THE EARTHQUAKE OF 28 JULY 1957 IN MEXICO CITY

by Emilio Rosenblueth*

Introduction

The earthquake that originated near the Guerrero coast on 28 July 1957 demands attention: It caused great damage in two widely separated cities, producing relatively insignificant effects elsewhere. Damage in the City of Mexico far exceeded that of all previous ground motions, and the City of Chilpancingo was all but destroyed.

Some small towns in Guerrero also suffered important damage, particularly San Marcos.

Earthquake history of Mexico City prior to this motion is summarized in Fig. 1. The catalogue** on which the figure is based is incomplete for earthquakes of intensities I-IV. Gaps were assumed to follow the distribution for the years in which information was available. The straight line in Fig. 1 fits the data closely. (It should not be extrapolated indefinitely, for it would predict infinite seismic energy per unit time.)

Most of the important earthquakes felt in Mexico City originate near the states of Guerrero and Oaxaca, along the Pacific coast. Indeed, according to Gutenberg and Richter, "For shallow shocks, the seismicity of the Pacific coast of central Mexico is the highest in the western hemisphere."(1) Other memorable earthquakes of this century have been:

Date	Modified Mercalli Intensity	Epicentral Distance(2)
7 June 1911	VIII	474 km
23 December 1937	VI	343 km
15 April 1941	VII	452 km
22 February 1943	VII	388 km

The Focus

The time in Mexico City was 2:40:51 a.m.(3) Tacubaya reports the epicenter at 16°21'N, 99°13'W, 358 km south of the city (Fig. 2).(2) The underwater fault in question belongs to the Middle American Trench and may be regarded as a continuation of the San Andreas fault.(4)

The epicenter is designated as No. 16 in the seismic chart of the Republic. Its neighborhood within a radius of 60 km is responsible for 277 recorded earthquakes between 1927 and May 1957.(2)

In the Gutenberg-Richter scale, Berkeley assigns this seism a magnitude of 7-1/4 to 7-1/2, Pasadena 7-1/2, and Tacubaya 7.5. The focal depth has been estimated at 25 km.(5)

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*Director, Institute of Engineering, University of Mexico.

**Kindly supplied by J. Figueroa, Head, Seism. Service, National Observatory at Tacubaya, Inst. of Geophysics, Univ. of Mexico.

Fourty-six aftershocks were registered on the day of the earthquake; 44 the next day; 8 on 30 July; 16 on 31 July; 3 on 1 August.[9] The one of greatest magnitude (6.25) took place at 14:16:21 on 4 August.

Isoseismals

Curves in Figs. 2 and 3 are based on witness accounts, appraisal of damage, and, in Fig. 3, on horizontal accelerations computed from the dimensions of brick fence walls that collapsed.[6] The method for computing horizontal accelerations is debatable but practically all the walls had the same height and thickness in compliance with Federal District regulations. This makes the procedure appropriate for comparative purposes.

The salient feature of isoseismals lies in the anomalies at Chilpancingo and Mexico City. These locally high intensities have been ascribed to the combination of long prevailing periods of the quake and local softness of the ground.[7]

A similar phenomenon is noticeable in Mexico City itself (Fig. 3). J. Merino y Coronado[6] has remarked that curves of maximum intensity coincide roughly with those of maximum rate of settlement of one foot per year in response to extensive pumping of underground water;[8] zones subsiding at a faster rate were associated with softer ground (higher water content) and denser population. Precisely the same factors led to greater earthquake damage.

Damage in Mexico City

The favorable time of night at which the shock struck accounts for the relatively small number of lives lost. Estimates[2,9] placed the death toll at 54 and material losses at between 25 and 160 million dollars.

The writer's experience with estimates of material losses in practically all the buildings for whose repairs he was retained as consultant was that the actual cost of repairs ranges between 120 and 450 percent of the original estimate.

Before enumerating damages in Mexico City one should describe subsoil conditions. Zones of the Valley of Mexico are well defined in terms of subsoil conditions. The following description as well as Figs. 4 and 5 were taken from Ref. 10. The figures are slightly modified in the light of more recent information.[8]

With basis on information from over 400 borings, there are three distinct zones (Figs. 4 and 5). Essentially the same division was proposed as early as 1953:[12]

Zone A is located on the old Texcoco lake bed. In it one finds from 32 to more than 700 ft of highly compressible volcanic clay strata. Their void ratios range from 1 to 16. Their average water content is 300 percent. The mean compressive strength of upper layers is 0.6 kg/cm^2 (9 psi), generally increasing with depth. Near the surface one finds 7-20 ft of clayey and silty sands of relatively low plasticity.

At times one also encounters artificial fills.

Beneath the clay formation the subsoil consists of dense silt and medium to hard clay strata alternating with deposits of sand and gravel. This succession exceeds 1700 ft in thickness.

Base rock lies 3300 ft below the surface and in parts of the city it is even deeper.(13)

<u>Zone C</u> includes the hilly southern and western slopes of the Sierra. The subsoil consists of volcanic tuffs, conglomerates, cemented or dense sands, and similar relatively incompressible material, except for a few areas covered by mildly compressible silty or clayey soils. The unconfined compressive strength usually exceeds 5 kg/cm^2 (70 psi).

<u>Zone B</u> constitutes the transition zone between A and C. Overlying the dense deposits mentiones, one finds sandy clays and silty sands of low to medium compressibility and occasional layers of highly plastic soft clay with an accumulated thickness smaller than 32 ft.

Damage to buildings can be described roughly as follows.(10,11)

- About 1500 cases of damage were reported to the Building Department. Of these nearly half were buildings that underwent very minor damage (hairline cracks in plaster) or none at all. Probably an additional 200-300 buildings suffered appreciable damage but were not reported. This brings the total of damaged buildings to about 1000. Less than 0.3 percent were located in the hilly zone, 4 percent in the transition zone, and the rest in the lake zone (Fig. 4; compare with Fig. 3). The percentages of number of constructions in the hilly, transition, and lake zones were approximately 11, 16, and 73 in 1957 (excluding shanties). Accordingly relative damages were in the ratios 1.4:18:100.

- Few buildings erected before 1930 suffered damage although in many instances these stocky bearing-wall constructions had been badly weakened by cracks due to differential settlements. The majority of the modern buildings in the lake zone suffered at least plaster cracks or glass breakage and accounted for most cases of severe damage.

- Collapses included one modern warehouse, two modern apartment and office buildings, two fairly old bearing-wall apartment buildings (one of them about to be demolished), two modern private homes (one of them almost completed), one cinema built about 15 years earlier, and part of one market place (about to be inaugurated). One theater collapsed on 29 July during an aftershock of magnitude 6.2. Condemned by the authorities were six private homes, four office or apartment buildings, one gasoline station, and a one-story commercial building. At least two buildings were condemned by their owners. All buildings that collapsed or were condemned lay in the lake zone.

- No damage to elevated tanks, silos, or chimneys was reported.

Correlation between damage and ground compressibility is well established for this earthquake. However, it is likely that earthquakes characterized by shorter period waves, probably originating at closer foci, would cause relatively heavier damage in the transition zone. Those with even shorter waves would more seriously damage constructions on the hardest ground, particularly buildings of the most rigid type. Available literature(14) substantiates this stand.

Also firmly established is the correlation of type of construction with damage. The fact that old stocky wall-bearing buildings fared far better than more modern structures has been ascribed to the former's greater energy absorbing capacity.(15) The greater energy reserve of the massive edificies is unquestionable when comparing them with framed structures whose connections and apparently non-structural details do not display the requisite care. It is debatable when comparing with framed structures whose design and execution meet modern concepts of aseismic construction. Buildings in this category escaped the temblor unscathed.(16)

A second plausible hypothesis has been advanced to account for the lesser damage suffered by the older, more massive structures.(17) There is little doubt but that the soft mantles of lacustrine clay dampened out the shorter-period waves, magnifying those whose periods lay close to the principal natural periods of the formation. Similar phenomena have been amply verified qualitatively(18) and quantitatively(19) in other regions of the globe. Hence structures with short natural periods would be expected to suffer less damage than the more flexible modern office or apartment buildings of moderate height. One may controvert the argument by citing the collapse of one modern single-story house and the excellent performance of some tall colonial constructions. But one need go no farther than to notice that the static lateral-load resistance of the house was practically nil(20) whereas that of the colonial buildings is truly large.

Careful weighing of the evidence favors both assumptions: the older buildings fared better because of their sizeable energy reserve and because of their usually shorter natural periods.

Attempts at establishing other correlations have led to debatable conclusions. Thus, the distribution of severely damaged buildings as a function of their number of stories gives a peak at 14 stories.(21) One may conclude that the distribution reflects either the prevalence of a certain ground period or that it is a consequence of tendencies in construction practices. Even less convincing have been efforts to prove that steel frames perform in a way superior to those of reinforced concrete or vice-versa, or that structures on certain types of piles are practically earthquake-proof.

<u>Principal Types of Damage</u>

Following the earthquake there sprang two schools of thought. Each was ardently defended. Some stated that the earthquake's violence was far above what one should reasonably design for. Others maintained that practically no damage occurred except in very poorly designed or built structures.[22]

It is the writer's contention that the earthquake intensity in some areas of the city was considerably above the degree VII of Mercalli's Modified scale it was officially assigned. The official estimate seems reasonable for the transition zone. But in that light, at least VIII should be reserved for the lake zone. Yet, as shown in Fig. 1, earthquakes of this same intensity can be expected to strike the city every 40 years on the average. One should demand that private homes and office and apartment buildings, not to mention more important constructions, be designed to withstand, with no more than minor damage, earthquakes of even slightly greater intensity.

It is a matter of definition that a structure which fails was not properly designed or built. Yet, to honor truth and fairness, one should say that there was lack of experience. Excessive confidence had met with the reward of economy and expediency. And no sobering event had as yet proved that it was dangerous to proceed thus.

The main causes of failure are listed below in decreasing order of importance. Some such as resonance phenomena are peculiar to very soft ground; most are observed during an earthquake on every type of soil. In this list the term failure is loosely applied.

Not listed are blatant mistakes and omissions.

When specific references are not given, cases reported are based on Refs. 11 and 20.

Disregard of Relative Rigidities. It was general practice in Mexico, and still is in many parts of the world, to assume that only the frame of a structure resists lateral forces, and to neglect the contribution of masonry walls and partitions. This is but the commonest version of the practice that neglects the contribution of all structural elements whose explicit consideration would lead to complication or doubt. If the frame is designed for, say, one fortieth of gravity, it may require deformations many times greater than the drift at which masonry walls and partitions crack.[23] To the high rigidity of these brittle elements and neglect to consider them in design must be ascribed their extensive cracking in many a building (Fig. 6). If the strength of a masonry element is found insufficient to resist the stresses that its rigidity calls for, it should either be reinforced or isolated from the rest of the structure.

Neglect of the stresses taken by asymmetrically disposed masonry walls led to high torsions in several structures. This was common in corner buildings, of which two sides were usually covered completely with brick and the other two with glass (in many of these there was extensive glass breakage). In some instances the fact that damage was greatest at and near the entire perimeter of a story indicated that its center of rotational deformations had been inside the building, a condition only possible if the dynamic torque had greatly exceeded that computed statically.

One building about to be inaugurated suffered such severe damage from torsion that the cost of repairs was not far from that of the entire building. Torsion came primarily from a solid masonry wall not considered in design. It was increased because of a mezzanine, covering a small part of the ground floor, that was not included in the original design but put in as an afterthought. The mezzanine halved the length of several first-story columns, raising the rigidity of frames near the brick wall. Additional torsional oscillations induced by the mezzanine were apparently propagated several stories up the building.

In one slender eleven-story building the entire lateral load in one direction was supposed to be taken by a single diagonally braced bay. Its rigidity in shear alone was indeed much greater than that of the rest of frames in that direction, but it became almost negligible when considering axial strains in columns due to overturning moment. The result was excessive drift, considerable breakage of unreinforced partitions, and minor damage in the unbraced frames, which were not designed for lateral load.

Lack of Aseismic Design. Prior to 1942 there was no requirement to design buildings against earthquakes. The macroseism of 1941 prompted not very severe requirements in the 1942 code. Buildings up to four stories or 16 m (52.5 ft) tall still did not have to be designed against earthquakes. Failure of some houses and small buildings can be ascribed to this cause, as well as severe damage in some taller buildings erected before 1942. Figure 7 shows a typical girder-to-column connection in a 16-story steel-frame apartment building constructed in 1942. The building was poorly designed even for vertical loads. It was damaged by the earthquake of 1943 and hastily repaired. The reason it did not collapse is that it possessed a dense array of hollow tile partitions that had not been considered explicitly in design.

Insufficiency of Reinforcement Away from the Supports. Girders of some reinforced concrete structures were checked only at the supports. The small seismic coefficients required by the building code,[24] coupled with a 33 percent increase in allowable stresses over those for static load often resulted in designs that required no additional steel to resist lateral load. Since most upper-layer bars were cut or bent a short distance from the support the girders were left unable to resist any appreciable bending moment of unfavorable sign throughout most of their span. This explains collapse of at least one office building whose columns were capable of resisting many times the seismic coefficient called for by the code. It also explains the appearance of numerous diagonal tension cracks in girders whose web reinforcement was apparently adequate. It is worth noting that even proper application of small seismic coefficients partially offset by low working stresses led and will often lead to similar cracking and unbalance between unnecessarily strong and dangerously weak sections and members.

Diagonal tension cracks constituted the most commonly observed type of damage in reinforced concrete members (Fig. 8). Insufficient web reinforcement was mostly responsible for the situation. Even if the causes for overstress are in each instance to be found elsewhere, the earthquake's selectivity for diagonal tension cracks leaves little doubt but that 3 percent of f_c' (plus 33 percent) is not conservative in comparison with other allowable stresses.

Reinforcing and strengthening of girders diagonally cracked during the earthquake was often attained through the use of external prestressed stirrups (Fig. 9). These were later covered with high strength grout having an admixture to prevent shrinkage.

Pounding. City regulations did not require separation between neighboring structures. Hence buildings were usually erected next to each other and severe pounding resulted in many instances (Fig. 10). Consequences were structurally important only when the floors of adjoining structures were situated at different elevations, for in that case the impact of a slab sometimes caused fracture of a column. It is likely that the phenomenon was intensified by rocking of foundations on the soft clay.

Previous Differential Settlement. Several old cracks due to differential settlement, which had been slightly repaired, reopened during the earthquake. Collapse of one building was ascribed mainly to the weakening effects of previous differential settlement. Yet in other buildings such weakening was surprisingly ineffective considering the magnitude of differential settlements. On the other hand, collapse of the cinema previously mentioned was due to the fact that differential settlements had tended to tilt the longitudinal bearing walls away from each other leaving little support for the transverse steel roof trusses. Fig. 11 shows walls and columns still standing and discloses failure of anchor bolts of the roof trusses.

Resonance. The word is used here to denote the coincidence of the first natural period of a structure with the local prevailing ground period or pronounced peak in the acceleration spectrum. Without recourse to this concept it is difficult to explain the fall of the Angel of Liberty and the collapse of a large portion of a new market place occupying four square blocks. The Angel had successfully withstood all ground motion since its inauguration in 1911. Revision of its static lateral-load capacity indicated it could easily have resisted 0.1 g.[11] Yet in 1957 the Angel fell to the ground and the stone facing of the column was severely disrupted.

The structure for the market place (part of which is shown schematically in Fig. 12)was designed by ultimate-load theory to resist 0.1 g at failure. It was excellently built. Failure occurred by fracture of the column bases. Damage was more severe in parts built on virgin ground (former streets) than on preconsolidated soil and more in the recently poured concrete than in that which was a few weeks or months old. Foundation compliance under the relatively small footings may have added in bringing about the resonant condition.

Outside of these structures, both of which may be regarded as inverted pendulums with low energy-absorbing capacity, there were no clear-cut instances of resonance. The fundamental period in both cases may have been between 1.5 and 2.0 sec. Probably the prevailing period of the soft soil in the valley was within this range (some favor 0.5-2.5 sec). The conjecture is substantiated by the prevalence of the second natural mode (period = 1.54 sec[25] in the shear distribution measured in the Latino Americana Tower during this as well as other earthquakes.[25,26] There are witness accounts favoring much shorter periods for the transition zone. Other inverted-pendulum structures, such as umbrella shells and elevated tanks, did not suffer damage although their lateral-load capacity was in many cases considerably smaller than 0.1 g.

Excessive Sway. Panic was exceptionally widespread in the lake zone. Reasons may be found in the long duration of the strong-motion phases of the quake (90 sec in Tacubaya) and in the length of prevailing ground periods. Witnesses claim that the type of motion they endured caused far greater inconvenience than earthquakes of comparable intensity in other cities. Quite possibly ground accelerations in the lake zone were smaller than during earthquakes of the same intensity on hard ground but ground displacements probably had greater amplitude due to the length of prevailing periods. Evidence of excessive sway may also be found in the shift of objects inside buildings.

In comparable buildings which withstood the earthquake undamaged and had been designed for the same seismic coefficient, drift was usually larger in those with steel frame than in those with concrete frame. It was larger in those with riveted than in those with welded connections.

The extent to which minimization of panic should influence the design of an office building is debatable. Certainly it should be of prime concern in the design of a large theater or shcool building.

Whip Effect. There were cases of failure of the uppermost story (Fig. 13). In some buildings the phenomenon should be ascribed to faulty anchorage of pent-house steel columns on a concrete structure. In others, perhaps, to a sudden decrease in rigidity or to a design based on uniform horizontal acceleration. Its manifestations were not sufficiently frequent to justify a more drastic increase in design accelerations with height than the linear variation which the code of San Francisco specifies.

Foundation Failures. The sidewalks around some buildings on piles were shaken and fractured. The plate covers of some construction joints fell to the ground leaving a wide gap plainly visible (Fig. 14). These phenomena led observers to conclude that the earthquake had caused several foundation failures.[27] Actually not one single case of foundation failure was confirmed, either in the soil or in the foundation itself. Phenomena observed had either taken place before but had gone unnoticed or else were very superficial and unimportant manifestations of the motion. All settlement curves for buildings in the lake zone known to the author and covering the date of the earthquake are similar to those in Fig. 15. In all cases they show a small sudden drop but after a short time attain the normal settlement curve. Reasons for this behavior are not clear. In some cases, particularly of buildings on point-bearing piles, there are indications that slight tilting or convex bowl-type settlements (due to negative skin friction) which would have taken place without the earthquake were merely anticipated by it.

Behavior of patented control cells in some piles affords an exception that may be classed as foundation failure. The cells are wood cubes whose stress-strain curve in compression is relatively flat although quite sensitive to the rate of loading.[28] The cells are placed between steel plates as shown in Fig. 16. The club sandwich thus formed transmits pile loads to a pair of I beams or channel sections. These in turn are held down by two bolts anchored to the foundation slab. Usually 25 cells are placed in each layer. Plastic behavior of the wood insures that the load transferred from the building to the pile (of the order of 60 metric tons (132 kips)) remains almost constant despite subsidence of the city and gradual emergence of the piles. The bolts allow occasional adjustments and cutting off of short lengths of pile.

This system is ideal for carrying vertical loads but is not very stable under earthquakes, for the dynamic increment in load alters suddenly the distribution of loads taken between piles and soil. This will cause sudden tilting of the bolts when they were not strictly centered and plumb at the outset of the motion. Several such instances occurred in July 1957, bringing about local settlements and tilting of some buildings. The structures were easily straightened, however, and the system has since been improved by the addition of concrete pedestals that insure its stability.

Overturning. Not one case of actual overturning was observed. However, failure of some concrete columns took place under a combination of shear, flexure, and change in axial load. Mere inspection does not disclose the relative importance of these stresses (Fig. 17). It would not be surprising if many cases reported as diagonal-tension failures of columns had been primarily due to overturning moment.

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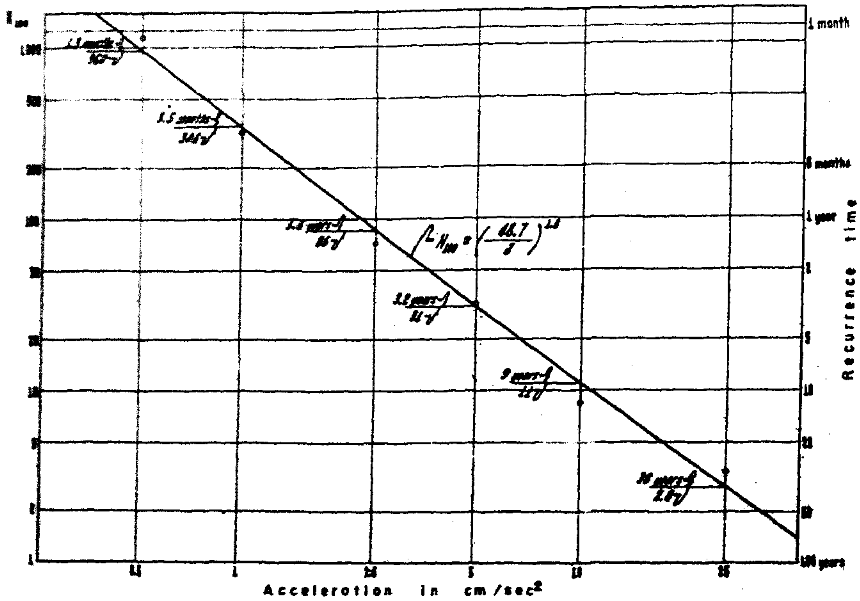

FIG. 1. RECURRENCE TIMES OF ACCELERATIONS IN TACUBAYA

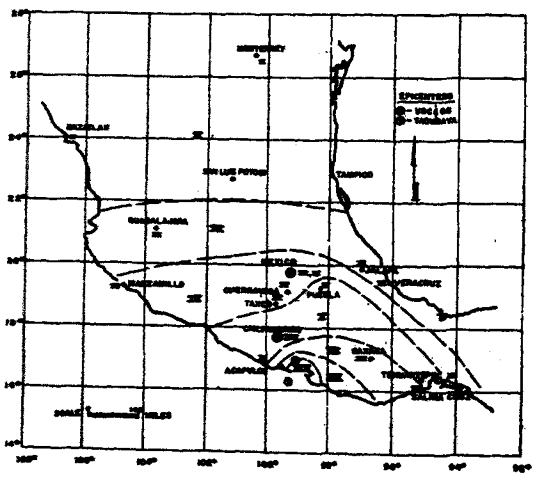

FIG. 2 ISOSEISMAL MAP, MEXICO EARTHQUAKE OF JULY 28, 1957. MODIFIED MERCALLI INTENSITY SCALE, 1931

(After Duke and Leeds)

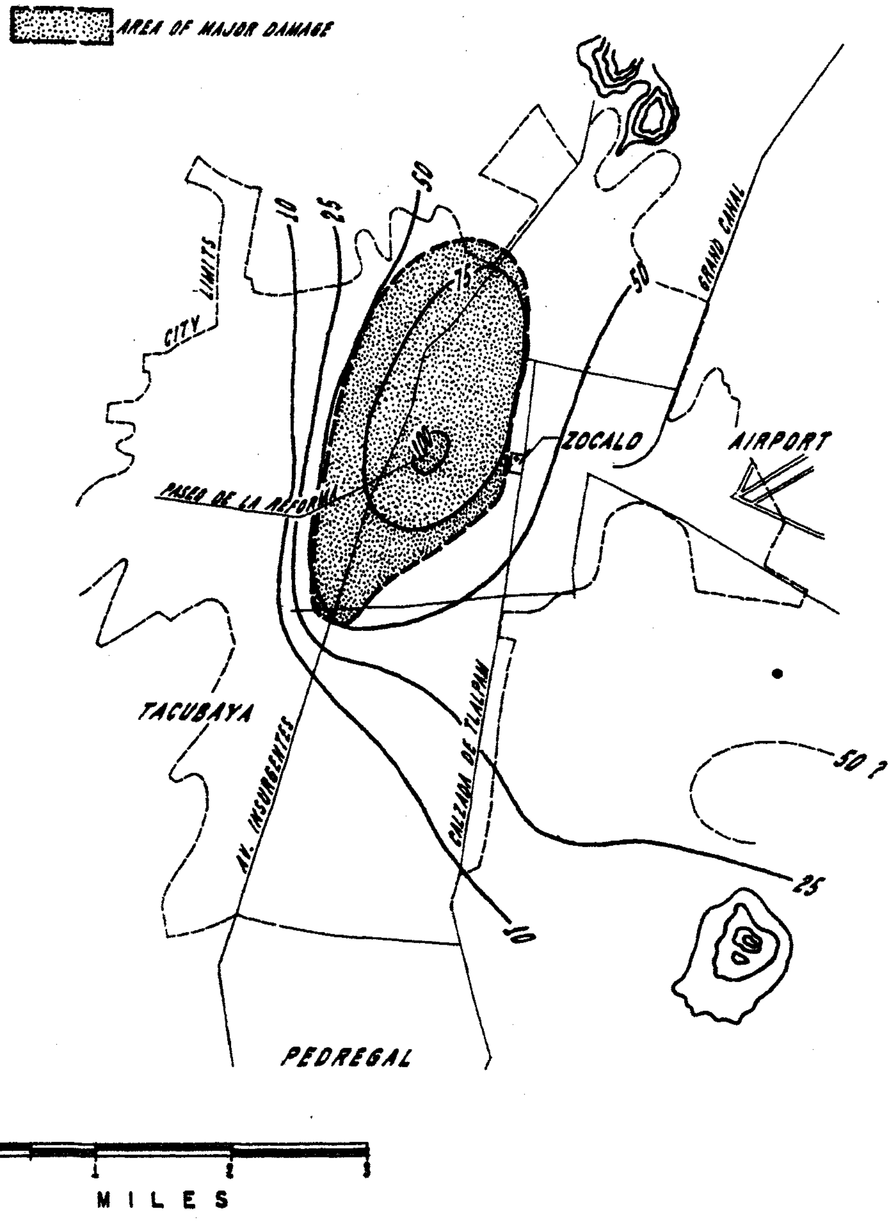

FIG. 3.- ISOSEISMAL MAP, MEXICO CITY, EARTHQUAKE OF JULY 28, 1957. ACCELERATIONS IN CENTIMETERS PER SECOND PER SECOND COMPUTED FROM DAMAGE OBSERVATIONS. (AFTER MERINO Y CORONADO)

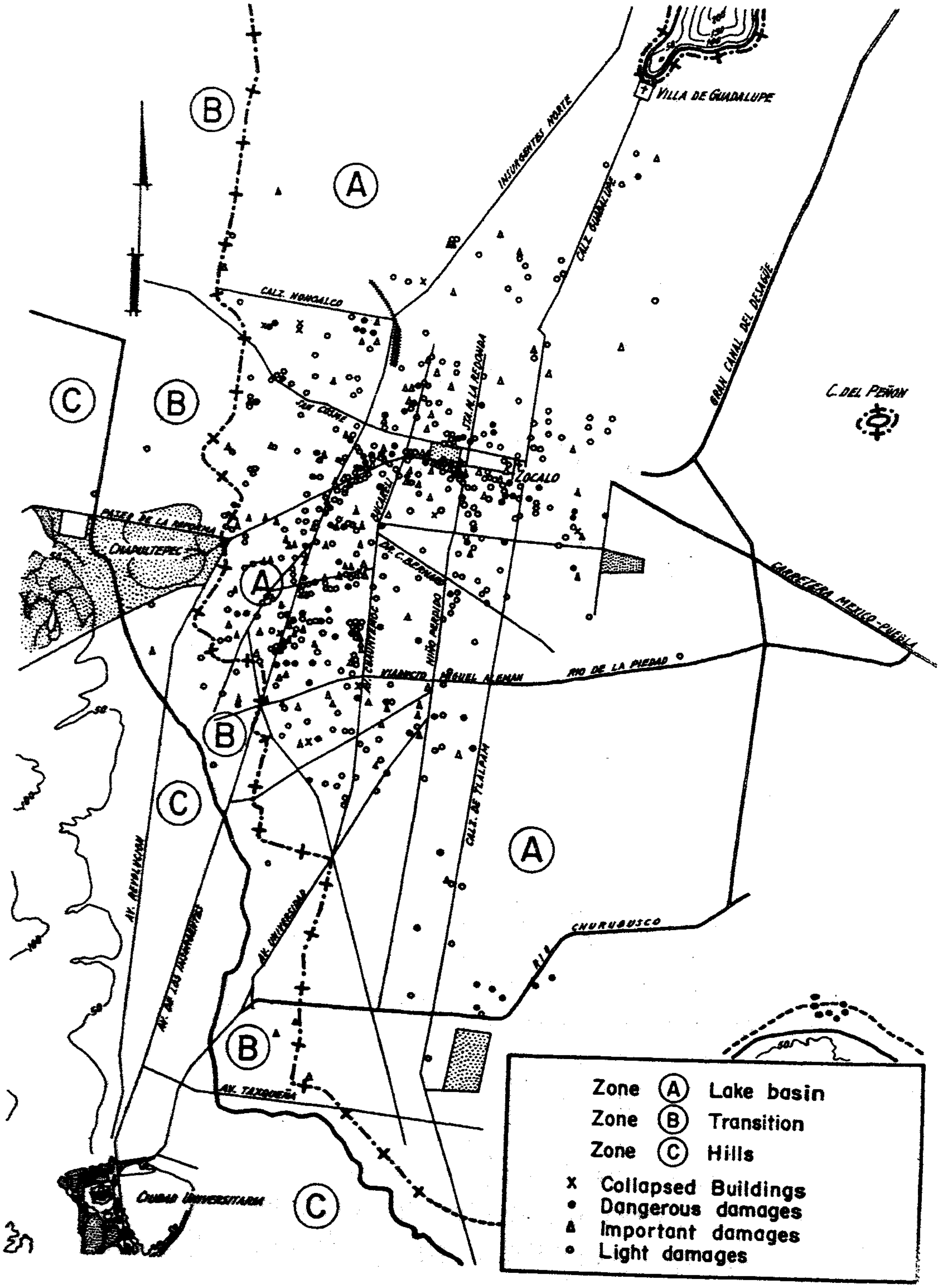

FIG. 4. MEXICO CITY'S ZONE CLASSIFICATION ATTENDING TO THE MECHANICAL CHARACTERISTICS OF THE SUBSOIL, AND DISTRIBUTION OF THE DAMAGES CAUSED BY THE EARTHQUAKE OF 28 JULY (After Marsal)

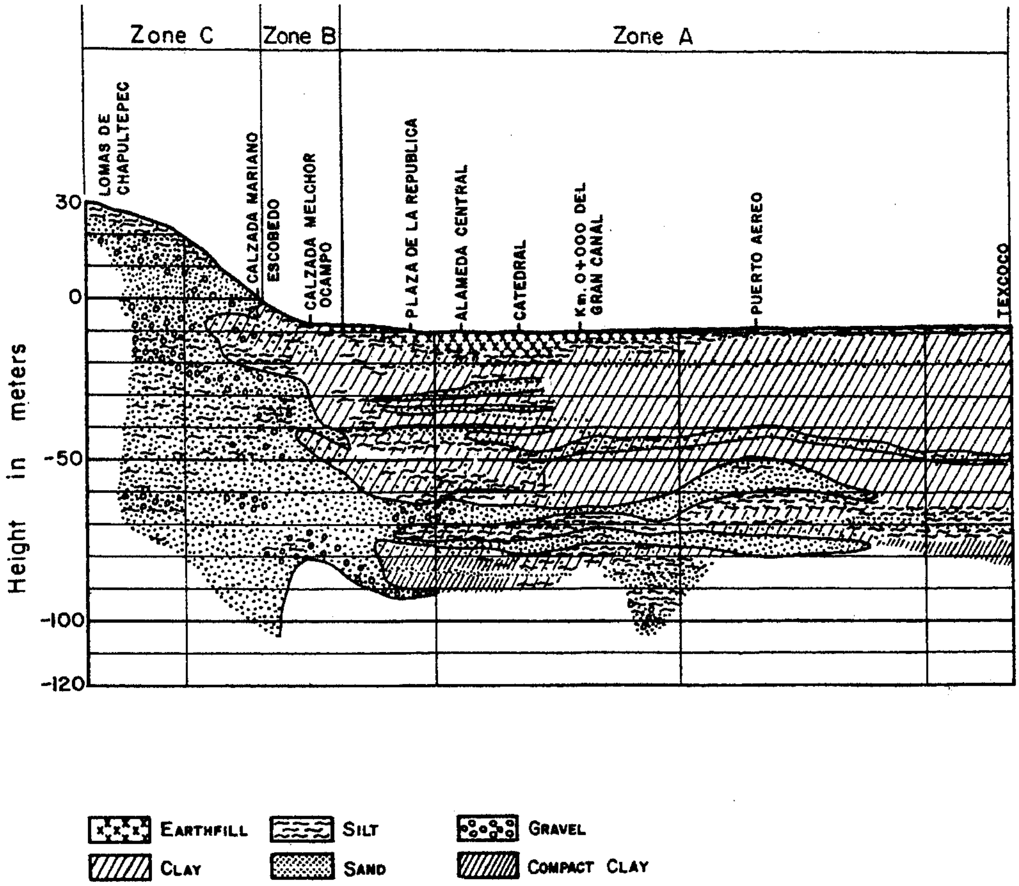

FIG. 5. SECTIONAL VIEW E-W OF THE SUBSOIL OF MEXICO CITY THROUGH THE COLUMBUS MONUMENT (After Marsal)

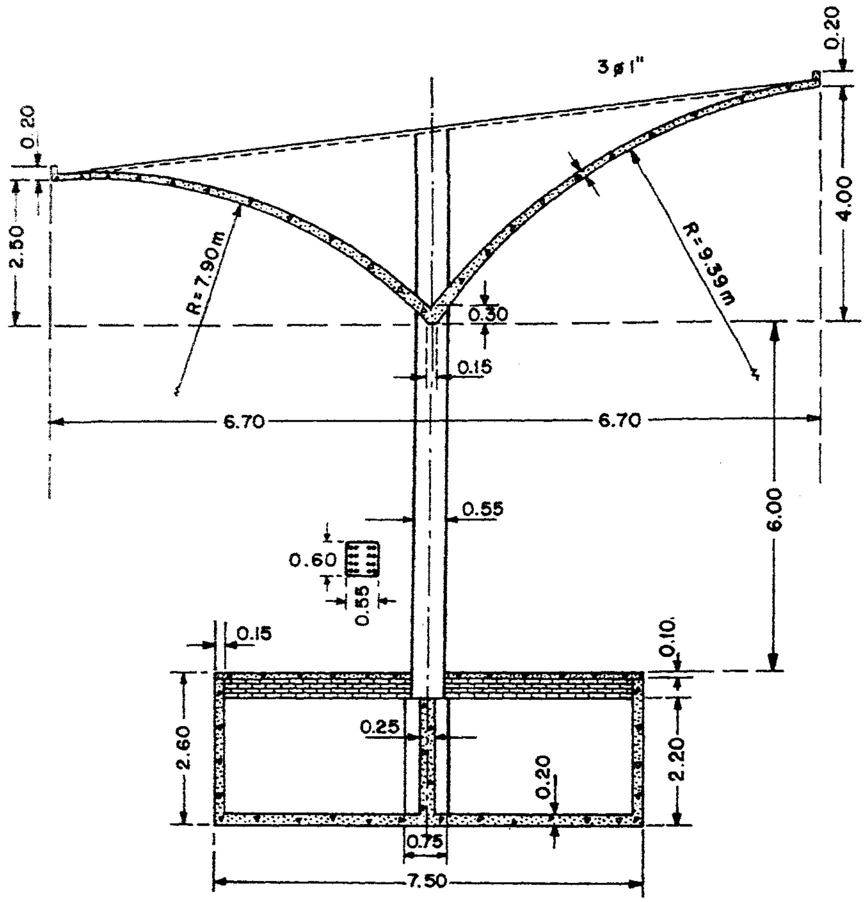

FIG. 12. SECTIONAL VIEW OF THE LATERAL SHELL IN LA MERCED MARKET

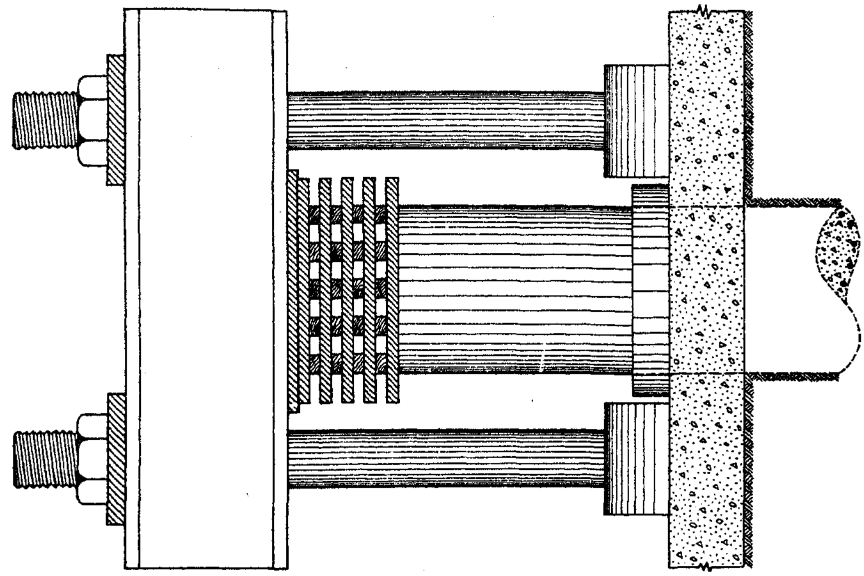

FIG. 16. PILE WITH LOAD CONTROLLING MECHANISM

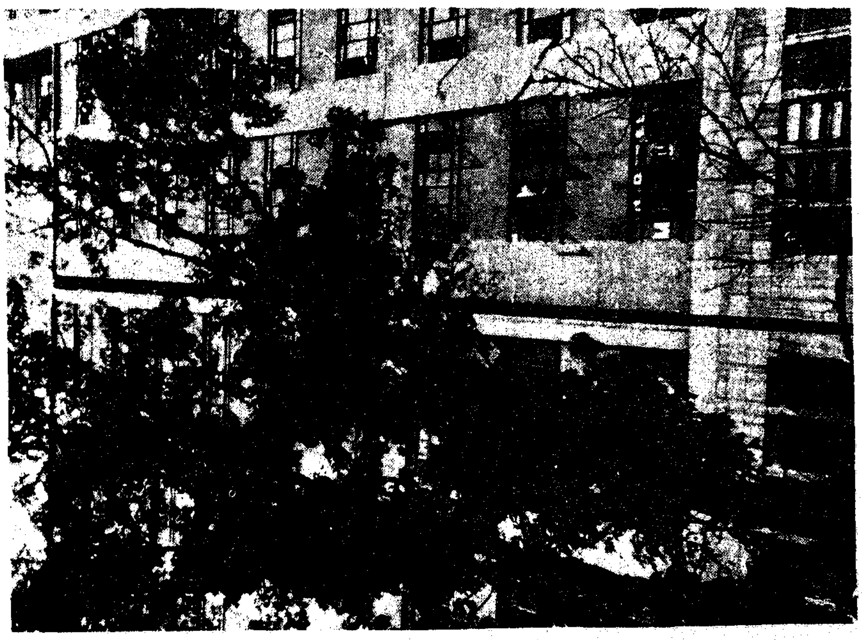

FIG. 14. SPLICE PLATE THAT FELL OFF

FIG. 13. FAILURE OF THE TOP STORY

FIG. 17. FAILED COLUMN

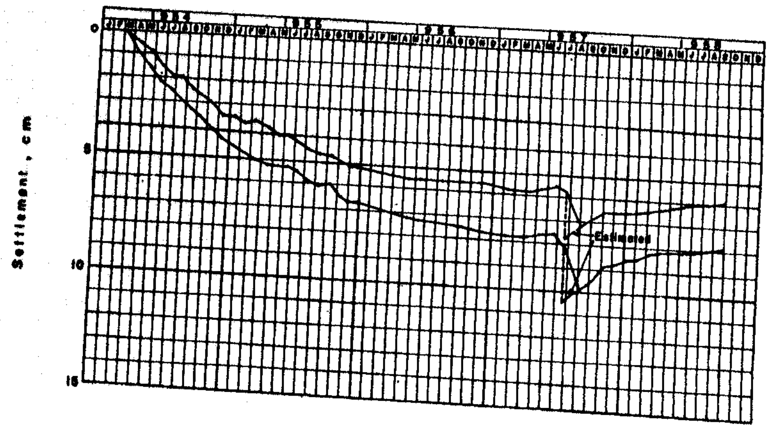

FIG. 15. TYPICAL TIME - SETTLEMENT CURVES

DISCUSSION

V. A. Murphy, New Zealand Railways, New Zealand:

Referring to the points raised by Mr. Engle on the question of ground periods and the assumed dominant periods from 1 second to 2½ seconds:-

"Could the lack of damage in the Cathedral constructed of masonry be attributed to the fact that the natural period of vibration of the Cathedral was something much less than 1 second, while the damaged buildings were probably in resonance?

E. Rosenblueth:

The writer expresses his thanks to the discussors for their comments. He agrees with Mr. Engle's observation.

In response to that of Mr. Murphy, the natural periods of Mexico's Cathedral have not been measured to the writer's knowledge. The first natural period may indeed be much smaller than one second and this may be one reason for its excellent seismic behavior. On the other hand its strength is undoubtedly very high owing to the quality of its constituent materials and care in construction; it is not unlikely that, with this strength, the Cathedral would have performed satisfactorily independently of its natural periods of vibration.

Although the natural periods of many buildings which were damaged exceeded one second, there is little doubt but that a large number of buildings which collapsed or were damaged in various degrees had their fundamental period in the range 0.3 - 1.0 second.