EARTHQUAKE RISK AND PROTECTIVE MEASURES

by

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Introduction

Earthquake risk may be defined as the potential life loss and property damage from seismic forces in any selected area over any given period of time.

First and foremost in earthquake risk protection is the protection of life. Experience in recent earthquakes, including the 1964 Alaskan shock, has shown that reasonable life safety is provided by good earthquake resistive design. The term "good" is a subjective one, and, of course, each structural designer considers that his work is superior. Unfortunately, earthquakes do not recognize these personal prejudices.

Too frequently a number of generally neglected details can cause significant life hazards in buildings which are considered to be earthquake resistive. I can cite examples such as window glass from multistorey glass walled buildings with moment resisting frames and also florescent light fixtures of which literally miles have come down in earthquakes. High and unanchored shelving has fallen and "non-structural" rigid masonry in flexible frame structures has caused life loss. The case histories of the earthquake performance of buildings in Alaska and elsewhere in North America will be discussed later on, and each of you will be able to define the term "good" for yourself based on the observed damage.

Second to life hazard, and rightly so, is earthquake damage control. Actually, it is often difficult to separate between damage control design and life safety design. It is important to point out that the earthquake provisions in the Uniform Building Code can allow considerable damage, both structural and non-structural. The Seismological Committee of the Structural Engineers Association of California, in discussing this code which they were instrumental in developing, stated in their published commentary on the code: ". . . the code does not assure against non-structural damage . . .". Their commentary also states that their code provisions are "intended to safeguard against

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major structural failures". Later in this lecture you will see slides of the damage to the 14-storey Mt. McKinley Building in Anchorage as a result of the 1964 shock. This damage has been reliably placed at 40% of the building's replacement value. I have heard it argued by one engineer that this building completely filled the code requirements, namely, that no one was killed or injured as a result of building damage.

Adequate design for damage control has many problems of detail. Fundamental to this is the understanding of the deformations that a building experiences due to earthquake motions. These deformations often cause damage which we like to call "damping". Fortunately, the term "damping" doesn"t sound destructive. From theoretical considerations, large damping values reduce the stresses in the structural frame, but it is the grinding of partitions, cracking of precast concrete "non-structural" walls, racking of heating and air conditioning systems, and the like which may have monetary value losses greater than the structural frame which is being protected by "damping". Obviously, damage control is a problem in economics, but the extent of the cost is a function of the engineer's understanding of the practical seismic design problems.

One of many bases for developing professional judgment in the field of earthquake risk and protective measures is to evaluate the experiences of previous earthquakes. The balance of this discussion will be directed to a brief review of several North American earthquakes which are significant to earthquake engineering. Due to obvious time limitations only one or two selected significant features can be given for each earthquake.

San Francisco, California
April 18, 1906

Let us arbitrarily start with the 1906 San Francisco earthquake for today's discussion. Vibration damage to buildings which did not have poor foundation problems was usually not spectacular. This is not to say that this damage was not serious.

The first slide on the screen (781*). illustrates downtown San Francisco with the fire advancing towards us. The masonry on the sidewalks indicates damage, and quite likely structural damage. The next slide (776*) shows significant structural damage, but not building collapse. Certainly the earthquake came no where near leveling San Francisco after the wood frame construction had been burned. What is not recognized by engineers who have not been on damage surveys is that earthquake damage often is not spectacular up to the point of collapse, although this non-spectacular damage may require that the structure be demolished. This observation will come up again in this discussion. A critical restudy of the 1906 earthquake damage has shown that this earthquake damage was about 20% of the total damage, with the remaining 80%

being fire damage. This fire loss occurred to a city which was described in an insurance report published a year before the earthquake as follows: ". . . San Francisco has violated all underwriting traditions and precedent by not burning up."

Long Beach, California, Earthquake
March 10, 1933

The 1933 Long Beach earthquake marks a major turning point in the field of earthquake resistive design and construction for much of California. Earthquake provisions up to that time were not contained in any of the metropolitan Los Angeles building ordinances, including that of Long Beach. There had been controversy regarding the potential earthquake hazard to the Los Angeles area. One book authorized by a prominent geologist and published five years before the earthquake, stated, "The accumulative weight of data substantiates beyond a doubt my deduction that Los Angeles is in no danger of a great earthquake disaster". The 1933 Long Beach disaster brought the debate to a close.

Property damage was estimated to be from $40 million to over $50 million. This shock was not of major proportions from the seismological standpoint. However, the Modified Mercalli intensity reached IX, and its occurrence in a highly populated area makes it of engineering significance. The ratio of damage to value on business, industrial and residential property in the City of Compton, for example, has been reliably placed at 29%. This is heavy by any North American standard. The shock ranks third to the most destructive earthquake in United States history.

Slide (1002*) shows building damage by class of construction which followed age-old patterns. Structures with walls of brick masonry having sand-lime mortar and with wood roofs and floors suffered severely. An interesting study by R. R. Martel on the performance of brick-joisted structures led to the conclusion that damage to buildings of this type was somewhat less on soft, water-logged soil than those on more firmly consolidated soil. This is curious and interesting in view of contrary observations in other earthquakes.

Slide (1006*) is of a public school. Public schools with unit masonry construction deserve special mention. Exterior walls of many schools were brick, or in some cases hollow clay tile. Roofs and supported floors were wood. The destruction to this type of school construction was most spectacular. Slide (1007*) is another example. Fortunately, the earthquake occurred after school hours and a potentially catastrophic situation was averted. However, the destruction was so extensive that the legislature of the State of California passed a bill which became law on April 10, 1933. This law, known as the Field Act, required all new public school construction to be highly earthquake resistive. Structures built under this law have performed excellently in subsequent shocks. It would be well for all other political jurisdictions in seismic regions to give thought to similar laws.

Imperial Valley, California, Earthquake
May 18, 1940

The next slide (1153) shows offset rows of orange trees which resulted from the 1940 Imperial Valley earthquake with its 40 miles of surface faulting. The right lateral fault displacement reached almost 15 feet near the United States-Mexico border. This shock had a Richter magnitude of 7.1. The maximum Mercalli intensity was X. Total direct life loss has been given as 8 or 9.

Of major engineering importance was the record obtained from the strong motion instrument in the city of El Centro. This accelerograph record has been of exceptional influence since it was a good recording of the ground motion near the epicentral region of a damaging earthquake.

The next slide (1069*) shows the wreckage of a two-storey wooden hotel at Brawley. Contemporary accounts state that the maximum Modified Mercalli Intensity was IX. In El Centro, a number of old brick buildings were so much damaged that they were condemned. Damage was chiefly to old brick construction, to walls that were not reinforced or tied into the structure, and to balconies projecting over sidewalks. But far from all buildings of these types were damaged. Damage in Imperial and Brawley was greater than in El Centro, although the epicenter was closer to El Centro. It is to be regretted that the structural engineering aspects have never been published in detail, so that it would be possible to attempt to correlate damage with the instrumental record.

Kern County, California, Earthquakes
July 21, 1952; and aftershocks of August 22, 1952

The July 21, 1952 Kern County, California, earthquake and its aftershocks are of particular engineering interest since they constitute the first major test in North America of structures of earthquake resistive design. The July 21 shock had a Richter magnitude of 7.7 and developed at least 14 miles of surface faulting. The time of its occurrence, 4:52 a.m., undoubtedly was a factor in keeping the life loss to the relatively low figure of 12. The August 22nd aftershock with a Richter magnitude of 5.8, was not the largest aftershock based on magnitude. However, its epicenter was located near Bakersfield, and the shock therefore caused extensive damage to many already earthquake weakened structures.

Damage in the White Wolf fault zone as a result of the surface faulting was primarily confined to the Southern Pacific Railroad tunnels since very few other man made structures were in the fault zone.

Slide (753) shows fault damage to a tunnel. Of the then total of 15 tunnels between Bakersfield and Tehachapi, the four which were seriously damaged were in the fault zone. It is to be expected that very heavy loss, if not total loss, must occur to buildings, canals, tunnels, etc., which are astride ground breakage resulting from fault movement. This need not be true for structures located in the immediate vicinity of the fault trace; this subject is discussed in more detail from the 1959 Hebgen Lake shock.

Slide (41) shows damage to a brick building. Unreinforced brick bearing wall structures, with its conventional sand-lime mortar in general use until the advent of earthquake resistive reinforced grouted brick masonry, was common in the older sections of the cities of Tehachapi, Bakersfield, and Arvin. Damage to this class of construction was severe just as it has been in all previous major earthquakes. This performance of non-earthquake resistive brick masonry is of interest since it may be compared with the successful performance of masonry materials put together in a manner to resist seismic forces.

Let us look closely at this slide. The bearing walls are gone and collapse is prevented by the non-structural walls. Slide (42) is another example of the second line of defence sometimes provided by "non-structural" walls. Slide (49) shows the ceiling of the second storey now supported by the piano and chairs in this lodge hall assembly room. In the next slide (51), the support given by the piano is obvious. In this case, the lack of non-structural partitions materially increased the life hazard.

Earthquake resistive brick masonry walls, built with a technique known as reinforced grouted brick masonry, performed excellently. The technique involves two wythes of brick laid in a cement mortar. The wythes are separated by usually 1 or 2 inches and the spade is filled with small aggregate reinforced concrete.

The best example of the performance of reinforced grouted brick masonry is the Arvin High School in Arvin. Slide (79) shows undamaged buildings. This school consisted of about 15 buildings constructed in the period 1949-1951. The three construction contracts, totalling $2,800,000, indicate the extent of the one and two storey buildings at this site. Reinforced grouted brick masonry was used as the principal wall material on most major buildings. The design and construction was done under the jurisdiction of California's Field Act. Minor or negligible damage was found in most buildings and none of it constituted a life hazard. However, the two storey Administration Building had significant damage to one 8-1/2 inch thick reinforced grouted brick wall as a result of the July 21st earthquake. The next slide (300) shows the general location of the damaged two storey wall. Slide (77) shows the shear cracks which went through the brick rather than through the mortar joints. Subsequent aftershocks increased the damage. While seriously damaged, collapse was not imminent. Workmanship errors in the damaged wall were apparent as may be seen in the core samples shown in slide (331). The overall damage to all buildings was less than 1% of their value. The performance of these buildings was a milestone in the development of a material previously associated with collapse and large life loss.

The few other known examples of earthquake resistive design using reinforced grouted brick masonry had no structural damage.

The case history of one precast concrete structure is worth summarizing at this point. A plot plan is shown in slide (780). This Lockheed plant in Bakersfield survived the July 21st shock, but the August 22nd aftershock caused significant structural damage to one of the two buildings at this site. As shown in slide (380) which was taken during construction, both structures had a precast concrete roof on precast concrete beams and girders, in turn supported by precast concrete columns. Exterior walls were also precast concrete as may be seen in slide (379). The earthquake design considered the roof to be a rigid diaphragm taking lateral forces to the exterior walls which in turn were to be used as shear walls. However, the joints between the precast concrete roof panels were filled with mastic instead of grout and other significant deviations from the plans were noted. As a result, the building swayed violently, breaking interior columns at the point of high bending moment at the floor line as shown in slide (751), and leaving one structure out of plumb. Precast concrete requires careful attention to the joinery between precast elements, and superior field supervision is necessary to insure the proper execution of a good design.

Damage in Los Angeles as a result of the July 21, 1952, shock was generally confined to steel and concrete frame fire resistive structures over 5 or 6 stories high. A few isolated instances of minor damage to one-and two-storey buildings were noted, but they were not significant. This pattern of damage was opposite to that which was experienced to Kern County on July 21st and in Bakersfield on August 22, 1952, in that there the one- and two-storey brick bearing wall buildings were not affected as compared to the multistorey steel and concrete frame buildings.

The occurrence of long period ground motion at long distances from an earthquake's epicenter has been well known, but building damage therefrom was not well recognized until the 1952 earthquakes. Since then, other examples of quasi-resonance have been clearly identified.

Port Hueneme, California, Earthquake
March 18, 1957

It is at this point that it is of interest to look at a quite different type of earthquake. The Port Hueneme, California, earthquake of 1957, with a small Richter magnitude of 4.7, caused exceptional damage for a shock of such a low magnitude. The maximum ground motion as recorded by an accelerometer was 18 per cent of gravity. An analysis of the strong motion record showed the earthquake to be essentially a single pulse of energy. This type of single pulse energy release appears to be unusual from a damage standpoint, and is quite the opposite to the more commonly known large magnitude earthquakes having a destructive duration of up to possibly as much as three minutes such as apparently occurred in Anchorage. The Agadir, Morocco, earthquake of 1960, which caused very severe destruction to a poorly built city, also appeared to have its energy release in a single primary pulse.

The full engineering implications of an earthquake releasing its energy in a single pulse are not well understood.

Mexico City, Mexico
July 28, 1957

Mention should also be made of the long period effects which occurred in the Mexican earthquake of July 28, 1957. Slide (1113) shows the collapse of a 7-storey reinforced concrete building. Details of the "pancake" failure are shown in slide (1133). Slide (1116) shows another collapsed multistorey building. Predominent ground motion has been estimated at 1.5 to 2.0 seconds at Mexico City. The shock's epicenter has been reported as 170 miles from Mexico City by one source and 220 miles by another. The fact that the collapsed as well as the seriously damaged multistorey structures were usually quite weak or of defective design should not obscure the observation that the gound motion did not proportionately damage weak and poorly designed low rigid buildings.

Hebgen Lake, Montana, Earthquake
August 17, 1959

Slide (1456) is one view of the extensive and complex fault scarp system which was formed during the Hebgen Lake earthquake which had a Richter magnitude of 7.1. The next slide (1510) shows a landslide containing about 43 million cubic yards of material and which dammed Madison Canyon.

Bedrock beneath Hebgen Lake warped, rotated, and caused a seiche in the lake. The water surface of Hebgen Lake dropped slightly over 10 feet due to the geologic changes. Precise relevel measurements by the U.S.C. & G.S. found a maximum drop of a benchmark near the north shore to be 18.8 feet. We must conclude that this was a major shock from geologic and seismologic evidence.

However, building damage in the epicentral region was singularly unspectacular insofar as vibratory forces were concerned. Slide (1471) shows a wood frame building almost on the fault. Wooden buildings could be considered as earthquake resistant, except for their unreinforced masonry chimneys and for the apparent fact that these structures were rarely anchored to their foundations. Chimney damage was general but far from universal, and few wooden structures left their foundations. Hollow concrete block buildings as a class did not perform quite as well as did wooden structures. Again damage was remarkably small considering the usually complete lack of construction features held mandatory in earthquake resistant design. One example, located almost on the fault, is shown in slide (1493); I would be very surprised if there was any wall reinforcement.

We may conclude from this earthquake, as well as from other earthquakes, that proximity to a fault is a secondary consideration to earthquake resistive design. The hazard becomes excessive only when a structure is across a fault.

Puget Sound Earthquakes
April 13, 1949; and April 29, 1965

The Puget Sound earthquakes of April 13, 1949 and of April 29, 1965 are of interest since their focal depths were deeper than those which generally occur in California.

For example, the focal depth of the 1965 earthquake has been tentatively placed at 36 miles. In view of the 10 mile focal depth commonly given for the majority of California earthquakes, the deeper focal depths for these two shocks in Washington suggest a somewhat more moderate intensity over a wider area than comparable magnitude but shallower California earthquakes. A general view of the 1965 preliminary damage data tends to confirm this observation.

Pockets of high earthquake intensity, as typified by damage such as fallen chimneys, could almost always be associated with the local geology. Damage was the most pronounced in what is commonly termed "poor ground" areas. These pockets were distributed over a large region, with the areas around these pockets having damage so slight that it was difficult to find it.

In summary, earthquake focal depths and local geologic conditions can greatly vary damage patterns for any given magnitude earthquake.

Alaskan Earthquake
March 27, 1964

Probably the most interesting earthquake to us is the recent Alaskan earthquake of March 27, 1964 with its disastrous seismic sea wave, with its landslides in Anchorage, and with its building damage in Anchorage. This earthquake, with its epicenter about 75 miles in an easterly direction from the City of Anchorage and in the Prince William Sound area, caused significant structural damage to modern buildings in Anchorage.

The earthquake's large magnitude of 8.4 ranks this shock close to the largest ones for which instrumental magnitudes are known. Additionally, almost all major buildings were constructed under a building code which required earthquake bracing. Under these circumstances, a review of the performance of buildings in Anchorage is very important. This discussion will be limited to vibrational effects to selected buildings in Anchorage and vicinity.

A forerunner to the 1964 earthquake was the 6-3/4 magnitude earthquake of October, 1954 with its epicenter about 50 miles due south of Anchorage. This earthquake caused minor structural damage to two 14-storey reinforced concrete bearing wall buildings and to the 6-storey airport control tower in Anchorage. These buildings were severely damaged in the 1964 shock.

A good general index to the earthquake resistance of buildings within a city is its building code seismic provisions. In 1950, Anchorage adopted the 1949 edition of the Uniform Building Code, including its lateral force provisions. The population at that time was about 11,200. Seismic zone 2 was in force from 1950 until about the middle of 1954, with zone 3 provisions in effect thereafter. During the period from 1950 until 1961, the plans for all major buildings were checked for Uniform Building Code compliance, including its seismic provisions, by the plan checking staff of the International Conference of Building Officials. It is apparent, then, that the plans for most of the major buildings in Anchorage were supposed to have been reliably checked.

Regrettably, no strong motion recording instruments were in Anchorage at the time of the earthquake. Therefore, the characteristics of the ground motion and its duration can only be estimated from vibrational effects on objects and the usually subjective accounts of persons who experienced the event.

The predominent period range of ground motions as generally described by the most objective observers in non-landslide areas is associated with period motions of 1/2 second and longer. Slide (2418) shows the shelves of a store across the street from the Fourth Avenue landslide. It will be noted that the globe is overturned. One or two items fell from the middle shelves. Elsewhere in the store, the bulk of the merchandise remained on the shelves although it usually shifted. This lack of damage is in sharp contrast to the damage to many large structures which were intended to be earthquake resistive. The frequently found lack of extensive shifting of small, unanchored objects on shelves also suggests that the short period motions were not predominent. The next slide (2037) shows wood frame buildings which are undamaged as far as one can see. Incidentally, the snowman on the right survived the shock.

Strong motion records of the aftershocks having epicenters about as far away as the main shock also show that the longer period motions predominated.

In summary, then, the predominent periods of the horizontal ground motions in Anchorage appeared to be in the order of one-half second and longer. This should definitely subject the longer period structures, genrally the tall buildings and very large area buildings, to heavier lateral forces and, with all factors being equal, to significantly more vibration damage.

The duration of the damaging intensity, as opposed to the total duration of the felt motions, has been estimated to be perhaps as long as three minutes. Since the duration of the severe motion in the 1906 San Francisco shock has been often given as 40 seconds and this is often used as a standard, the 1964 Alaskan earthquake appears to be exceptional. An earthquake's duration is one fundamental in damage. Many repeated excursions into the yield range will eventually bring destruction to steel. Hairline shear cracks in reinforced concrete will become larger with an extended duration and finally the enlarged cracks can bring failure. It is interesting to note that many observers of building collapses in Anchorage stated that the buildings that they were watching failed during the last stages of strong motion.

As you have seen, wood frame structures such as the conventional single family dwellings survived nicely as a class of construction when not located in landslides.

Also, as you have seen, unit masonry construction has usually been associated with large life loss and property damage in past earthquakes. Anchorage proved to be an exception to this rule. When small in floor area, when one or two storeys high, andwhen not located in the land movement areas, damage was usually no more than slight to moderate at most. The hollow concrete block was almost always reinforced with steel, although too often it was quite poorly done.

Slide (2404) shows a typical damaged hollow concrete block building. Slide (2407) shows the corner detail, and we can readily observe that the cells containing steel were not grouted. Slide (2384), taken at the partially collapsed Gay Airways, clearly shows a cell which was not grouted.

Due to time limitations, I would now like to limit the discussion to case histories of several major buildings in Anchorage. Considerable additional information will be published by the U.S. Coast and Geodetic Survey in about one year.

Hill Building:

As a first example, let us consider the Hill Building which is shown in slide (2125). Exterior walls are metal skin. The two central cores of this 8 storey building are of poured-in-place reinforced concrete. The core walls are bearing walls and carry loads from the steel beams which frame into these walls. All core walls are 8-inches thick and contain only a single curtain of steel. Floors are 5-inch thick one way slabs of poured in place reinforced concrete on structural steel beams.

The lateral forces are resisted by the two interior stair and elevator cores which are located 10'6" apart and which do not have a common footing.

The bulk of damage was found in the two central cores. The reinforced concrete walls had their principal damage in the first storey as seen in slide (2139), with damage becoming progressively less in the upper storeys. The next slide (2142) shows movement along a construction joint in the first storey, indicating that the reinforced concrete walls did not develop their full design stress. However, the basic cause for building damage was not found until repairs were in progress and until the first floor slab, which was on grade, was removed. Slide (2137) shows the ground floor slab removed; note the buckled reinforcing steel. The corner of this building went down 4-3/8 inches, and the steel buckled due to the concrete "mushing" out. The bottom of the foundations remained true.

Laboratory analysis of the concrete in this "mushed" area indicated excessive organic material. Certainly the strength was low, since it was possible to dig the material out with a finger. It appears that exceptionally poor concrete was placed in a layer varying from about 6 inches to zero inches. During the earthquake when the concrete "mushed", the seismic loads were all thrown to the other core tower with its poor construction joint, and the damage ensued. Slide (2126) shows partition damage in the first storey. The next slide (2132) shows damage at a beam connecting the two core walls. These two independent cores are about 10 feet apart. The excessive lateral deflections of the cores caused excessive bending and shear stresses to the beam between these core towers. This damage was reflected to the top storey as may be seen in slide (2124).

The representative damage that you have seen amounted to about 20% to 25% of the replacement value of the building. The primary cause of the damage appeared to be an isolated case of poor workmanship at a critical location in a shear wall.

<u>Mt. McKinley Apartment Building</u>:

The 14-storey Mt. McKinley Apartment Building, shown in slide (2057), is entirely of poured-in-place reinforced concrete construction. Floors are generally 5-1/2 inch reinforced concrete one way slabs having maximum clear spans of 17'-9". These floors are supported by the exterior walls serving as bearing walls. The walls are 8" thick in the top storeys and become 12" thick in the first storey. Some of the 10" walls have only a single curtain of steel.

The next slide (2062) shows the damaged spandrel walls having rustications which are nominally one inch thinner than those without rustications. However, the rustications are about one inch deep, thereby reducing an 8-inch wall to about 6 inches at the rustications; similar reductions apply to thicker walls. Another example of wall damage may be seen in slide (2075). Note the X-cracks indicating diagonal tension failure. A close-up of typical damage may be seen in the next slide (2074). Looking up the wall as in slide (2077), we note that the damage becomes less in the upper storeys. The next slide (2095), shows that the interior shear walls around the stairs were also damaged; note that these horizontal cracks represent slippage along construction joints.

The north wall damage, shown in slide (2084), was most interesting. It can be seen that one shear pier has failed. This is more clearly shown in slide (2089). Slide (2096) shows the inside of the damaged shear pier. A complete air gap extends along the entire length of this failed shear pier.

The mechanism which caused this vertical alignment of damage may be more clearly understood if the end wall shown in slide (2078) is considered to act as a vertical cantilever. If, at your convenience, you draw a deflection diagram of this vertical cantilever, it will become apparent that large shear stresses must be developed in the spandrels. Perhaps the next slide (1878) showing idealized deflection arrangements will make the problem chearer. In slide (1877), if the spandrels are rigid and weak, then we may expect a vertical alignment of damage as shown on the right.

This vertical alignment of shear damage, in one form or another, was found in multistorey buildings elsewhere in Anchorage.

Damage to this building has been placed at 40% of the building's replacement value.

Slide (2105) shows another essentially identical 14 storey building to the one just discussed. If you look closely in the lower left hand corner you will note a fractured shear wall similar to that found in the Mt. McKinley Building. The next slide (2110) is a closer view. Slide (2115) shows once again the vertical shear failure. This building had damage which approached about 30% of the replacement value of the building.

Anchorage-Westward Hotel:

Slide (2147) shows the 14 storey Anchorage-Westward Hotel which is located at the edge of the Fourth Avenue landslide. As a result of this earth movement, the building is about one foot farther to the north.

Cracks in the basement floors and walls can be attributed to this land movement. The 14 storey tower was built in two stages, with the full basement and the lower 8 storeys completed in about 1960. The upper storeys of this building were almost ready for occupancy at the time of the earthquake.

The building has a partial steel frame. Floors are 5-1/2" to 6-1/2" reinforced concrete on metal deck, in turn welded to steel beams. As may be seen in slide (2149), the walls are metal skin although some exterior walls are reinforced conrete. Earthquake bracing is in the form of interior as well as exterior shear walls.

If we look closely, we can see that the two buildings pounded together. Slide (2154) shows the effect of this pounding. The next slide (2155) shows pounding effects in the rear of the building. Details of this pounding damage may be seen in slide (2157). It will be noted that the steel has badly buckled, and some has apparently snapped.

Slide (2177) shows damage to an interior shear wall. Slide (2182) shows a classic example of movement along a construction joint; the vertical bars have snapped.

Doorway lintels were damaged as may be seen in slide (2159). This is at room 441. The next slide (2160) is at room 741. Slide (2161) is at room 941. From the room numbers, it is apparent that there is a vertical alignment of damage from storey to storey. This also occurred over other doorways. Here again we see the results of vertical shear damage similar to that described for the exterior walls of the last two buildings.

Overturning forces were also present. Slide (2165) shows a buckled column in a lower storey which is best explained by overturning.

Four Seasons Apartment House:

Slide (2214) shows the collapsed Four Seasons Apartment Building. As you will note, the two central core shear towers have turned over. This building was in the last stages of construction at the time of the earthquake, with only the non-structional elements being incomplete.

Slide (2213) shows this building at the extreme left. In the foreground is the graben of the "L" Street landslide. Note in the center of this slide that a wood frame dwelling, along with its brick chimney, remain standing. This question has been raised more than once: "Why did the older buildings constructed by carpenters perform better than those very new buildings which had architects and engineers?"

Slide (2216) allows us to count the number of storeys, namely 6. The supported floors and roof were of prestressed lift slab construction.

Slide (2216) allows us to count the number of storeys, namely 6. The supported floors and roof were of prestressed lift slab construction. Columns were structural steel in the 10-inch wide flange series. Slide (2220) shows that the column loads were transferred from the prestressed slabs by means of special shear heads. It will be noted that the metal did not fail, rather the concrete sheared. This has led to the strong recommendations by some that a better column to slab connection must be developed, particularly by means of dowels welded to the shear head. Coming back to the slabs as shown in slide (2225), we see that upon failure the slabs were buckled and stacked like pancakes. These 8-inch floor and roof slabs were post tensioned with draped tendons before being lifted into place. The tendons were not grouted after being tensioned as may be seen in slide (2227).

After the slabs had been lifted into place, the two core towers were constructed of poured in place reinforced concrete. As shown by the next slide (2690), the slabs did not have the full 1/2" specified seat but this deviation did not in my opinion bring about primary failure.

Observers who watched the collapse generally stated that it occurred at the latter stages of this long duration earthquake.

Collapse probably occurred when the shear towers overturned due to bond failure in the shear walls at the first floor line. Calculations tend to confirm this observation regarding bond failure. As the increasing lateral deflections of the core towers continued in this long duration earthquake, damage at the first floor line probably became progressively greater. Finally, the two core towers overturned to the north. It seems most likely that general slab failure started as the towers began to overturn since the stacked slabs were offset from each other.

In looking again at the stacked floor slabs as shown in slide (2226), the potential life loss is appalling. Let us briefly summarize our findings without taking time to establish proofs. The shear distribution between core towers would be 50-50 on a simplified net shear area distribution basis, and this is a common procedure. Similar results may be obtained on a gross moment of inertia basis. Under these circumstances, the building probably will meet the seismic code provisions.

However, a more exact analysis would consider the rigidities of the lintels over the doors which contain deformations from vertical shear forces as has been described in the Mt. McKinley Building and the Anchorage-Westward Hotel. Under these conditions, one core tower is considerably stiffer than the other. This more "exact" analysis requires a number of non-exact assumptions, and probably no two engineers will come up with the same numerical answers. Indeed, I am afraid that most practicing engineers would not attempt this complex analysis.

If we don't make this type of complicated analysis, then the framing system should be simplified. One solution is a symmetrical shear wall layout. Alternatively, a series of shear walls can be located and designed that if failures do take place, then the remaining elements are symmetrical and can take loads in a known proportion.

Penney Building:

Slide (2699) is a general view of the often discussed Penney Building. We are looking at the collapsed northeast corner. It is instructive to take a look at all of the exterior elevations. The next slide (2040) shows the south elevation at the left and the east elevation at the right. The left wall is a blank wall. Movement can be seen along the full length of this wall at the second floor line. The right wall has numerous openings. The one remaining precast wall panel on this elevation has a poured in place wall behind it. Slide (2678) is the west wall which had several hollow concrete block panels where the Penney Building met the adjoining structure. This view was taken during the demolition of the Penney Building.

Slide (2039) shows the front elevation after the debris from the precast concrete facing panels had been removed. It is worth looking at this slide carefully. The roof and floors are of 10" reinforced concrete. There are no beams. So this strufture doe not have a frame. I think that it becomes obvious that a rigid precast concrete skin would have difficulty remaining in place on these flexible supports.

There were no interior shear walls and, therefore, the torsicnal characteristics of this structure were poor. In slide (2042), we note that damage is heavy at the second floor line and that the upper storey walls no longer line up over the first storey walls. The next slide (2048) is a closeup and shows that temporary posts were put beneath the upper storey walls to prevent collapse. Difficulties in this particular wall were compounded by panel anchorage access holes as may be seen in slide (2047). Slide (2050) clearly shows that the net shear area remaining in this wall was low due to these access holes.

Turning again to the south wall, slide (2055) shows a detail of the construction joint. Note the misplaced pocket of grout which should have been evenly distributed along the wall. The clean line along the construction joint clearly indicates that the concrete was not monolithic, probably due to the non-removal of laitance from this joint. It is quite unfortunate that the practice of sandblasting construction joints, or chipping them, in order to obtain good bond, was practically never followed in Anchorage.

Chrysler Center:

The Chrysler Center Building, shown during the course of construction in slide (2696), is typical of the number of buildings having prestressed single tee-beam roofs. The building was about 157' by 73' in plan, and divided into two by a center shear wall. Structural walls were 8" hollow concrete block. The front of this building was structurally open due to the large glass areas, and therefore large torsional forces existed.

Slide (2377) shows the same general area after it collapsed. The next slide (2379) shows the failed connections between the tee-beams. The 1-1/2" thick flange of these tee-beams has been criticized as being too thin from a practical standpoint, and I tend to agree with them. One school of thought has questioned the lack of blocking, or its equivalent, at the tee-beam seats in the glass wall areas.

Slide (2381) shows the damage at the rear of this building. We are looking at a side wall, with the end wall being to our left. The same damage occurred to the opposite wall. This damage is explainable if the roof diaphragm connections failed at the rear wall. This damage is explainable if the roof diaphragm connections failed at the rear wall, thereby allowing the roof tee-beams to punch out the side walls.

We climbed up on the roof. There was no visible damage. So we then removed the wood plate as you see in slide (2382). Here you see the plate turned over and removed from the top of the wall. We were amazed to find that the 1/2" bolts at 8' centers had sheared, but the tar and gravel roofing between the roof slab and the wood plate was undamaged. Here was a case where the roofing was a stronger element than the 1/2" bolts at 8' on center.

At this point, I would like to make some comments about precast roof tee-beams in general insofar as Anchorage experience is concerned. I have looked at most of the buildings, and have studied a number of them in detail. The largest completely undamaged building with a tee-beam roof diaphragm had an area of about 6500 square feet. The only instance of <u>internal</u> diaphragm damage not related to building collapse occurred to the Pepsi-Cola Plant with its diaphragm area of 13,000 square feet. The welded connections between roof tee-beams apparently failed at the Pepsi-Cola plant.

Diaphragm boundary connections were often troublesome, either at the actual connection, or adjacent and within the supporting system. The tee-beams performed excellently as individual structural components, but buildings containing these members did not perform as well as similar area buildings having similar wall materials but different roof or floor systems.

Some standard construction details need more thought and seemingly revision. Certainly, a mesh reinforcing concrete topping slab will add diaphragm reliability in the event that the flange tips are to be kept to 1-1/2" minimums. Too often metal-to-metal connections failed without elongation of the metal parts, and these joints usually were not ductile in the commonly accepted definition of the term.

<u>Selected Bibliography</u>

This bibliography has been selected on the basis of engineering interest. It has been restricted to North American sources. Obviously, important geological and seismological references usually have been omitted unless they supplemented the engineering studies. The papers elected in this bibliography are considered to be the more important of those of engineering interest. The bibliography by E. P. Hollis, listed below, is of particular importance to the student wishing to explore the subject further. The extensive references in Richter's "Elementary Seismology" will also be of value.

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PHOTOGRAPHS

It has not been possible to reproduce all the slides illustrated in the text. The following photographs were selected as representative samples by the author.

No. 781

April 18, 1906

San Francisco, Calif.

No. 776

April 18, 1906

San Francisco, Calif.

No. 1002

March 10, 1933

Long Beach, Calif.

No. 1006

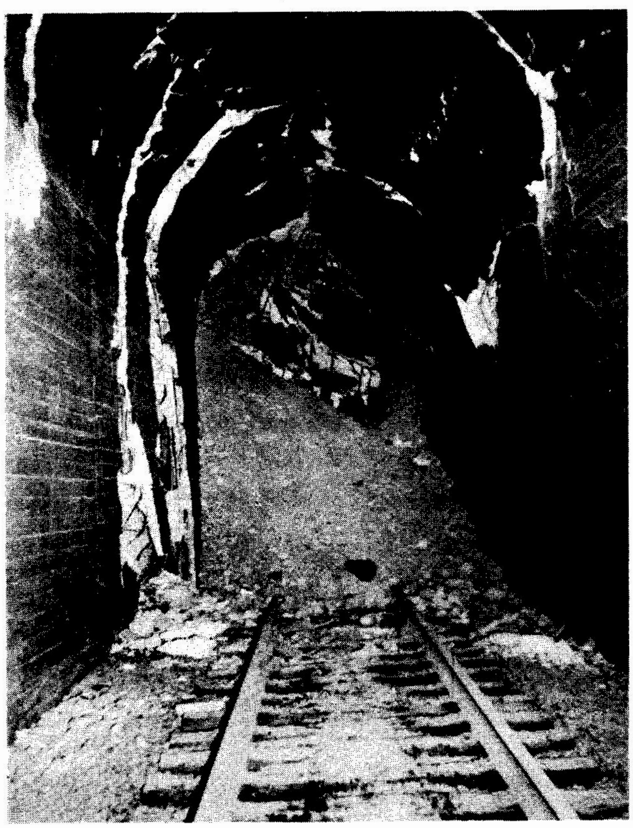
March 10, 1933

Long Beach, Calif.

No. 1007

March 10, 1933

Long Beach, Calif.



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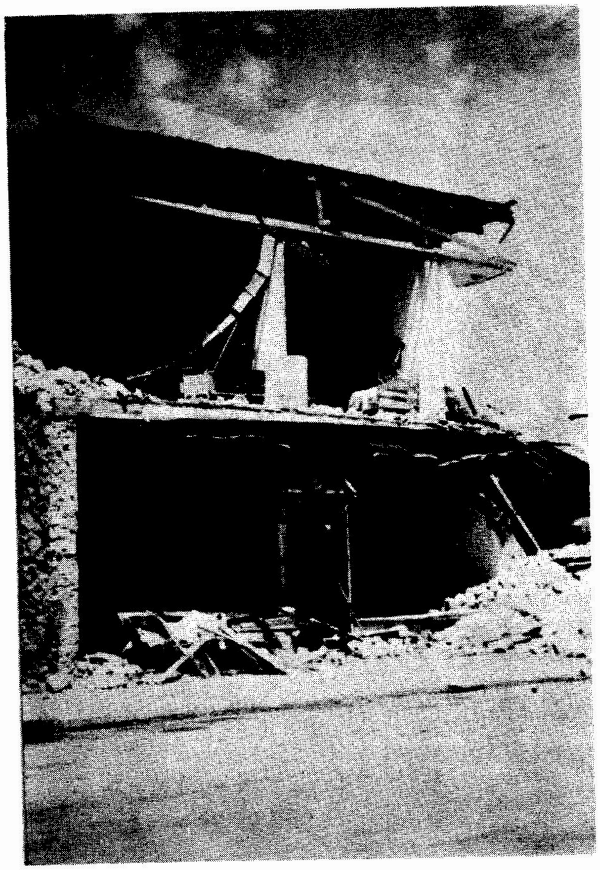
July 21, 1952

Kern County, Calif.



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July 21, 1952

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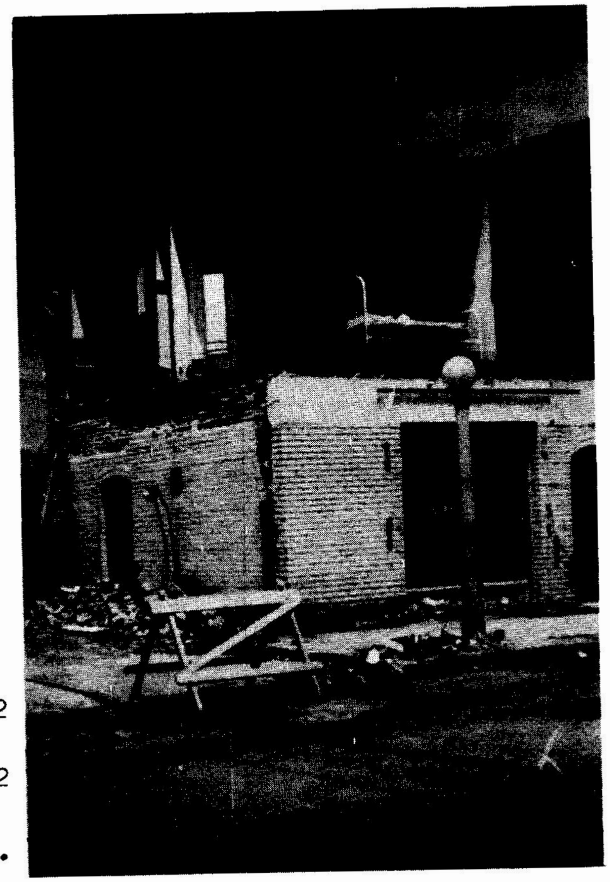

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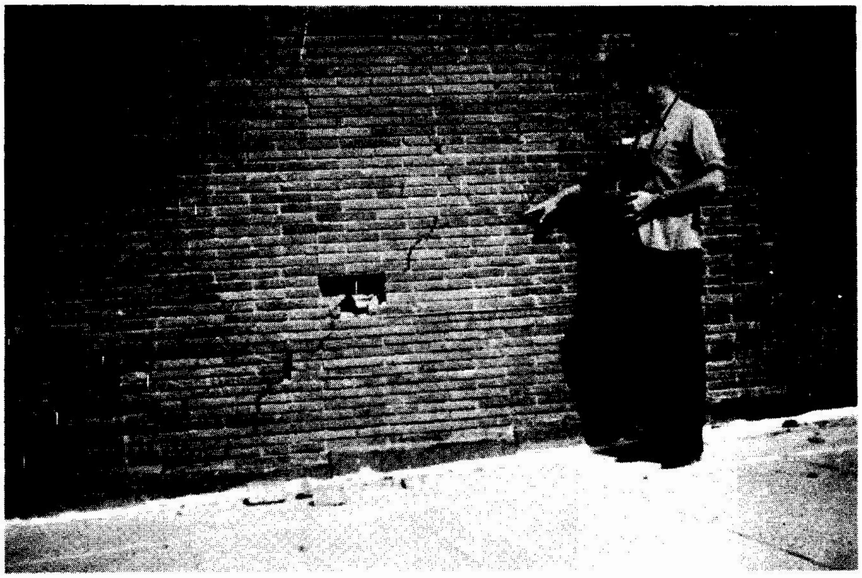

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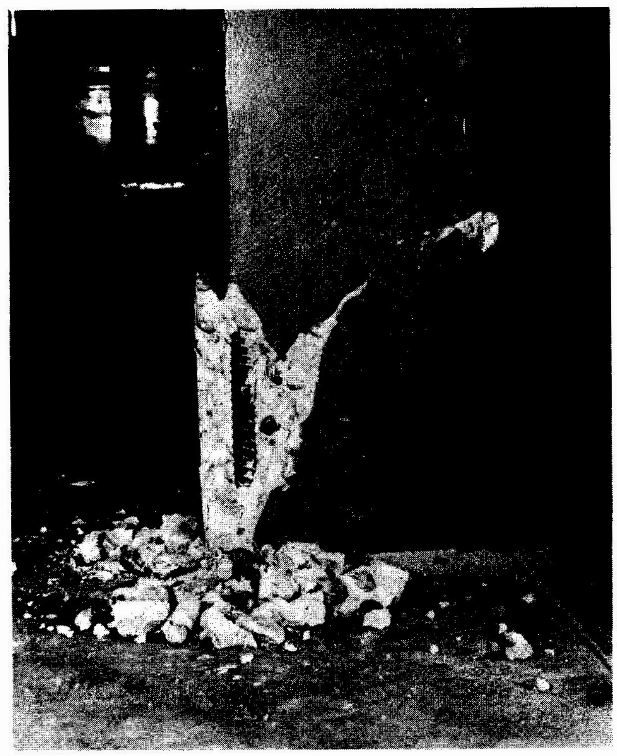

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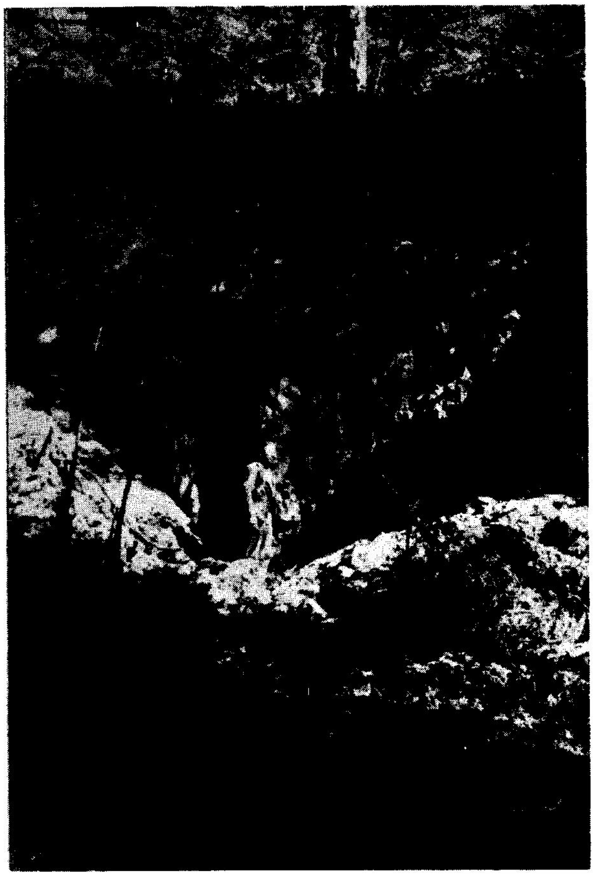
July 28, 1957

Mexico City



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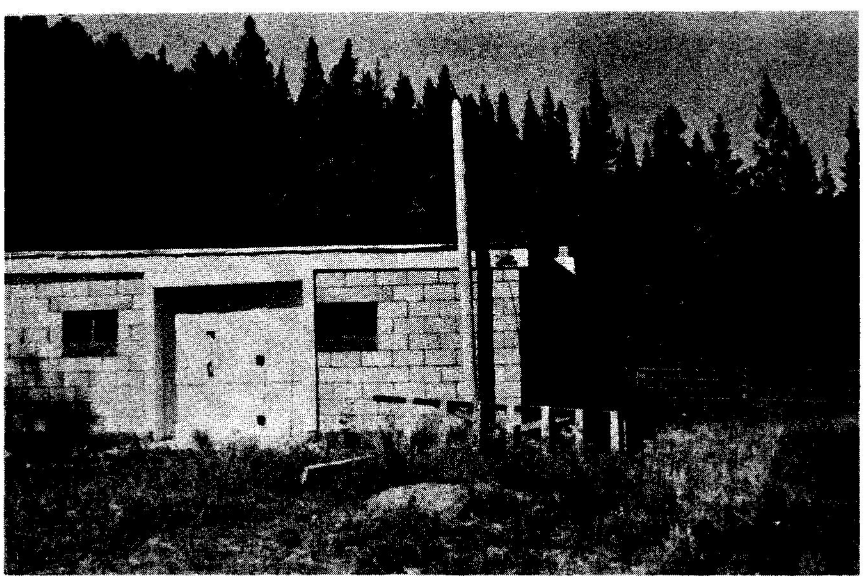
Hebgen Lake, Montana

No. 1510

August 17, 1959

Hebgen Lake, Montana



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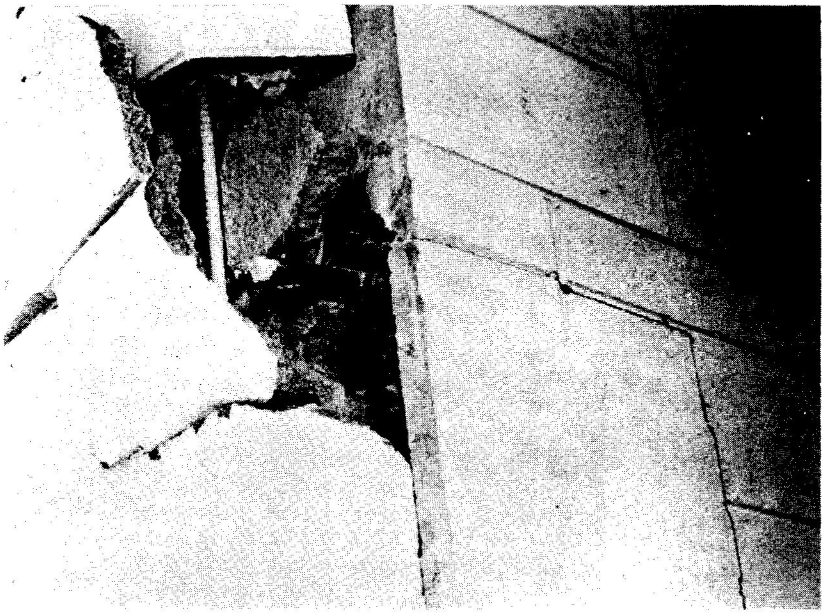

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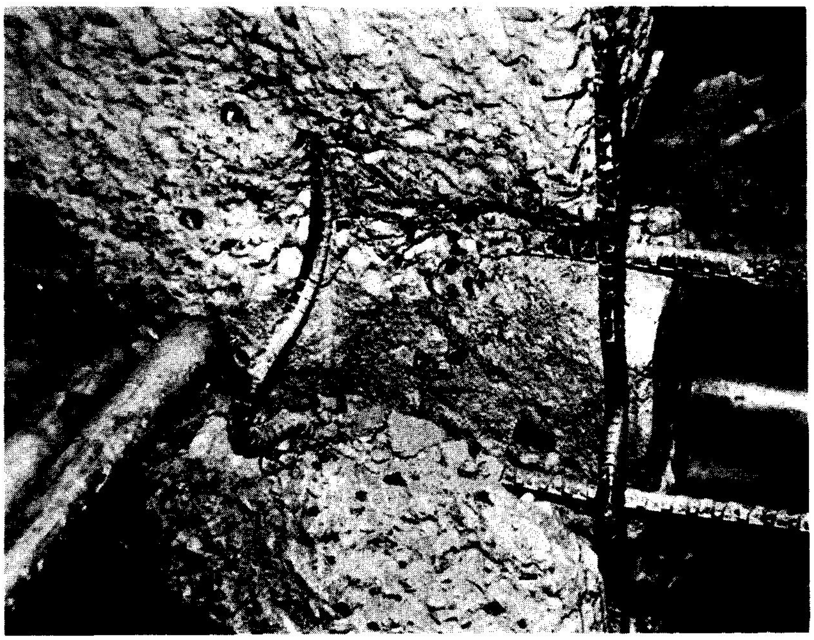

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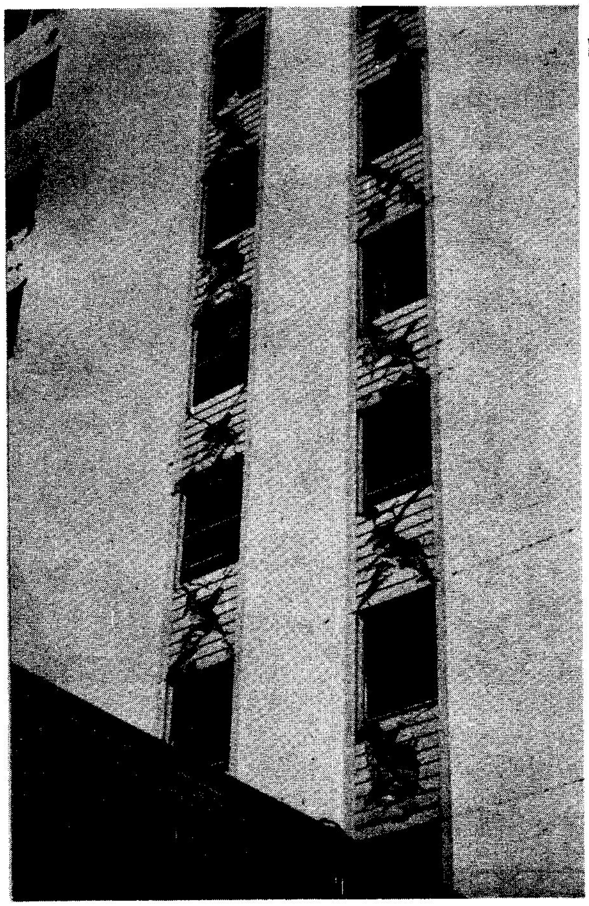
Anchorage, Alaska



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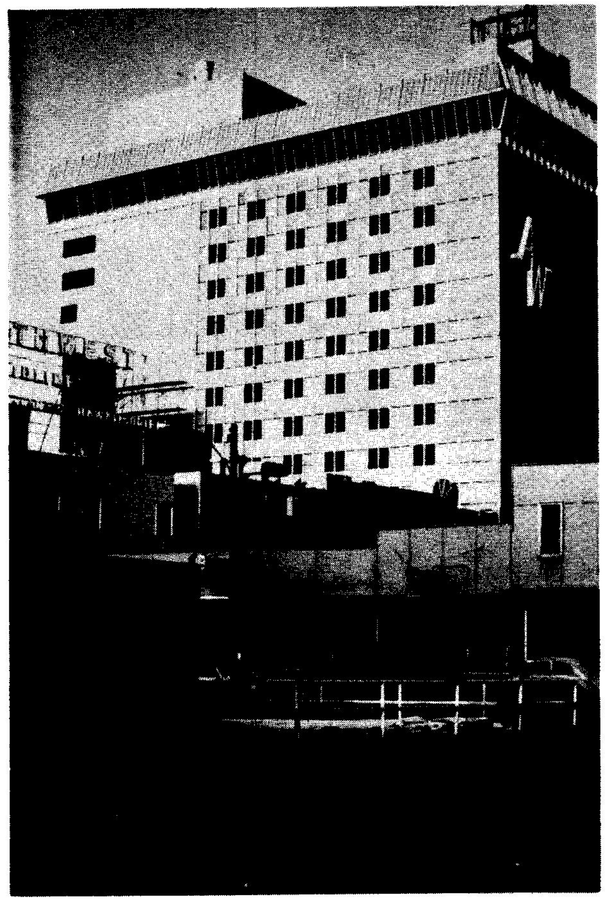
Anchorage, Alaska



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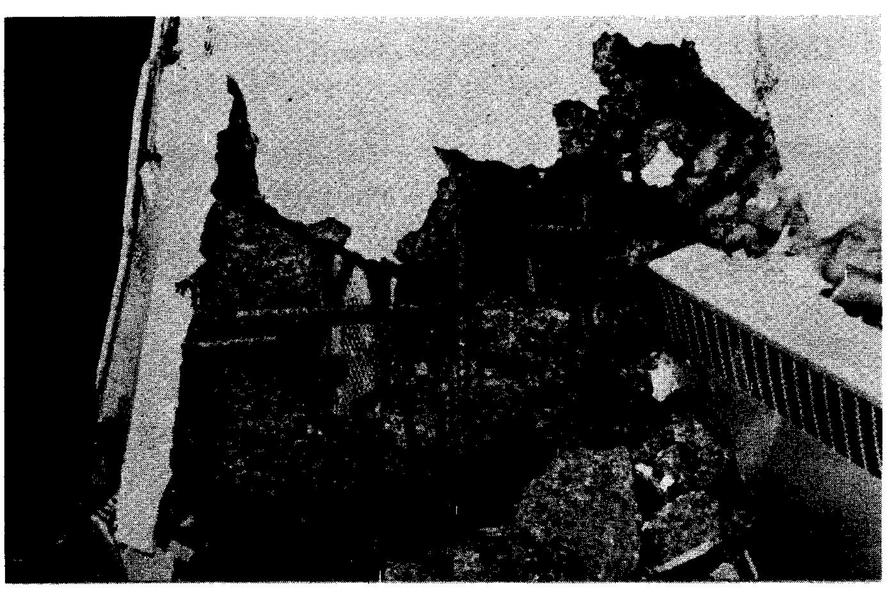
Westward Hotel

Anchorage, Alaska



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March 27, 1964

Westward Hotel

Anchorage, Alaska

No. 2165
March 27, 1964
Westward Hotel Anchorage, Alaska

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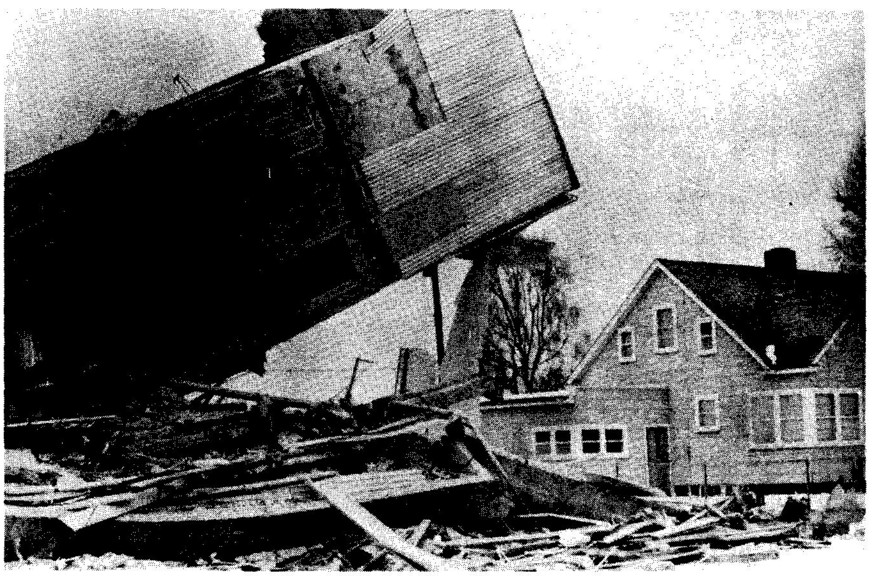
Westward Hotel
Anchorage, Alaska



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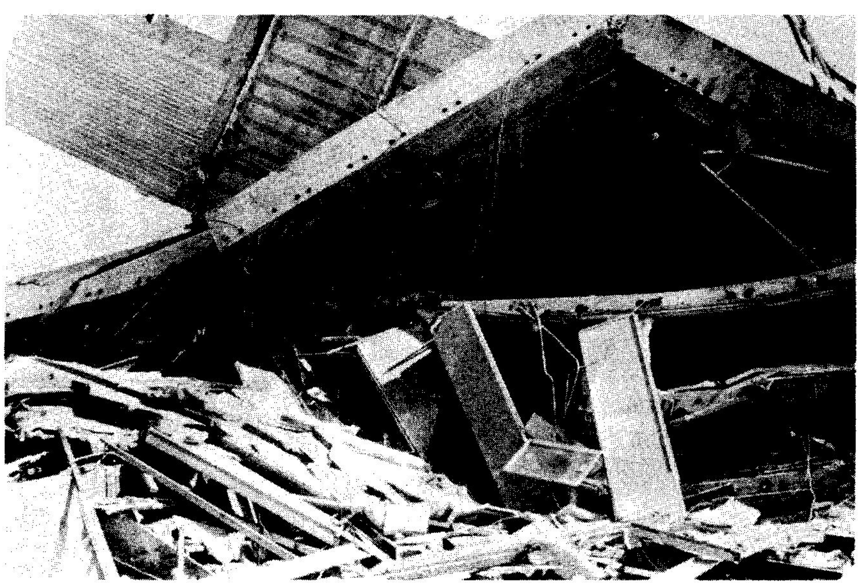

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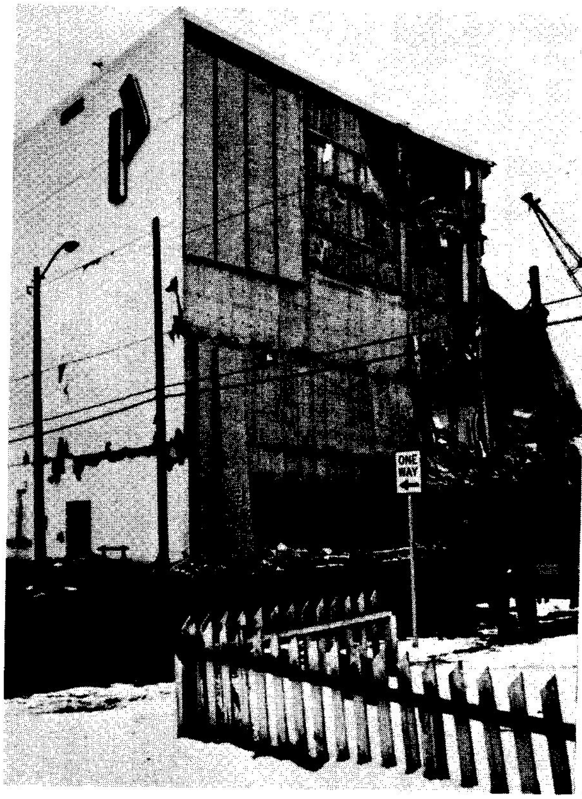

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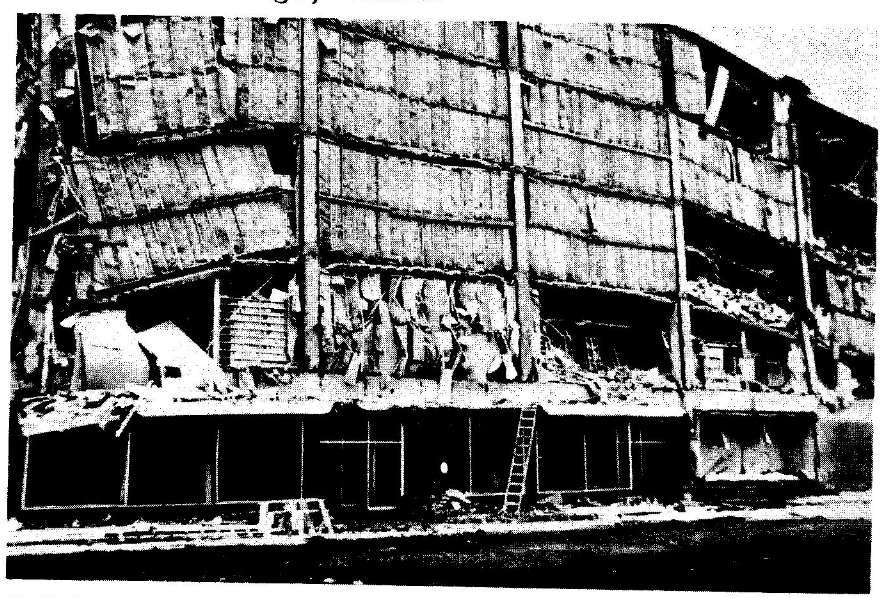

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Penny Bldg.
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No. 2377
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Chrysler Bldg.
Anchorage, Alaska