EARTHQUAKE EXPERIENCE IN NORTH AMERICA, 1950-1959

by

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North America experienced a number of damaging earthquakes during the decade starting with 1950. Figure 1 and Table 1 indicate 13 of the more important shocks from the engineering standpoint. This paper is a critical review of some of the information obtained and conclusions presented by various authorities as a result of their field investigations. Unpublished as well as published material has been freely drawn upon.

The subject matter is restricted to field observations of buildings and allied structures, and to observations relating to ground intensity. Supporting data are found in the Bibliography. Strong motion instrumental investigations have been largely excluded, but references are included in the Bibliography.

The functions of a field examination are: (1) To gather accurate data for use in the design of structures and, (2) For the comparison of existing theory with actual behavior and development of basic theory. The first aspect may lead to empirical rules based on considered judgment while the second may or may not find practical application for years. Since earthquakes occur without notice as to time, place, and size, it is difficult if not impossible to be adequately prepared for a field investigation, and many published reports are not as thorough as their authors might have liked.

The authors would like to acknowledge the helpful suggestions made by Mr. Donald Moran during the preparation of this paper.

Earthquake Intensity:

Intensities of past earthquakes in a given area are valuable to engineers and to building department officials in setting up or revising standards for earthquake resistive construction. The application of intensity values from isoseismal maps is becoming much more complex due to the ever increasing number of building materials, construction techniques, and design methods. The problem is typified by the definition for MM VIII which includes "Damage slight in structures (brick) built especially to withstand earthquakes" (#16). Some communities with earthquake provisions in their building codes will allow unreinforced brick masonry while others require reinforcing steel; the performance of these two types of "structures built especially to withstand earthquakes" will be quite different. Uniformity in interpreting the Modified Mercalli scale is mandatory if isoseismal maps of various earthquakes are to be compared. This uniformity can best be achieved by having these maps always prepared by one agency in each country.

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Long period ground motion, often described as a "long slow rolling motion," has selectively damaged multistory buildings but relatively few low rigid buildings. The 1952 Kern County earthquake damaged numerous structures over 5 or 6 stories in height which were located 70 to 90 miles from the epicenter (#4, pp. 399-429). The 1957 Mexico shock is another example of damage to multistory structures in Mexico City without comparable damage to low rigid structures. These long period effects need clarification on isoseismal maps.

It would appear that the Modified Mercalli scale is in need of development to meet present day conditions. Improved isoseismal maps, in conjunction with an increased number of records from strong motion instruments, is an apparent answer to an improved microregionalization of areas such as metropolitan San Francisco and Los Angeles (#25, #26). A recent classic example of the problem is the 1957 Mexico earthquake with its marked change from MM IV to MM VII over a short distance within Mexico City (#14).

Earthquake history of large areas of western United States and Canada is meager. One approach to the establishment of engineering design criteria in these areas is the correlation which has been developed between Modified Mercalli intensity and the earthquake's Richter magnitude (#7, p. 353):

Magnitude	2	3	4	5	6	7	8
Maximum Intensity	I-II	III	V	VI-VII	VII-VIII	IX-X	XI

The foregoing is a rough correlation "For ordinary ground conditions in metropolitan centers in California," and its use elsewhere must be with caution. Examples of some limitations are:

1. Effect of ground. The best recent example was the 1957 Mexico earthquake at Mexico City (#13); see also Figure 2.

2. Variations of the earthquake's focal depth. While good examples with respect to damage are lacking, it is reasonable to expect intensity variations as a function of focal depth. The depth of focus probably influences the energy release per unit surface area.

3. Principal fault motion. Coastal California faults have had predominately strike-slip movements as witnessed by the classic 1906 San Francisco shock. But historic faulting in Montana and Nevada have had predominately vertical movements. One explanation for the relatively little damage in the 1959 Hebgen Lake, Montana, earthquake is based on the observed vertical faulting; vertical motion from vertical faulting would find the usual "non-earthquake resistive" building quite adequate for heavy vertical overloads.

4. Duration of strong shaking. This "duration" may be the result of one shock or the cumulative effects of several shocks. The 1952 Bakersfield earthquake found many structures which had suffered slight cracking or loosening in the principal shock which had occurred a month earlier. The apparent intensity of the aftershock was greater as the result of cumulative effects.

   It is possible to have the strains in the earth's crust released so slowly that damage will occur without a noticeable earthquake. A curious case exists on the San Andreas fault where a building across the fault has been progressively torn apart at a rate of about $\frac{1}{2}$" per year (#18). An example of the other extreme is the March 18, 1957, earthquake at Port Hueneme, California, which was essentially a single pulse with all its energy concentrated in the pulse (#19).

Ground Effects:

Damage may be considered from three standpoints with respect to the ground beneath a structure:

1. Surface faulting. This appears to be more common than previously supposed (#28); eight recognizable instances have been recorded from 1950 through 1959. We question the propriety of developing population centers in the San Andreas, Hayward(s), and other fault zones as is being currently done.

   Structures in or across a fault zone which undergoes surface breakage can usually expect to have severe damage as did the railroad tunnels in the 1952 Kern County shocks (#4, pp. 283-292) or the hollow concrete block structures on Culligan's Blarneystone ranch in the 1959 Hebgen Lake earthquake.

2. Surface effects (a. slides, b. lurching, and c. slumping).

   a. Slides, Numerous small slides have occurred in all major earthquakes. The aftershocks of the 1952 Kern County earthquake caused spectacular dust clouds arising from landslides on Bear Mountain (#4, pp. 295-296; #5, pp. 6-7). The 1958 Lituya Bay, Alaska, shock resulted in a rockslide which in turn was the primary cause of an enormous water wave reaching a maximum height of 1700 feet (#21). This water wave diminished in height as it swept towards the mouth of Lituya Bay, stripping the forest from the shore. A slide accompanying the 1959 Hebgen Lake shock has been tentatively estimated to contain between 35,000,000 to 50,000,000 cubic yards of principally rock. This slide dammed the Madison River and created a lake. Loss of life (28 estimated) in the 1959 Hebgen Lake earthquake was unusual in that all deaths were the result of landslide and related effects rather than building collapses.

      The potential slide hazard warrants more consideration than has been given to it. Recent earthquakes have usually occurred in dry weather when the slide hazard was minimum, and the sliding that took place was in largely uninhabited regions. A number of large metropolitan areas are expanding into hillside regions where landslides have been noted during wet seasons in the past. A major shock during a wet season is potentially disastrous unless more stringent foundation engineering control is developed for these new areas.

   b. Lurching. Earth lurching may be defined as surface cracks due to horizontal vibratory forces (as opposed to gravity forces associated with slides). This cracking usually is found on level ground having incompetent soils. These fissures have been quite damaging to underground utilities, drainage channels, and roadways, but not particularly to buildings. Good examples of lurch cracking were seen after the 1954 Fallon - Stillwater shocks (#8).

   c. Slumping. Ground slumping along with possible lurch effects appears to be a reasonable explanation for intensified damage to a group of about 450 wood frame houses in the 1957 San Francisco shock (#12). Construction and foundation conditions were essentially the same as those in surrounding areas which received considerably less damage. The principal damage area was on a high bluff (up to about 550 feet above the Pacific Ocean), and afterward the compression effects in sidewalks and underground piping indicated ground shortening, which in turn is reasonable if the entire bluff slumped.

3. Vibratory effects with respect to foundation material. In general, vibratory effects were more severe on poor ground than on very firm ground or rock. The one outstanding example of this was the 1957 Mexico shock in Mexico City.

Other earthquakes were less conclusive. While wide variations in foundation conditions exist at Eureka (California), the building damage pattern in the 1954 shock was confused due to the pre-earthquake differential settlement damage. It is a common error for untrained investigators to confuse pre-earthquake damage (as cracks from thermal stresses, shrinkage, differential settlements, etc.) with those caused by the shock.

There have been pockets of damag often located miles away from the instrumental epicenter, which have not been fully explained. Damage to the wood frame dwellings in the Interlaken District of Watsonville (California) in the April 25, 1954, shock is the best of several such examples; nearby dwellings of apparently poorer construction suffered significantly less damage.

Energy transferred from the ground to a structure may be dissipated by the structure through some form of internal damping, or the energy may be returned to the earth as evidenced, for example, by the building's rocking. Undoubtedly both effects are present to some degree. Known instances of building rocking through measurable arcs have been few, possibly because the evidence is not prominent and is easily lost. Building rocking requires vertical movement of the foundations, and it should not be confused with the rather common shear distorsions associated with partition and wall damage. In both cases, the roof of the building has a lateral deflection component which can result in pounding between buildings. The 8 story reinforced concrete Hotel Padre apparently rocked enough to cause a $\frac{1}{4}$" to $\frac{1}{2}$" vertical displacement at a building face in the Kern County or Bakersfield earthquake (#4, p. 353). Unpublished information in the authors' files on the 1957 Mexico earthquake in Mexico City also indicates rocking of a tall multistory building. The damage to the four 10 story Stonestown buildings in the 1957 San Francisco earthquake is best explained by building rocking, although no physical evidence could be found.

In regions where faults have been mapped in some detail, as Los Angeles and San Francisco, the question arises regarding the best location for structures. Broadly speaking, the farther that a building is away from a fault on which an earthquake occurs, the less will be the intensity. However, there is a line of reasoning, supported by some data (#17), indicating that vibration intensity adjoining a fault zone need not be significantly different from that several miles distant, provided geologic conditions are uniform. This was apparently borne out in the 1959 Hebgen Lake shock. A well designed structure is of more importance than is its distance from a fault.

Long Period Effects:

It is well known that the longer period ground motions will travel longer distances than will shorter period waves. However, that long period waves from strong distant shocks can result in significant damage and occasional collapse has not been too widely recognized. Structural damage to reservoirs and tanks at Sacramento, California, about 185 miles from the 1954 Dixie Valley - Fairview Peak earthquakes was the result of quasi-resonance between the fluids and the ground motion. Strong motion instruments recorded a ground motion having a period from 6 to 8 seconds, and a maximum double amplitude of 5.5 inches. The computed periods of the reservoirs and tanks were in the same range as the ground period (#9).

As mentioned in previous paragraphs, the 1952 Kern County earthquake resulted in numerous instances of non-structural damage to multistory fire resistive frame buildings in Los Angeles and Long Beach which were located about 70 to 90 miles from the epicenter. Generally the affected buildings were 10 to 12 stories high and had a natural period of 1 to 2 seconds, but buildings as low as 6 stories were also damaged. Again, the period range of the ground motion more nearly coincided with the periods of the taller buildings than it did with the shorter periods of the low rigid buildings. The damage to multistory structures in Long Beach was probably intensified by the fact that effective repairs were not made to many of these after the 1933 Long Beach shock (#3; #4, pp. 399-429).

Ground motion with an estimated period of 1.5 to 2.0 seconds damaged a number of multistory buildings and caused several to collapse in Mexico City in 1957. The shock's epicenter has been reported as 170 miles from Mexico City by one source (#14) and 220 miles by another (#13). One reinforced concrete shell structure collapsed and, while of low height, it had long period charactertistics due to its inverted pendulum form. Another factor in the intensified damage at Mexico City is the previously discussed poor foundation conditions. The fact that the collapsed as well as the seriously damaged multistory structures were usually quite weak or of defective design should not obscure the observation that the ground motion did not proportionally damage weak and poorly designed low rigid buildings. While theorectical studies are not part of this paper, it is still of interest to consider observed damage effects in Mexico City (#13) with Gutenberg's findings for metropolitan Los Angeles (#23).

Types of Construction - General:

Certain types of construction which were not specifically designed to resist strong earthquakes have performed considerably better than others. In recent years the development of earthquake resistive design has greatly reduced the damageability of many types of construction. The relative damageability of various types of non-earthquake resistive construction has been compiled, and non-earthquake resistive construction has been broadly related to earthquake resistive construction (#4, p. 221). This type of information is particularly useful for earthquake insurance purposes.

Wood Frame Structures.

Wood frame dwellings too often have less construction supervision than do larger structures. In some respects, dwellings are improving as a class as may be witnessed by the general acceptance of foundation anchor bolts and continuous concrete foundations. Masonry chimneys, too, are being reinforced; in the authors' entire experience only one reinforced masonry chimney was seen down (San Jose shock of September 4, 1955). No doubt other reinforced chimneys have fallen, but the over-all performance has been excellent even with questionable workmanship.

On the debit side are items such as the increasing use of structurally weak wall sheathing and dwellings with excessive wall openings. In the latter category, the relatively minor 1957 San Francisco shock caused disproportionate damage to numerous dwellings having excessive first story front wall openings (#12). In summary, as a class the small area one and two story wood frame structures such as dwellings performed well. The damage was usually under 5% of the total value and usually consisted of damage to wall and ceiling finishes and to unreinforced masonry chimneys. Severe damage to dwellings was rare and usually was associated with the building moving off its foundations.

Reinforced Concrete Structures:

Reinforced concrete structures may be subdivided into three types: 1. Rigid monolithic construction, 2. Flexible monolithic construction, and 3. Precast construction. The first two types will be considered as poured in place.

Rigid monolithic construction may be identified by the use of shear walls. This type of structure generally experiences very minor damage until the shear walls have fractured. Several examples of the performance of rigid monolithic construction may be cited:

Brock's Department Store in the 1952 Bakersfield shock: This three story plus basement reinforced concrete structure was the only one of its type to have suffered serious damage in the past decade. The design deficiencies and damage to the shear walls may be noted in Figure 3. The "L-shaped" shear wall layout also added serious torsional stresses (#4, pp. 337-343). A number of other rigid structures such as the Hotel Padre in Bakersfield performed well.

Kenai Peninsula, Alaska, earthquake of 1954: Two 14 story reinforced concrete shear wall buildings located at Anchorage about 50 miles from the epicenter suffered quite minor partition cracking (less than $\frac{1}{2}$ of 1% of value), although the building motion caused water to slop out of toilet bowls.

Stonestown Apartment Buildings in the 1957 San Francisco shock: These four 10 story apartment buildings had window glass damage which formed an interesting pattern and which could be explained by rocking of the structure and deflection of concrete walls. Several minor cracks were found in the concrete walls at unimportant locations (#12).

Rigid shear wall construction will result in lower building periods than will flexible construction, and this in turn will subject the shear wall construction to heavier earthquake forces. However, the published earthquake record for the last 10 years shows no collapses, serious damage to one poorly designed structure, and no significant damage to numerous other poorly designed shear wall structures. A word of warning appears to be in order. Many building codes are now allowing much higher unit shearing stresses in concrete shear walls than was considered satisfactory some years ago, and the factor of safety has been materially reduced. The good performance of shear wall construction has probably been due to low unit stresses.

Flexible monolithic construction is usually identified by the use of the concrete frame to resist lateral forces through moment resistance. This type of construction had spectacular damage and several collapses in the 1957 Mexico earthquake at Mexico City. Quite poor construction and/or design were obviously the major factors. To date, no detailed studies have been published on the construction details of the collapses. Non-structural damage to this class of construction has been noted in all major shocks. In general, life safety and damage control have been more difficult to achieve in this type of construction than in rigid construction, and structural damage is difficult to repair.

Precast reinforced concrete, of course, covers the flexible and rigid types previously discussed. Main concern is in the effectiveness of the joints between precast reinforced concrete units. The 1952 Kern County and the 1952 Bakersfield shocks damaged several structures:

> Both buildings of Lockheed's Bakersfield plant were damaged in the 1952 Bakersfield shock (#4, pp. 359-363). The buildings are one story, 220 feet by 280 feet and 200 feet by 320 feet in plan, and are entirely of precast reinforced concrete. The precast concrete roofs were designed to act as diaphragms with exterior walls acting as shear walls. The substitution of mastic for grout at the joints between the precast roof panels made the roof ineffective as a rigid diaphragm, and interior columns failed in bending due to excessive lateral deflections of the roof. On the other hand, the weld plates and welding which connected the precast members proved to be satisfactory.
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> Another example is the Di Giorgio Winery's performance in the 1952 Kern County earthquake (#4, pp. 385-390). The precast roof panels were not specifically designed to act as elements in a diaphragm and the panel interconnections were therefore weak. The roof supports were of varying lateral rigidities. While the earthquake damage could be attributed to relative rigidities and rotational effects, such damage as did occur was found to be to the supporting members and practically none to the precast roof.

Precast concrete has not resulted in a "collapse of a house of cards" as predicted by some, but it has brought out the need for closer engineering supervision and closer attention to design details such as joints.

Precast wall construction with other roof materials will be discussed in later paragraphs.

<u>Steel Frame Structures</u>:

Steel frame buildings have performed better than have reinforced concrete frame structures in cases where neither were designed to resist strong earthquakes. No steel frame building collapses are known. One building under construction and only partly of steel frame collapsed in the 1957 Mexico shock. The nearest example of collapse of a steel frame was the 13 story Casa Latino Americano Building in the 1957 Mexico earthquake. Some of the columns were highly overstressed due to normal vertical loads, and apparently the earthquake brought several to the point of collapse.

The fact that steel frame construction has not suffered as severely as concrete frame has been evidence to some that the greater ductility of steel makes it a more reliable material for tall structures. Undoubtedly poor construction has more effect on concrete than it does on structural steel.

The foregoing comments relate to steel frame structures which have gone considerably beyond their design stresses. However, earthquake resistive multistory steel frame structures in which the lateral forces are taken by frame continuity have had significant amounts of non-structural damage in some instances. For flexible steel frame structures with unit stresses below yield, there usually was no significant difference in the cost of repair between structures designed to be earthquake resistive and those not so designed. An interesting example occurred in Los Angeles as the result of the 1952 Kern County shock; a 15 story steel frame earthquake resistive hotel suffered non-structural damage in percentage not significantly different from many nearby non-resistive steel frame structures (#4).

There have been several instances where the structural engineer has taken special care to reduce the damageability to the architectural, mechanical and electrical features in a flexible multistory building. Best example of the effectiveness of such a design was the negligible damage to the 43 story Tower Latino Americana in Mexico City during the 1957 Mexico earthquake (#24). It seems in order to again point out the fact that the energy delivered by the ground to the structure must be dissipated by damping within the structure or be returned to the soil. The problem of damage control requires careful attention if the building is to absorb energy with minimum damage.

Masonry Walls with Varying Roof and Floor Materials:

Unit masonry walls have suffered severely when not specifically designed to be earthquake resistive. Current interest lies in the effectiveness of modern techniques intended to make unit masonry into an earthquake resistive material by giving it reliable strengths in tension and shear as well as in compression. Details of construction techniques to make brickwork and hollow concrete block into earthquake resistive materials may be found in pamphlets such as Reinforced Grouted Brick Masonry - Field Inspectors Handbook and Reinforced Hollow Concrete Masonry - Field Inspectors Handbook.

One instance of effective use of reinforced grouted brick masonry was the Arvin High School in the 1952 Kern County earthquakes (#4, pp. 367-375). The design and construction of the group of structures were done under the jurisdiction of the Field Act, a California law which has proven effective in making public schools highly earthquake resistive. Over-all damage was less than 1%, although the nearby mercantile district in the City of Arvin had losses up to 100%. Damage was noted at roof to wall connections in several buildings, but the fracturing of the reinforced grouted brick masonry wall in the two story Administration Building shown in Figure 4 is of greatest interest. Cores from this wall taken prior to replacement indicated adequate compressive strength; however, observation during demolition indicated unsatisfactory bond between the mortar and brick. Also, it appeared that metal lath had been placed in some of the bed joints contrary to specifications.

Other reinforced grouted brick structures performed well. This type of material assembly is acceptable for earthquake resistive purposes, provided that competent full time resident inspection is maintained during the construction period. In vivid contrast was the damage to sand-lime unreinforced brick walls in the Kern County earthquake (#4, pp. 250).

Adequately reinforced hollow concrete block also performed equally well with reinforced grouted brick masonry. Causes for structural damage to hollow concrete block were more often attributable to faulty construction than to design, although the design may have also been poor. Such serious construction errors as insufficiently lapped reinforcing bars and the absence of grout in cells containing reinforcing steel were noted. Also, the lack of cleanout holes at the bottom of filled cells may have been contributory. Unreinforced hollow concrete block suffered similar damage as unreinforced sand-lime mortar brick masonry in all earthquakes which the authors studied.

Reinforced concrete walls with various roof and floor materials performed as well or possibly somewhat better than did reinforced unit masonry. Reinforced concrete walls, although intact even when the roof suffered serious damage, were not without their flaws. Classic in poor design, workmanship, and materials was the Cummings Valley School which collapsed in the 1952 Kern County earthquakes (#4, pp. 258-259). A more typical damage feature was the indications of movement at construction joints in reinforced concrete walls.

Tilt-up construction, in which the precast concrete walls are cast horizontally and later tilted into place, has performed well. The 1952 Kern County, the 1952 Bakersfield, the 1954 Eureka, the 1957 San Francisco, and other shocks have subjected a hundred or more of this type to strong ground motion. The majority of these structures had poured in place reinforced concrete pilasters. When damage occasionally occurred, it was usually in the form of "working" between the panel and its pilaster (normally where shrinkage cracks had already formed). While poured in place concrete pilasters have been effective, wood and steel columns and their attachments to the precast tilt-up walls remain largely untested by earthquake.

Diaphragms:

A horizontal diaphragm is usually considered to be a floor or roof which can resist lateral forces in a manner analogous to a plate girder. The roof or floor is assumed to act as the web while the boundary members of the roof or floor are assumed to act as the flanges. When effective boundary members or chords are lacking in a diaphragm, then all such lateral force resistance as might exist is provided by the analogous web alone.

Certain assemblies, as wood boards laid at right angles to their supports (i.e., straight sheathed), have not been considered suitable as diaphragms for lateral forces. Numerous examples of straight sheathed roofs with masonry walls built prior to the enactment of earthquake ordinances have experienced strong shocks. Almost without exception, such failures as did occur to these roofs were at their boundaries rather than internally. Diagonally laid wood sheathing has performed better as a diaphragm since it is stiffer than straight sheathed wood decks and since it is frequently used in earthquake design. Plywood as a diaphragm material has also been quite satisfactory. In general, damage related to wood diaphragms has occurred at their boundaries, particularly at masonry walls.

A number of metal roof decks have been subjected to heavy seismic forces in the last decade, and several of these have had poor earthquake design. Again, the authors know of no <u>internal</u> failures of metal decks however poorly tied together. Failure has occurred in several instances at the diaphragm's boundary, with the metal deck supports pulling away from the walls. Internal diaphragm stresses should not be neglected in the design despite the lack of internal damage to wood and metal decks; it should be apparent that the boundary has been the weakest element and not necessarily the only weak element. Also, an excessively flexible deck may be the cause of boundary failures due to large deflections.

Poured in place reinforced concrete as a diaphragm material has performed well, but calculations normally show concrete stresses to be quite low. The performance of precast concrete roof panels has been mentioned in previous paragraphs.

Damage to X-bracing used in place of diaphragms has often been the result of faulty workmanship, but in some cases it was related to design. Brief mention of several instances of defective construction may be of interest. In the 1957 San Francisco shock, failures occurred where anchor bolts at connections of X-bracing to concrete walls were improperly placed despite considerable inspection (#12). A similar situation occurred at the Arvin High School garage in the 1952 Kern County shock (#4, pp. 367-375). Stripped threads in rod X-bracing occurred at the Tehachapi Valley Union High School in the 1952 Kern County shock and was due to improper use of dies (#4, pp. 366-367). In summary, X-bracing systems are effective, but experience has shown the need for field supervision of critical details by the design engineer even though a resident inspector may be present. Incidentally, the permanent bending of rigid angle X-bracing has been seen in several shocks, notable the 1957 San Francisco shock (#12).

It is appropriate to mention that a number of material assemblies have been tested for their diaphragm characteristics, but only the tests on wood have been made generally available.

Special Structures:

Steel elevated water tanks, such as the rather conventional four legged 100,000 gallon tank on a 100 foot steel tower, have suffered significant structural damage although often designed to be earthquake resistive. The damage to the anchor bolts and to the rod bracing clearly indicates that present standards are not conservative (#4, pp. 436-454; #11, pp. 136-137).

Liquids in large ground level tanks will surge and in doing so may damage the tank tops. Floating roofs are particularly susceptible. It is not always recognized that a tank shell will deflect vertically as well as horizontally when liquids surge. Rigid piping which is closely coupled between tanks and pumps or similarly stable objects often breaks due to these shell distortions (#4, pp. 454-462; #12). In addition to the shell distortions, these tanks have been known to slide in very heavily shaken regions. Large vertical accelerations which may accompany horizontal accelerations make frictional resistance unreliable. Special consideration to deflections and sliding, as well as to shell stresses, should be given to tanks having contents which must not be lost in a shock.

Tall steel processing towers, such as are commonly found at oil refineries and chemical plants, have also experienced seismic forces in excess of those given in building codes (#4, pp. 270-279).

Relatively few tall stacks have experienced strong earthquake forces in recent years. Although several were damaged, none collapsed (#4, #11).

Summary and Conclusions:

Earthquake intensity scales, of the subjective type such as the Modified Mercalli scale, are becoming much more difficult to apply due to factors such as:
1. Definition of terms (i.e., definition of "good" construction varies markedly from

one community to another since earthquake resistive design standards vary); 2. Multiplicity of building materials and construction techniques; and 3. Long period effects. With these complexities it becomes more important to have the scale applied by a single agency within each country in order to achieve uniformity of interpretation.

The distance that a structure is from a fault on which an earthquake occurs is not as important as proper earthquake resistant design of the foundations and superstructure. Recent damage observations tend to confirm the theory that the intensity adjoining the fault's surface trace may not be significantly different from that several miles away from the fault, provided the geologic conditions are reasonably uniform and provided the fault dip is reasonably vertical. There is need for verification through the use of strong motion instruments.

Several major communities have active faults within their city limits, but most have not considered these faults in their city planning. Surface faulting has been recognized in 8 instances between 1950 and 1959, and its apparent frequency certainly warrants study in city planning.

Long period ground motion from strong distant shocks have been selectively damaging to long period structures due to quasi-resonance. Principally affected were multistory structures and fluids in large tanks. Low rigid buildings, although weak, have performed relatively well in the long period motion.

Monolithic poured in place reinforced concrete was a satisfactory construction material in multistory buildings when properly designed and constructed. Faulty design and construction were always apparent where serious structural damage occurred. Shear wall construction reduced non-structural damage much more effectively than did concrete frame construction. The record indicates that a carefully conceived and executed precast concrete design can be effective.

With respect to collapse or partial collapse of structures not earthquake resistive, steel frame construction performed better as a class than did reinforced concrete. Steel frames with shear walls generally had less non-structural damage than those without shear walls.

Reinforced grouted brick masonry and reinforced hollow concrete block masonry have proven effective in resisting the tensile, compressive, and shear forces from earthquakes. Workmanship is critical, and good construction is more difficult to obtain than for structural steel or poured in place concrete.

Roof and floor diaphragms have had damage along their boundaries, but internal diaphragm damage was quite rare. The connection of the diaphragm to a dissimilar wall material is critical both in design and workmanship. X-bracing instead of diaphragms was satisfactory, but the connections to masonry walls were critical.

Competent full time inspection should be provided on all work which has structural significance as well as for all major construction. In addition to the resident inspector, the design engineer should field inspect particularly critical and complex details. This is especially true for some newer non-conventional structures, and for unit masonry construction.

Reasonable life safety is provided by good earthquake resistive design, but a number of often neglected details can cause hazards (as broken glass, falling light fixtures, etc.). Damage control of non-structural items has been successfully applied to some buildings, although many consulting engineers still neglect any seismic problems other than the stability of the structural frame.

It is mandatory to develop a much greater network of strong motion seismic instruments for best progress in earthquake engineering.

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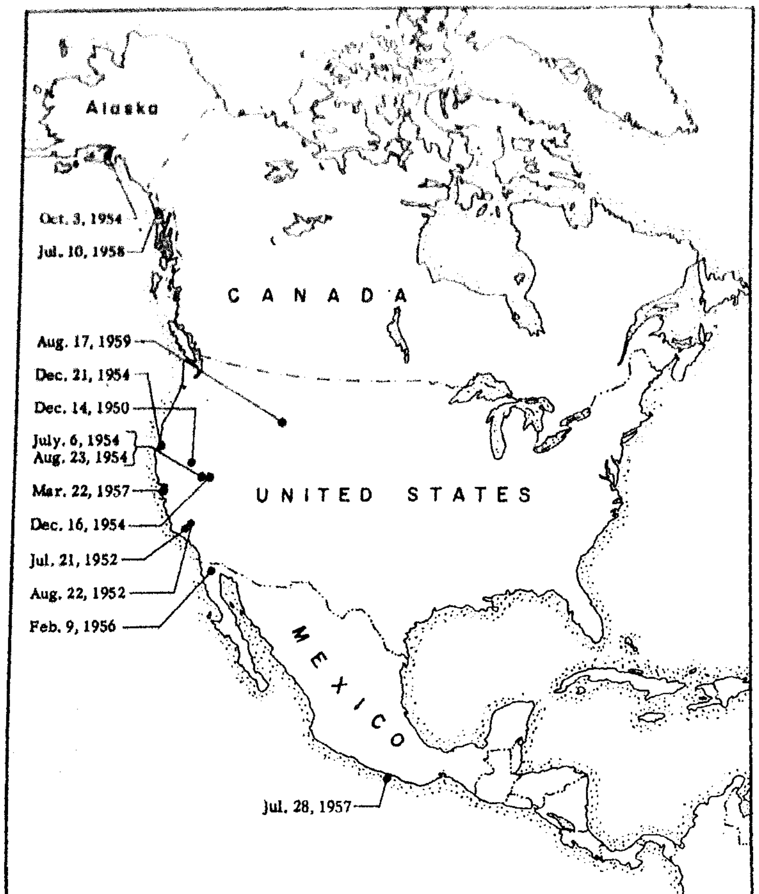

Figure I. Epicenters of principal earthquakes having engineering interest.

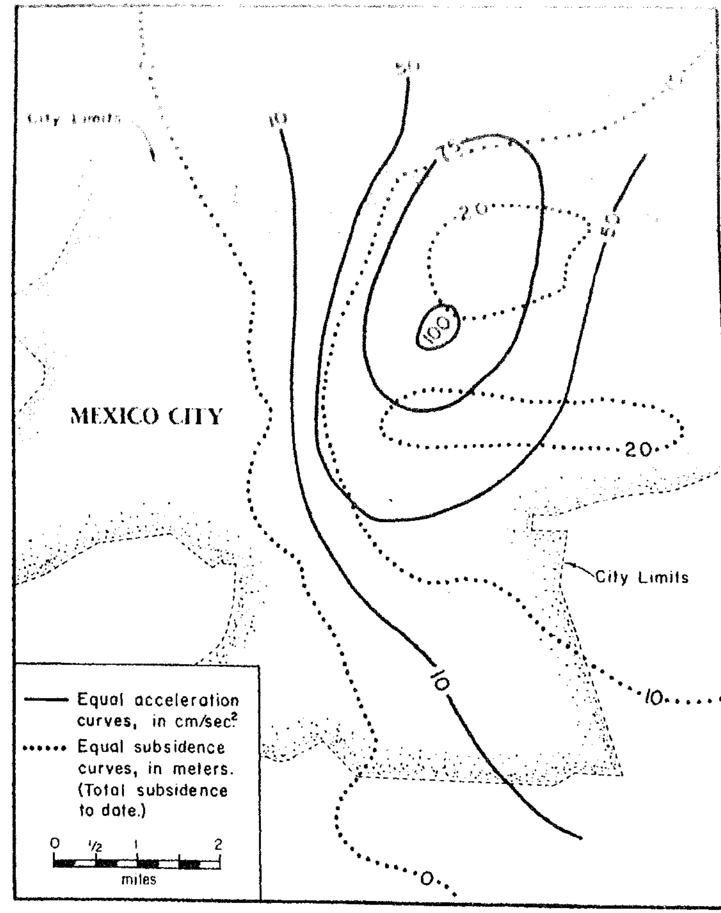

Figure 2. Relation between subsidence and acceleration in the July 28, 1957, shock. Subsidence is indicative of geologic conditions. After J. Merino y Coronado.

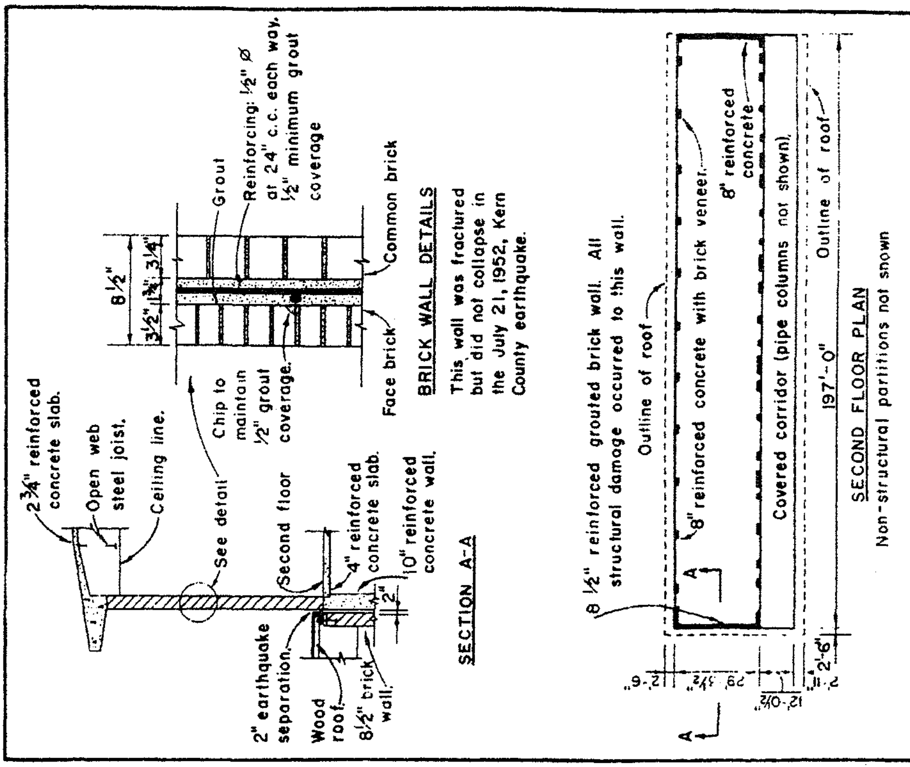

Figure 4. Administration Building, Arvin High School

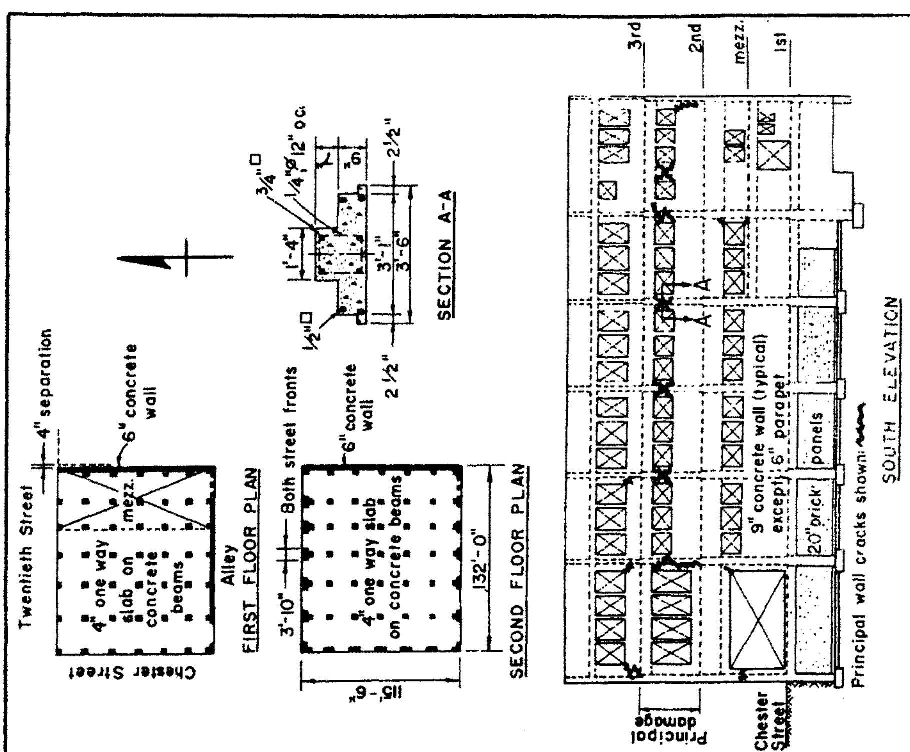

Figure 3. Brock's Department Store, Bakersfield.

TABLE 1 ' NORTH AMERICAN EARTHQUAKES, 1950-59
Selected for Engineering Interest

Date[1]	Name and Location	Richter Magnitude[2]	Modified Mercalli Intensity[3]	Surface Faulting[4]	Bibliography
December 14, 1950	Fort Sage Mtns., California	5.6	VII	5.5 mi.	1, 2, 3
[5] July 21, 1952	Kern County, California	7.6	X[14]	14 mi.	3, 4, 5, 6, 7
[6] August 22, 1952	Bakersfield, California	5.8	VIII	None	3, 4, 5
July 6, 1954	Fallen - Stillwater, Nevada	6.6	VIII	11 mi.	3, 7, 8
[7] August 23, 1954	Fallen - Stillwater, Nevada	6.8	VIII	19 mi.	3, 7, 8
[8] October 3, 1954	Kenai Peninsula, Alaska	6.75	VIII	---	3
[9] December 16, 1954	Dixie Valley - Fairview Pk., Nev.	7.1, 6.8	VII	35 mi., 30 mi.	3, 9
December 21, 1954	Eureka, California	6.6	VII	None	3, 11
[10] February 9, 1956	San Miguel, Baja Calif., Mexico	6.8	VII	12 mi.	3, 10
March 22, 1957	San Francisco, California	5.3	VII	None	12
[11] July 28, 1957	Mexico	7.5	VIII	---	13, 14, 15, 27
[12] July 10 1958	Lituya Bay, Alaska	8	---	18 mi.	21
August 17, 1959	Hebgen Lake, Montana	7.1	---	14 mi.[13]	22

[1] Local time.
[2] Slight variations will be found in various publications.
[3] As determined by the U. S. Coast and Geodetic Survey (for U. S. locations). Intensity given is the maximum for vibratory effects; higher intensities usually exist at surface faulting.
[4] Length of surface faulting is not precise in most instances, and may have been considerably longer in several cases.
[5] Faulting probably longer, but covered by deep alluvium.
[6] Aftershock of July 21, 1952, earthquake.
[7] Second of two shocks in same area.
[8] Unpublished report on file at the authors' offices.
[9] A second shock followed the first by four minutes. The Modified Mercalli intensity applies to both considered as a single event.
[10] M. M. intensity assigned by authors of this paper.
[11] Principal engineering interest was in Mexico City damage (MM=VII). Unpublished report on file in authors' offices.
[12] Information on this shock is preliminary.
[13] Faulting complex, and regional warping occurred.
[14] Maximum intensity for buildings was VIII.