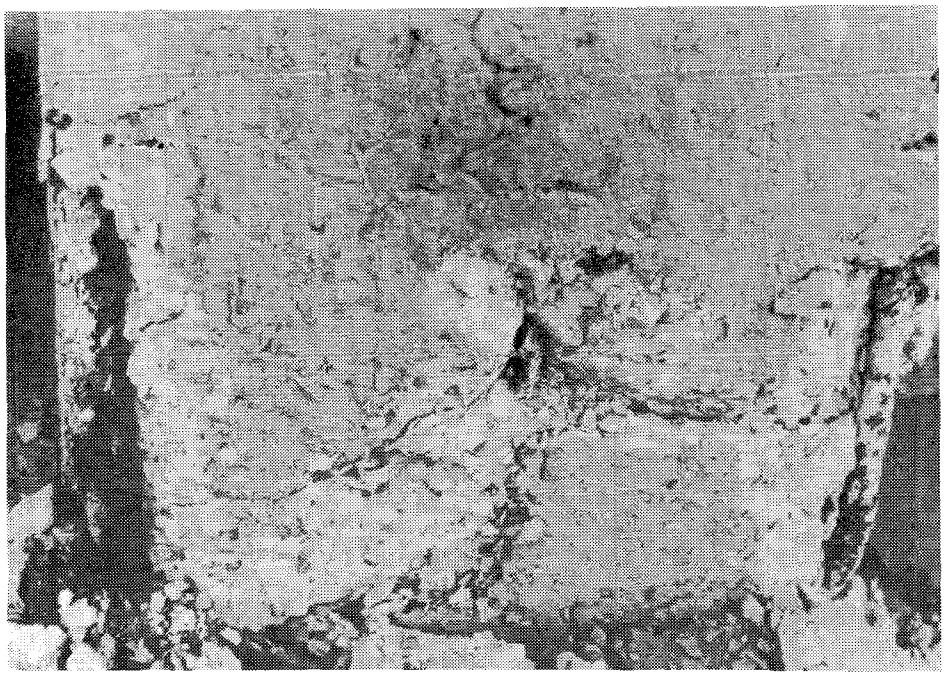


(b) Lack of Confinement and Poor Quality Concrete

Fig. 4.17 Collapsed Two-Story Houses Under Construction, Boucaa Sahnoun



(a) Concrete Removed by Kick of a Boot



(b) Honeycomb in Concrete and Placement of Reinforcement with Practically No Cover

Fig. 4.18 Examples of Poor Quality of Concrete in Columns of Cité An Nasr Building

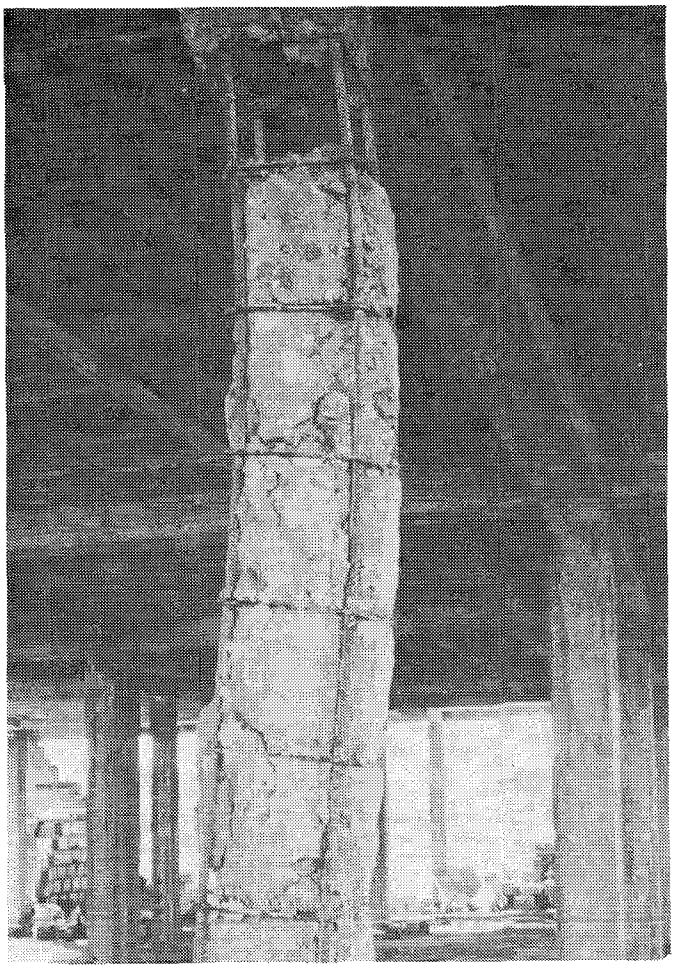


Fig. 4.19 Concrete Column That Disintegrated When Subjected to Overstress

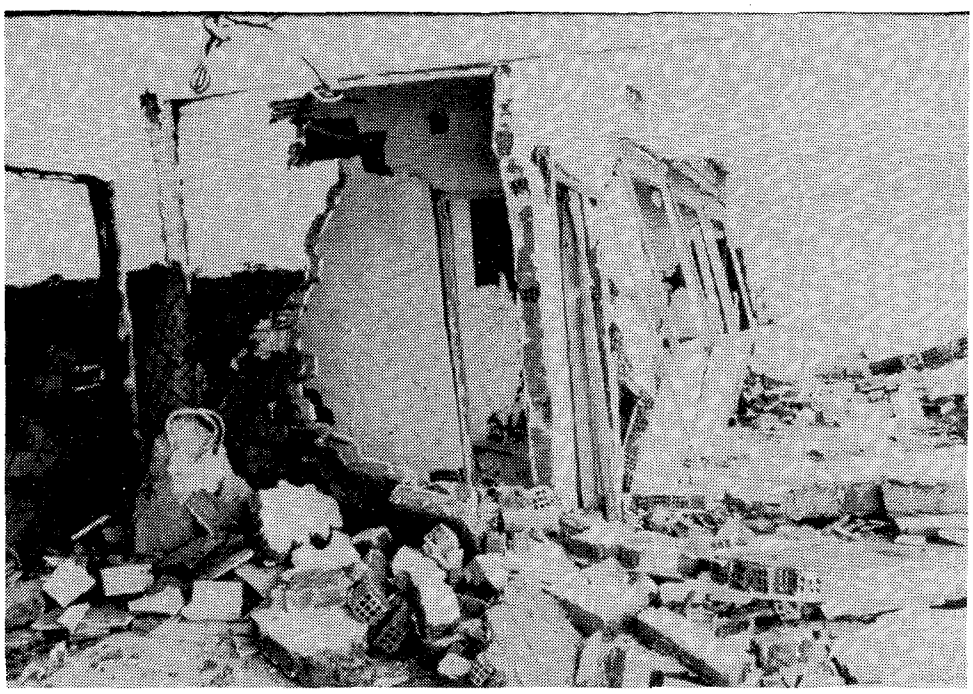


Fig. 4.20 Heavy Damage to Interior Partitions of Buildings That Remained Standing

construction apparently overtaxes the availability of competent contractors, skilled workers, and quality control personnel. Although good concrete material is available, the demand is so extensive that good concrete is often mixed with other, poor quality, material. It should be noted that most concrete is mixed at the site with the foreman in control of the mix.

For the main reinforcing steel, deformed bars with yield strength of about 2800 kg/cm^2 (40,000 psi) were generally used. However, in several buildings, twisted deformed bars were also observed; and in some buildings built after the 1954 earthquake, e.g., Cité An Nasr, plain bars were found. In general, small bars [smaller than 29 mm (#9)], were used as main reinforcement and in small amounts, i.e., the concrete was only lightly reinforced, particularly in the columns. The transverse reinforcement in most columns consisted of plain 6 mm (#2) bars, usually spaced at 180 mm (7 in.) or more.

4.1.3 Use of Nonstructural Elements. Building construction in the region of El-Asnam was characterized by the use of heavy, stiff architectural components and systems. As already pointed out, the roofing system was heavy (Fig. 1.16). In addition, tall, heavy, unreinforced parapets were built on the roof (Fig. 4.10). In some new buildings, heavy ornamentation was added to the facade (Figs. 4.5, 4.6, 4.8, and 4.9). The majority of stairways were constructed of reinforced concrete and attached to the columns at midheight of each story, thus creating short columns which failed during the quake (Figs. 4.12 and 4.14). Furthermore, because of their unfortunate location and stiffening effects, these stairways caused significant torsional moment in the whole building during earthquake ground motions.

Perimeter walls and internal partitions were generally constructed of heavy, stiff, and unreinforced hollow clay tile brick and concrete block masonry. In some older buildings, solid brick, stone, or adobe were used. These walls and partitions, which divided the building into rooms 3 x 3 meters (10 x 10 ft), had significant effect on the behavior and safety of the structures. The walls and partitions were not properly integrated with the structural system: RC frame. These walls were usually infilled between the RC framing elements although they were sometimes constructed outside the columns without any attachment. In some new buildings in which the structure remained standing, the amount of debris produced by the collapse of these unreinforced filler walls and partitions was so extensive that it prevented the team from entering the rooms (Fig. 4.20). Large portions of collapsed partitions blocked the stairways of several buildings, making evacuation difficult.

4.1.4 Inadequate Proportioning and Detailing of Structural Elements, Connections, and Supports. It is well known that there are large uncertainties in estimating the demands and strengths in earthquake-resistant design. The best approach to overcome these uncertainties is proper selection of the building configuration, its structural layout, and proper proportioning and detailing of the structural elements and their connections and supports. Unfortunately, the importance of these aspects, which are pointed out in AS 1955 Recommendations (Appendix A), was ignored in the reconstruction of El-Asnam after the 1954 Orléansville earthquake.

As previously noted, the proportioning and detailing of columns relative to the girders led to a weak column/strong girder system. Furthermore, in several cases it was observed that the amount of main reinforcement in the column was very low; for example, 45 x 52 cm (18 x 20.5 in.) columns having only eight D19 mm* (#6) bars as main reinforcement, i.e., less than 1 percent of steel.

*D19 is size in millimeters.

Cases of columns with only 0.8 percent total main reinforcement were found. This, together with the use of poor quality unconfined concrete, led to the building's collapse because of the initial failure of the columns. The spacing, size of the ties, and the hooks used in columns were not according to recommendations for achieving close ties (hoops), a prerequisite for achieving confined concrete in seismic-resistant construction. Usually 6 mm (#2) ties at 200 mm (8 in.) were used.

In general, the girders were provided with sufficient longitudinal reinforcement although the amount and detailing of the transverse reinforcement in the critical regions (ends of the girders) were not adequate.

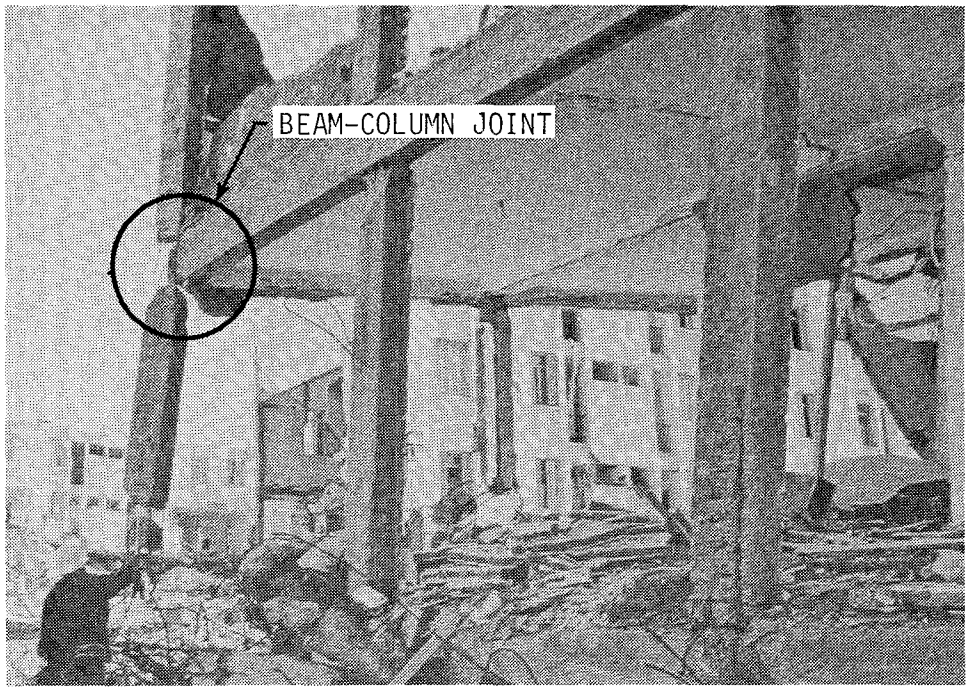
The design, detailing, and construction of the column-girder connections were inadequate in many cases (Figs. 3.27 and 4.21). The main drawbacks were the following: 1) little or no transverse reinforcement at the joint, 2) inadequate anchorage of the bars (this was particularly noted in the anchorage of the column bars in the roof girders), and 3) absence of hooks and so short an embedment length that the column bars pulled out completely from the girder (Fig. 4.22). Splicing of the main longitudinal reinforcement of the column was also observed to be poor. Lap splicing was generally done at the bottom or top of the column and was inadequate (Fig. 4.23).

4.1.5 Poor Inspection and Construction Techniques. The quality control of the structural materials and the workmanship were inadequate, and these deficiencies contributed significantly to building failures. As previously noted, the quality of the concrete was poor. Further, placement of the reinforcement bars was inadequately controlled. Cover of the main reinforcing varied from 0 to 8 cm (3 in.) (Fig. 4.18), and spacing of the ties was not uniform, which clearly reveals that the steel cage was either 1) incorrectly constructed or 2) not placed correctly or not kept in its correct position during placement of the concrete. It is believed that these features of poor concrete construction could be eliminated by adequate inspection during construction.

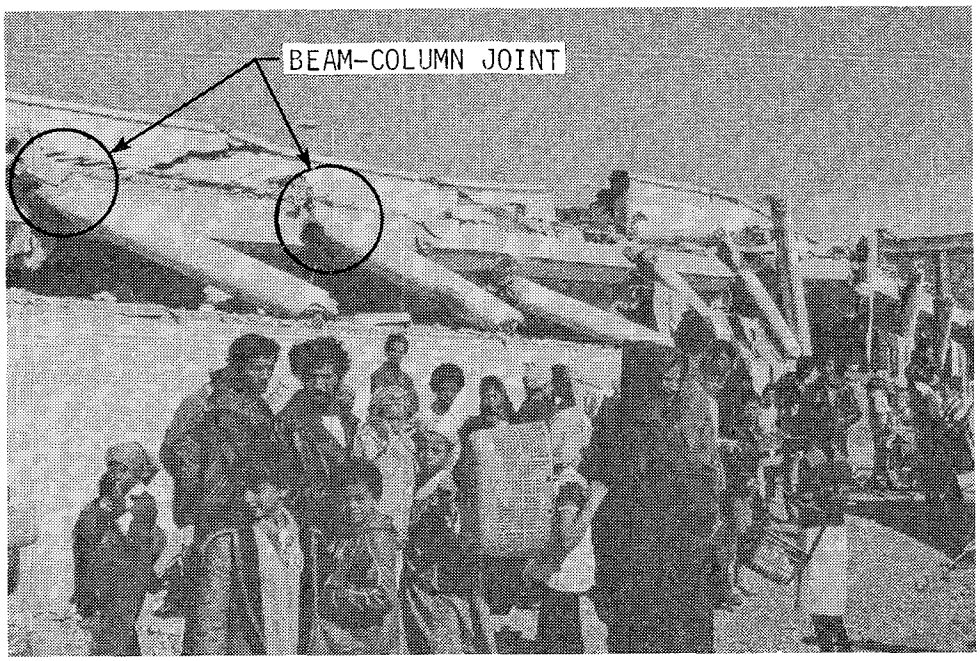
Several two and three-story buildings under construction failed during the earthquake because of the construction technique used. This technique consisted of first constructing the RC frame and then building the masonry walls and partitions, beginning at the top story and proceeding downward (Figs. 4.13 and 4.14). This technique led to destruction when the earthquake found a building with a soft first story, as illustrated in Fig. 4.24.

4.1.6 Inadequate Building Maintenance. In many cases it was obvious that the lack of adequate cover or the presence of large cracks caused significant corrosion of the reinforcing steel. Considerable corrosion of the column steel was observed in some columns at Cité An Nasr.

4.1.7 Concluding Remarks. A confirmation of the belief that the extensive building destruction in El-Asnam was due more to general inadequacy in design and construction rather than the severity of ground motion can be found in the fact the some buildings survived without significant damage. Some of these buildings were designed against seismic forces, as was the case at the cement plant, while in others the construction appears to have followed basic seismic-resistant design and construction principles although these buildings were not designed against seismic forces. Examples of this were the upgraded building shown in Fig. 1.14 and the house occupied by CTC (Fig. 4.25).

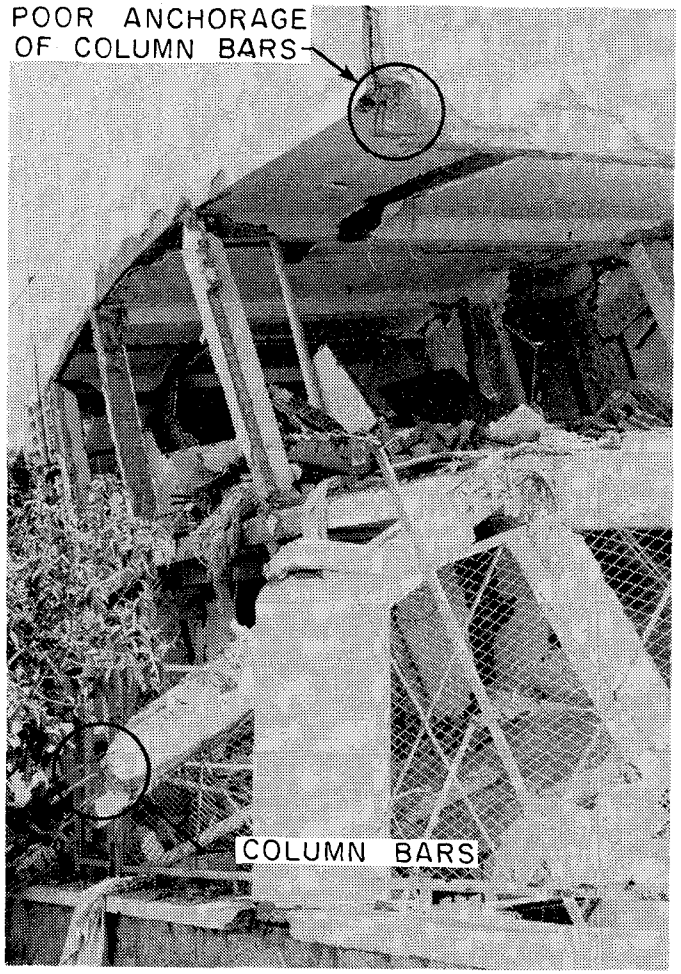


(a) Apartment Building

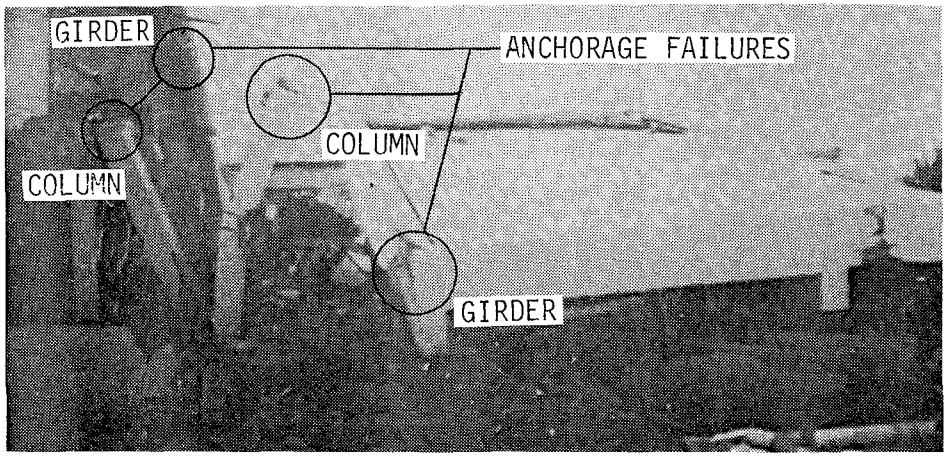


(b) School (see Fig. 3.27)

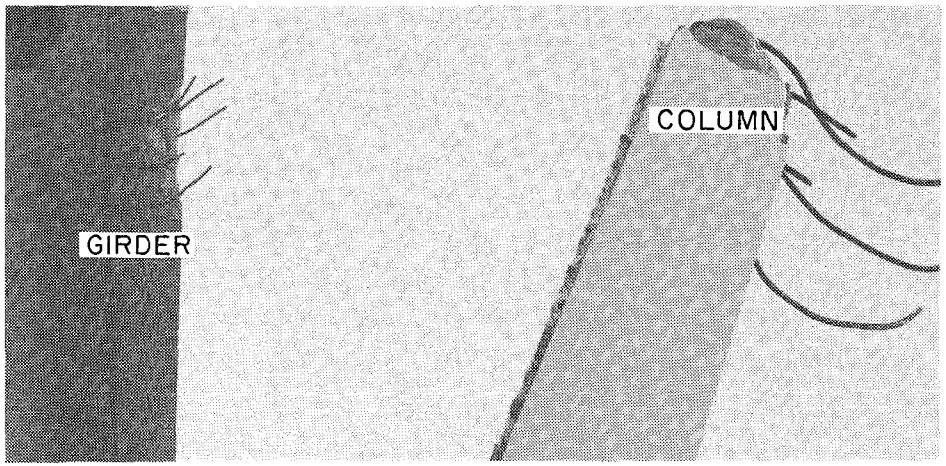
Fig. 4.21 Examples of Poor Beam-Column Joints



(a) Collapsed Two-Story House



(b) Collapsed Industrial Plant Office Building
(see Fig. 3.39)



(c) Closeup of Anchorage Failure

Fig. 4.22 Inadequate Anchorage of Column Reinforcement in Roof Girders

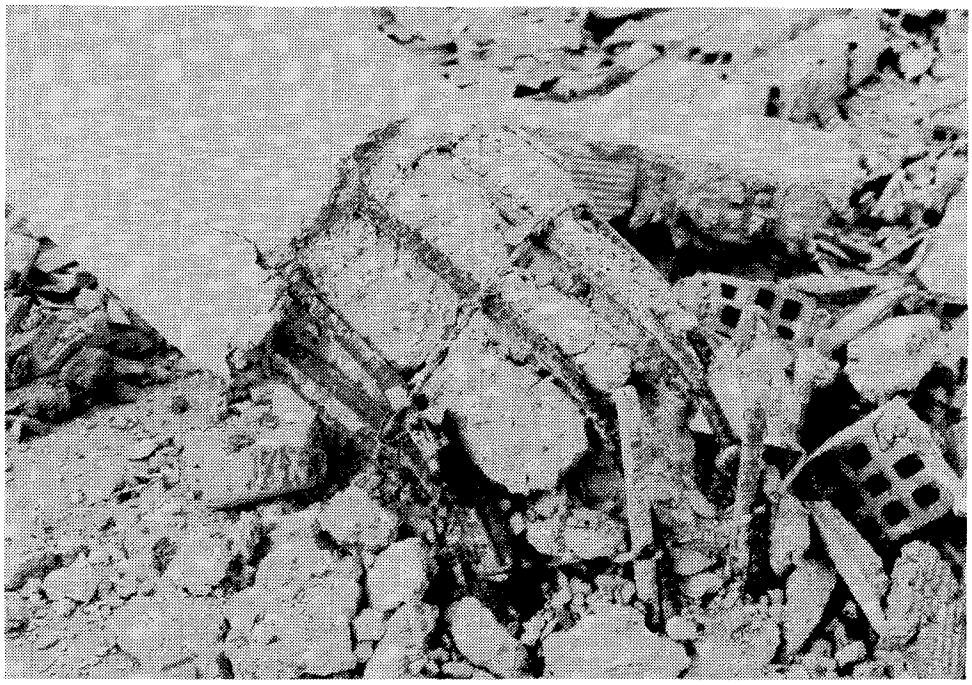


Fig. 4.23 Inadequate Lap Splicing of Main Reinforcement at Bottom of Column

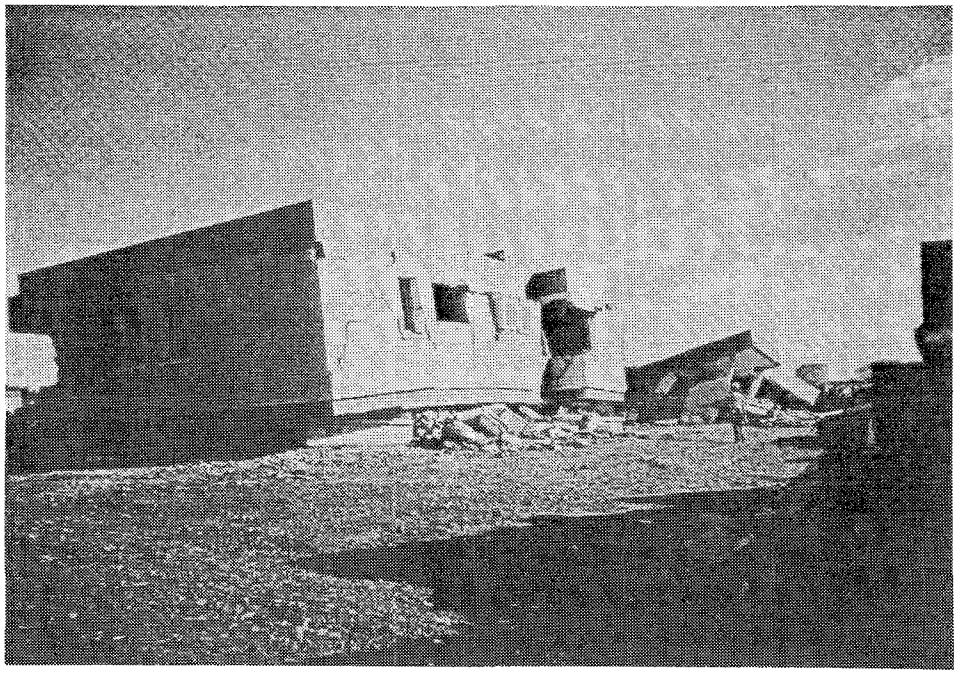


Fig. 4.24 Collapsed Two-Story Houses Under Construction in Boucaa Sahnoun That Had Soft First Story

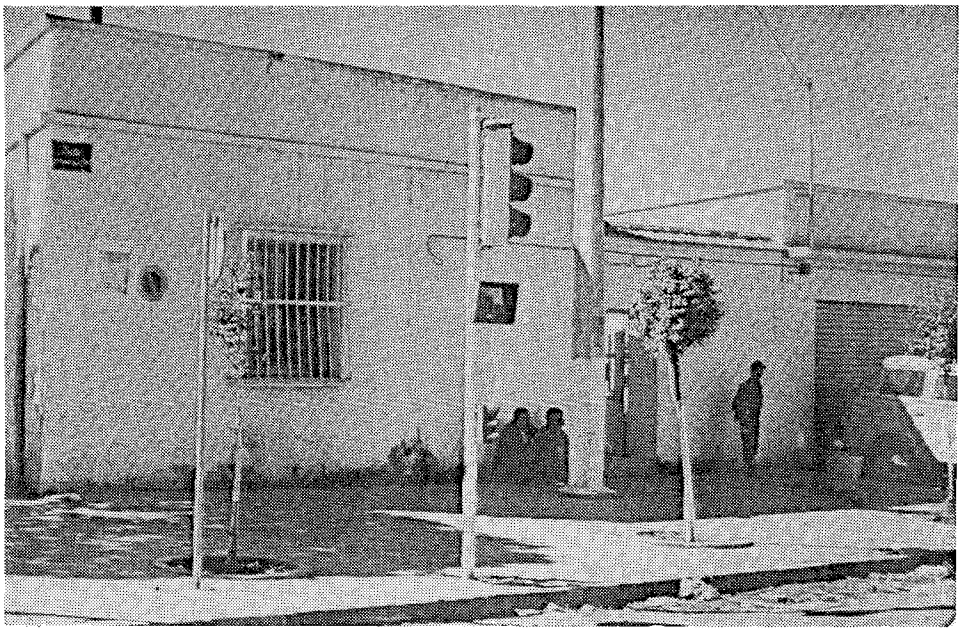


Fig. 4.25 House Occupied by CTC That Performed Very Well

Although Orléansville (El-Asnam) was destroyed by an earthquake in 1954, buildings were typically redesigned and constructed without following elementary rules of seismic-resistant design and construction formulated as a consequence of the 1954 damage. These rules were contained in the seismic code specification AS 55 developed for Algeria by the French (see Appendix A). Furthermore, in January 1955, Rothe in an article published in *La Nature* discussed the characteristics of the September 1954 earthquake and stressed the need for seismic-resistant construction in Algeria, particularly in the region of Orléansville (El-Asnam). Rothe formulated rules to attain adequate seismic-resistant construction. His suggestions and the AS 55 rules were generally not rigorously enforced in the rebuilding of Orléansville (El-Asnam).

4.2 Case Studies: Performance of Two Buildings

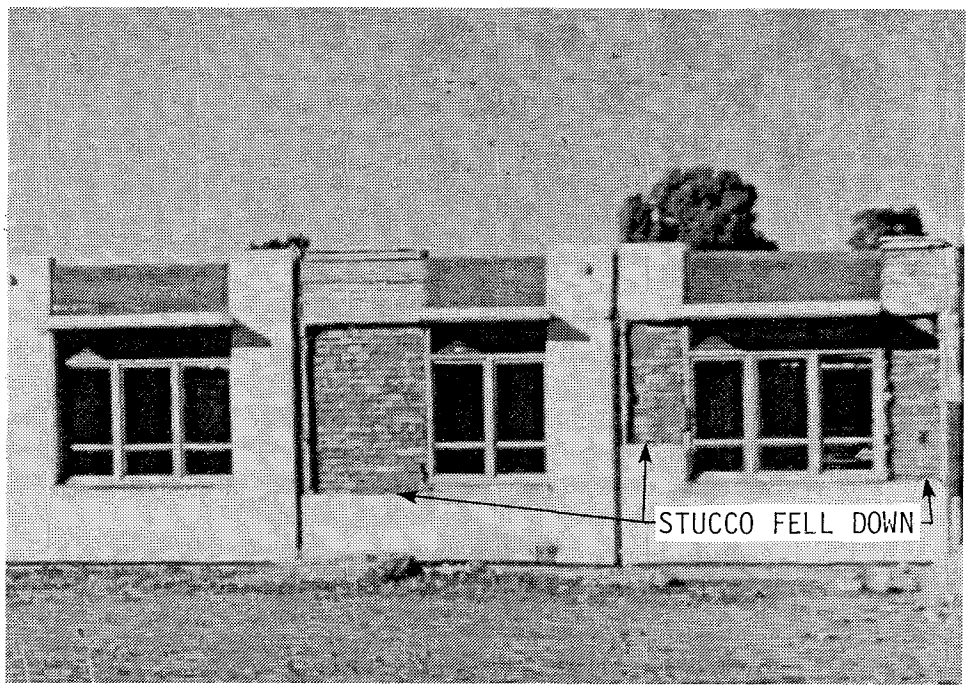
No record was obtained of the main October 10, 1980 shock or aftershock, and no design calculations or design and construction drawings were available to members of the two reporting teams. Therefore, it was not possible to conduct detailed numerical analyses of building performance. However, approximate analyses of the following two buildings were performed and are discussed in the following paragraphs.

4.2.1 Primary School 5 km East of El-Asnam. This school comprised several one-story buildings belonging to category 2 of the classifications listed in section 1.6. The classroom buildings were very low, and their structural systems consisted of RC frames infilled with unreinforced masonry. Except for small cracking along the boundaries of the infilled walls, some typical shear diagonal cracking in these walls, and crumbling of stucco on the facade, no other nonstructural damage was observed (Fig. 4.26a).

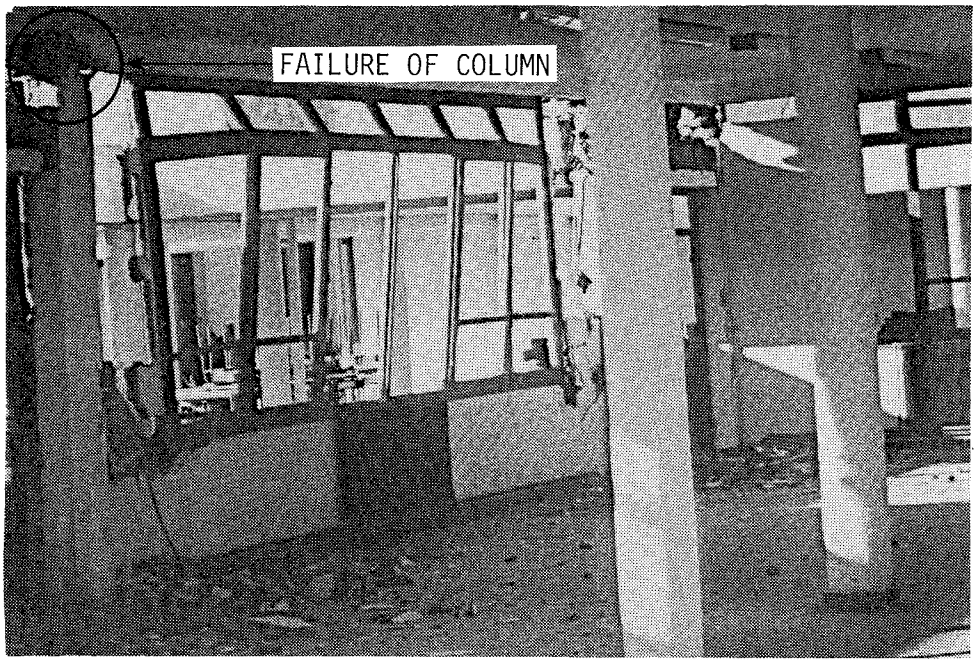
However, there was significant structural damage in the main hall of the school and on several of the canopies. This main hall, which had large window openings, was severely damaged, including structural damage, as illustrated in Fig. 4.26b.

The other important structural damage occurred in canopies over corridors along and between classrooms. These canopies were supported by one row of cylindrical columns 35 cm (13.8 in.) in diameter and having a clear height of 2.5 meters (99 in.) (see Fig. 4.27). While some of these canopies collapsed, as illustrated in Fig. 4.28, others located between two adjacent classroom buildings remained standing but were thrown out-of-plumb, with some significant permanent lateral deformation (Fig. 4.29). The questions are: Why did some canopies collapse while others did not? What force (or ground acceleration) could produce such collapse? Or, even more difficult to answer, what ground motion intensity would produce this type of behavior?

Because these canopies are bare structures and simple to model mathematically, it was decided to use their performance as transducers to evaluate the severity of the ground motions. From the dimensions of these canopies and from an estimation of the mechanical characteristics of their materials, it has been possible to calculate the seismic force required to produce such a performance. From this calculation it was possible to estimate the potential ground motion that could induce such force. The computations given in Appendix C show that the effective horizontal peak acceleration could have been between 0.35 g for the derived 1971 Pacoima Dam accelerogram (impulsive ground motion) and 0.55 g for the 1940 El Centro accelerogram.

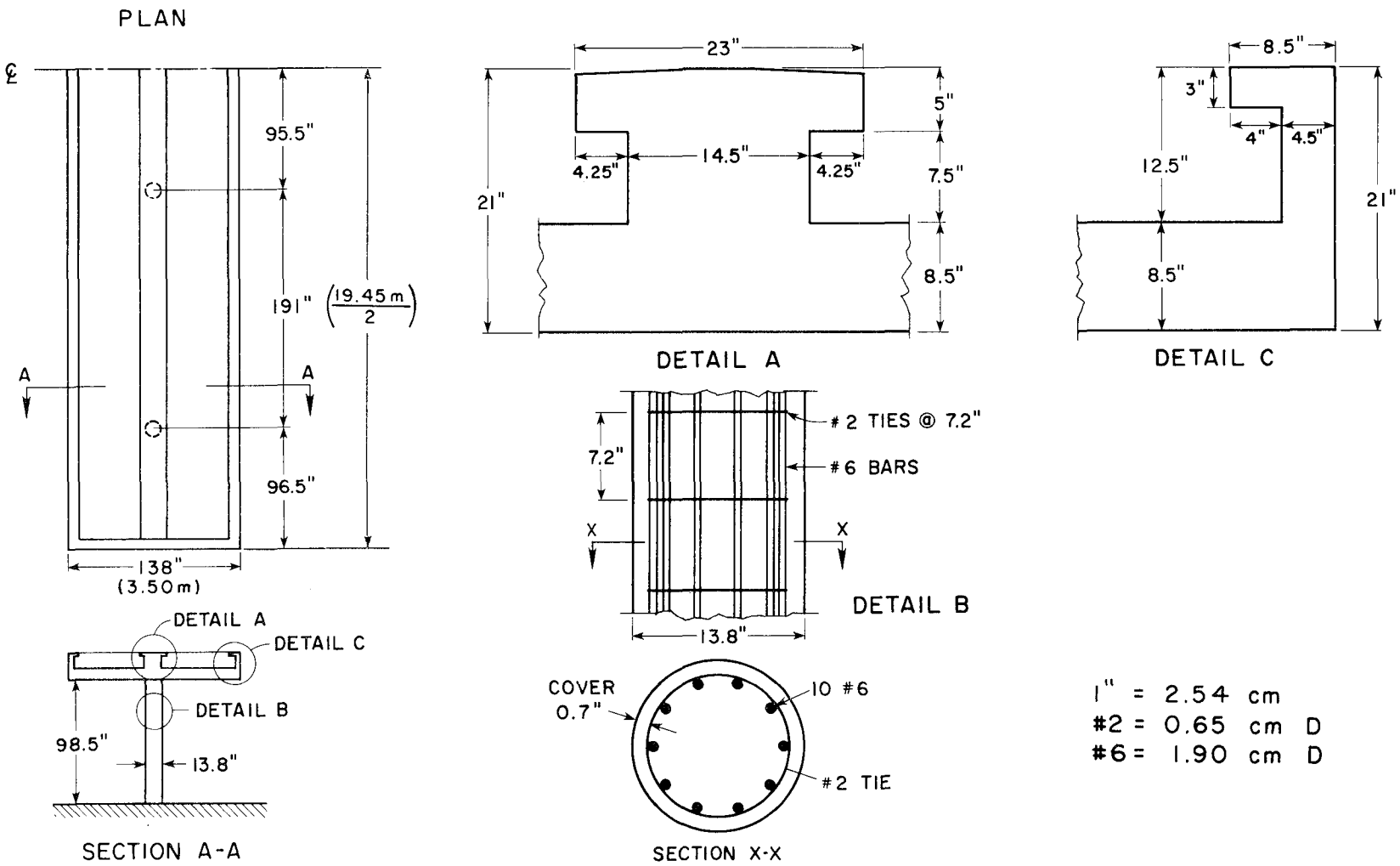


(a) Classroom Buildings with Only Minor Nonstructural Damage



(b) Main Hall, Showing Broken Windows and Structural Damage

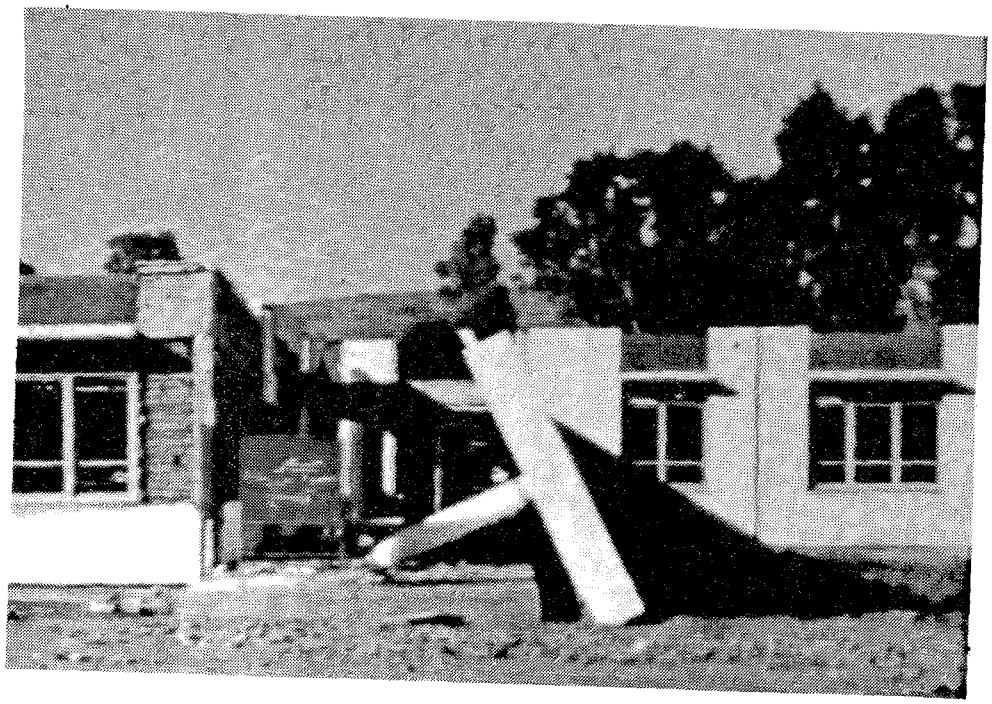
Fig. 4.26 Primary School



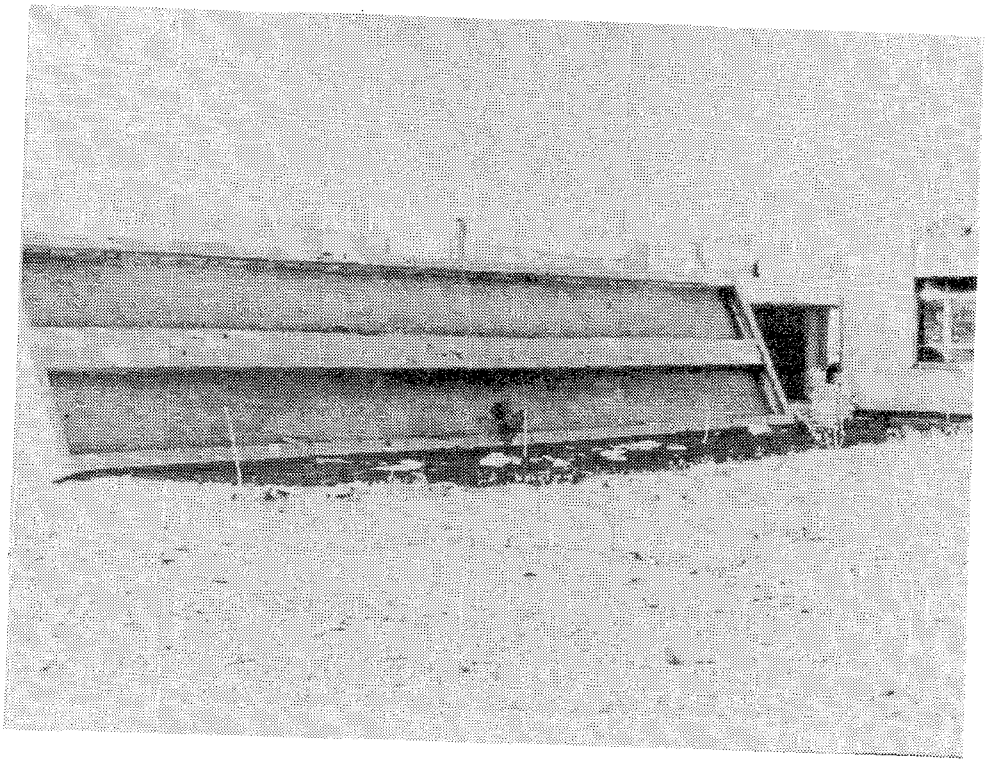
(a) Structural Drawings

(b) Details

Fig. 4.27 Primary School Canopies

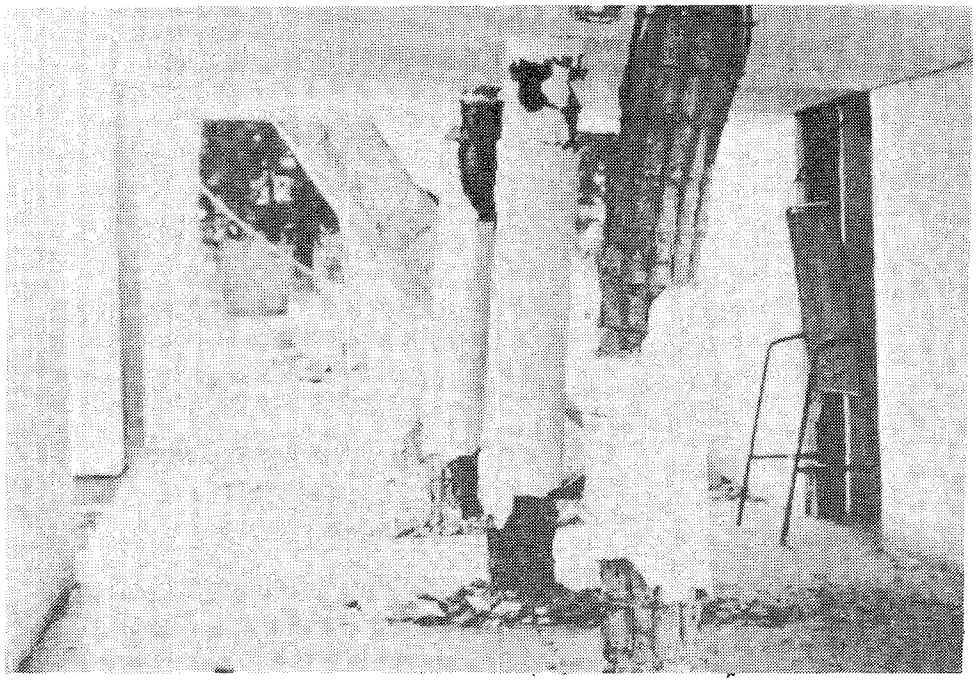


(a) Front View

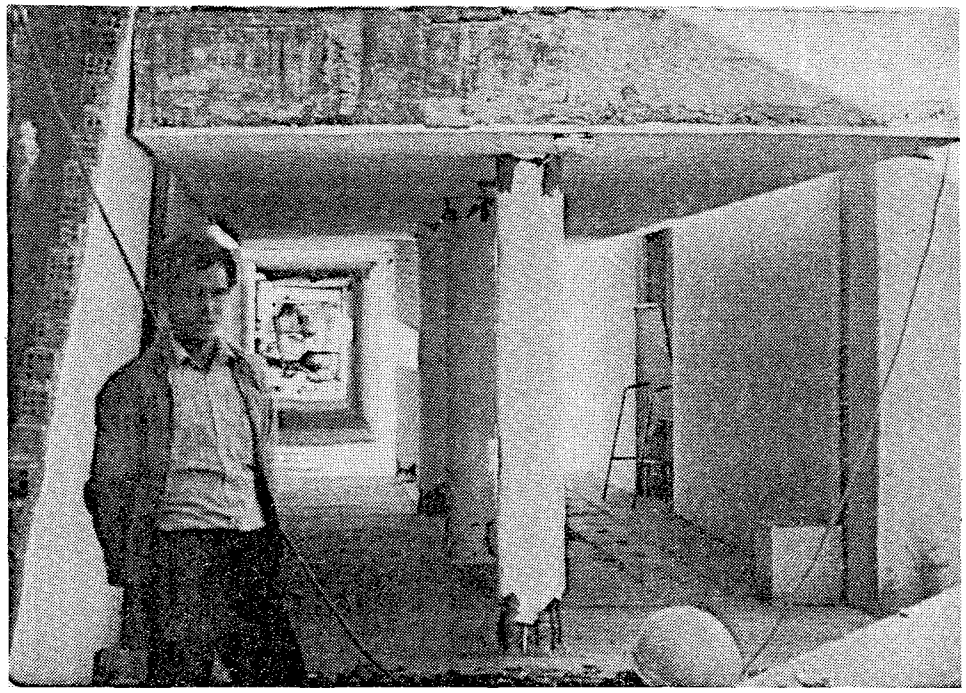


(b) Side View

Fig. 4.28 Primary School Collapsed Canopy



(a) Overstressed Columns with Significant Permanent Deformation



(b) Damaged Facade of Adjacent Buildings Due to Hammering of Canopy

Fig. 4.29 Canopy That Remained Standing

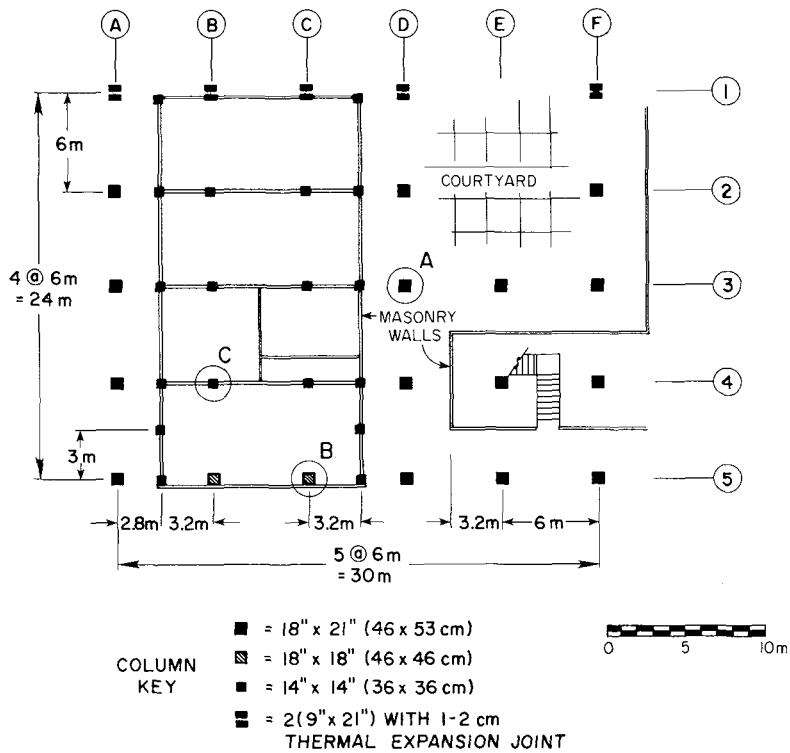
As noted, the canopies that remained standing were placed between two adjacent buildings (one on each side), while those that collapsed were flanked by a building on one side only. The lateral separation between the canopy roof and the adjacent building was approximately 15 cm (6 in.) on each side. From the observed damage it is clear that the canopies that remained standing hammered against the facades of the adjacent buildings, producing damage in the facades (Fig. 4.29b). The damage in the columns of the standing canopies and the analyses presented in Appendix C indicate that the adjacent building restrained the lateral deformation of the canopy roofs, thereby preventing their collapse.

The intensity of shaking was great, effective peak acceleration of 0.35 g or more horizontally, with a vertical acceleration component that could have been of the same intensity or even higher; nevertheless, it is believed that damage of these canopies could have been avoided by proper design and construction. Both the shape selected (inverted pendulum) and the use of unnecessarily heavy mass at the top were unwise decisions. These design flaws were aggravated by the lack of good confinement of the concrete [circular ties of D6 mm (#2) at 180 mm (7 in.) spacing], as can be seen in Figs. 4.27 and 4.29a.

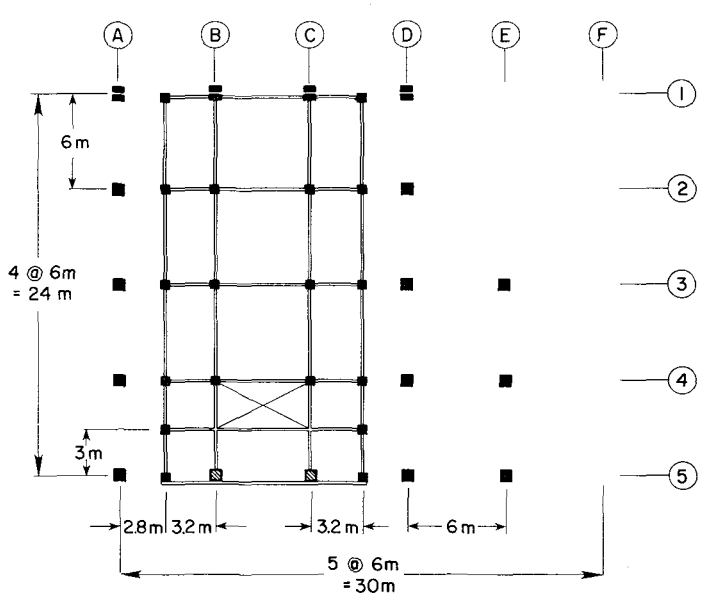
4.2.2 Cité An Nasr Market Complex. This large complex belongs to category 2 buildings (defined in section 1.6). It was designed in 1956 according to the seismic provisions of the AS 55 Recommendations, i.e., two years after the 1954 major earthquake, and construction was finished in 1962. As can be seen in Figs. 1.17, 3.23, and 3.47, the market site was close to the steeply sloping southern bank of the Cheliff River. Although there was evidence of some soil movement (cracks in the pavement) along the main street north of the police station and city hall, no soil failure was observed at Cité An Nasr. There were also reports (unconfirmed) that the building site was an old creek bed (parallel to the Cheliff River) and, therefore, filled with compacted soil. Although it is possible that soil conditions amplified ground shaking at this particular site, from the inspection conducted by both teams it appears that the collapse of this building complex was for reasons other than soil failure.

As shown in Figs. 1.17 and 3.23, the complex comprised three large buildings covering an area larger than 150 x 150 meters (492 x 492 ft). At the center of building 18 there was a mosque. This building and the two buildings (No. 14) were made up of several units separated by thermal expansion joints of 10 to 20 mm (0.4 to 0.8 in.). Most units were two stories high, but some were three stories; in the ground story there were restaurants and a variety of shops. A first-story mezzanine had residential apartments, as did the second and third stories. With the exception of one corner, the two buildings (No. 14) collapsed within a few seconds (Figs. 3.24, 3.34, and 3.35). The corner that remained standing is marked (i) in Fig. 3.23. The most severe shock occurred at noon, when people were at home in their apartments or in cafes; it was estimated that close to 3000 people were in this complex, most of whom were missing and believed dead. Bodies were still being removed on day 19 after the earthquake, and the search continued.

Fig. 4.30 illustrates the several structural arrangements and dimensions (approximate) of the corner (i) of the market complex that remained standing. As can be seen from sketches and photographs in Figs. 3.24, 4.4, and 4.30, in the first story the main structural systems consisted of a waffle slab supported on columns. The waffle slab had a total thickness of 45 cm (18 in.) and the exterior columns were either 46 x 53 cm (18 x 21 in.) or 46 x 46 cm (18 x 18 in.). These columns were spaced about 6 meters (20 ft) apart and had a clear height of 4.75 meters (15 ft). The exceptions were some columns in which the height was cut in half by beams required to support the floor system of the 2.2 meter clear height mezzanine (Figs. 4.4 and 4.31). As shown in Fig. 4.30, the interior columns C were of smaller cross-sections, usually 36 x 36 cm (14 x 14 in.). As illustrated

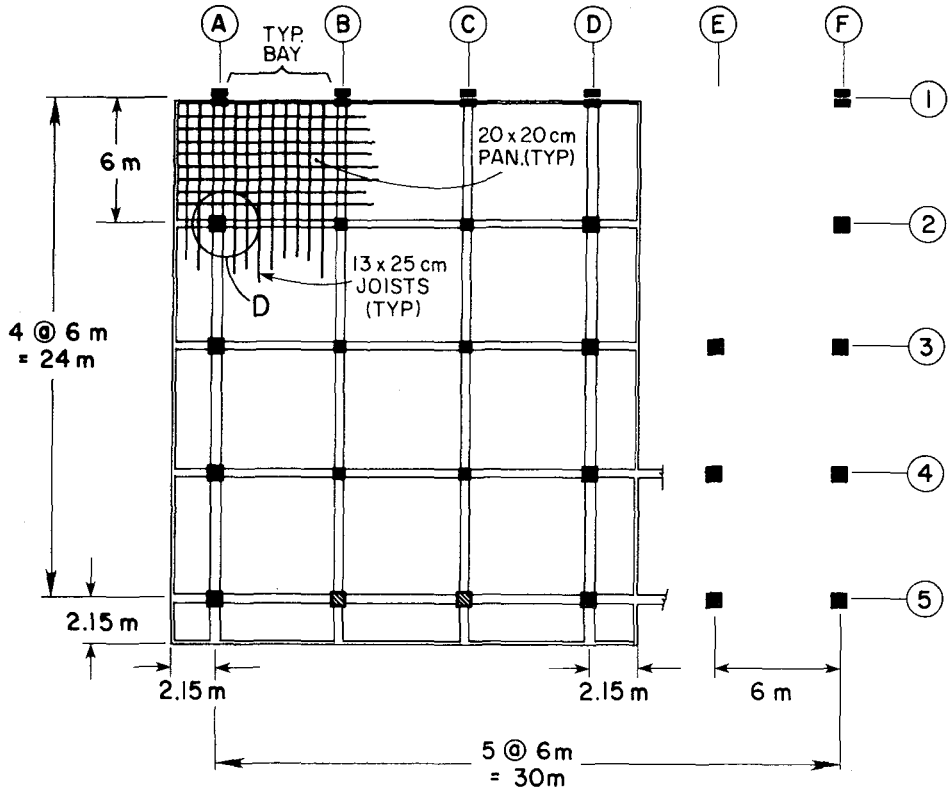


(a) Structural Plan of Ground Story

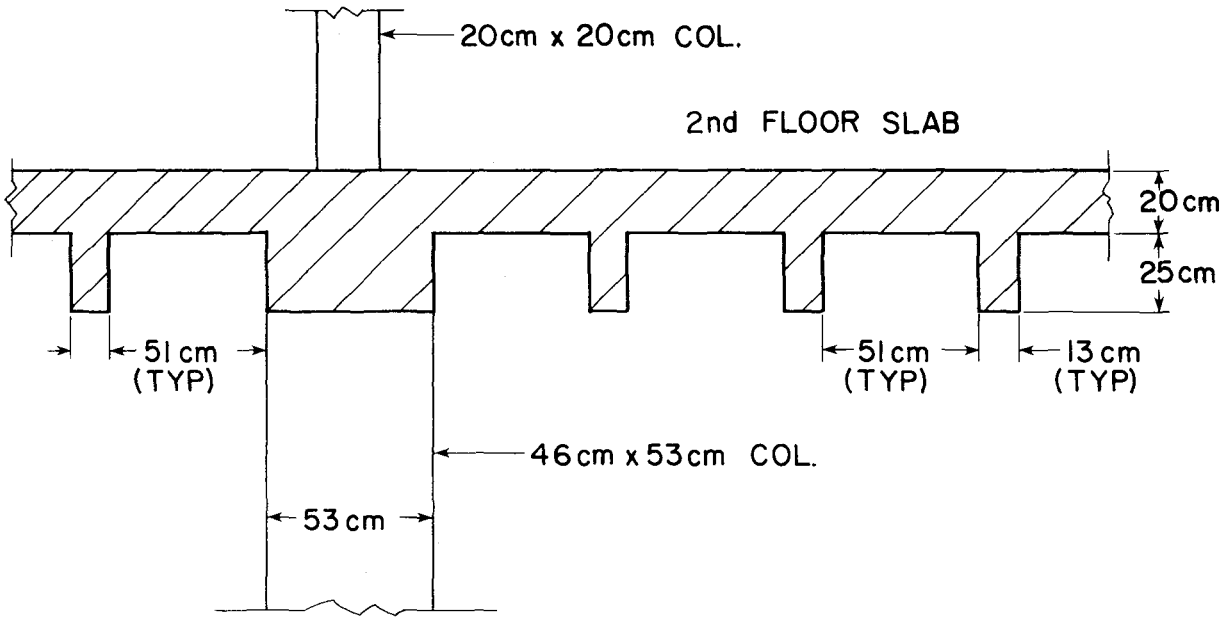


(b) Structural System for Mezzanine at 2.25 Meters

Fig. 4.30 Structural Plans and Details of Corner of Cité An Nasr That Remained Standing

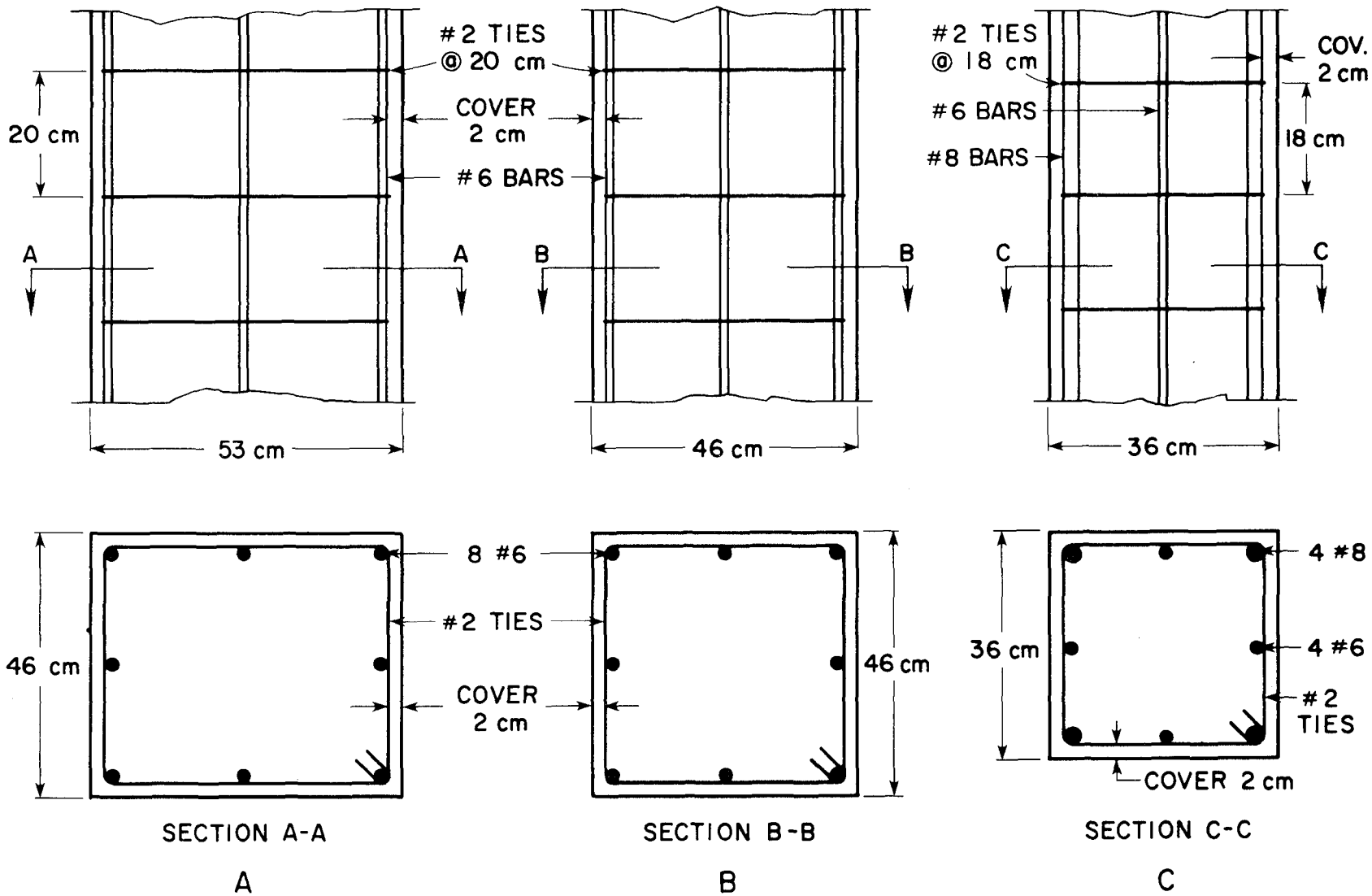


(c) Plan of Second Floor System



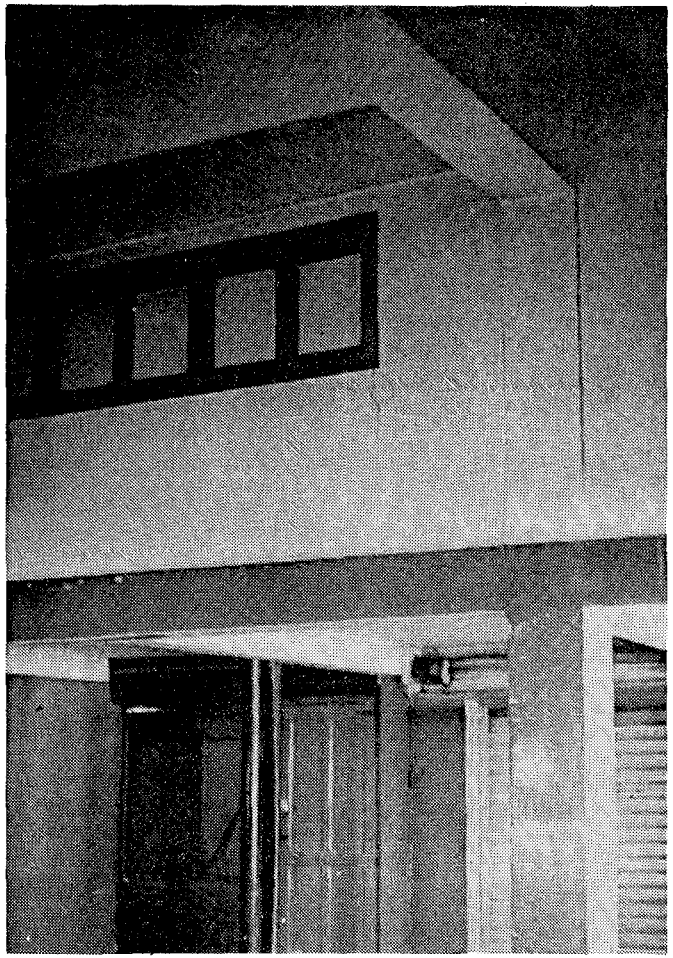
(d) Detail of Column/Waffle Slab Connection

Fig. 4.30 (cont.)

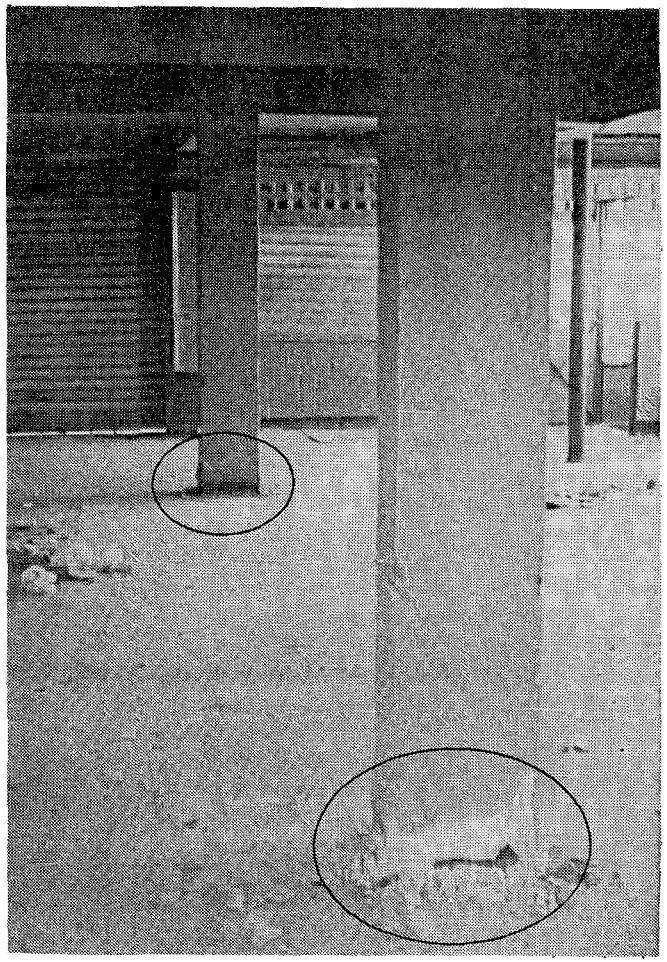


(e) Column Details

Fig. 4.30 (cont.)



(a) Mezzanine



(b) Damaged Columns

Fig. 4.31 First-Story Short Columns Supporting Mezzanine

in the sketches of Fig. 4.30e, reinforcement of the exterior columns consisted of eight D19 mm (#6) bars, giving a steel content below 1 percent for the largest columns. The transverse reinforcement consisted of ties of D6 mm (#2) plain bars spaced at 20 cm (8 in.).

It should be noted that the waffle slab serving as the second floor was cantilevered about 2.15 meters (86 in.) at each side of the building; at the tip of this cantilever, there was an external wall at the second story which was unreinforced, unrestrained, and not contained by any column in its plane. The columns in the second story had the same distribution as in the first story except that some were considerably smaller. Most of these columns were 20 x 20 cm (8 x 8 in.) with just four D19 mm (#6) bars, which resulted in a tremendous change in the column stiffness and strength in the second story, where all the columns had at least one main axis infilled with masonry. Significant concrete honeycomb was observed in several columns (Fig. 4.18).

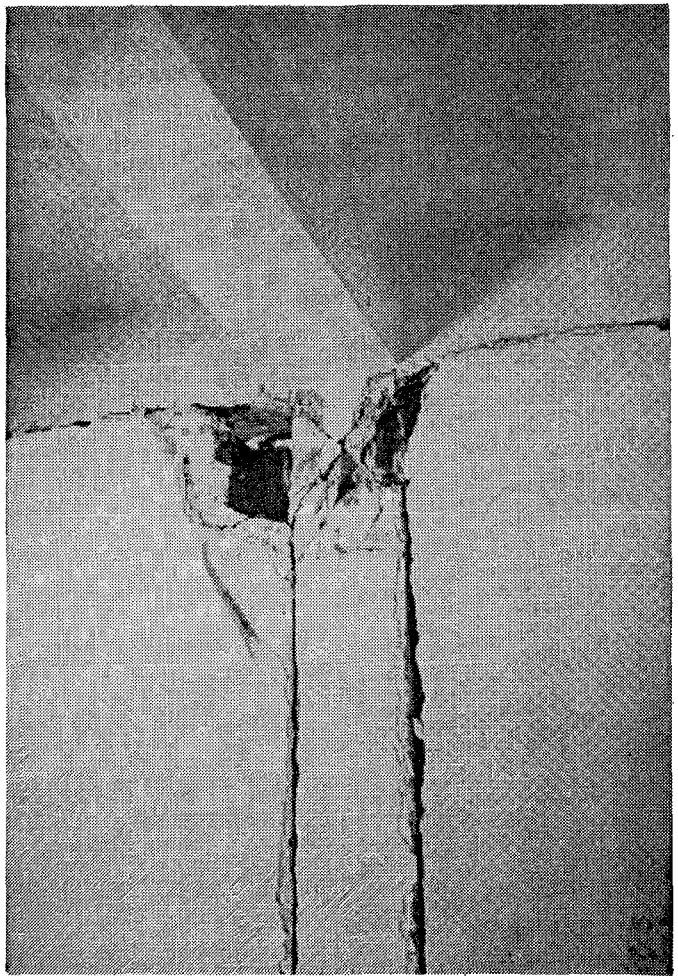
Although the roof system varied, it generally consisted of a waffle slab with tall parapets constructed on the roof (Fig. 4.10). In one corner there was a two-way RC slab on beams covered with insulation and a waterproofing membrane, which added considerable weight.

The corner of the building that remained standing was structurally separated from the rest of the building by a thermal expansion joint 10 to 20 mm (0.4 to 0.8 in.) thick. At this joint the column widths were one-half the width of the other columns. Although this corner was still standing after the earthquake, the structural damage was so serious that the building had to be demolished. It should, therefore, be considered as failed.

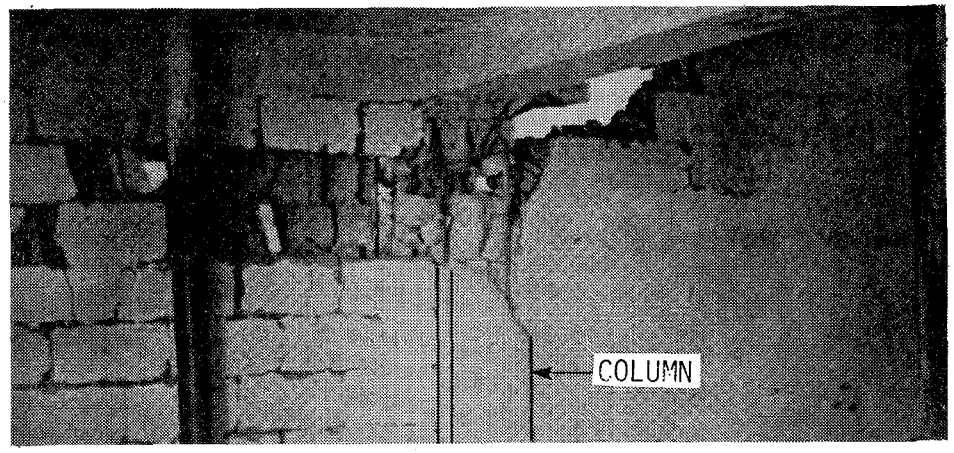
From the damage observed it appears that the mechanism of failure was as follows: The second story gave the impression of solid construction because of its many partitions. Structurally, however, it was very weak. The heavy roof was supported by weak columns. The masonry walls, especially the external ones on the tip of the cantilever part of the waffle slab, were unreinforced and not tied to the structural system. Consequently, these walls could not help the structure resist the effects of the earthquake. Similarly, the infilled partitions failed to work with the columns in resisting the seismic forces; and, therefore, the second-story columns began to shear off (Fig. 4.32a). Because of the lack of proper shear reinforcement [ties D19 mm (#2) plain bars at 180 to 200 mm (7 to 8 in.)] and weak concrete [probably with ultimate strength lower than the specified 210 kg/cm^2 (3000 psi)], the columns could not resist the large inertial forces that developed at the roof once the walls began to fail.

Collapse of the second story led to collapse of the first story, which should have started with the failure of the interior columns (36 x 36 cm or 14 x 14 in.). These columns, because they were captive by the infilled partitions and because of the presence of beams supporting the mezzanine, had to resist the major part of the lateral shear at the first story. Because of inadequate transverse reinforcement as well as poor concrete, these short columns could not resist such shear and failed. Fig. 4.32b demonstrates this failure. The first-story interior column failure led to cave-in of the floor (waffle slab) and failure of the external columns. Commencement of failure of the exterior columns after failure of the interior columns (and cave-in of the waffle slab) is illustrated in Figs. 3.34, 4.4b, and 4.32c.

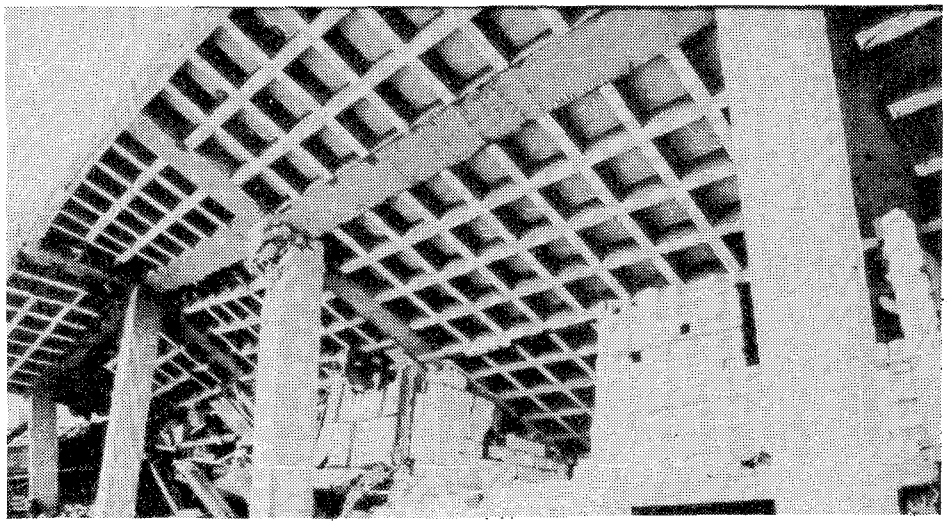
To summarize, the main reasons for failure of this huge building complex were: 1) poor seismic-resistant structural layout, particularly in height, which caused too sudden a change in stiffness and strength between the first and second stories, and between the exterior and interior columns of the first story; 2) long cantilevers; 3) heavy roof and floor systems; 4) heavy parapets



(a) Shearing of Second-Story Column



(b) Shearing of First-Story Short Interior Column. Note lack of adequate transverse reinforcement.



(c) Failure of First-Story Columns, Leading to Cave-in of Second-Floor Waffle Slab

Fig. 4.32 Failed Columns in Cité An Nasr Building

on the roof, and heavy unreinforced walls and partitions improperly anchored to the structure; 5) poor design and detailing of structural members; and 6) poor quality and placement of concrete, particularly in columns.

The collapse of the Cite An Nasr building complex again demonstrates that designing according to seismic code provisions does not guarantee seismic-resistant construction.

4.3 Probable Causes of Some Major Building Failures

Probable reasons for the collapse or poor performance of major buildings have been deduced from field observations by members of the two teams during the survey of damage, and by observations by CTC published in a preliminary report (CTC, November 17, 1980) in which some technical reasons were formulated.

In presenting the probable technical causes for building failures, the authors found it convenient to classify the buildings into two major groups: buildings designed to a seismic code and those which were not.

4.3.1 Buildings Whose Design Was Based on Seismic Code Provisions. Few buildings in El-Asnam had been designed according to seismic code provisions. The Cité An Nasr and Hotel du Cheliff were exceptions. These two buildings were designed in compliance with the seismic provisions of the AS 55 Recommendations. The Maison de la Culture (cultural center) was designed using the seismic provisions of the PS69 Recommendations. As the reasons for the Cité An Nasr failure have already been discussed, only those reasons concerning the other two buildings are now presented.

Hotel du Cheliff. The two-story Hotel du Cheliff had an irregular plan (Fig. 1.17). Its seismic-resistant design was according to AS 55, and construction was finished in 1962. The structural system consisted of moment-resisting frames with weak columns/strong girders. Very heavy masonry walls and partitions added considerably to the mass, as did the roof system. The building was destroyed because of failure of the columns, which had not been reinforced properly to resist the large shear that was induced.

Maison de la Culture. The cultural center consisted of four building units or blocks, four stories each, which were under construction when the earthquake occurred. The structural system of each unit was almost complete; and some of the masonry walls, particularly the external ones, were finished (see Figs. 4.8 and 4.33). It was estimated that 60 percent of the construction was completed. The structural system consisted of moment-resistant frame but of irregular nature in plan and elevation. As illustrated in Fig. 4.8c, the upper story had a long cantilever whose facade was loaded by deep RC spandrel beams and heavy decorative elements. There were changes in the dimensions of each story in an asymmetrical way (see Fig. 4.8c), and the columns in each story had different stiffnesses. The external facade columns were considerably shorter than the others (Fig. 4.8b) and at the same time weaker than the deep beams they supported.

As shown in Fig. 4.8a, although the three lower stories of one building unit pancaked, the unit beside it remained standing. There were some differences in overall dimensions of these two units (the one that collapsed had six bays in the longitudinal direction, while the one standing had four bays); however, the structural systems appeared to be identical. It is important to note that there was an RC stair shaft alongside the unit that remained standing (Fig. 4.33). Although this stair shaft was structurally independent of the unit, it appeared to have been constructed so close to the unit that it supported or constrained the deformation of the adjacent building unit.

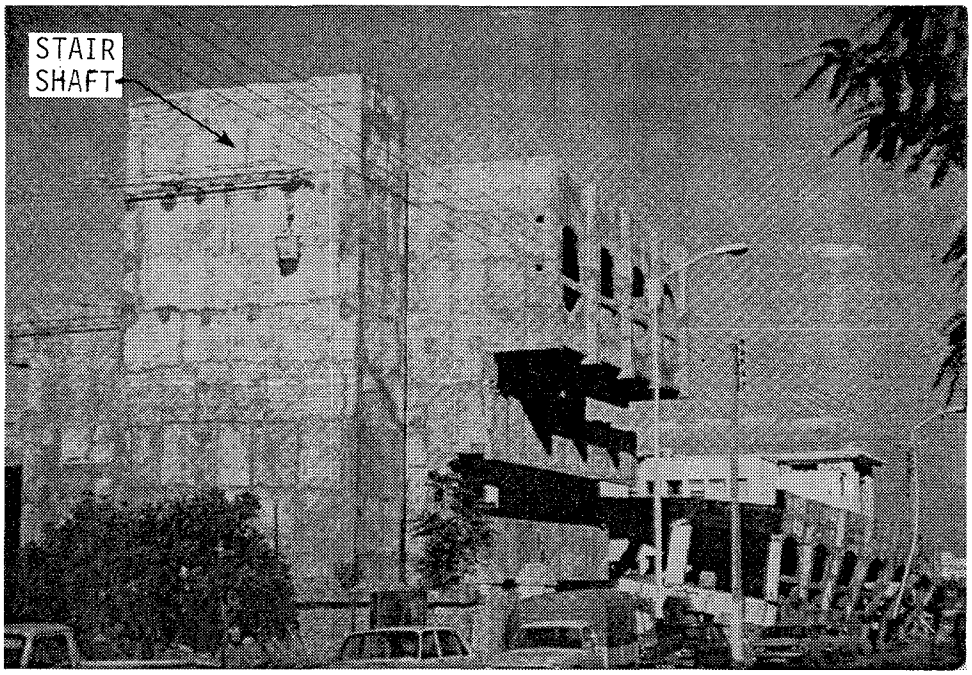


Fig. 4.33 Cultural Center. The stair shaft performed well; the middle building had serious damage; the building in background collapsed.

Inspection of the building that remained standing showed that this unit was on the verge of collapse. As illustrated in Figs. 4.8b and c, all front columns had sheared off. Therefore, it is concluded that the main reason for collapse of the adjacent unit was shearing of the columns at the facade, i.e., the shear demands were higher than the shear supplied to the columns. The development of high shear in the short facade columns resulted from a combination of defects in the conception of the building configuration and structural layout, and in the design, detailing, and construction of the structural components. To summarize, the main technical defects that triggered the shear failure of the columns were as follows:

1. **High Column Shear Demands**

a. Lateral story shear and the shear induced by considerable torsional effects created by the irregularities in elevation of the building, particularly the long cantilever, about 4 meters (13 ft), overhanging the top story, and aggravated by the heavy decorative mass added at the tip of this cantilever (Fig. 4.8d).

b. Creation of short columns in building facades because of the existence of deep beams (Fig. 4.8b).

2. **Relatively Lower Shear Strength Available**

a. Mediocre quality concrete.

b. Inadequate transverse reinforcement. The transverse reinforcement consisted of 6 mm (0.24 in.) ties spaced at about 20 mm (8 in.), which was equal to the effective depth of the column section. (It should be noted that this is the spacing allowed by AS 55.)

From inspection of the buildings that were designed according to provisions of a seismic code (AS 55 or PS69), it became clear that while it is possible that the designer followed code recommendations for computations of the seismic forces, obviously what was not followed were the basic rules in AS 55 or PS69 for proper conception or selection of building configuration and structural layout and for detailing of structural components. Further, the construction of the structures and of the building was improperly executed. The failure of the three code-designed buildings described above is new proof that numerical design against seismic forces code is not necessarily sufficient. In earthquake-resistant construction it is more important to pay close attention to conceptual design, proper detailing, and to construction and maintenance aspects than to numerical computations that satisfy code requirements.

4.3.2 Buildings Whose Design Computations Did Not Include Effects of Seismic Forces. According to official statements by CTC (Report No. 3, November 1980), most buildings in El-Asnam were designed without computations on the effects of seismic forces. Moreover, as inspection results indicated, most buildings were neither conceptually designed nor constructed as seismic-resistant structures although in 1954 the city had been devastated by a similar earthquake. Despite the lack of conceptual and numerical design against seismic forces, it is believed important to briefly describe the technical reasons for failure of some of these buildings. The significance of the conceptual design, detailing, and construction and maintenance is clearly demonstrated.

Galerie Algérienne. This relatively new building finished in 1978 consisted of two blocks of four stories plus a basement. The two blocks were separated by a thermal expansion joint. As illustrated in Fig. 4.34, while the three upper stories of one of the blocks collapsed, the other block remained standing but with significant nonstructural damages. The structural system of the block that collapsed consisted of a moment-resisting space frame. The block that remained standing had, in addition to the moment-resisting space frame, a well infilled RC frame shaft for the stairway (Fig. 4.35). Although this shaft introduced considerable torsional forces, its stiffness and strength were sufficient to avoid collapse of this block.

The interior bays of the frames were infilled with panels of hollow brick masonry. These panels, which were neither reinforced nor anchored to the frame, had excessive dimensions: 5.2 meters (17.1 ft) long by 4.4 meters (14.4 ft) high. It should be noted that AS 55 recommends that the masonry panels be framed (confined) by RC horizontal and vertical elements at a distance not exceeding 5 meters (16.4 ft). Today it is recognized that these panels are too large and it is preferable not to exceed 3 meters (9.8 ft). Some of these panels exploded in the earthquake, as shown in Fig. 4.35.

In one of the external sides of the building heavy masonry ornamental elements protruded from the facade about 200 mm (8 in.) and about 1.5 meters (5 ft) in width (Fig. 4.36). These protruding elements were supported by short RC brackets at each floor level.

Regarding the mechanism of failure of the block that pancaked, it appears that because of the heavy roof and heavy masonry, as well as the considerable increase in stiffness that infilling of the frame introduced, the story shear increased beyond that which the columns alone could resist once the masonry exploded. As the columns were weaker than the girders and poorly reinforced against shear, they sheared off. The hinging and shearing of the ends of the second-story columns can be seen in Fig. 4.37.

49 Villas CNEP at Boucaa Sahnoun. There were 49 two-story housing units under construction in a district near El-Asnam. These villas had a moment-resisting space frame as a structural system. The floor system consisted of heavy slabs supported on strong beams. The roof was similar to the floor but considerably heavier because of the addition of sand as insulation material. The columns were weak compared to the beams. They had a square section of about 20 x 20 cm (8 x 8 in.). While the first story was completely open, the second story was closed by heavy masonry walls (Figs. 4.13 and 4.38) plus heavy masonry partitions (see Fig. 4.39, which also illustrates the weak columns and strong beams). It is not known whether it was planned to close the first story with walls and partitions or if it had been designed to remain open as an architectural solution for peasant farmer housing (see Fig. 4.15a).

The failure of these units was triggered by hinging of the weak first-story columns and their shearing off, as shown in Fig. 4.13. In some units the effect of the impact when the first story collapsed, in addition to the effect of lateral shear in the second story, produced complete pancaking of this story, as shown in Figs. 4.17a and 4.40. These photographs depict the complete disintegration of the columns. This disintegration was due to poor design and detailing, as well as extremely poor quality concrete. These facts are illustrated by Figs. 4.17, 4.41, and 4.42. In Fig. 4.41 it can be seen that in a 20 x 20 cm (8 x 8 in.) column the spacing of ties was about 15 cm (6 in.) and the concrete had coarse aggregates larger than 7.6 cm (3 in.). Not only was the qranulometry very poor, but the aggregates appeared to be dirty and there was evidence of a lack of cement. Fig. 4.42 points out the inadequate detailing of the external beam-column joint.

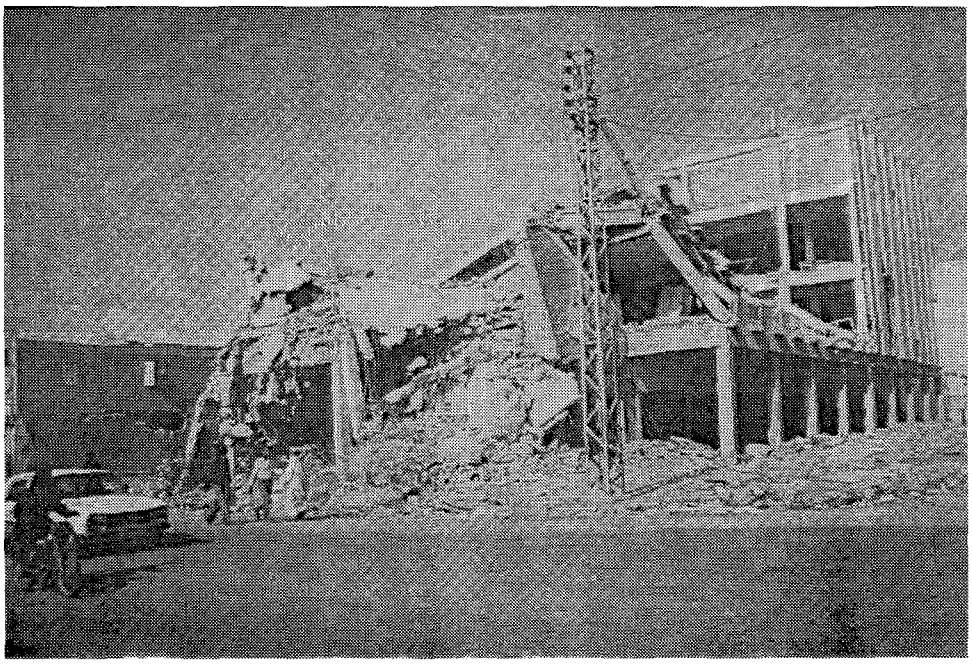


Fig. 4.34 Four-Story RC Galerie Algérienne. Unit that collapsed is in foreground, unit that remained standing in background.

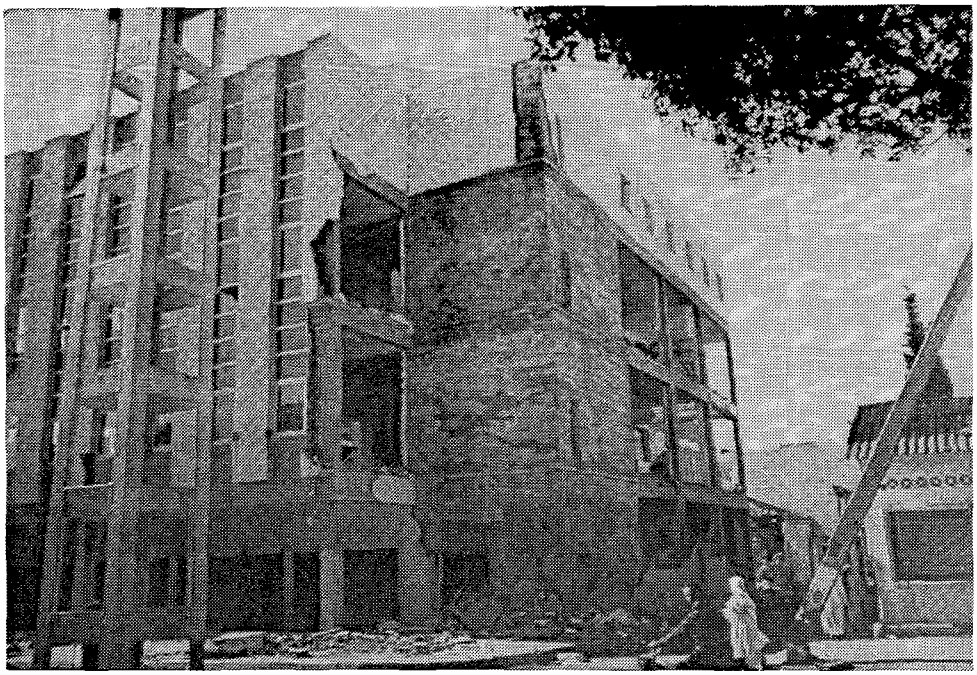


Fig. 4.35 Galerie Algérienne Unit That Remained Standing

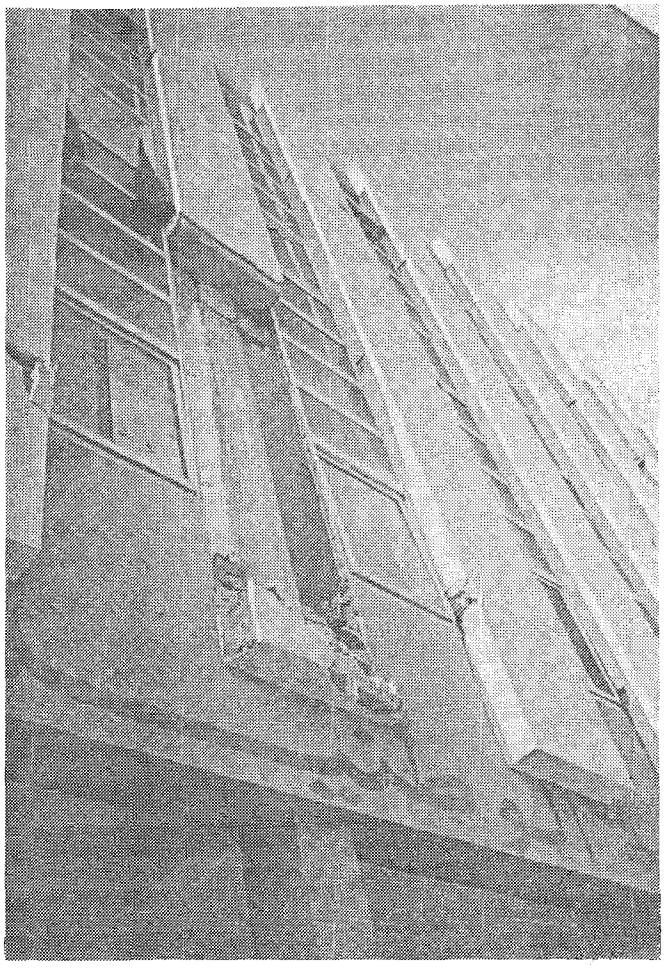


Fig. 4.36 Galerie Algérienne Protruding Masonry Ornament Supported on RC Brackets

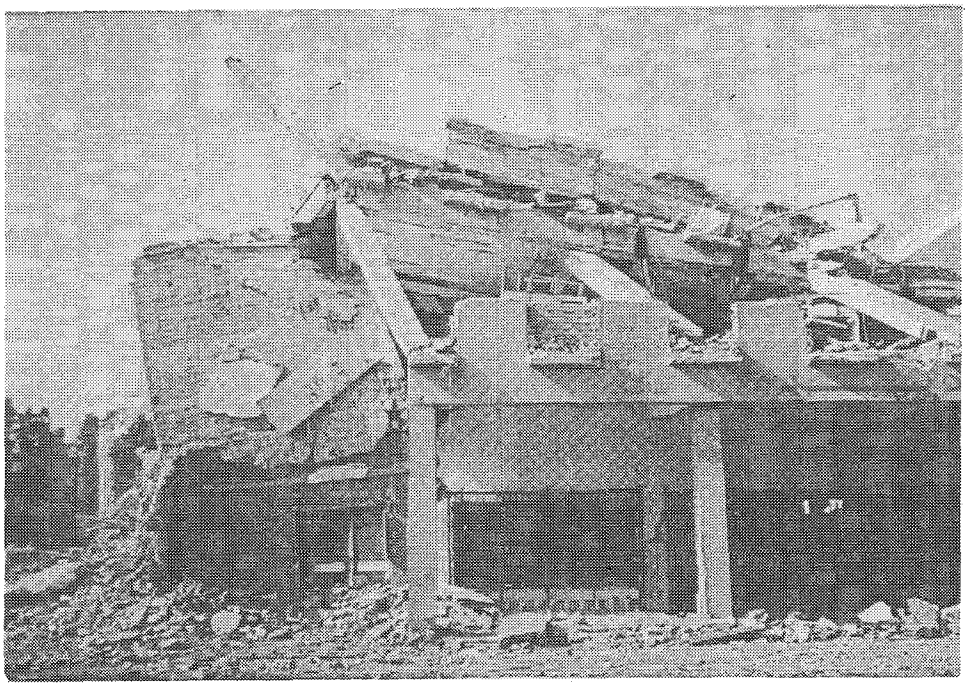
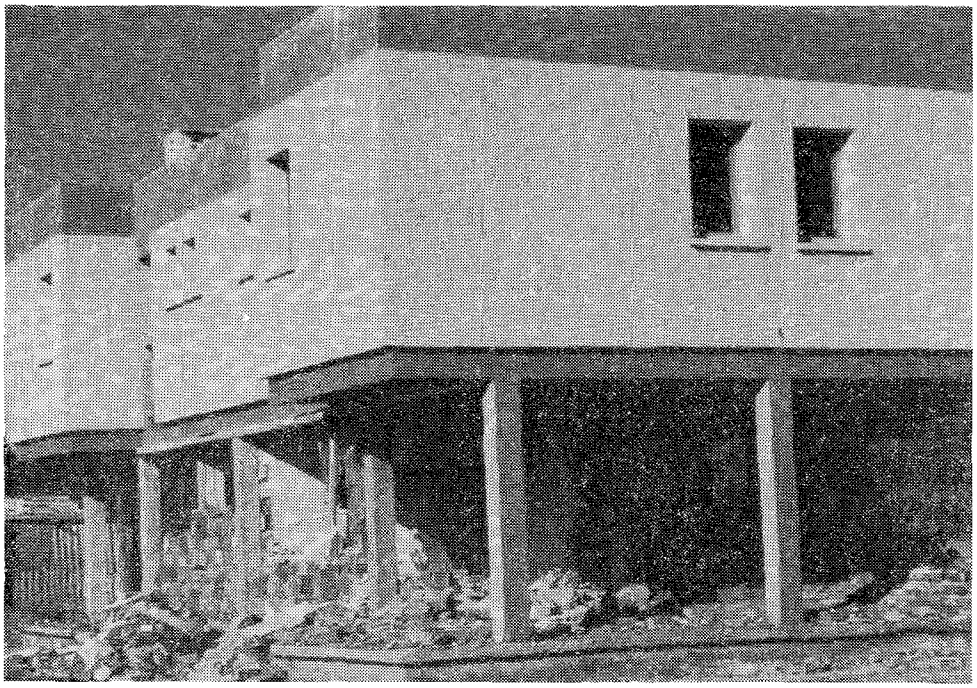


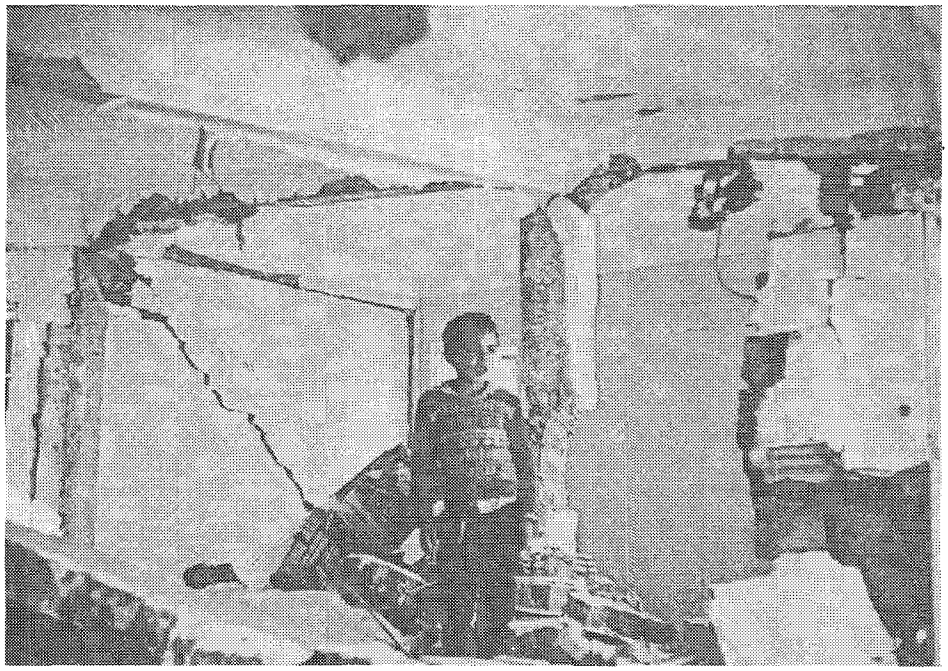
Fig. 4.37 Closeup of Galerie Algérienne Unit That Failed, Showing Hinging and Shearing of Second-Story Columns



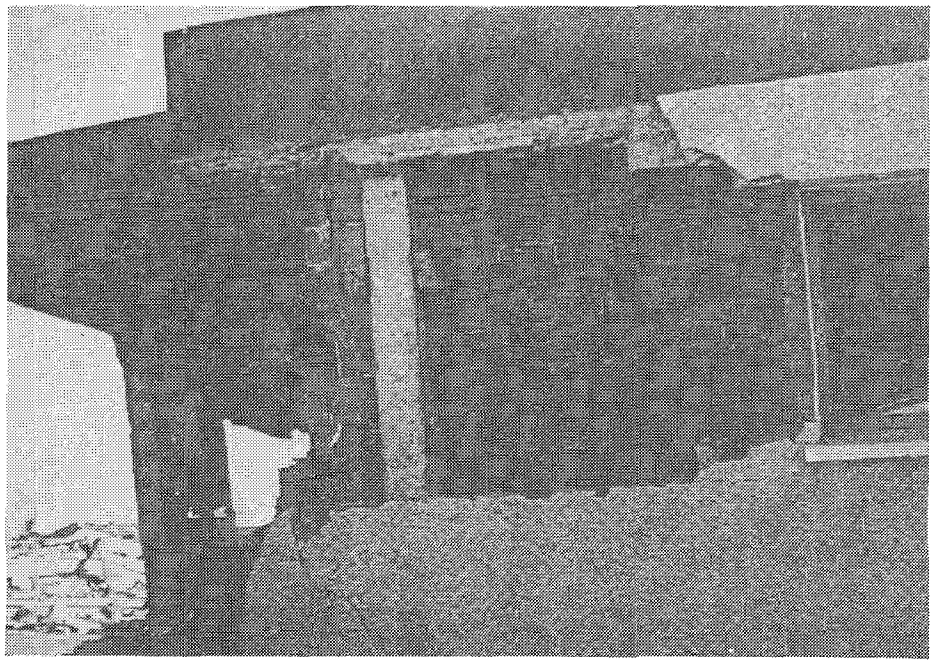
(a) Two-Story Houses: RC Frame with First Story Open and Second Story Infilled with Masonry

(b) First Story Weak Columns/Strong Girders: Note column concrete disintegrated.

Fig. 4.38 Boucaa Sahnoun Villas



(a) Internal Partitions



(b) Internal Partitions, Weak Columns/Strong Girders

Fig. 4.39 Boucaa Sahnoun Villas

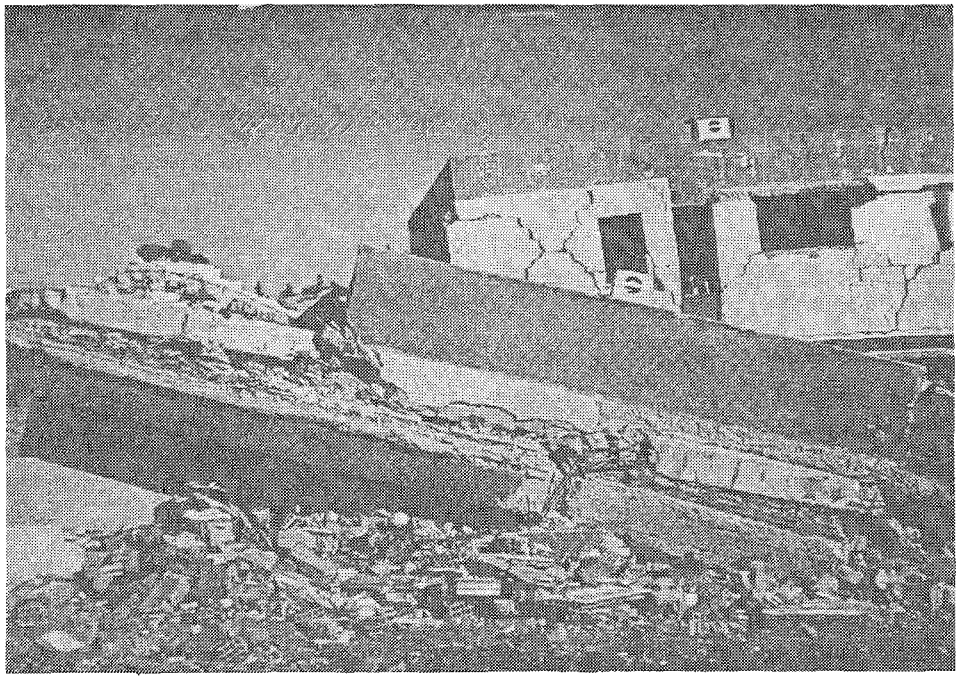


Fig. 4.40 Boucaa Sahnoun Villas. Shearing of first and second-story columns led to complete pancaking of several houses.

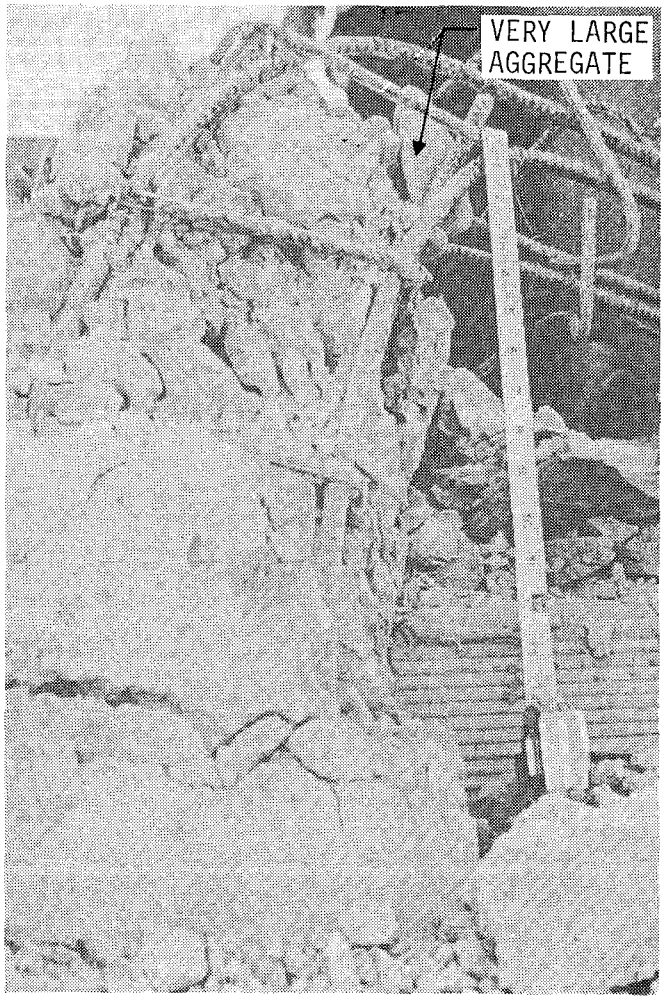


Fig. 4.41 Failed Boucaa Sahnoun Villa RC Column. Note lack of adequate transverse reinforcement and poor quality of concrete, particularly large size coarse aggregate.

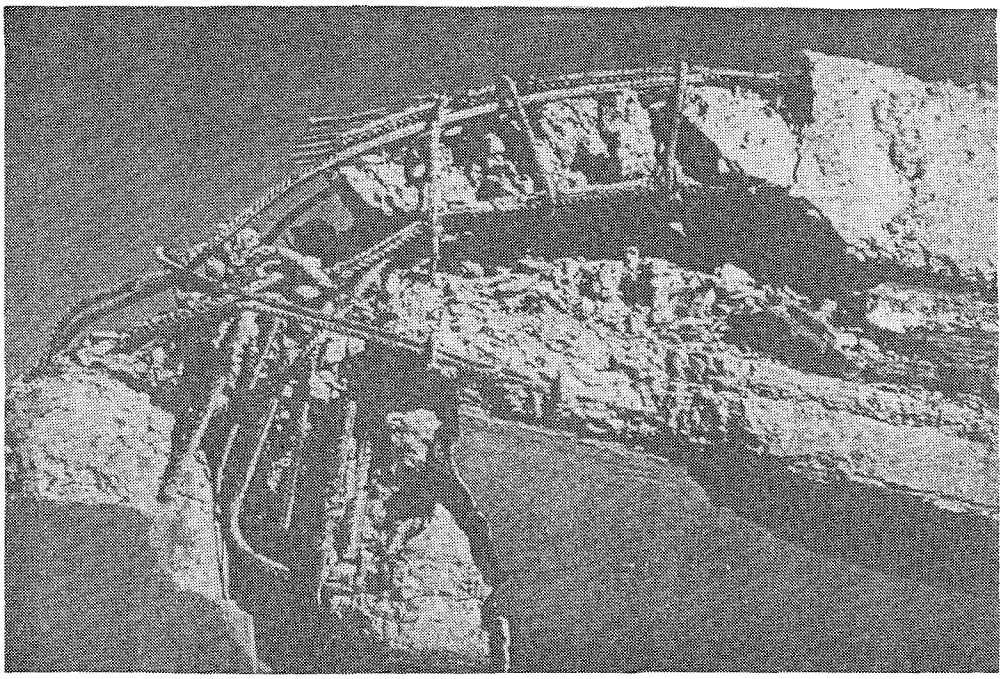


Fig. 4.42 Failed Boucaa Sahnoun Villa Column-Beam Joint.
Note absence of reinforcement.

V. SOCIO-ECONOMIC ASPECTS

The following discussion of socio-economic aspects of the El-Asnam earthquake is subject to several limitations. These include the teams' lack of knowledge of either Arabic or Berber, lack of fluency in French, a relatively short post-earthquake visit to Algeria, as well as the absence of a local information center. Given these circumstances, the observations on socio-economic aspects presented here are based on a limited number of interviews with officials in the El-Asnam area and Algiers, local and foreign journalistic accounts, and personal observations made by the reconnaissance and investigating team members.

5.1 Human Aspects of the Disaster

The October 10, 1980 El-Asnam earthquake struck on a Friday, the day of rest in this Moslem country. That fact had a significant impact on the number of people killed or injured. Fortunately, schools, public buildings, and major stores, although totally destroyed, were empty and so were not the primary sites of death and injury. However, as it was the hour of prayer, the Grand Mosque was full. Many people were at home having their noon meal; others were strolling in the parks or streets. The single area with the largest casualty toll was the Cite An Nasr apartment-cafe-market complex (Figs. 3.23 and 3.24). Three thousand people lived in the complex, and the search for bodies buried under the collapsed remains continued long after the earthquake.

In the countryside, where so much of daily life takes place out-of-doors, relatively few were killed in spite of the total collapse of numerous dwellings. Many villages were isolated by road cutoffs because of fissures, landslides, or destroyed bridge access.

The first shock, with a Richter magnitude of 7.2, lasted 40 seconds. It apparently came without warning* and left the populace overwhelmed, confused, and faced with a transformed landscape. Most key buildings collapsed or were in ruins. The city and *wilaya* offices, the police station, courts, major market, and hotels were destroyed, as were lifeline services of water, electricity, and gas. Telephone service was also interrupted.

Descriptions of the earliest events following the quake indicate that although an earthquake contingency plan was not available, an effective response happened surprisingly quickly. Each organization did what it could. The National Liberation Army (FLM) immediately began coordinating efforts and by the fifth day was officially in charge of all operations. Although lack of a contingency plan created some delays in providing assistance in the distribution of water and food, and in providing information about casualties, the rescue and care operation for life-preserving human needs was rapid and effective.

*One person interviewed stated that TV or newspaper interviews indicated that one person felt a shock a week earlier but ignored it since there are frequently small tremors in El-Asnam.

5.2 Rescue and Relief Operations

Dedicated efforts by individuals and organizations were clearly evident in El-Asnam. The victims themselves handled the first rescue effort with whatever tools were available. The Army handled the emergency situation effectively and with a firm hand. The *wilaya* of El-Asnam was immediately closed to non-emergency vehicles. Without the express permission of the military or the government, no person was allowed to enter El-Asnam. All civilian population was evacuated from the city, and orders were given to shoot looters. Three days after the earthquake, Shah of the reconnaissance team reached El-Asnam and found that the emergency rescue teams, the medical teams, and the military were already working effectively and in control of the situation. The Army conducted an aerial reconnaissance of the surroundings; helped in cleanup and rescue operations, and later arranged for evacuation of the injured; and set up a receiving and distribution supply center, a military hospital, and tent cities to shelter the homeless (Fig. 3.38).

Paralleling the rescue efforts were national and international medical and paramedical personnel. Collection of blood was organized the first day. By the second day, foreign teams with specialized sonar equipment arrived on the scene to help locate and rescue victims buried alive. By October 14, day 4 after the main shock, *Le Monde* reported the presence of several dozen medical teams. The injured were transported by ambulance or helicopter to the closest hospital — field hospitals as well as hospitals in Algiers and Oran because the one major hospital in El-Asnam was destroyed by the earthquake. Rescue activity continued day and night. By the fifth day after the disaster, the main medical emergency period had passed and emergency hospitals were shifting to provision of routine medical care.

The dead were covered by a transparent plastic spray which allowed identification but controlled disease and odor problems prior to burial. The principal medical concern became the prevention of disease and epidemics, as corpses still buried in rubble began to decompose. Two main medical thrusts proceeded concurrently: one was a large-scale vaccination program to prevent disease outbreaks; the other was the provision of sanitary living conditions, latrines, and safe water supplies for the homeless.

Tent cities were organized and shanty towns erected by the victims themselves. These were constructed of whatever materials were readily available and furnished with household goods salvaged from the debris.

An interesting debate, reflected in *Algérie Actualité*, a weekly newspaper, concerned whether morale was better in the well-organized tent cities or in the more variably organized shanty towns created by the victims' own initiative. All agreed that the greatest need was to provide shelter and warm clothes for the victims as winter was approaching.

When the CND/EERI investigating team arrived two weeks after the devastating earthquake, many of the essential tasks had been accomplished or were well under way. The medical emergency had passed, and vaccination of the population was proceeding. Access to the region was controlled and handled in a routine manner. Tent cities had been erected, and the distribution of food and water was well organized. Courts had been established to deal with looting and profiteering, and severe penalties were imposed. Planning teams were completing the analysis of sites of temporary, semipermanent, and permanent housing. Train service had been reestablished between Oran and Algiers, and highways had been repaired. The enormous tasks of removing the debris and sheltering the homeless were far from over, however, especially in the mountain and rural areas.

El-Asnam had been restructured by the Army into 11 sections, each with 14 islands containing 10 to 15 families. Each family had a card and each island a chief who maintained records of the people residing in his section, according to age and sex. Such basic organization was necessary for all civil records had been destroyed.

The major emerging concerns were reestablishing the basic community administrative functions, returning the children to school, and resuming normal activities. Temporary mobile shelters were erected near the railroad station for the administration, and plans were under way to provide elementary schooling in El-Asnam and secondary schooling at more distant points.

Considering the immensity of this disaster, the population and the government organizations, including the military personnel who were in charge of administering the rescue mission, performed admirably. Authoritarian rule and discipline imposed by the military administrators helped return the city of El-Asnam to some semblance of order within two weeks of the destructive October 10, 1980 earthquake.

VI. LESSONS LEARNED, CONCLUSIONS, AND RECOMMENDATIONS

6.1 Socio-economic

There were no contingency plans for earthquake emergencies. However, the rescue, care, and relief operations were organized surprisingly quickly.

6.2 Geologic and Tectonic

The 7.2 Richter magnitude El-Asnam earthquake of October 10, 1980 was caused by displacement on the Oued Fodda fault, a northeast-trending thrust fault dipping to the northwest. At its closest approach, the surface trace of the Oued Fodda fault is about 7 km southeast of El-Asnam. Although the Oued Fodda fault was not recognized before the October 10, 1980 earthquake, there was abundant geologic and geomorphic evidence indicating the presence of an active fault. Therefore, the fault could have been identified prior to the earthquake. Although no surface faulting was reported on the Oued Fodda fault following the Richter Magnitude 6.7 event in the same region in 1954, the 1954 earthquake was probably generated by displacement on the Oued Fodda fault at depth. Furthermore, moderate to large magnitude earthquakes can be expected to recur on the Oued Fodda fault in the future, and they can be expected to occur on other active faults in the region as well.

The main surface rupture on October 10, 1980 occurred along the primary thrust fault within a broad zone on the upper plate of the thrust. Most of the severe damage from intense shaking was concentrated near the epicentral area, not along the fault trace. The zone of surface fault rupture was primarily in agricultural areas; and damage was limited to collapsed huts in small villages along the surface trace and to broken or seriously impaired roads, railroads, pipelines, and irrigation canals across the fault zone. Many adobe huts that were not on the fault but adjacent to it survived without significant damage. El-Asnam, the city most severely affected by the earthquake, was not within the zone of surface fault rupture.

6.2.1 Liquefaction. Clough et al (1981), after inspecting seven sites (Fig. 3.17), reported that the liquefaction phenomena observed generally followed trends that have been reported for other earthquakes. Numerous examples of sand boils, both conical and linear, ground subsidence, and lateral spreading of the ground were observed. Although, strictly speaking, no new lesson has been learned with respect to liquefaction, the following behavior was unusual:

1. Depression of trees into the sand and the passage of linear sand boils between trees.

2. Formation of a large earthquake lake, in part due to liquefaction subsidence.

3. Formation of sand boils through a clay overburden at site 4 (refer to Fig. 3.17) with a thickness of 6 meters (minimum). Grain size analyses of sand samples taken from sand boils at this site showed a larger percentage of fines (27 percent passing the No. 200 sieve) than have been reported in the literature.

4. Distinctive coloring of the sand boils at site 5 (Fig. 3.17) due to the presence of layers of red and yellow sands.

5. Water spouts up to 2 meters high in many of the sand boil areas.

6.2.2 Landslides. Clough et al (1981) reported that the landslides were generally found within 10 km of the major fault systems. As with the instances of liquefaction, the landslides were in sparsely populated rural areas. Most of the behavior observed was common to that reported for other earthquakes. The most striking occurrence was reactivation of the large old landslide near Beni Rached, which led to generation of a geyser that spouted 20 to 40 meters into the air. The mechanism for forming the geyser and flowslides has been formulated by Clough et al (1981).

6.3 Engineering Design and Construction

The lessons learned regarding building design and construction are considered of utmost importance because practically all loss of life and property damage were caused by building failures, which in turn were mainly due to ground shaking. The large number of casualties resulted from a combination of the many buildings that collapsed (pancaked) and the high density occupancy of these buildings. It is ironic that buildings designed and constructed to provide shelter and protection from the hazards of the natural environment suddenly became the cause of death.

Construction in El-Asnam was predominantly low rise (one to two stories) with several buildings in the three to five-story range. A large percentage of buildings were relatively new (constructed after the 1954 earthquake had devastated the city), and most of the three to five-story buildings were erected after 1970. The majority of buildings were constructed of modern materials (reinforced concrete for the structural framework, infilled with masonry of hollow brick or concrete block).

Although the ground motion was strong, with a particularly high vertical acceleration component, the primary reason for collapse of a large number of buildings was the inadequacy of their design and/or construction for resisting earthquake ground motion. This inadequacy resulted from 1) lack of enforcement of seismic-resistant regulations and of building code provisions for normal loading conditions; 2) buildings apparently designed by professionals and built by contractors without adequate knowledge of seismic-resistant construction, resulting in poor selection of building configuration, structural layout, and/or construction methods; and 3) poor quality control of structural materials and poor workmanship, both of which were perhaps the result of lack of proper inspection during construction.

No new information was obtained about the structural performance of buildings during this earthquake. On the other hand, the performance observed strongly reaffirmed knowledge gained from investigations of previous earthquakes, i.e., the design of a structure must be according to basic principles governing ductile seismic-resistant design, and proper seismic-resistant construction practice and maintenance of buildings must be followed, or the use of strong structural material like reinforced concrete can prove an expensive disappointment. As noted, many

buildings in El-Asnam collapsed because they were not architecturally designed and engineered to withstand the effects of strong earthquake ground motion, not because of any economy on structural materials.

Knowledge gained from investigating previous earthquakes and reemphasized by the El-Asnam earthquake of October 10, 1980 points out the importance of paying close attention to the following design and construction principles and practice:

1. **Recognition of the possibility of very severe earthquake ground motion.** The severity of ground motion that can occur in the epicentral region of a Magnitude 7.2 earthquake has been estimated from the performance of various structures. Horizontal components of ground motion (acceleration) have been estimated with an effective peak acceleration larger than 0.40 g in certain areas. The October 10, 1980 El-Asnam earthquake was characterized by vertical components of ground acceleration of larger intensity than the horizontal components in the epicentral region. The largest peak effective accelerations appeared to have occurred at certain distances (in some cases up to 5 km) from the surface faulting.

2. **Site selection that takes into account the suitability of soil conditions for earthquake-resistant construction.** No reliable information on geologic soil conditions or local topography of El-Asnam was available to the investigating team. Comments by Papastamatiou (1980) and Moriya (1980) suggest that the soil had some effect on the severity of ground motion at different locations throughout the city. In general, it is believed that none of the important failures was caused merely by soil failure; at least this was not apparent during the field survey.

3. **Sound seismic-resistant architectural conception of building.** The adoption and use of architectural styles and building configurations developed for nonseismic regions in a region of high seismic risk, as El-Asnam, is one of the main reasons for building failures. The importance of this architectural conception, which cannot be overemphasized, was dramatically illustrated in this earthquake. Certain architectural methods for improving protection against climatic conditions (namely, heavy roofs and thermal expansion joints) and sanitary conditions (*vide sanitaire*) were employed which aggravated the effects of seismic ground motion. These styles exhibit a lack of concern about the importance of symmetry of building mass and lateral resisting elements or the danger of open ground floors with shear walls terminated on the second level. In several cases, the choice of building configuration based solely on architectural style resulted in buildings with irregularities in plan and elevation; sudden changes in mass, stiffness, strength, and ductility; excessive torsion; soft stories; or unreasonably long cantilevers.

4. **Sound seismic-resistant structural layout (systems), considering the interaction with nonstructural components.** The bad seismic-resistant features of the soft story concept (weak columns and strong girders) were demonstrated by this earthquake, as was the improper use of so-called "nonstructural elements."

5. **Proper detailing of structural members and connections.** Even in those buildings designed according to a seismic code, detailing of members was poor, particularly of column reinforcement and joints with beams. *In no other earthquake have there been so many beam-column joint failures as in the 1980 El-Asnam earthquake.* Most of these failures were due either to poor anchorage of the main reinforcing bars or just lack of adequate transverse reinforcement at the joints.

6. **Satisfactory construction techniques.** Several two or three-story buildings under construction collapsed or suffered serious structural damage as a result of the unsatisfactory construction method used. That method consisted of infilling with masonry the RC frame in the upper stories and leaving the first story open until completion of construction. This type of failure was another typical feature of this earthquake and convincing evidence of the need to modify construction techniques.

7. **Quality control of materials.** It is well known that since reinforced concrete is a composite of steel and concrete (which in turn is a composite material), the attainment of a good seismic-resistant RC material is difficult. The performance of RC buildings during the El-Asnam earthquake emphasized the importance of having good quality control of construction materials. The use of dirty fine and coarse aggregates, together with poor gradation, contributed considerably to the fabrication of very poor concrete.

8. **Satisfactory workmanship.** The lack of good workmanship in the El-Asnam region was readily apparent in the fabrication of the concrete. Poor gradation of materials; use of only a small amount of cement; poor mixing of the materials; and poor placement, vibration, and curing of the concrete; together with inferior quality aggregates; contributed to the attainment of very poor quality concrete that did not offer a good bond to the steel and which disintegrated under relatively low stress. It appears that in the El-Asnam region the demands of construction (i.e., the large quantity required) overtaxed the availability of competent contractors, skilled workers, and quality control personnel. Most of the work just completed or still under construction seemed to have been done hurriedly, without sufficient attention to details.

9. **Thorough inspection during all stages of design, construction, and service.** Many of the failures in El-Asnam were due to the combined effects of poor design, poor construction, and poor maintenance, which could have been avoided through competent inspections.

10. **Adequate maintenance of completed buildings.** In some buildings that failed, the reinforcing steel was corroded, which induced a detrimental effect on the surrounding concrete.

6.4 Reconstruction Planning

The first question asked was whether it was advisable to reconstruct the city of El-Asnam on the same site that had been devastated twice in 27 years. The second question was, if it were rebuilt on the same site, what must be done to avoid future devastation?

Regarding the first question, there was insufficient data concerning the local geology to warrant a definite answer. As discussed in section 2.5, El-Asnam is built on alluvium, cut by the Oued Cheliff River, and recent deposits of loose sand. There are several terraces; except for part of the lower terrace, the city is built on soil sufficiently firm to found buildings. Furthermore, the topography of the site does not present any serious problems since the slopes are slight. A thorough study of soil conditions at the present city site should be conducted to detect areas where the soil is unsuitable for building construction. In regions where the soil is firm, it would be possible to erect buildings following the basic guidelines of seismic-resistant construction. In this sense the CND/EERI recommendations presented in section 6.5 were formulated. It is to be noted that in February 1981 CTC issued some improvements to the existing seismic-resistant specifications. An English translation of these improvements is presented in Appendix D.

Before presenting the authors' recommendations for steps needed to prevent similar death and destruction from future earthquakes in the El-Asnam area, it may be instructive to compare events in El-Asnam with what happened in San Juan, Argentina, which was devastated by an earthquake in October 1894.

After the 1894 tragedy, it was decided to rebuild the city of San Juan a little to the south although Dr. Bodenbender, a geologist and professor at the University of Cordoba, pointed out that the solution was not to move the city from one place to another (Castellanos, 1944). The entire region around San Juan was covered with faults; thus moving the city would move it away from one fault but closer to another. Bodenbender suggested changing the methods of building construction, but his suggestions were ignored. A new city was erected on the same site using the same construction methods, and it was again devastated by an earthquake January 15, 1944. The same type of damage that occurred in 1894 was again observed in 1944.

In view of this new tragedy, the authorities decided to rebuild the city according to a new seismic code formulated by specialists. (See International Association for Earthquake Engineering, "Earthquake Resistant Regulations," A World List, 1980.) The code was a simple one clearly specifying the way that buildings, particularly one, two, or three-story housing units, should be constructed using material available in the region (reinforced concrete and masonry). Until 1972, the code was strictly enforced through rigorous inspection covering all phases of design (numerical computation and drawings), materials quality control, and field examination during construction; then inspections ceased.

On November 23, 1977, San Juan was again shaken by an earthquake, with surface magnitude $M_S = 7.4$, inducing ground shaking similar to that of 1944. This time dwellings and buildings generally withstood the earthquake practically without damage. The only significant structural damage occurred in a new building constructed after the group controlling building design and inspection had been abolished.

This is a good example of how effective comprehensive but simple seismic code recommendations can be when stringently enforced throughout the period of design and construction. These recommendations should include not only coefficients to obtain forces and equations to do analyses, but recommendations on proper design criteria, building configuration, and structural layout, and use of proper structural detailing, high quality materials, proper construction techniques, skilled workmanship, and adequate inspection.

6.5 Recommendations

As a result of the field investigations and subsequent analyses of the devastating October 10, 1980 El-Asnam earthquake, the authors have made the following recommendations to mitigate damage from future earthquakes. The recommendations are directed at two levels: 1) the public level, and 2) the scientific and engineering level. Implementation of these recommendations at both levels should provide a significant degree of disaster preparedness and damage mitigation. It should be borne in mind that some recommendations under the "Scientific and Engineering Level" heading must be accomplished before certain steps can be taken on the "Public Level."

6.5.1 Public Level

1. **Establish chain of command and control.** Develop earthquake contingency plans paralleling other natural disaster contingency plans specifying who is in charge and who is responsible for each emergency operation: search and rescue; medical care; road access; and provision of water, food, and other supplies. Preparations should be made at both the national and local levels.

2. **Provide adequate education.**

   a. Government officials

   b. Practitioners, technicians, and facility owners

   c. General public

   Awaken the seismic consciousness of the public through basic education. Special higher education courses should prepare those whose careers will most directly involve them in activities related to earthquake mitigation measures.

   Through a process of continuous education, provide programs for training technicians, building inspectors, and local contractors. Require of engineers and architects entering the field of design and construction of seismic-resistant structures, special registration and professional examination on seismic-resistant design and construction.

   Offer university courses in earthquake engineering for architectural and engineering students, and provide a program of continuing education in this field.

3. **Revise seismic-resistant building regulations.**

   a. Zoning

   b. Planning

   c. Building design and construction

   d. Quality control (materials and workmanship)

4. **Enforce new regulations.**

   a. Require permits for compliance with code design regulations and zoning regulations

   b. Perform inspection during construction for compliance with permitted design

   Enforce new seismic-resistant regulations and code building provisions for normal loading conditions through capable review of the design computations and inspection of field construction.

5. **Repair and/or upgrade existing hazardous structures.** All countries with regions subject to seismic hazards should initiate detailed investigations regarding the most efficient methods for repairing and/or upgrading (retrofitting) existing hazardous structures. Utilization should be made of the experience gained in retrofitting in other countries.

6.5.2 Scientific and Engineering Level

1. **Conduct site studies —** geologic and seismic (geologic planning)

   Conduct detailed studies of the local geology in high hazard seismic areas in order to determine where ground failures could occur in future earthquakes as a result of surface faulting, landslides, and liquefaction. In these regions, construction should be limited.

   Several instances of the unusual liquefaction behavior could be fruitfully studied to develop a better understanding of liquefaction phenomena and the seismicity of the El-Asnam area. The site of the earthquake lake and the nearby site 4 with the substantial clay overburden are two such cases. In addition, site 5, with its distinctive layers of red and yellow sands, is also a strong candidate for further study.

2. **Determine economic geography of area —** transportation, commerce, industry, agriculture, or other use.

3. **Review existing building code** in light of structural performance in 1980 and 1954.

   Review existing seismic-resistant regulations and improve them according to the information collected, and knowledge and experience gained from analyzing the effects of this earthquake as well as past earthquakes.

4. **Develop revised building code** incorporating adequate regulations to provide seismic resistance of structures. This may require fundamental changes in the prevalent architectural concepts.

   Develop comprehensive recommendations for seismic-resistant design of buildings and other civil engineered structures for seismic regions of Algeria, and other countries considering the local seismicity and the present building construction technology level of the country.

   It is not merely a question of formulating new seismic codes to design against more severe seismic forces. It is more a question of the architectural building style configuration, structural layout, quality control of materials, proper detailing, use of good workmanship and construction upon which the code recommendations must insist.

In summary, it should be noted that it is difficult to radically change the industry and professions of a country within a short time; therefore, engineers should develop seismic-resistant code specifications that realistically meet their country's needs rather than nonselectively adopting codes developed by more technologically advanced nations. The authors believe that by proper conceptual design, proportioning, and detailing of structural and nonstructural elements, followed by careful construction and maintenance, it is possible to significantly reduce earthquake damage and casualties without significantly increasing cost.

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*See "Postscripts" section for a list of selected papers describing the surface deformation and seismicity associated with the October 10, 1980 El-Asnam earthquake.

POSTSCRIPT

Nearly two years after the October 10, 1980 earthquake in El-Asnam, Algeria, the region around El-Asnam is slowly coming back to social and economic normalcy. The city of El-Asnam has been renamed Ech-Cheliff.

The process of reconstructing the damaged and collapsed buildings is not yet complete. A new code is being drafted by the Algerian Contrôle Technique de la Construction (CTC) to serve as the basis for repairing the damaged structures. Most of the population is still living in prefabricated, temporary housing. International bids to conduct microzonation studies for the Ech-Cheliff region were requested in May 1982. The three fundamental needs of these proposed studies are to develop:

1) A seismic hazard model of the Ech-Cheliff region

2) Seismic microzonation maps of the urban sites in the Ech-Cheliff region

3) Codes and land use regulations for use by building designers, contractors, and land use planners.

In May 1982, the Government of Algeria purchased 90 strong-motion instruments from Kinemetrics, Inc. (SMA-1's) to install in the Ech-Cheliff region.

The lake that was created by the earthquake has been drained, and farming activities of the villages in and around Ech-Cheliff have been resumed. All roads, bridges, and other lifelines are now functioning. Incorporation of state-of-the-art earthquake engineering in rebuilding the commercial and residential structures in this area is one of the challenges faced by the planners and engineers of Algeria. The greatest remaining challenge is to rebuild destroyed El-Asnam into a socially and economically vibrant Ech-Cheliff.

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APPENDIX A
AS 1955 RECOMMENDATIONS FOR BUILDING IN SEISMIC REGIONS*

Preamble

ZONES IN ALGERIA

Two zones have to be distinguished in Algeria:

Zone A — low seismicity

Zone B — high seismicity

The limits of these zones are shown in Fig. A.1, excerpted from the report of November 28, 1955 by Laffitte and Gourinard, Professor and Assistant at the Laboratory for Applied Geology, Algiers Faculty of Science.**

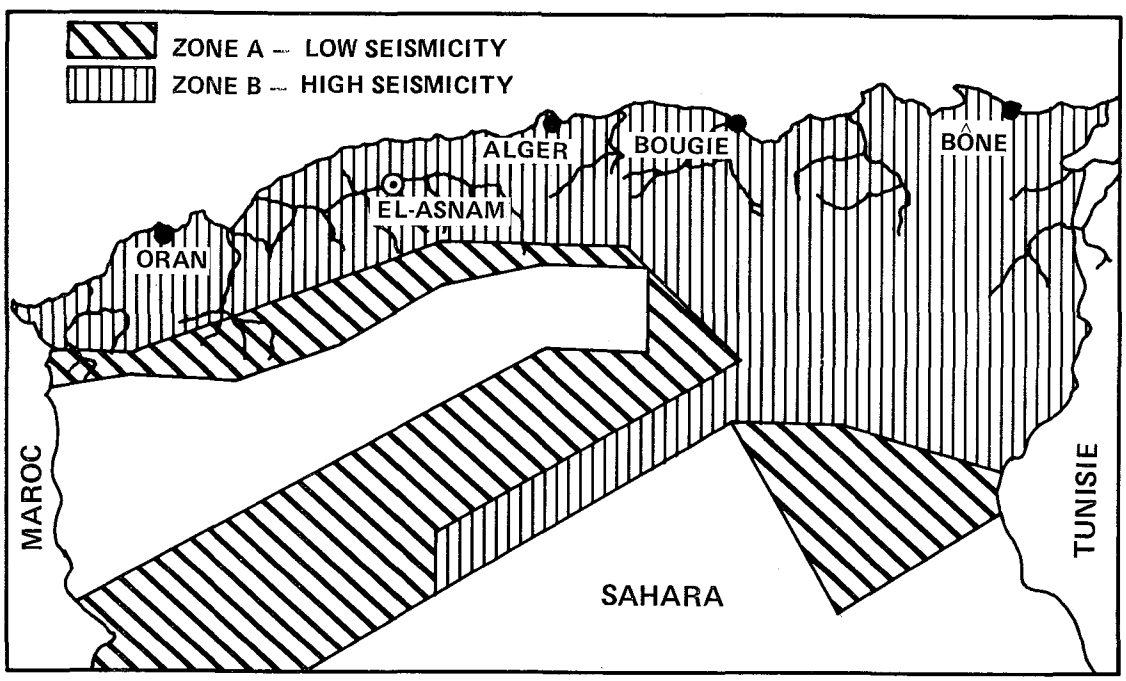

Fig. A.1 Algeria Seismic Zones

*French Housing and Reconstruction Ministry, May 1955; translated from French by V. V. Bertero, 1982, incorporating portions of translation published by A. Brenier in 1960.

**This version of the map of seismicity in Algeria is taken from "Earthquake-Resistant Regulations, 1963."

1. GENERAL RECOMMENDATIONS CONCERNING DESIGN

In order to have buildings that can withstand earthquakes as strong as those which have occurred till now, without an unacceptable increase in costs, compliance with the following rules is recommended.

1.1 General Conception of Buildings

1.1.1 Decrease as much as possible the height of buildings, especially the ratio of height to width (the smallest distance between outside walls). If that ratio exceeds 2.5 for Zone A or 2 for Zone B, special justifications have to be provided.

1.1.2 Avoid structures poorly balanced relative to height or inertia. For building with T or L plan shape, avoid too large branches. Avoid too large openings.

1.1.3 Design, if possible, a basement as wide as the building, or foundations that are deep and bulky or reinforced in such a way as to anchor the structure to the soil.

1.1.4 Avoid nonreinforced arches and, generally speaking, structures or parts of structures that become unstable with small movements of the supports.

1.1.5 Avoid cantilevers, brackets, and cornices with projections, and, generally speaking, all elements poorly fastened to the framework.

1.1.6 On roofs and ceilings, avoid poorly hung (not well fixed) elements, even if they are small.

1.1.7 Design exits to allow a quick way out in case of an earthquake. Each stairway should be as stiff as possible, well tied to the landing and the framework.

1.2 Foundations

1.2.1 Choose, if possible, compact soils; avoid saturated soils, fills, slumps, thin or recent alluvium.

1.2.2 Design deep foundations, carefully tied, and reaching the hard pan, particularly to withstand lifting forces due to the earthquake.

1.2.3 Avoid any heterogeneous foundation.

1.2.4 Design a very strong link between foundations and structure.

1.3 Structure

1.3.1 Decrease the vertical loads in the upper parts and lower, as much as possible, the center of gravity of the building. Avoid heavy terraces and roofs.

1.3.2 In framework constructions, provide rigid joints and ensure the rigidity of the framework with efficient bracing in all directions; for example, in the case of walls close enough to one another, and rigid cladding, these elements should be linked to each other or to the framework with well-anchored reinforcement.

The arrangement of the reinforcement at the joints of the framework in reinforced concrete should allow correct pouring of the concrete.

In columns, overlappings of the bars should be at least 50 times the diameter of these bars, and must be completed without hooks.

1.3.3 Ensure efficient binding between the different parts of the structure with horizontal, vertical, or slanting links (tie rods or tie beams of reinforced concrete or steel) that can withstand traction and shear, and forces resulting from possible torsion of the entire structure.

Reinforced concrete members subject to shear, except for slabs without opening and foundation caps, should be provided with transverse reinforcement whose spacing is to be not greater than the effective depth of the member considered.

1.3.4 Take special care to ensure those links in the case of precast elements. Do not have a floor with one set of parallel joists (ribs) without an RC slab poured in place. Ensure efficient ties between the joists and the slab.

1.3.5 In masonry construction, it is strongly recommended to gird (frame) the masonry panels with horizontal and vertical reinforced concrete tie beams whose spacing (between parallel members) should not exceed 5.00 meters.

When links are not provided and when the masonry with brick is laid horizontally, in certain parts of the construction one can provide reinforcements in the masonry joints, provided that those reinforcements are carefully anchored in vertical elements of the structure or in orthogonal walls. Such reinforcements have to be laid in horizontal thick joints (3 to 4 cm thick) at the most every 50 cm, and their cross section should be about 1 cm^2 in each joint.

Masonry should be pugged with cement mortar, made with clean sand without grains smaller than 0.4 mm. All masonry materials should be wetted thoroughly just prior to the building process.

Avoid isolated piers and narrow masonry walls between windows.

1.3.6 Provide around the openings reinforced elements connected to the structure or to the tie beams.

1.3.7 Take special precautions for structures having angular shape (especially in regard to bracing), and in case of adding new stories or introducing modifications where stability of old and new parts has to be verified.

1.3.8 Avoid transmission of large forces resulting from vertical or horizontal accelerations on small surfaces (punching or ram effects at the extremities of beams).

Provide elastic buffers in the thermal expansion joints between parts of same or similar inertia. Separate with wide joints parts of different inertia.

1.3.9 Provide nonbrittle gas and water pipes and carefully protected electric wires.

1.3.10 In scheduling work, avoid too large lapses of time between completion of the framework or supporting walls and completion of inside or outside walls which contribute efficiently to the lateral stiffness (bracing) of the structures.

2. COMPUTATION RULES

Stresses shall first be computed considering the effects of dead loads, service live loads, and climatic loads, according to regulations in effect at the time of construction.* Then seismic effects have to be computed as follows:

2.1 Earthquake Forces

Inertia forces created in a building due to the seismic ground motions may act in any direction. It will be sufficient to consider simultaneously or successively the effects of one horizontal component and the vertical component, which are defined below.

2.1.1 Horizontal Component. For one element of the building, this horizontal component of whatever direction applied to the gravity center, is σP, where σ is a seismic coefficient equal to the product of $\sigma_1\ \sigma_2\ \sigma_3$ (see Tables A.1 and A.2). σ_1 is a zone coefficient which up to 10 meters above ground level should be taken as

0.035 for Zone A**

0.070 for Zone B

For heights over 10 meters, the above values have to be increased 2 percent per meter; for example, for a height of 16 meters:

$$\sigma_1 = 0.07\,(1 + 0.02 \times 6) = 0.0784 \text{ for Zone B.}$$

*The current (1955) enforceable regulations are those of the Ministry of Housing and Reconstruction, known as Règles BA 1945 (BA45) for reinforced concrete structures, CM 1946 (CM46) for steel stuctures, and NV 1946 (NV46) for computation of the effects of snow and wind.

**These values correspond to the base coefficients:
0.05 for Zone A
0.10 for Zone B
multiplied by a reducing factor of 0.7 that accounts for the fact that the seismic effects of earthquakes (on which the intensity varies rapidly with time) are assimilated in the computations by static forces of long duration.

AS 55 Recommendations

Zone A

TABLE A.1. SEISMIC COEFFICIENTS FOR STATIC COMPUTATIONS IN DESIGN OF BUILDINGS IN EARTHQUAKE REGIONS

	Height Above Ground Level, m	Soils of Medium Consistency		Rocky Soils		Soft Saturated Soils	
		Deep Foundations	Shallow Foundations	Deep Foundations	Shallow Foundations	Deep Foundations	Shallow Foundations
Horizontal	≤10.00	0.0350	0.0437	0.0262	0.0328	0.0437	0.0547
	≤ 20.00	0.0420	0.0420	0.0315	0.0394	0.0552	0.0656
	≤ 30.00	0.0490	0.0612	0.0367	0.0459	0.0612	0.0765
	≤ 40.00	0.0560	0.0700	0.0420	0.0525	0.0700	0.0875
Vertical	Any height	0.0700	0.0875	0.0525	0.0656	0.0875	0.1094

AS 55 Recommendations

Zone B

TABLE A.2. SEISMIC COEFFICIENTS FOR STATIC COMPUTATIONS IN DESIGN OF BUILDINGS IN EARTHQUAKE REGIONS

	Height Above Ground Level, m	Soils of Medium Consistency		Rocky Soils		Soft Saturated Soils	
		Deep Foundations	Shallow Foundations	Deep Foundations	Shallow Foundations	Deep Foundations	Shallow Foundations
Horizontal	≤10.00	0.0700	0.0875	0.0525	0.0656	0.0875	0.1094
	≤20.00	0.0840	0.1050	0.0630	0.0787	0.1050	0.1312
	≤ 30.00	0.0980	0.1225	0.0735	0.0919	0.1225	0.1531
	≤ 40.00	0.1120	0.1400	0.0840	0.1050	0.1400	0.1750
Vertical	Any height	0.1400	0.1750	0.1050	0.1312	0.1750	0.2188

σ_2 is a ground coefficient which depends on type of foundation. Usually this coefficient would be 1; however, it will vary between the following two extreme values:

0.75 for foundations on rock

1.25 for foundations on loose and water-saturated soils

σ_3 is a foundation coefficient depth which is 1 when the basement is as large as the structure or when the foundations are deep and bulky or reinforced, otherwise it is 1.25.

P is equal to:

1) Dead (permanent) loads for dwelling structures

2) Dead loads plus half of the live loads acting directly on the member considered for stores, warehouses, and industrial structures

3) Dead loads plus total live loads acting directly on member considered, for tanks and silos

2.1.2 Vertical Component. The vertical component is equal to $\pm 2\ \sigma P$, where σ and P are defined as in 2.1.1. For σ_1, take the value for 10 meters or less without any increase above that height.

2.1.3 In the case of structures with floors, horizontal forces are applied at the level of each floor, and P should take into account the dead loads and possibly the fraction of live loads applied to the floor, as defined above.

2.1.4 For isolated structures (factory chimneys, tanks, fence walls, etc.), the above coefficient has to be doubled.

2.1.5 For the shaft or stack of chimneys above the last floor and for corbels on outside walls (projecting elements of balconies, cornices), the above coefficient has to be tripled. For a balcony, computations have to be carried out for the railing and for the whole balcony.

2.2 Allowable Stresses

Stresses should be computed considering the simultaneous effects of:

— dead (permanent) loads
— service live loads
— seismic loads (excluding climatic)

2.2.1 In certain computations, zero surcharge (live loads) can have a less favorable effect, and one has to remember the possible upward direction of vertical seismic forces.

2.2.2 All elements should be verified using a rupture (ultimate strength) method based on experimental studies with sufficient reliability to ascertain that, for the above loads, the ultimate strength of the structures or part of the structures is at least equal to the maximum computed ultimate strength demand.

Temporarily, regarding rupture methods, if there are no regulations available (limit analyses) one can use classical methods, i.e., elastic methods.

Computations have to be carried out with allowable stresses being equal to:

— The conventional elastic limit, for the structural steel of steel framework or for the steel reinforcement of reinforced concrete

— 0.8 of the compressive strength (stress measured after 90 days on 14.1 cm or 20 cm wide cubes) (the tensile strength being considered as zero), for the concrete of flexural elements of reinforced concrete structures

— 0.6 of the compressive strength (measured as above) for concrete pieces whose sections are entirely under axial compression

— Three times the normally allowable stress for masonry and non-reinforced concrete

— Three times the normally allowable stress for sound rocks

— Normally allowable stress for loose and water-saturated soils

— Twice the normally allowable stress for soils different from above

2.2.3 Panels between beams and columns have to be taken into account for the stability analysis of the structures if these panels are completely girded (framed) by elements of the framework or if they are of masonry constructed following the rules of 1.3.5.

APPENDIX B
DAMAGE EVALUATION FORM

Inspectors Code | DAMAGE EVALUATION FORM (EL-ASNAM) | CTC

Date:

IDENTIFICATION OF STRUCTURE:

Sector: | Zone: | Structure Designed for Earthquake Resistance: Yes - No

Address or Means of Identification: | Inspected Construction: Yes - No

STRUCTURE USE

Residential | School | Commercial
Administrative | Hospital | Industrial
Socio-Cultural | Recreation | Water Reservoir
Others (describe): .....

SUMMARY OF DAMAGE

Approximate Age:
Number of Stories:
Number of Separation Joints:
- in elevation
- in plan of superstructure

Sanitary Crawl Space: Yes - No ( )
Basement: Yes - No ( )
Exterior Independent Elements:
(stairways, shed, covered walkways)
.....

SOIL PROBLEMS AROUND STRUCTURE

Faulting: Yes - No
Liquefaction: Yes - No

Subsidence - Uplift: Yes - No
Landslide: Yes - No

FOUNDATIONS - SUPERSTRUCTURE

Foundations:
- type of foundation:
- type of damage
  settlement: Yes - No
  sliding: Yes - No
  overturning: Yes - No

Superstructure (for the case of sanitary crawl space or Basement)
- continuous concrete wall: 1-2-3-4-5
- concrete columns with infill: 1-2-3-4-5

STRUCTURAL SYSTEM

Vertical Load Carrying Elements
- masonry walls: 1-2-3-4-5
- concrete walls: 1-2-3-4-5
- concrete columns: 1-2-3-4-5
- steel columns: 1-2-3-4-5
- wood columns: 1-2-3-4-5
- others: 1-2-3-4-5

Lateral Load Resisting Elements
- masonry walls: 1-2-3-4-5
- concrete walls: 1-2-3-4-5
- reinforced concrete frames: 1-2-3-4-5
- steel frames: 1-2-3-4-5
- cross-braced frames: 1-2-3-4-5
- others: 1-2-3-4-5

Floors - Flat Roof
- reinforced concrete: 1-2-3-4-5
- steel joists: 1-2-3-4-5
- wooden joists: 1-2-3-4-5

Sloped Roof
- steel truss: 1-2-3-4-5
- wood truss: 1-2-3-4-5
- tile roof 1-2-3-4-5
- asbestos cement sheet roof: 1-2-3-4-5
- corrugated metal roof: 1-2-3-4-5

( ) Circle the appropriate description, in the case of numbers: one or more numbers can be circled.

SECONDARY DAMAGE

Stairways		Exterior Wall Panels	
concrete:	1-2-3-4-5	- masonry:	1-2-3-4-5
steel:	1-2-3-4-5	- precast concrete:	1-2-3-4-5
wood:	1-2-3-4-5	- corrugated metal:	1-2-3-4-5
		- others:	1-2-3-4-5
Other Interior Elements		**Exterior Elements**	
- ceilings:	1-2-3-4-5	- balconies:	1-2-3-4-5
- partitions:	1-2-3-4-5	- railings:	1-2-3-4-5
- glass:	1-2-3-4-5	- overhang:	1-2-3-4-5
		- parapets-cornices:	1-2-3-4-5
		- chimneys:	1-2-3-4-5
		- others:	1-2-3-4-5

INFLUENCE OF ADJACENT STRUCTURES

The structure endangers another structure:	Yes - No
The structure endangered by another structure:	Yes - No
The structure may be a support for another structure:	Yes - No
The structure may be supported by another structure:	Yes - No

VICTIMS

Yes - No - Maybe - if yes, how many?

COMMENTS CONCERNING THE NATURE AND PROBABLE CAUSE OF DAMAGE

	Traverse Direction	Longitudinal Direction
- plan symmetry:	good - average - poor	good - average - poor
- elevation regularity:	good - average - poor	good - average - poor
- redundancy of bracing elements:	good - average - poor	good - average - poor

OTHER COMMENTS

FINAL EVALUATION

General Level of Damage	Color to be Assigned		
1 - 2 - 3 - 4 - 5	GREEN	ORANGE	RED

DAMAGE LEVELS

1. NO DAMAGE:

   Except for overturned furniture and broken glass.

2. LIGHT DAMAGE:

   Cracked interior partitions

   Cracked ceilings

   Damage to plumbing, electrical, lighting systems

   In summary, isolated non-structural damage.

   Remarks: Take the most unfavorable case and ma e comments if necessary.

3. MODERATE DAMAGE:

   Significant damage for the non-structural elements and slight damage for the structural elements.

   * Non-Structural Elements:

   All the architectural elements and those elements which are not part of the structural system.

   Structural Elements:

   Load bearing system (walls, frames with infilled walls, or combinations of these)

   Remark: For the case of the collapse of the short columns of the sanitary crawl-space, and if the building has settled or tilted due to this support failure, even if the super-structure is undamaged, this damage should be classed as Category 4.

4. MAJOR DAMAGE:

   Very significant non-structural damage and considerable structural damage.

   X-cracking in shear walls, and spalling in beam-column joints.

   Remarks: Be sure to accurately choose between the levels 3 and 4. Do not hesitate to ask the opinion of other engineers.

5. CONDEMNED OR COLLAPSED BUILDINGS:

   For example:
   - a story has pancaked
   - a building tipped over
   - too many beam-column joints are fractured

   In general, buildings to be condemned are those which have experienced too much deformation, or where the repair cost would be equal to the initial cost of the structure.

   Conclusion:

   Green color: ..... Level 1 and 2

   Orange color: ..... Level 3 and 4

   Red color: ..... Level 5

APPENDIX C
ANALYSES OF SEISMIC PERFORMANCE OF CANOPIES ON A PRIMARY SCHOOL BUILDING

INTRODUCTION

General Remarks

A primary school located about 5 km east of El-Asnam was constructed with RC canopies covering the corridors between the one-story classrooms. Some of these canopies collapsed during the October 10, 1980 El-Asnam earthquake; however, most remained standing but with severe structural damage to the columns. Because of the simple structural system used in these canopies and because no record of the ground motion originated by this earthquake was obtained, it was considered of interest to conduct a detailed analysis of the performance of these canopies with the following objective.

Objective and Scope

The main objective in conducting numerical analyses of the performance of the RC canopies was to use this performance as a transducer, if possible, to have a gross estimation of the intensity of the ground motion (peak effective acceleration).

To attain this objective, a series of nonlinear dynamic analyses of the behavior of these canopies has been conducted, varying certain parameters on which no reliable information was obtained. Because of the lack of reliable information about the real mechanical characteristics of the materials used and of the sensitivity of the dynamic response of the structure to the dynamic characteristics of the ground motion, it must be recognized that the results obtained should be interpreted just as a crude estimation of the peak effective acceleration.

The following is a summary of studies that are being conducted by J. Cartin and V. Bertero (1982).

RC CANOPY STRUCTURAL SYSTEM

Actual Structural System

As illustrated in Fig. 4.27, each canopy is about 3.50 x 19.45 meters in plan, supported by a row of four centrally located cylindrical columns. The average nominal dimensions obtained for field measurements are given in Fig. 4.27. Although it was not possible to obtain information regarding the columns' foundation from an inspection of the performance, it was clear that the foundation had not suffered significant movement, i.e., it acted as a rigid foundation. From the way the canopy collapsed, as well as from the damage and permanent deformation observed in

those canopies that remained standing, and from preliminary estimations, it became clear that the only significant response occurred in the transverse direction of the canopies. Therefore, only the response under a horizontal ground motion acting in the direction perpendicular to the longitudinal axis of the canopy will be considered here.

Analytical Modeling of Structural System

As illustrated in Fig. 4.28, the structural failure of the canopies was triggered by the failure of the columns at their base. This failure was, in turn, triggered by flexural yielding of the main reinforcement followed by crushing of the concrete. In view of this type of behavior, the actual structural system illustrated in Fig. 4.27 has been idealized as the one shown in Fig. C.1, where

$L = 2.67\text{m}$

M = concentrated mass at roof = 1167 kg-sec^2/m

m = distributed mass of columns = $\dfrac{23.5 \text{ kg-sec}^2/\text{m}}{\text{m}}$

I_o = mass moment of inertia of roof = 1228 kg sec^{2}m

EI = column flexural stiffness $\begin{cases} \text{Uncracked transformed cross section} & EI_{uc} = 1.85 \times 10^6 \text{ kgm}^2 \\ \text{Cracked transformed cross section} & EI_{cr} = 0.77 \times 10^6 \text{ kgm}^2 \end{cases}$

The above values have been estimated from the measured dimensions and from assumed mechanical characteristics of the material, as discussed below.

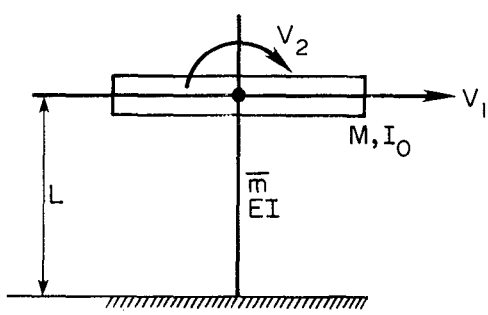

Fig. C.1 Analytical Model of Actual Canopy

PREDICTION OF MECHANICAL BEHAVIOR OF CANOPY COLUMNS

Mechanical Characteristics of Column Materials

Regarding these characteristics, no testing data nor design values were available. However, based on visual inspections of the materials used and on conversations with people who were involved in the construction, the following values were selected as good approximations of the actual values.

Concrete: f'_c = 2500 psi (176 kg/cm^2). Although only the results obtained with this basic maximum compressive strength are reported, analyses were carried out using a ±500 psi variation. Using this assumed unconfined concrete strength, the stress-strain relationship shown in Fig. C.2 was adopted for the prediction of the mechanical behavior of the canopy column.

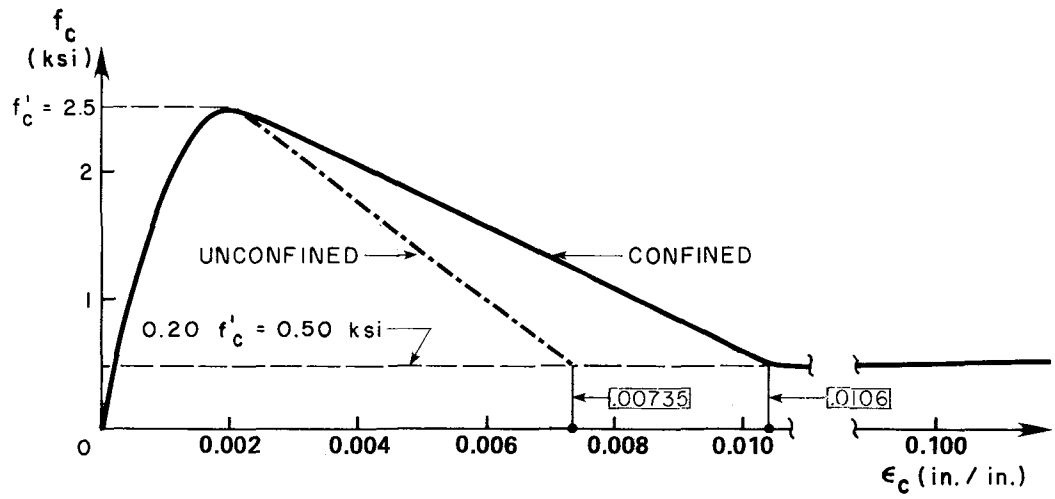

Fig. C.2 Stress-Strain Relationship Selected for Column Concrete (Kent and Park, 1971)

Reinforcing Steel: For the main longitudinal and the transverse reinforcement, different yielding strengths and stress-strain characteristics were considered. The results presented here are based on the stress-strain relationship shown in Fig. C.3, which is based on a yielding stress of f_y = 40,000 psi (2810 kg/cm^2).

Moment-Curvature (M – ϕ) Relationships of Column Cross Section

The RCCOLA Program (Mahin, Bertero, 1977) was used to obtain several M-ϕ relationships considering a series of variables besides variations of the mechanical characteristics of the materials stated above. Some of the variables considered are the following.

Axial Force, P. At the critical cross section of the column (bottom of column), the axial force produced by the weight of the canopy and column was estimated as 12 tons (26.4 K). In order to study the possible effect of the vertical component of the ground motion acceleration, estimated to be as high as 1.0 g, it was decided to evaluate the M-ϕ curve for P = +24 tons (52.9 K) and P = 0 tons. Fig. C.4 shows the M-ϕ diagrams for the three different values of P. Although the variation in P somewhat affected the stiffness and strength of the column, in view of the other uncertainties this variation can be neglected.

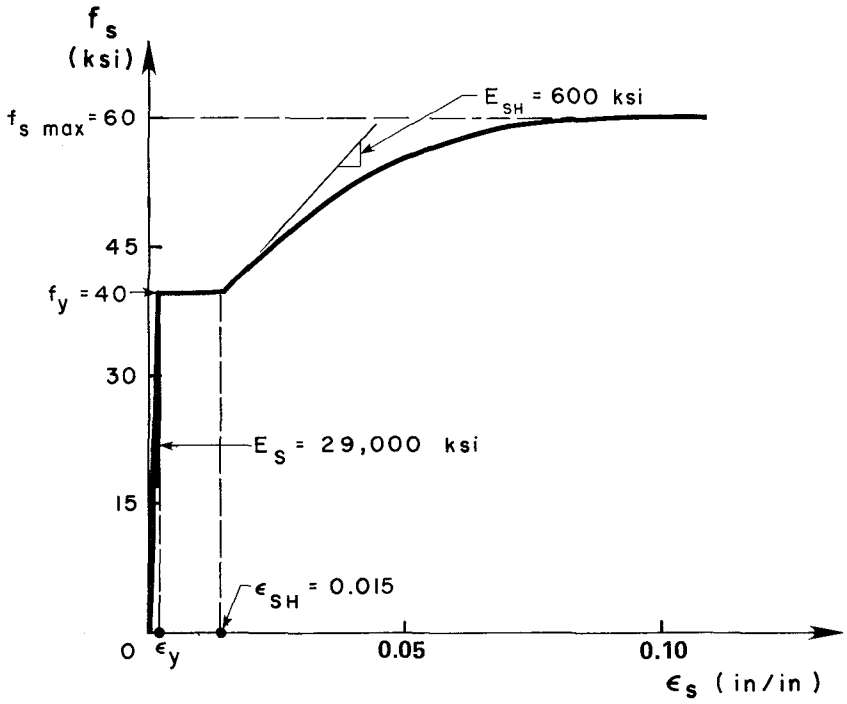

Fig. C.3 Stress-Strain Relationship Adopted for Column Longitudinal Reinforcement

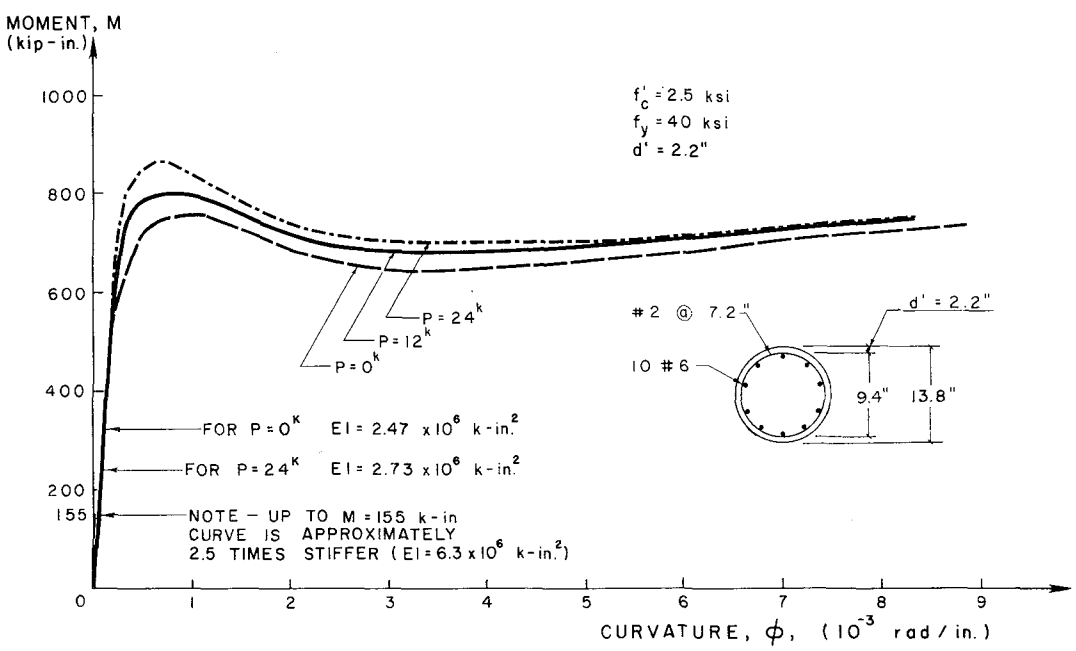

Fig. C.4 M-ϕ Diagrams for Column Cross Section Under Different Axial Loads, P

Different Cover Thickness. During damage inspection, measurements showed that while the total diameter of the cylindrical column was practically constant 13.8 in. (35 cm), the cover varied between 1.07 in. (2.7 cm) and 1.57 in. (4.0 cm). This gives as a result values of d varying from 1.7 in. (4.3 cm) and 2.2 in. (5.6 cm), where d is the distance from center of longitudinal steel reinforcement on tension side to outside edge of concrete on opposite side of column.

As shown in Fig. C.5, the main effect of an increase in cover is a decrease in yielding and maximum flexural strength of less than 10 percent.

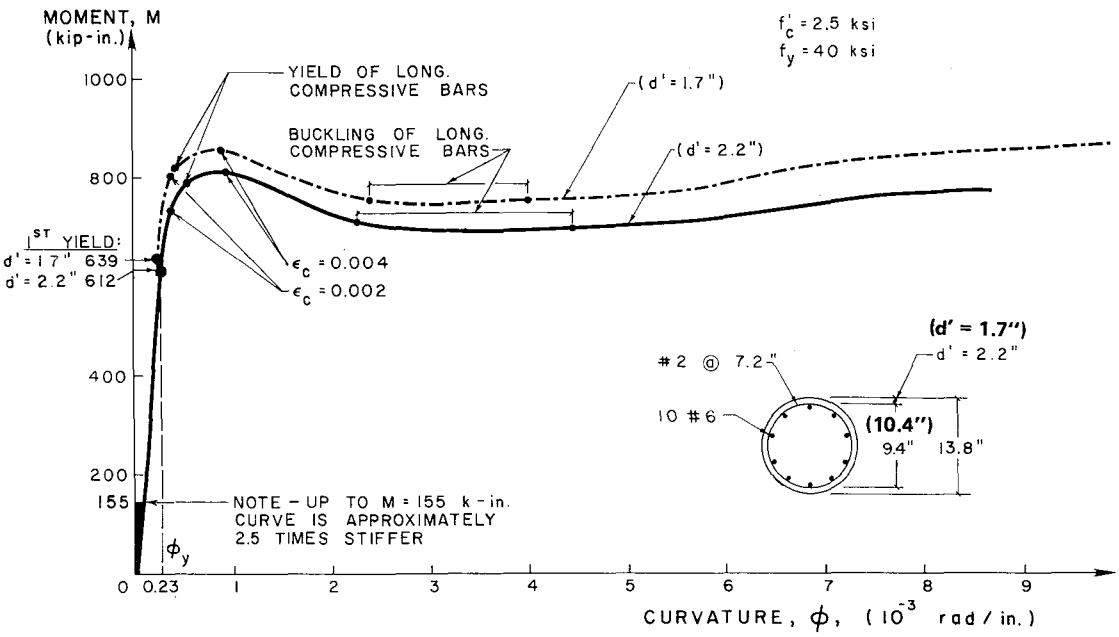

Fig. C.5 Effect of Cover Thickness on M-ϕ Relationship of Column Cross Section Under P = 12t

Shear Strength of Column

The available shear strength of the cylindrical column was estimated using the equations suggested by Capon and Diaz de Cossio (1965). Considering the resistance of concrete against shear as effective, the shear strength available was found to be considerably higher than the shear resulting from the lateral force required to induce flexural failure. However, if the 1979 UBC specification 2626 (f) 5 is applied, which requires that when v_c shall be considered zero when $P_e/A_g < 0.12\, f_c$, the column should fail in shear before it even starts yielding. As already pointed out, inspection of the damage in the standing canopies and the type of failure observed in the collapsed canopies revealed that the failure was of the flexural type. It was concluded that shear was not a problem, although according to present UBC recommendations it should have been the controlling strength.

Behavior of Critical Regions, Possibility of Buckling of Main Bars

Because the spacing of the circular hoops was relatively large, 7.2 in. (18 cm), larger than the 4 in. required by 1979 UBC for seismic zones 3 and 4 [2626 (f) 4B], and even larger than the eight bar diameters which for #6 bars implied a distance of 6 in. (15 cm) required for buckling, the possibility

of premature buckling after the unconfined cover of concrete spalls was also considered. (In a few of the canopy columns that remained standing, there was evidence that the bars might have just started to buckle.) To study the effect that buckling of main reinforcing bars can have on the behavior of the structure, it is necessary to study the behavior of the critical regions in the area where the bar may buckle. The logical method is to find the moment-rotation, M – ϕ), relationship along a region having the length along which the bar can buckle. To simplify the analysis it was assumed that the moment and curvature along the critical region will be constant and, therefore, it is possible to interpret the M – ϕ already found for the critical column cross section as the M-θ of the critical region, where θ is equal to ϕ multiplied by the length on which buckling can occur, in this case 7.2 in. (18 cm). According to this assumption, the M – ϕ at which buckling would start has been computed and is indicated in Fig. C.5. If the effect of the bars buckling is included in the determination of the M – ϕ_{av}, the diagram shown in Fig. C.6 is obtained.

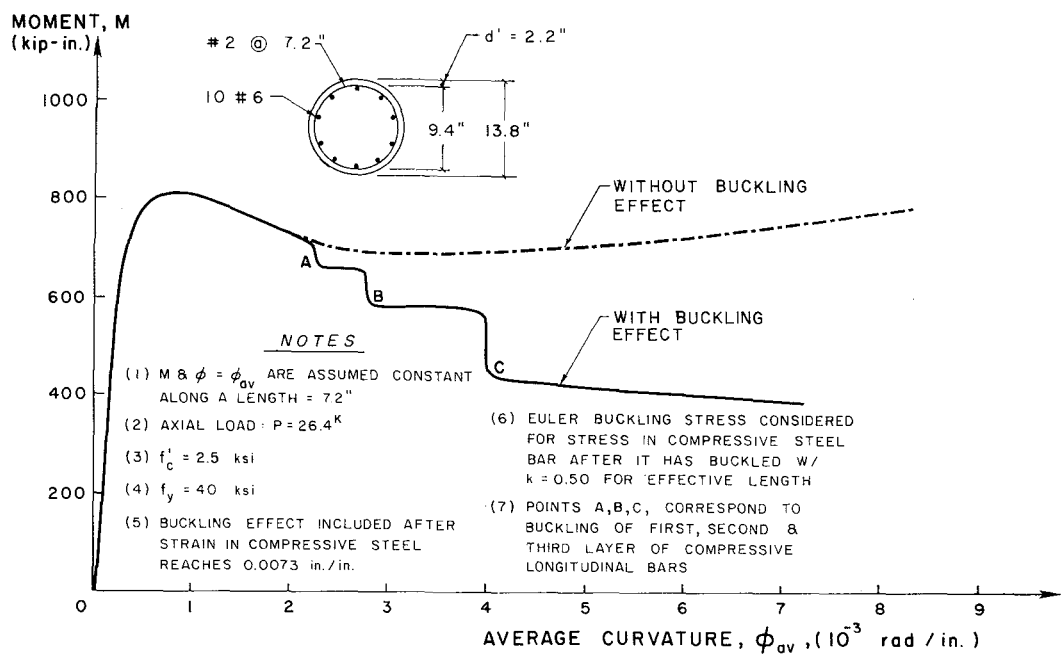

Fig. C.6 Effect of Compression Reinforcement Buckling on Moment – Curvature (M – ϕ) Relationship

DYNAMIC CHARACTERISTICS OF CANOPY

Canopy Analytical Model

The actual structural system has been mechanically idealized, as shown in Fig. C-1. Although this idealized system has infinite degrees of freedom, because of the relatively small value of m compared with M and the nature of the ground motion, for the practical application under consideration it is possible to represent the system by an equivalent two-degree-of-freedom system.

Periods, T

Considering the inertia forces shown in Fig. C.1 and using elementary beam theory to determine influence coefficients, the frequency equation is easily obtained from which the following different values of T have been estimated.

1) Using a constant column stiffness corresponding to the uncracked transformed section, EI_{uc}: $T_1 = 0.46$ sec, $T_2 = 0.11$ sec.

2) Using a constant column stiffness corresponding to the cracked transformed section, EI_{cr}: $T_1 = 0.72$ sec, $T_2 = 0.17$ sec.

Because of the nature of variation of moment along the column, it is expected that initially the period of the first mode of vibration was closer to 0.46 sec than to $T_1 = 0.72$ sec. Thus, it was decided to use $T_1 = 0.50$ sec in estimating effective peak acceleration. The above values were obtained assuming a fixed foundation, which according to the actual type of construction and performance of the structure, appears to be a good approximation.

Damping Ratio, ξ

Considering that this is a very clean building consisting of only the bare structural system, the damping ratio, ξ, for the first mode of vibration has been selected as 2 percent. However, to study what the effect of ξ might be on the effective peak acceleration, a value of $\xi = 5\%$ has also been considered.

ESTIMATION OF EFFECTIVE PEAK GROUND ACCELERATION

General Remarks

To estimate the effective peak ground acceleration, a_{ep}, different approaches can be followed. In the values reported here, an approximate procedure based on the charts developed by Bertero, Mahin, and Herrera (1976) was adopted. This procedure requires idealizing the hysteretic behavior of a one-degree-of-freedom structure as being linear elastic/perfectly plastic. Besides the estimation of the yielding strength, R_y, and the displacement at yielding, v_y, it is required to estimate the maximum displacement ductility available, μ_δ, and the damping ratio, ξ. Furthermore, the dynamic characteristics of the ground motion should be known or assumed.

Idealized Hysteretic Behavior

The estimated M-ϕ diagrams shown in Fig. C.5 have been used to estimate idealized lateral resistance functions, R vs. v_1, of the canopy. The linear elastic/perfectly plastic idealization of the R vs. v_1 used in estimation of a_{ep} is shown in Fig. C.7. In this figure a range of possible values for $v_{1\ max}$ is indicated. This range is limited by the values of ϕ_u, controlled by the buckling of the longitudinal reinforcing bars, indicated in Fig. C.5.

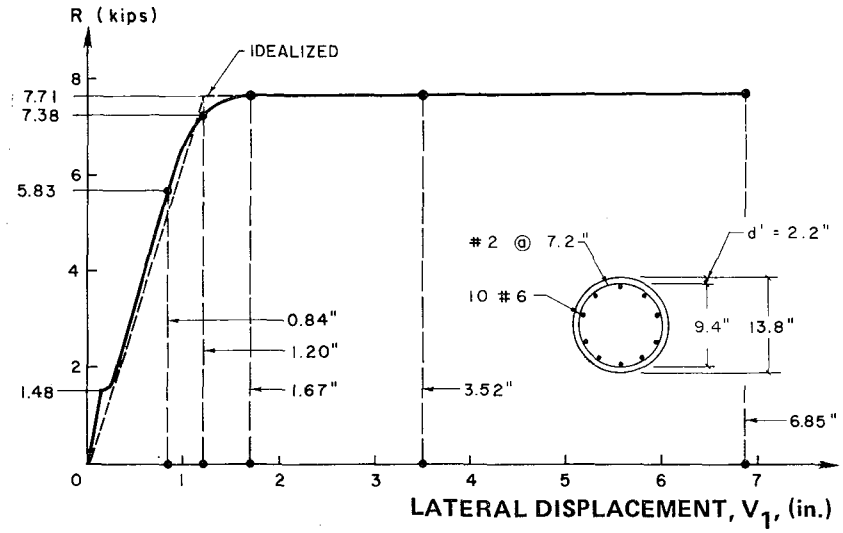

Fig. C.7 Lateral Resistance Function, R vs. v_1

According to the procedure suggested by Bertero et al (1976) to estimate a_{ep}, it is necessary to estimate the value of the maximum displacement ductility ratio, μ_δ, of the idealized linear elastic/perfectly plastic seismic resistance, R,-displacement, v, relationship. This μ_δ has been estimated using different approaches, two of which are summarized below.

Park and Paulay (1975) Approach. These authors offered an approximate solution for the relationship between curvature ductility and displacement ductility ratios in the case of a cantilever column with a lateral load at the end (Fig. C.8). Based on the assumed curvature distribution shown in Fig. C.8, the following relationship is derived.

$$\mu_\delta = 1 + \left(\frac{\phi_u - y\phi}{\phi_y}\right) \frac{\ell_p\,(\ell - 0.5\,\ell_p)}{\frac{\ell^2}{3}}$$

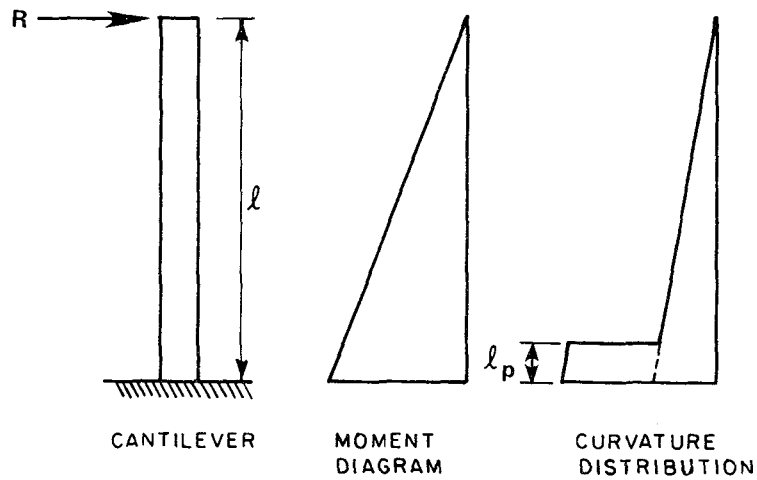

Fig. C.8 Curvature Distribution of a Cantilever Column (Park and Paulay, 1975)

Using the M-ϕ shown in Fig. C.5 for case d' = 2.2 in., the value of the equivalent length of the plastic hinge, ℓ_p, has been estimated as 26 in. (66 cm). With this value, the values of ϕ_u and ϕ_y given in Fig. C.5, the above expression yields a value of μ_δ = 6.80. Note that besides the assumptions of the curvature distribution and length of plastic hinge in this approach, the effect of vibration in the second mode on the bending moment distribution along the length of the cantilever column is neglected.

Second Approach. As discussed in the main text (sec. 4.2.1), the canopies were able to undergo lateral displacement of at least 6 in. (15 cm) without failure. Assuming this displacement as the v_u, and using for v_y the value of 1.2 in. (3 cm) estimated in Fig. C.7 as the equivalent yielding displacement for the linear elastic/perfectly plastic resistance function, μ_δ = 5 has been estimated.

Estimation of Effective Peak Ground Acceleration, a_{ep}

According to the procedure suggested by Bertero et al (1976), to estimate the value of a_{ep}, it is necessary to know the values of the T, ξ, μ_δ, the mass, m, and the yielding strength, R_y, of the idealized linear elastic/perfectly plastic behavior of the one-degree-of-freedom structural system. Furthermore, the dynamic characteristics of the ground motion must be specified. As no ground motion has been recorded, it was decided to use two different ground motions which can be considered as bound as far as inelastic deformations demands are concerned, the N-S component record of the 1940 El Centro earthquake and the S16E derived Pacoima Dam base rock record. From inelastic deformation demands, the El Centro record can be considered as of the resonance (harmonic) type of ground motions being a low bound in demands for structures with T $\geq$ 0.4 sec. The derived Pacoima record is of the impulsive type and constitutes an upper bound in demands.

Required a_{ep} for Normalized El Centro Record. Using T = 0.5 sec, ξ = 2%, and μ_δ = 5.0 from the graph prepared by Bertero et al (1976) shown in Fig. C.9, a value of $\eta \doteq 0.55$ is obtained. Considering that:

$$\eta = \frac{C_y}{\ddot{u}_{g\,max}/g} = \frac{R_y}{ma} = \frac{1}{\ddot{u}_{g\,max}/g} = \frac{R_y}{m\ddot{u}_{g\,max}}$$

and that for the equivalent single-degree-of-freedom system of Fig. C.1 the reactive mass to the horizontal component of the ground motion is:

$$m = M + m\,L/2 = 1167 + 23.5 \times 2.67/2 = 1198 \text{ kg-sec}^2/\text{m} = 67.02 \text{ lb-sec}^2/\text{in.}$$

and the yielding strength, R_y, is approximately = 7714 lb (3500 kg)

$$a_{ep} = \ddot{u}_{g\,max} = \frac{7714\text{ g}}{(0.55)\,(67.02)\,(386)} = 0.55\text{ g}$$

Note that the yielding seismic coefficient, C_y, of the canopy becomes R_y/mg = 0.29, a value considerably higher than required by present United States seismic codes. Repeating the above procedure in the case of ξ = 5%, a_{ep} becomes equal to 0.60 g.

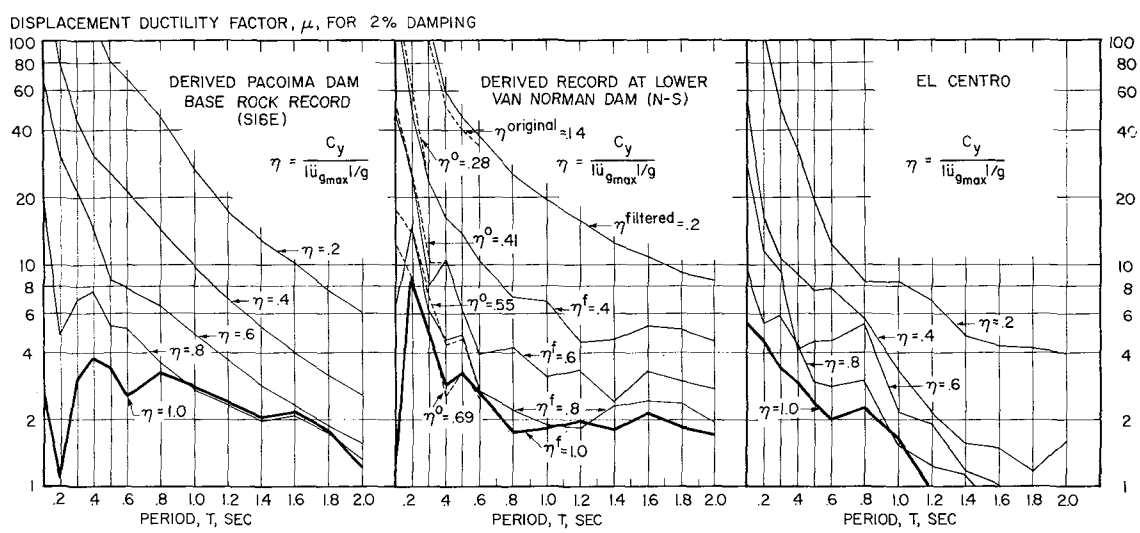

Fig. C.9 Inelastic Response Spectra for 2 Percent Damping

Required a_{ep} for Normalized Derived Pacoima Dam Record. Using the same values considered above but using the inelastic response spectra corresponding to the derived Pacoima Dam type of record (Fig. C.9), the following values are obtained for a_{ep}:

For $\xi = 2\%$: $a_{ep} = 0.35$ g

For $\xi = 5\%$: $a_{ep} = 0.39$ g

Concluding Remarks

The intensity, represented by effective peak acceleration (a_{ep}) of one horizontal component of the ground motion, has been estimated based on a series of assumptions. The analysis presented clearly illustrates the large number of uncertainties involved in the estimation of response of any structure and, consequently, in the estimation of the so-called effective peak acceleration. Given the many uncertainties and assumptions in the analysis, the values obtained should be considered as approximate estimations pointing toward a solution of the problem.

Considering the values obtained for $\xi = 2\%$, which the authors believe are more realistic, there is still a great difference in the estimated values for a_{ep}, depending on the dynamic characteristics of the normalized ground motions; 0.35 g for the derived Pacoima Dam type of ground motion to 0.55 g for El Centro. From analysis of the El-Asnam Earthquake aftershock records, it appears that most of these ground motions are closer to the El Centro type record than to the derived Pacoima Dam record. Thus, it would appear that a_{ep} at the site of the primary school canopy was closer to 0.55 g. It is obvious that the use of a_{ep} as the only parameter to define seismic risk and, particularly the intensity of the design earthquake for building seismic-resistant construction, is not sufficient.

It should also be noted that a nonlinear dynamic time-history analysis of the two-degree-of-freedom model shown in Fig. C.1 when subjected to the N-S component of the 1940 El Centro record normalized to a maximum peak acceleration of 0.50 g resulted in a maximum moment at the top of the column of 745 K-in. When this is compared with the value of moment at first yielding of steel (612 K-in.) and the maximum moment of 810 K-in. (Fig. C.5), it is clear that the effect of the rotational inertia of the mass M (Fig. C.1) cannot be neglected and some of the damage of the concrete along the total height of the column (Fig. 4.29) might be justified.

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APPENDIX D
MODIFICATIONS TO SEISMIC REGULATIONS*

PREAMBLE

As a result of the earthquake of October 10, 1980 at El-Asnam, and after surveying the damaged structures, the following requirements have been worked out as a first stage toward improving the existing rules in Algeria.

These requirements are compulsory. They cancel, complete, or take the place of those used previously.

The goal of this complement is to provide provisional requirements for the projects (designs) presently being studied and should be enforced until publication of the Algerian seismic regulations.

PRESENTATION

- The first column on the left displays the number of the article being modified.
- The second column on the left uses the code below:

  C: The requirements of this document complete the article under review.

  S: The requirements of this document take the place of the article under review.

CTC, "Compléments aux Règles Parasismiques, February 1981. Translated from French by V.V. Bertero.

RULES COMMON TO DIFFERENT TYPES OF CONSTRUCTION

1.1 C High Seismic Risk Soils

"High seismic risk soils" are soils with mechanical characteristics that can suffer significant changes when subjected to cyclic loading.

An earthquake may initiate very unfavorable effects in these soils, such as: settlement, loss of strength, or liquefaction.

For instance, the following are "high seismic risk" soils:

- Muds
- Clays with lime
- Loose saturated sands
- Certain kinds of underconsolidated fills

All these "high seismic risk" soils must be specifically studied.

Substructures

Definition and Purpose

The substructure is made up of the foundation and elements of the structure that are partially or totally below the ground level (all elements below the 0.00 conventional level).

The substructure shall consist of a rigid assemblage capable of the following functions:

- Supplying full fixity to the structure in the ground (sections have to remain plane, deformations shall be compatible).
- Transmitting to the foundation soil all the forces supported by the structure.
- Limiting the differential settlements to a reasonable value.

General Provisions

Avoid joints in the substructure. They should be allowed only in cases where significant differences exist in the vertical loads or in the soil conditions.

Elements of the substructure: floor slabs above *vide sanitaire* (crawl space) or basement, slabs cast on the ground, interior and peripheral walls, longitudinal girders and foundations, should be constructed of reinforced concrete cast in place in order to attain a monolithic assembly.

Precast elements can be used for the low level floor, provided that a compression slab of a minimum thickness of 6 cm is cast in place.

Substructure of Multistory Buildings

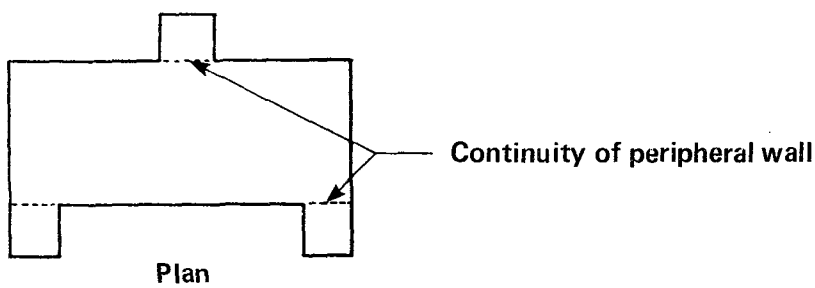
The substructure has to be made of the following elements:

a) A continuous peripheral wall* between the level of foundation (footings, foundation rafts, pile caps, etc.,) and the level of the first floor above the outside ground (0.00 conventional level).

Where the substructure consists of blocks divided by settlement joints, that wall shall gird (collar) each block.

This wall shall have the following minimum characteristics:

- Height ≥ maximum (1/10 of the height of the building; 0.80 meter).
- Thickness ≥ maximum (1/10 of its height; 0.15 meter).
- Longitudinal continuous superior (top) and inferior (bottom) reinforcement, whose section is ≥ 0.20 percent of the transverse section of concrete with overlappings 50 diameters, with reinforcements at right angle at the corners.
- Skin longitudinal reinforcement with section ≥ 2 cm^2 per meter of height for each face.
- The openings in this wall shall not noticeably decrease its stiffness.
- If the exterior (facade) walls have irregularities in the plane view, the foundation wall under these exterior walls should be continuous and straight throughout the length of the facade.



b) A continuous wall at right angles with each interior transverse and longitudinal component of bracing walls with or without openings. Each of these walls should be extended to permit its connection to a perpendicular wall of the same kind or to the peripheral wall.

The building requirements for interior walls are similar to those for peripheral walls.

c) For isolated foundations (footings or piers): a longitudinal girder or a continuous wall, at right angles with each internal transverse and longitudinal row that is made of columns only.

*"Wall" is used as an English translation of the French word "voile." It can also be translated as "web".

The longitudinal girders are not compulsory in zone I when the vertical distance between the top of the foundations and the lower point of the joists of the floor at 0.00 in. does not exceed 1 meter.

The longitudinal girders shall withstand an axial force at least as large as 10 percent of the largest vertical load.

d) In the case of continuous foundation footings: a longitudinal girder or a continuous wall at right angles with each row perpendicular to the continuous footing. Longitudinal girders may not be provided in the conditions of paragraph c above.

e) Floors above the crawl space (*vide sanitaire*) or above basement and slabs on ground shall be provided with a continuous reinforcement in one or two layers in the form of a grid or of welded mesh. These layers have to be anchored in the peripheral walls with a minimum reinforcement of 0.25 percent in each direction.

Substructure of Industrial Buildings (One Level Only)

In the case of a basement or of a *vide sanitaire,* the requirements relative to the substructure of the multistory buildings still apply.

In the case of a slab cast on the ground, the requirements relative to the substructure of the multistory buildings can be reduced as follows:

- Peripheral walls can be replaced by a longitudinal girder at the level of the foundation.
- Peripheral walls and longitudinal girders at right angles with interior rows are optional when those rows are frames with hinges at their bases.
- Walls and longitudinal girders at right angles with interior rows can be deleted, in the case of columns with fixed bases only if the soil characteristics of the foundation enable the designer to justify this fixity without the help of tightening elements in the substructure.

1.2 C Bracing Systems

Bracing systems for which the present document gives specifications to justify their resistance are the following:

- Self-stable (or self-lateral resistant) framework in reinforced concrete.
- Framework in reinforced concrete braced by concrete walls (webs).
- Framework in reinforced concrete braced by masonry walls.
- Concrete bearing walls molded with forms.
- Self-stable (self-lateral resistant) steel framework or braced with concrete walls (webs).
- Self-stable (self-lateral resistant) timber framework braced with concrete walls (webs).

Restriction of Use of Masonry for Lateral Bracing

Reinforced concrete frameworks braced by masonry panels and masonry bearing walls are allowed (provided that the seismic requirements of paragraphs 2.11 and 3.10.5 are followed) only for the structures R + 0 and R + 1 with a soil surface not exceeding 200 m^2.

Bracing Systems Different from Those Indicated Above

Other systems, such as those based on the use of heavy or light precast elements, prestressed concrete framework, . . . etc . . . can be accepted provided that the central agency has been previously satisfied with their lateral resistance.

Requirements Particular to Structures with Rigid Floors (or Diaphragms)

The structures made of floors or of horizontal diaphragms rigid in their plane shall have lateral bracing as similar as possible in both horizontal directions. In the zones of moderate and high seismic risk (zones II and III), the lateral bracing of those buildings shall be of the same kind in both directions.

1.10.4 S Strength Justification According to Existing Rules

Foundations:

For the most unfavorable case as defined in 3.10.3, the allowable stress in foundation soils without "high seismic risk" can be taken equal to the allowable stress for vertical loads increased by the following accounts:

- 50 percent for rocks
- 30 percent for soft soils

In the case of shallow foundations, one has to ascertain that the eccentricity of the resultant of the vertical loads does not exceed one-quarter (1/4) of the dimension of the foundation in the considered direction.

The strength justification of the "high seismic risk" soils shall be carried out in accordance with the specific study that is required (cf §1.1, - complement).

Self-Stable (Self-Lateral Resistant) Framework in Reinforced Concrete:

- **Columns:** The strength verification to the most unfavorable normal stresses due to combined bending shall be carried out with the concrete and steel allowable stresses of the first kind increased at the most by 25 percent.

  The verification for shear strength shall be carried out considering the following values for the shear force, T, and axial force, N:

  T = two times the computed shear force *
  = three times the computed shear force **

  $N = 0$

  $\bar{\tau}_b = 0.15\ \sigma_{28}$,

  $\bar{\sigma}_{at} = \sigma_{en}$

*If the slenderness in the considered direction is ≥ 15.
**If the slenderness in the considered direction is ≤ 15.

- **Beams:** The strength verification to the most unfavorable normal and shear stresses shall be carried out with the concrete and steel allowable stresses of the first kind increased at the most by 50 percent.

Reinforced Concrete Frameworks Braced by Concrete Walls (Webs) and Concrete Structures Molded with Forms

- **Thin Walls (Webs) and Solid Walls**

The strength verification to the most unfavorable normal stresses due to combined bending shall be carried out with the allowable concrete stresses of the first kind increased at the most by 25 percent and with the allowable steel traction stress at the most equal to σ_{en}.

The strength verification to shear stresses shall be carried out with

$$T = 1.5 \text{ times the computed shear force}$$
$$N = 0$$
$$\bar{\tau}_b = 0.12\,\sigma_{28}$$
$$\bar{\sigma}_{at} = \sigma_{en}$$

- **Thin Walls (Webs) and Walls with Openings in a Line**

The strength verification of the piers to the most unfavorable normal (due to bending) and shear stresses shall be carried out with

$$T = 1.5 \text{ times the computed shear force}$$

N is computed with the above T value

$$\bar{\tau}_b = 0.12\,\sigma_{28}$$
$$\bar{\tau}_b = 0.75\,\sigma_{28}$$
$$\bar{\sigma}_a = \bar{\sigma}_{at} = \sigma_{en}$$

- **Columns and Beams**

The strength verification to the most unfavorable normal and shear stresses shall be carried out with concrete and steel allowable stresses of the first kind increased at the most by 50 percent.

Reinforced Concrete Frameworks Braced Laterally by Masonry Panels and Structures with Masonry Bearing Walls

- **Masonry Panels and Masonry Bearing Walls**

The strength verification of the masonry to the most unfavorable stresses shall be carried out with an allowable stress of the first kind increased at the most by 25 percent in the case of solid elements of masonry and with no increase in the case of masonry of hollow units.

- **Columns and Beams**

The strength verification to the most unfavorable normal and shear stresses shall be carried out with the steel and concrete allowable stresses of the first kind increased at the most by 50 percent.

Steel Frameworks

The strength verification of steel frameworks to earthquakes shall be carried out by following the requirements of the existing computation rules of steel work (cf § 1-123 "exceptional circumstances").

Timber Frameworks

The strength verification for timber frameworks to earthquakes shall be carried out with allowable stresses of the first kind increased at the most by 50 percent*.

1.11 C Computation Principles

The simplest equivalent static computation, based on the seismic coefficients (α, β, γ, δ) determined for the single fundamental mode, is authorized only if the following conditions are met:

- The structure or the unit under study is less than 45 meters high in zones I and II or 30 meters in zone III.
- The configuration (layout) in plan of the structure or of the unit under study is simple, symmetrical, and close to a rectangle, with recessed and projecting parts not larger than 25 percent of the general dimensions.
- In the case of irregularities in elevation, the change of dimensions in both directions does not exceed 25 percent between two adjacent levels and decreases with increasing height.
- The distance between the center of mass and the center of rigidity (torsion) does not exceed all levels 20 percent of the effective width of the structure or of the unit under study (this width has to be measured perpendicular to the direction of the considered seismic action).
- The ratio of the mass to the stiffness of the adjacent levels shall not vary more than 25 percent in each direction.
- The damping ratio is similar at all levels of the building or unit. Particularly in the case of self-stable (lateral resistant) frameworks with masonry infills, the infills between columns of the framework at all levels must have a density of the same order of magnitude.
- The structure does not have several degrees of freedom in the same horizontal plane (cf 3, 114-2).
- The fundamental period of poorly damped structures with several degrees of freedom is at the most equal to 0.5 sec.

*When those frameworks are braced by concrete thin walls (webs), those webs have to be verified according to the provisions relative to the bracing webs of reinforced concrete frameworks.

1.11 S Torsion of Whole Structure

For all structures made of floors or horizontal diaphragms that are rigid in their plane, it shall be assumed that, at each level and in each direction, the eccentricity of the resultant of the horizontal forces relative to the torsion center is the largest of the two following values:

- 5 percent of largest dimension of the structure at that level
- theoretical eccentricity based on the structural plan

1.12-12 C Intensity Coefficient, α

Group	Risk Factor	Examples	Zone I	Zone II	Zone III
1	Acceptable risk	Dwellings, offices, factories	0.5	1	1.6
2	Special risk because of high occupancy and importance to region	Schools, stadium, theaters, power plants	0.75	1.4	2.2
3	Safety is of prime importance	Hospitals, barracks	1	1.8	2.6
4	Destruction poses a high danger	Liquid gas plants	Each case should be studied individually		

1.12-13 C Response Coefficient, β

Selection of the coefficient β is limited to *low or medium (moderate) damping.* The decrease of β in the case of large layers of soft soil is no longer allowed.

For the standard type of structures with framework on walls, the damping shall be considered as *medium.*

For industrial structures, the bare frameworks, the elevated water tanks, the chimneys, the steel towers, the isolated staircases and all structures different from the average structure, the damping shall be considered as *low.*

1.11.2 2 S Seismic Coefficient for Vertical Direction

At a given level, the vertical seismic coefficient is taken equal to the largest of the two horizontal coefficients ($\alpha\beta_1\ \gamma\delta$ or $\alpha\beta_2\ \gamma\delta$).

3.22 S Horizontal Deformations

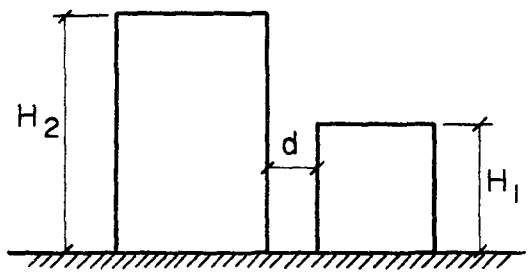
For the dwelling structures and equivalent (offices, school, etc.), it must be verified that $\Delta \leq 1.3\ h/1000$, where h is the height of one story and Δ is the relative displacement of a level estimated with the design forces assuming linear behavior (uncracked stiffness in the case of reinforced concrete).

For industrial buildings, warehouses, etc., without brittle infills (steel boarding, for instance), the relative displacement of a level can exceed the maximum value above. However, if $\Delta \geq 2h/1000$, the second order effects shall be taken into account to determine the stability of the structure.

21 S Width of Expansion Joints

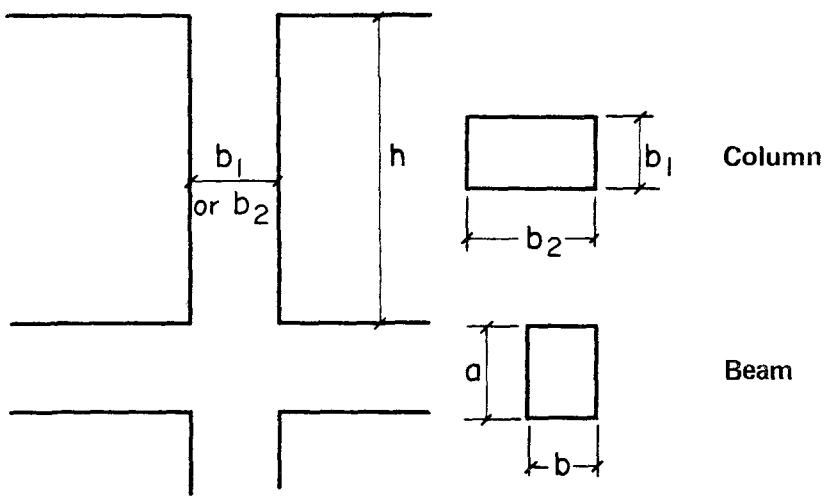
The width of the expansion joints shall allow horizontal deformations due to the computed (design) forces multipled by 1.5 to take into account the nonlinear deformations. The minimum allowable width for a joint is 2 cm.

In the absence of computations and only for dwelling structures and similar equivalents, one can take an outright width $d \geq H/500$, where H is total height of building.



31 S Self-Stable (Lateral Resistant) Framework in Reinforced Concrete

32 S Requirements Relative to Framework



- **Rectangular Columns:** a) $A = b_1, b_2 \geq KN/\sigma_{28}'$

  with $K = \begin{cases} 5 \text{ in zone III} \\ 4 \text{ in zone II} \\ 3 \text{ in zone I} \end{cases}$

  b) $M_{in.}(b_1, b_2) \begin{cases} 25 \text{ cm in zones I and II} \\ 30 \text{ cm in zone III} \end{cases}$

  c) $1/3 \leq b_1/b_2 \leq 3$

  d) $M_{in.}(b_1, b_2) \geq h/20$

- **Circular Columns:** a) Diameter, $\Phi \begin{cases} 25 \text{ cm in zones I and II} \\ 30 \text{ cm in zone III} \end{cases}$

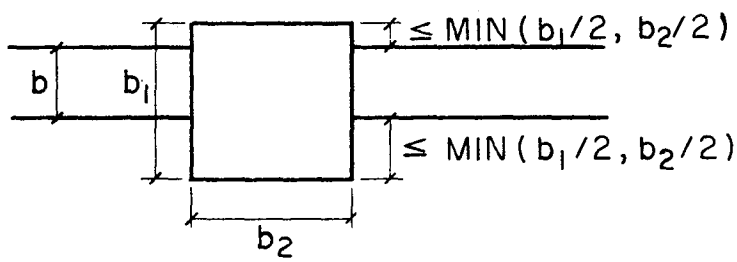
  b) Circular columns cannot be taken into account to equilibrate the horizontal forces.

- **Beams:** a) $b \geq 20$ cm

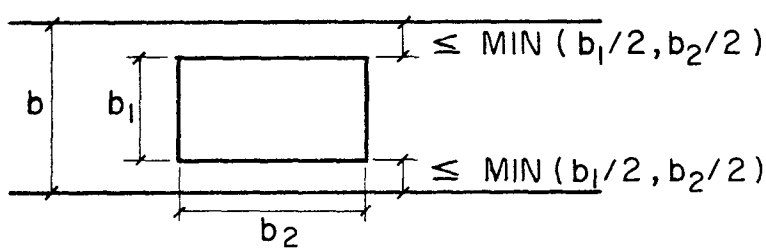
  b) $a \geq \begin{cases} 30 \text{ cm in zones I and II} \\ 40 \text{ cm in zone III} \end{cases}$

  c) $a/b \leq 3$

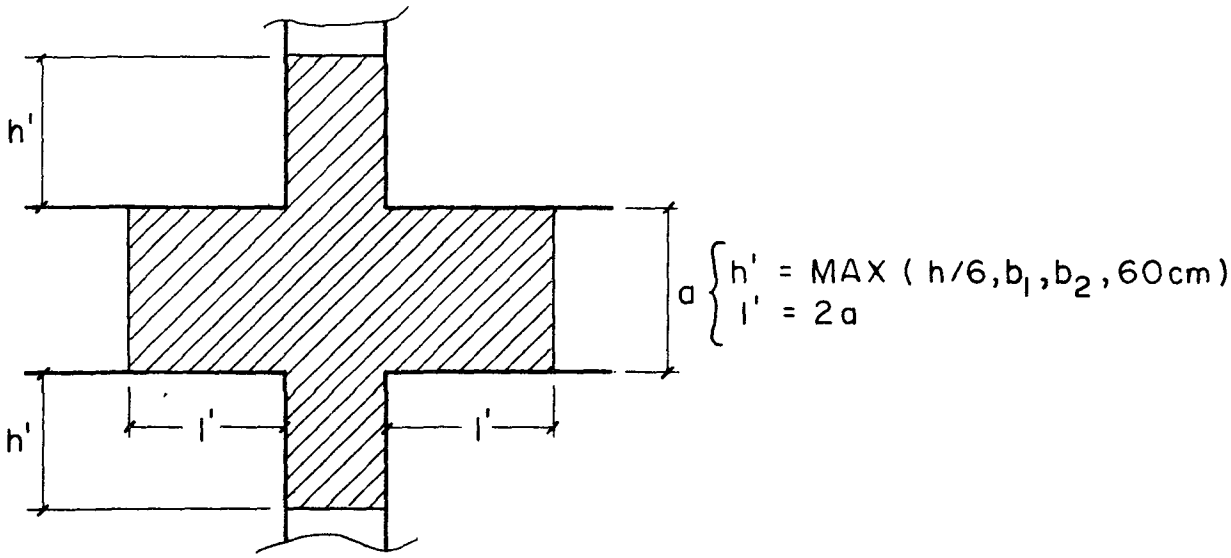
  d) Beams narrower than columns



  e) Beams wider than columns



- **Definition of Nodal Zone:**



Requirements Relative to Reinforcement

- **Columns**

  - **Longitudinal Reinforcements**

a) Longitudinal reinforcements shall be of high bond bars.

b) Minimum diameter is 12 mm in zone I and
14 mm in zones II and III

c) Minimum percentage of the reinforcement is

Zones I and II	0.8% for interior columns 0.9% for columns in facade 1.0% for corner columns
Zone III	1.0% for interior columns 1.1% for columns in facade 1.25% for corner columns

d) Maximum percentage of the reinforcement is 4 percent in the overlapping regions and 2.5 percent in between.

e) Hooks are prohibited at the overlapping places of the longitudinal reinforcement.

f) Minimum lapping length is

Zones I and II	50 diameters
Zone III	60 diameters

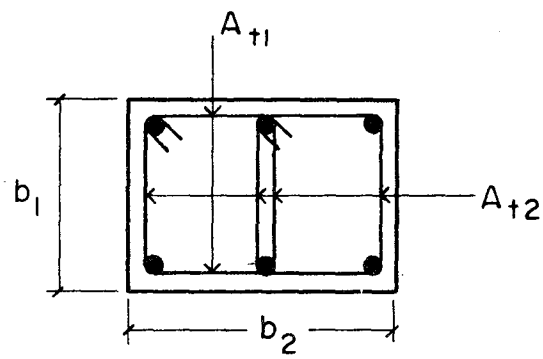
g) Distance between vertical bars on the face of a column shall not exceed 25 cm in zones I and II, and 20 cm in zone III.

- **Transversal Reinforcements**

a) Minimum diameter is 6 mm for high bond steel and 8 mm for plain steel.

b) Transversal reinforcement shall be closed ties and stirrups (2 strands).

c) The percentage ϱ_{t1} and ϱ_{t2} of the transversal reinforcements in a nodal zone defined below shall be at least equal to the following minimum percentages



Minimum Percentages ϱ_{t_1} and ϱ_{t_2}	
Zone I	0.3%
Zone II	0.4%
Zone III	0.5%

$A_{t1}/b_1t = \varrho_{t_1}$

$A_{t2}/b_2t = \varrho_{t_2}$

t = tie spacing

For interior columns, the condition of minimum percentage of transversal reinforcements does not apply in the height of the beam column node. Along this height, outside ties shall be kept with spacing conforming to the following:

d) The spacing, t, of the transversal reinforcements shall be determined as follows:

- In nodal zone (including the height of the node)

  Zones I and II $t \leq \min(10\Phi, 15 \text{ cm})$

  Zone III $t \leq 10$ cm

- In standard zones

  Zones I and II $t \leq 12\Phi$ (ϕ = diameter of vertical reinforcement

  Zone III $t \leq \min(b_1/2, b_1/2, 10\Phi)$

e) The ends of ties and cross ties shall be 135° hooks with a straight length of 10Φ minimum.

f) The ties and stirrups shall allow vertical chimneys (Φ chimney ≥ 12 cm) so that concrete can be vibrated correctly throughout the height of the columns.

- **Beams**

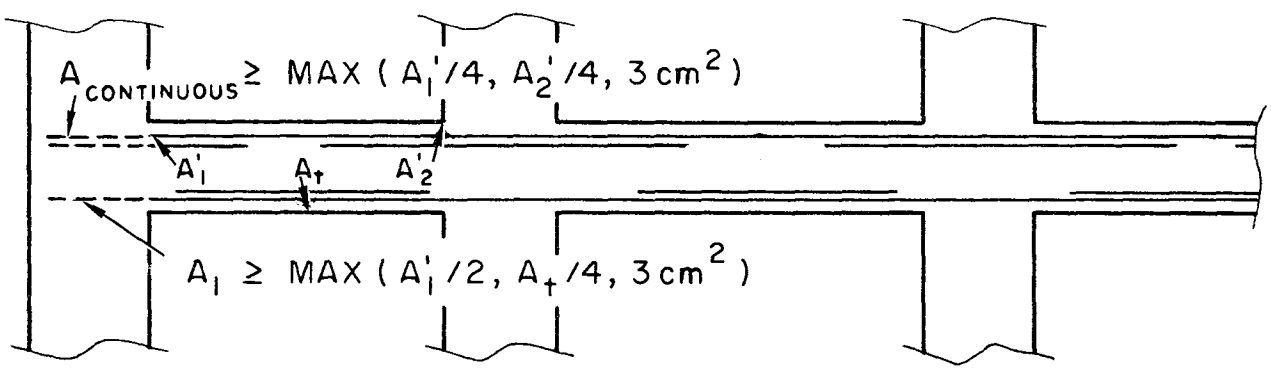
  - **Longitudinal Reinforcements**

a) Sections of the longitudinal reinforcements and ends of the bars shall be computed with the help of charts showing the envelopes of the most unfavorable bending moments resulting from vertical loads and earthquakes.

b) Minimum total percentage of the longitudinal reinforcements throughout the length of the beam shall be 0.3 percent for high bond steels and 0.5 percent for plain steel bars.

c) Maximum total percentage of the longitudinal reinforcement shall be 2.5 percent.

d) Beams sustaining vertical loads of floors shall be made of continuous reinforcements (bottom and top), having a minimum section as shown below.

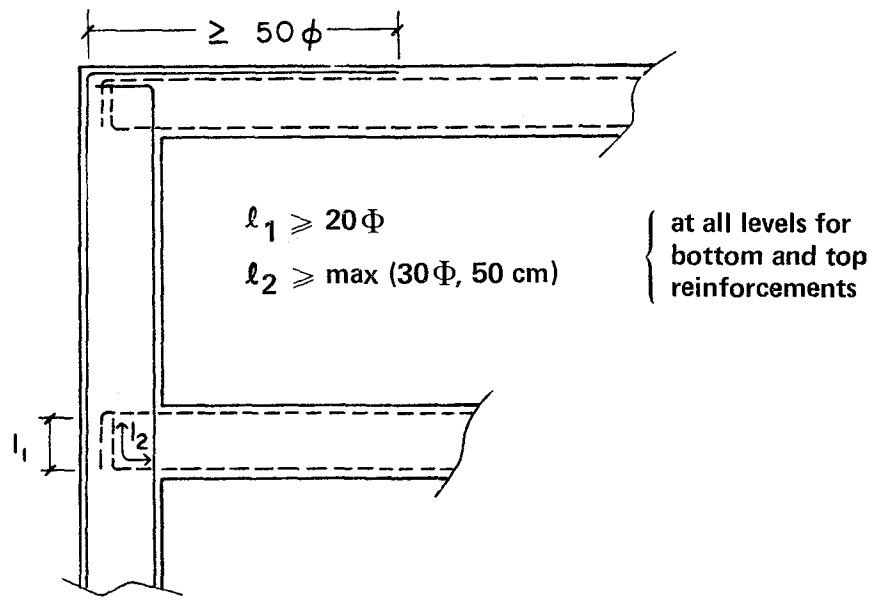


e) Beams sustaining low vertical loads and undergoing mainly lateral seismic loads shall be made of symmetrical reinforcements, whose section (amount) at middle span is at least equal to the half of the section (amount) on the supports.

f) Splicing by overlapping shall withstand the maximal traction (tension) force of the bars.

g) In zone III, splicing by overlapping shall be outside of the beam column nodes.

h) Anchorage of top and bottom longitudinal reinforcements in the side (external) and corner columns shall be carried out as indicated in the figure below.



- **Transversal Reinforcements**

a) Transversal reinforcements shall be computed with the help of charts showing the envelope of the most unfavorable shear forces resulting from vertical loads and earthquakes.

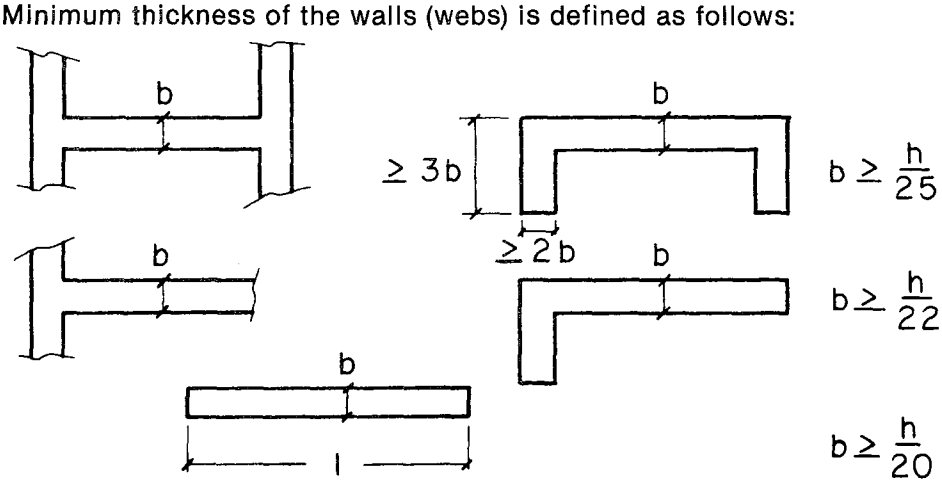
b) In the nodal zone, the transversal reinforcements shall surround and restrain laterally each longitudinal bar in order to prevent its buckling. Outside the nodal zone, at least one longitudinal bar out of two shall be restrained against buckling.

c) Maximum spacing of the transversal reinforcements is

- In nodal zone, $M_{in.}$ (0.3 h, 12Φ)
- Outside nodal zone, 0.5 h

C Lateral Bracing Walls (Webs) and Concrete Bearing Walls Requirements Relative to the Formwork

a) Minimum thickness of the walls (webs) is defined as follows:



The thickness shall always be greater than 15 cm.

b) Walls (webs) are considered as lateral bracing wall (web) if they satisfy the conditions: $\ell \geq h$

where ℓ = length of wall (web)
h = height of story

Requirements Relative to Reinforcement

- **Continuous Part of Solid Walls (Webs) and of Piers**

a) Continuous parts have to be reinforced by grids of bars in two layers, whose minimum percentage is given below.

$$\left.\begin{array}{ll} \text{For } \tau_b \leq 0.25\,\sigma'_{28} & 0.15\% \\ \text{For } 0.025\,\sigma'_{28} \leq \tau_b \leq 0.12\,\sigma'_{28} & 0.25\% \end{array}\right\} \text{ in each direction}$$

b) When part of the wall (web) undergoes tension under the vertical and horizontal loads, the reinforcement shall withstand all the tensile forces.

The minimum percentage of vertical reinforcement throughout the zone under tension is 0.5 percent.

It is possible to concentrate tensile reinforcement at the ends of the wall (web) or of the pier. The total section of vertical reinforcement of the zone under tension shall be at least 0.5 percent of the horizontal section of the concrete under tension.

c) The spacing of the horizontal and vertical bars shall be less than the smaller of the two following values

$$s \leq 1.5\,b \qquad \text{or} \qquad s \leq 30\,\text{cm}$$

d) The two layers of reinforcement shall be linked together with at least four cross ties per square meter. In each layer, the horizontal bars should be placed outside.

- **Ends of Solid Walls or Piers**

Edges of solid walls or of piers must have vertical reinforcements in the shape of little columns whose dimensions are b x 1.5b.

Reinforcement requirements of these little edge columns shall be at least those of the facade columns of the self-stable (self-lateral resistant) frameworks in zone I.

Vertical reinforcements of these little columns can be used to withstand the tension forces in the zone of the wall under tension. In that case, the overlapping lengths shall be 70 diameters (70Φ).

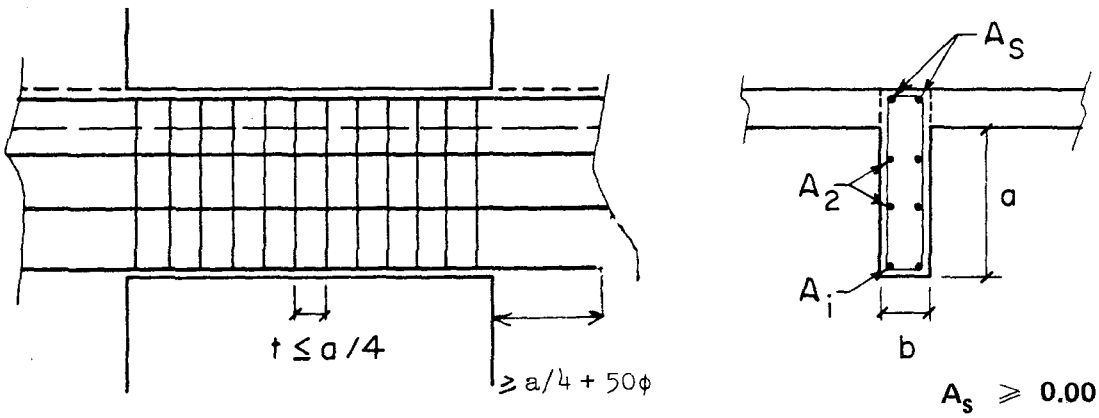
- **Horizontal Joints Where Pouring of Concrete Is Restarted**

Along these joints, vertical stitching reinforcements should be provided, uniformly distributed in two layers, that can withstand all shear forces with $\sigma_a = \sigma_{en}$ and an anchorage length equal at least to 50 diameters (50Φ) in the walls.

In the absence of computations, one can provide outright a stitching reinforcement section of 0.5b per linear meter.

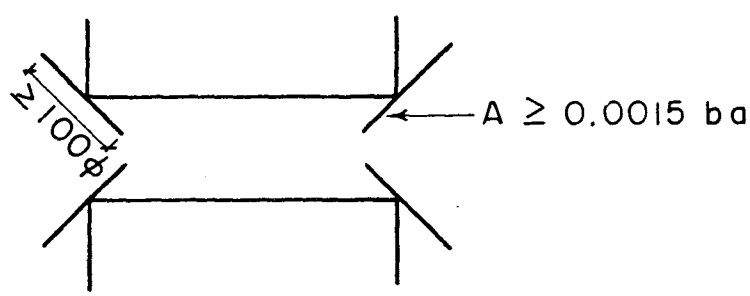
- **Lintels**

a) Reinforcements shall be designed and anchored in the piers, as shown in the following sketch:



$A_s \geqslant 0.0015\ ba$
$A_i \geqslant 0.0015\ ba$
$A_2 \geqslant 0.002\ ba$
$A_t \geqslant 0.0025\ bt$

b) For $\tau \geqslant 0.06\ \sigma_{28}$, supplementary reinforcements have to be provided in the corners, as shown in the following sketch:



2.2 C Beam-Column Frameworks Braced by Walls (Webs)

2.3 C

- **Bracing Walls**

All the above requirements must be followed.

- **Columns-Beams**

All requirements relative to the self-stable (self-lateral resistant) framework in zone I, as defined above, must be followed.

2.5 C Steel Works

Assemblages of steel must be designed to withstand the ultimate strengths of the assembled sections.

APPENDIX E
TEAM PERSONNEL

Name	Affiliation	Specialty
Vitelmo V. Bertero (Investigating Team Leader)	Professor, Civil Engineering University of California Berkeley, California	Structures
Amar Chaker	Organisme de Contrôle Technique de la Construction (CTC) d'Algérie Algiers, Algeria	Structures
Wayne G. Clough	Department of Civil Engineering Stanford University (at Virginia Polytechnic Institute since August 1982)	Geotechnical
Lloyd S. Cluff	Vice President Woodward-Clyde Consultants San Francisco, California	Geology
Nicholas F. Forell	President Forell/Elsesser Engineers, Inc. San Francisco, California	Structures
Peter Gergely	Professor, Structural Engineering Cornell University Ithaca, New York	Structures
H. Max Irvine	Massachusetts Institute of Technology Cambridge, Massachusettes	Geotechnical
Christian P. Mortgat	TERA Corporation Berkeley, California	Structures
Braham Rebzani	Organisme de Contrôle Technique de la Construction (CTC) d'Algérie Algiers, Algeria	Structures
Thomas Saarinen	Professor, Geography University of Arizona Tucson, Arizona	Socio-economics
Ahmed Sendjani	Organisme de Contrôle Technique de la Construction (CTC) d'Algérie Algiers, Algeria	Structures

Name	Affiliation	Specialty
Haresh C. Shah (Reconnaissance Team Leader)	Professor, Civil Engineering Stanford University Stanford, California	Structures, risk
Frank H. Swan, III	Senior Project Geologist Woodward-Clyde Consultants San Francisco, California	Geology
Henry T. Taylor	Associate Harding-Lawson & Associates San Francisco, California	Geotechnical
Farouk Tebbal	Director General Organisme de Contrôle Technique de la Construction (CTC) d'Algérie Algiers, Algeria	Structures
Marcy L. Wang	Associate Professor, Architecture University of California Berkeley, California	Architecture
Thomas D. Wosser	President H.J. Degenkolb Associates San Francisco, California	Structures

APPENDIX F
GLOSSARY

AS 1955	French provisional recommendations for building in seismic regions
CND	Committee on Natural Disasters, National Research Council
CTC	Organisme de Contrôle Technique de la Construction d'Algérie
daira	Small regional administrative entity (six in wilaya of El-Asnam)
EERI	Earthquake Engineering Research Institute
IZIIS	International Institute of Seismology and Earthquake Engineering, Skopje, Yugoslavia
pilotis	Columns
vide sanitaire	Sanitary void — crawl space between ground and first floor, supported by stubby columns
wilaya	Regional administrative entity, similar to a province