A PROGRESS REPORT

STRUCTURAL ENGINEERING ASPECTS OF THE ALASKAN EARTHQUAKE OF MARCH 27, 1964

by

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The Alaskan earthquake of March 27, 1964, with its epicenter near Prince William Sound, ranks among the great earthquakes in the world. The earthquake's Richter magnitude has been given by various authorities as follows:

Pasadena:	8.4
Berkeley:	8 1/2 to 8 3/4
Palisades:	8.6
USC & GS:	8.5

Geologically the earthquake is also notable since apparently tens of thousands of square miles either dropped or rose in terms of many feet. Major faulting was found on Montague Island and beneath the adjoining seas in Prince William Sound, indicating that portions of the island had been uplifted by as much as 33 feet.

The earthquake was also very important from the structural engineering standpoint since major buildings in Anchorage, about 75 miles from the epicenter, experienced significant damage. The fact that almost all of these structures had been designed (or were supposed to have been designed) to be earthquake resistive in accordance with the earthquake provisions of the Uniform Building Code gives added importance to the damage studies.

Detailed structural engineering studies of the earthquake damage to buildings in Anchorage, Alaska are in progress at this writing, and the author is involved in these continuing studies. Most of these studies are to be published by the U.S. Coast and Geodetic Survey.

This paper, then is a summary report of a portion of a detailed publication to be published in a year or two.

Compared with other major earthquakes, the loss of life was small. Occurring at 5:36 P.M. on Good Friday, schools were closed and many people were either at home or on the way home. Of the 115 reported on May 4 as dead or missing in Alaska, 59 have never been found, their bodies in most cases having been carried away by seismic sea waves. The greatest life loss, 32, occurred in Valdez, 23 in Chenega, a small native village, 19 in Kodiak, 13 each in Seward and Whittier, 9 in Anchorage, and from 1 to 3 in five other smaller communities. Ten lost their lives at Crescent City, California and 4 along the Oregon coast.

Damage estimates to May 4 were placed by the Federal Office of Emergency Planning at $537.6 million, of which $74.6 million is estimated damage to federal facilities, $244 million to state and local public facilities, and $219 million to private property. Roughly 60% of the total damage, by preliminary estimates, is believed to have occurred in the Anchorage area, perhaps 10% in Seward, Valdez, and Kodiak combined, and the remainder elsewhere.

Broadly speaking, damage and life loss in the Alaskan earthquake may be studied from two standpoints: (A) Seismic sea wave or "Tsunami" effects, and (B) Earth vibration effects. The principal population center in the heavily shaken area, and the location of the greatest number of buildings, was the metropolitan Anchorage area and therefore the remainder of this paper is being restricted to metropolitan Anchorage. The seismic sea wave was not significant at Anchorage, and damage principally was the result of ground vibration.

LANDSLIDES IN ANCHORAGE:

A pre-earthquake study of foundation conditions in and around Anchorage is found in U.S. Geological Survey Bulletin 1093 "Surficial Geology of Anchorage and Vicinity, Alaska," by Miller and Dobrovolny, published in 1959. This publication describes the sands and gravels found on the ground surface for much of Anchorage, and it points out that these sands and gravels usually cover a material known as Bootlegger Cove clay. The authors stated that caution should be used in founding large structures on this clay. Also pointed out were the numerous areas covered by landslides, earth slumps, or earth flows, and these were mapped and published. With respect to these slide areas, they state on page 103 "...The Bootlegger Cove clay--an unstable material when wet, that can be dislodged by some triggering action--underlies stratified sand and gravel" Their viewpoints, published 5 years before the earthquake, were prophetic.

Building damage was greatly intensified in the areas where landslides occurred or in poor ground areas which were badly fissured. However, it should be pointed out that the landslide areas were highly localized, and only by chance did portions of the business, industrial, and residential sections of Anchorage experience landslide effects. Large sections of the city and its surrounding area, although underlain by the Bootlegger Cove clay, were (and are still considered to be) quite safe for structures when using commonly accepted engineering design methods.

The damage due to landslide and earth movements was spectacular, and tended to overshadow the less spectacular damage. Typical dwelling damage as a result of landslide in the Turnagain residential area is shown in the accompanying figures.

The "L" Street land movement was principally a horizontal movement towards Cook Inlet. Many buildings located on this shifted land mass were essentially undamaged. Buildings located at the boundary between the shifted land mass and the stable land mass were severely damaged.

The Fourth Avenue landslide occurred in part of the business district. The buildings were total losses and have been demolished.

A very thorough soils engineering study of the earthquake induced landslides in Anchorage has been made by the firm of Shannon and Wilson for the U.S. Army Engineer District. This "Report on Anchorage Area Soils Studies, Alaska" is a classic in its field. The following is quoted from its summary:

> Most of the Greater Anchorage area is built on a sand and gravel outwash deposit overlying a thick stratum of Bootlegger Cove clay. The upper and lower zones of this stratum are fairly stiff and competent, but the central portion is weak and sensitive and contains lenses and layers of loose sand. The strong ground motion waves from the earthquake were transmitted through underlying compact gravels and tills and generated shear waves in the Bootlegger Cove formation.
>
> It is probable that amplification of motion developed as a result of resonance, and that maximum shear strains were produced at the upper boundary of the weak and sensitive clay stratum. Where the upper zone of this clay also contained layers of loose sand, failure developed by liquefaction in these layers. In layers where sand layers were not present, failure developed as a result of shear stress reversals in the sensitive clay itself.

Space limitations do not allow a detailed review of this excellent report, but the report is certainly mandatory reading for all wishing to know more about soil effects as a result of earthquake motions.

Ground Motion:

The type of ground motion has a major effect on the damage pattern. It is well established that earthquake ground motions change their characteristics as they travel from their source. The amplitudes of the short period motions decrease much more rapidly with distance than do the longer period motions. This means that large earthquakes have motions at a relatively long distance from their epicenters (say 100 miles from the energy release) which are often described as a "rolling motion." The amplitude of the motion and its other characteristics may be greatly modified by the local geology. The epicenter of the Alaskan earthquake was about 75 miles from Anchorage. No strong motion instruments were in Anchorage, so the ground motions are not well known. However, available data strongly indicate that periods of ground motion 0.5 seconds or greater were predominant as compared to those less than 0.5 seconds. As a result, the earthquake tended to selectively damage certain classes of construction while leaving other classes relatively undamaged.

The duration of violent shaking is, of course, also a major factor in the damage pattern. The longer the duration, the greater the cumulative damage. The duration of the strongest shaking at Anchorage has been estimated at certainly not less than one minute, or possibly 3 or 4 minutes according to other sources.

When ground shifts and foundation failures were not a significant factor, earthquake damage was far more pronounced in the tall buildings than in the low buildings. Large floor area buildings were more often damaged than were small floor area structures, all other factors being equal.

The foregoing damage pattern is consistent with the type of ground motion usually experienced at a long distance from the source of energy of a major earthquake. The longer period ground motions in Anchorage tended to cause quasi-resonance with multistory buildings and large area buildings; this resulted in larger earthquake forces in the taller or larger structures. Exceptions, of course, would be poorly built buildings or very well-built buildings. Local geology, too, undoubtedly influenced the ground motions and caused variations in damage patterns. However, on a percentage basis, the tall structures received more damage than did the small buildings.

Building Damage:

Most of the principal buildings in Anchorage have been built since World War II. Currently, and during most of this period, the Zone 3

(most restrictive) earthquake requirements of the Uniform Building Code have been in force. When the earthquake provisions were first adopted, plans were sent out of state for review for building code compliance. In the last few years, this review has been done within the Anchorage Building Department. Therefore, the bulk of the major buildings in Anchorage should have had earthquake resistive design and construction.

Earthquake damage may be conveniently described by class of construction material, such as wood frame, all metal, etc. Historically, this has been an accurate method for classifying structures in accordance with their probable earthquake damage when not dealing with earthquake resistive construction.

Wood frame dwellings in Anchorage performed excellently when foundation failure was not a factor. Brick or hollow concrete block chimneys, in general not reinforced, often were undamaged. Houses, although often not well anchored to their masonry foundations, rarely shifted from these foundations. In various sections of Anchorage, dishes and similar items did not fall from shelves. Elsewhere in Anchorage, most objects were thrown from shelves. The foregoing indicates that, although the earthquake's intensity as measured by this type of damage was quite variable, the performance of wood frame structures was superior when located on firm ground.

All-metal buildings performed excellently as may be typified by the usual absence of damage to all steel gasoline service stations.

Some damage, however, was noted in all metal structures having large elevated masses. Equipment shifted, one bin fell, X-bracing broke, metal skin buckled, and steel columns twisted at the Chugach steam power generating plant located in the Ship Creek section of Anchorage; differential settlements indicated that foundation problems probably accentuated this damage. Nearby, the Elmendorf Air Force Base power plant had its principal damage restricted to the steel connections from a large elevated bin to the main structure. Overall damage to both power plants was estimated to be slight, and overall equipment damage probably could also be classified as being slight. A third power plant, owned by the City of Anchorage and also located in the Ship Creek area, had apparently negligible damage.

Hollow concrete block was a common construction material for small mercantile structures as well as for industrial structures. Roof and supported floors were usually wood. When small in area, when one story high, and when not located in the land movement areas, such damage as sometimes occurred was usually no more than slight to moderate. Some parapets fell, unanchored roofs punched out sections of the hollow concrete block walls, but collapse was uncommon. Wire webbing for reinforcement was usually laid in selected horizontal

joints of the hollow concrete block. The placement of vertical steel was inconsistent. This reinforcement, in general, appeared to be of a size and amount that would not be considered to be fully adequate in other sections of United States that are considered seismically very active. Workmanship was often poor where collapsed walls were noted; the concrete grout did not fill all of the cells in many observed instances.

Most of the foregoing observations of workmanship were made in connection with hollow concrete block structures which collapsed in the slide or land movement areas. It is reasonable to expect similar workmanship in the other buildings which survived in the largely undamaged sections of Anchorage elsewhere. While the earthquake was not of a type to create the heaviest forces in this kind of low rigid building, it was quite evident that the reinforcing steel played a major role in minimizing the life hazard and reducing property damage.

Poured-in-place reinforced concrete wall construction performed well for small buildings. Usually the roof and supported floor materials were wood for the small buildings, although not always so. Instances of metal deck and metal open web joist roof and floor systems were found. The performance of these structures was generally good and somewhat better than similar size structures of different masonry materials when not located in the land movement areas.

Penney Building:

The 5 story Penney Building was nearly square in plan (about 130 feet by 150 feet) and was entirely of reinforced concrete construction. The nearest point of the Fourth Avenue landslide was approximately 450 feet to the north, although cracks were found in the street pavement about 100 feet to the north and these cracks appeared to be associated with the landslide. No ground fractures were found beneath or around the Penney Store. Foundations were of the reinforced concrete spread type, with the bottom of the footings being about 4 feet below grade.

Roof and supported floors of the Penney Building were 10-inch reinforced concrete flat plates. Maximum roof and floor panel size was 22 feet by 26 feet. The reinforced concrete slabs were usually supported by 20-inch by 20-inch square reinforced concrete columns, except where bearing walls existed. The building, then, did not have a frame. Exterior structural walls, in general, were 8-inch reinforced concrete, although some 10-inch and 11-inch reinforced concrete walls existed and several 8-inch and 10-inch hollow concrete block walls were found on the west elevation. The two street fronts were faced with 5-inch precast reinforced concrete panels having exposed aggregates.

The precast panels were installed after the poured in place walls had been completed.

Lateral force bracing in the Penney Building was provided by the exterior reinforced concrete shear walls, plus several small reinforced concrete interior walls. The shear wall bracing system for lateral forces was reasonably symmetrical in the first story, but the north elevation was structurally open in the upper stories. The absence of north elevation bracing in the upper stories resulted in heavy rotational forces.

Earthquake damage was severe to the Penney Building and the building was torn down to its foundations before being rebuilt. An examination of the accompanying figures shows that the bulk of the damage occurred above the first story. Precast walls fell, shear walls collapsed, and columns were sheared. Also, major movements occurred along the construction joints, principally at the second floor level.

It is of interest to examine the precast concrete panel anchorage to the shear walls (see accompanying figure). Slots, or holes, were left in the shear walls to allow anchorage of the precast panels to the floor slabs. The holes were not grouted after the anchorage had been completed. It is obvious that the holes and slots greatly reduced the shear resistance of these shear walls.

South wall damage was largely associated with movement along the second floor construction joint. The wall movement along the second floor construction joint followed the pattern commonly found in Anchorage. The laitenance had not been effectively removed and the concrete wall was not truly monolithic.

The mechanism causing failure can be explained by torsional forces. Torsional forces were not a significant factor in the first story since shear walls were found along all street fronts. The upper stories, however, had a structurally open north wall, and large torsional forces would arise from the U-shaped shear wall bracing system when subjected to east-west lateral forces. These torsional forces gave the highest unit stresses in the second story.

It is appropriate to point out at this time that the conventional design methods for torsional analysis are not necessarily on the safe side. Conventional analysis places the center of rotation in cases such as these at or near the wall parallel to the open front. A more rigorous analysis would place the center of rotation at or near the shear center which would be some distance away from the conventional center of rotation. Additionally, theoretical dynamic analyses have shown that the static method underestimates significantly the magnitude of the maximum stresses.

Four Seasons Apartment Building:

The Four Seasons Apartment Building was in the last stages of construction at the time of the earthquake, with only the non-structural elements being incomplete. The building was located very close to the main graben of the "L" Street landslide. The ground cracking near and through the site is evidence of ground motions different and possibly more severe than that several blocks farther away on the more stable ground. However, this presumed greater intensity of ground motion was not apparent in the nearby wood frame dwellings since practically all of them had no more damage than those several miles away when not in the graben area or associated with other landslide manifestation.

The Four Seasons Apartment building was six stories high, plus having one basement used principally as a garage. The roof over the basement garage was poured in place prestressed concrete joist with a 3 1/2 inch prestressed concrete roof slab. The prestressing cables were 1/2 inch in diameter. The basement areas were damaged by the collapse of the 6 story structure, but suffered little if any direct earthquake damage.

The supported floors and roof of the six story tower section were of prestressed lift slab construction. Columns were structural steel in the 10 inch wide flange series. Column loads were transferred from the prestressed concrete slabs by means of special shear heads. The prestressed 8-inch thick floor and roof slabs were post-tensioned with draped tendons before being jacked into place. The tendons were not grouted after being tensioned. The prestressing was designed for dead loads plus a 10 psf live load with zero deflection, and no tension in the final prestress. Both principal axes of each slab were prestressed. The slabs were cast in two sections, having a reinforced concrete pour strip tying them together after the sections were jacked into place.

After the slabs had been jacked into place, the two core towers were constructed of poured in place reinforced concrete. The walls of these two core towers had been completed, and therefore the earthquake bracing was complete at the time of the earthquake.

According to persons observing the collapse, the actual collapse came at or near the end of the long duration earthquake. The collapse may have occurred at about the same time the "L" Street graben was formed.

An examination of the collapsed structure showed that the floors came down first, with both core towers overturning to the north after the floors were partly or mostly down. The reinforcing bar dowels, tying the floors to the core walls, sheared off in some cases. In other cases, the dowels broke free from the prestressed concrete slabs. Many prestressing cables in the floor slabs broke, and portions of the cables and their anchorages flew away from the building for varying

distances. There are conflicting accounts regarding the sequence of events in the failure of the prestressing cables, and no firm proof has been found to resolve this matter.

The two core towers which provided the earthquake bracing failed at the first floor level. It is readily conceivable that the structure experienced at least several hundred cycles of motion, and any minor cracks which formed during the first few cycles probably would become major cracks from this long duration motion.

Since it is reasonable that the primary failure occurred at the first story level to both core towers, it is of some importance to examine the stresses at this level. The evidence on hand indicates that the dowels did not shear. Tensile stresses did not cause necking on the few bars which could be observed prior to demolition. Evidence after demolition shows scraping marks in the slab, indicating that the steel which lapped the dowels tended to remain intact. The field evidence suggests either concrete compression failure or bond failure at bar splices.

There has been considerable discussion regarding the advisability of using unbonded tendons in prestressed slabs. It is maintained by some that bonded tendons provide a second line of resistance and the bonding would have prevented the loss of prestress at the Four Seasons Apartment Building. While undoubtedly improving the slab's factor of safety, it appears to be speculative to state that bonded tendons would have avoided collapse.

Hill Building:

The eight story Hill Building is located about 1100 feet from the 4th Avenue landslide and about 1200 feet from the "L" Street landslide.

The central core of this building is of poured in place reinforced concrete. The core walls are bearing walls and carry the loads from the steel beams which frame into these walls. All walls have only a single curtain of steel. Except for the core, this building has a structural steel frame. Plaster fireproofing was used throughout on the structural steel. Upper story floors are 5-inch thick one way slabs of poured in place reinforced concrete which are supported by the structural steel beams.

Exterior walls are 3-inch insulated porcelain enamel metal panels. Minor exceptions of hollow concrete block may be found in the first story.

The lateral force bracing system is simple in its concept. The 5-inch reinforced concrete floor slabs act as horizontal diaphragms

taking lateral forces to the central core walls. The core walls act as shear walls as well as bearing walls. The central core is so located as to minimize torsion.

There is no secondary lateral force bracing system. The steel beams are framed to the steel columns by web angles and only negligible moment resistance can be developed without excessive frame deformations.

The foregoing description is that of a structure of a relatively common modern architectural and engineering design, and one that has a common and acceptable lateral force bracing system. It therefore becomes of some importance to understand the earthquake performance of this building.

The principal structural damage was found in the central core. The reinforced concrete walls had their principal damage in the first story, with damage being progressively less in the upper stories. The damage pattern in the reinforced concrete central core may be divided into 4 distinct categories:

A. Failure of the concrete walls of the west unit of the central core below the first story floor slab.

B. Slippage along construction joints in the core walls.

C. X-cracks in the core walls as a result of diagonal tension stresses, and possibly overturning stresses.

D. Damage to the beams between the two units of the central core.

The most significant item of those listed above was the failure below the first floor slab in that it played the major role in the other damage. This failure in the west unit below the first floor slab was due to a horizontal belt of extremely poor quality concrete. This concrete did not fracture, rather it "mushed" out. The reinforcing steel acted as columns and buckled under compression loading. Laboratory analysis of this "mushed" concrete indicated excessive organic material which had resulted in extremely low strength concrete.

Other Buildings:

Space does not permit as lengthy a discussion on other major buildings which are of equal interest to those previously described, and therefore the following structures must be described in summary terms.

Two almost identical 14-story apartment buildings, the Mt. McKinley Apartments and the 1200 "L" Street Apartments, are interesting examples of damage. Exterior walls are reinforced concrete, and are bearing. The walls around the stairs are reinforced concrete as are interior columns. Floors are reinforced concrete and span up to about 18 feet. The earthquake resistance was provided by the exterior bearing walls and by the interior reinforced concrete walls, all acting as shear walls. Damage was usually in the form of X-cracking in the reinforced concrete spandrel walls; this is typical reinforced concrete shear failure. The vertical alignment of shear failures, termed for convenience "vertical shear," was particularly noticeable in the stories located in the middle third of each face of each exterior wall. The interior walls did not suffer as extensively as did the exterior walls. Building corners were supported by cantilever exterior walls, and these corners suffered extensive damage. Floors appeared to have suffered relatively little damage. One major bearing wall in each building completely failed in a lower story. Both buildings had a similar orientation and had very similar damage patterns although located about one mile apart. Shaking machine tests are in progress at this writing, and many additional data are to be expected in the near future.

An example of a low reinforced concrete building which partially collapsed is the West Anchorage High School. Principal damage was to the 2-story classroom section which was structurally separated from the adjoining auditorium and gymnasium. Shear walls failed in the classroom wing, probably then followed by column bending failure in this wing. Extensive pounding damage occurred between structurally separated units. Eye witness accounts tell of major building oscillations before the partial collapse occurred.

Precast reinforced concrete construction had perhaps more than its share of failures. Failures were almost invariably at the connections between precast elements. The Alaska Sales and Service Building is an interesting one-story failure in this category, having a precast prestressed reinforced concrete "T" roof system on precast reinforced concrete bents. Walls were also of precast concrete. Interconnections between precast elements were usually made by means of welded metal connections. The building was in the final stages of construction. Large sections of the building collapsed while much of the remainder was out of plumb and on the verge of collapse.

Other precast reinforced concrete buildings, or partially precast reinforced concrete buildings, also had serious damage. A precast prestressed "T" roof system on hollow concrete block supports partially collapsed at the Chrysler Center automobile sales agency.

While the interconnections between the precast elements failed, or the connections from precast elements to their backing wall failed, these failures were not always the primary cause of the collapse. For example, the hollow concrete block supports for the precast "T" roof at the Chrysler Sales apparently gave way first, with the spectacular roof damage as a consequence. Other cases could be cited. But whether the precast element failure was a primary or consequential failure, it was almost invariably the connections which were the weakest element.

The 14-story Anchorage Westward Hotel was built in two stages, with the 6-story second stage almost ready for occupancy at the time of the earthquake. It had essentially a complete steel frame, except that several reinforced concrete walls were designed, at least in part, to take column loads. Exterior walls were metal insulated panels ("metal skin"), except for the rear wall and a portion of one side wall which were reinforced concrete. Floors were metal deck with a reinforced concrete wearing surface. The earthquake resistance was provided by the exterior and interior reinforced concrete walls acting as shear walls. The metal skin had sufficient flexibility so that very little damage appeared from the exterior, but significant structural damage occurred to the more rigid structural elements within the building as may be seen by the figures.

The 6-story Cordova Building also had a steel frame and interior reinforced concrete shear walls. Damage was serious along the south wall to both the concrete "skin" and to the steel frame. The failure of the north reinforced concrete wall was largely due to faulty construction joints. The severe damage to the steel column in the southeast corner is interesting and was the primary result of a stair landing framing into the column, thereby greatly increasing the column's stiffness and the lateral force going to the column.

By no means have all of the interesting buildings been reviewed. Some, such as the new Providence Hospital, were notable for their lack of major structural damage.

Summary and Conclusions:

In summary, and with reference to the multistory buildings, a majority of the tall buildings had significant structural damage. The damage to reinforced concrete shear wall buildings was often found in the spandrel walls between shear walls, with this damage having a vertical alignment. It would appear obvious that this type of stress has been given too little design attention. Construction practices were usually a factor in the damage as well as design practices. Concrete construction joints often failed due to a complete lack of bond between the two concrete pours. Laitenance was rarely removed in construction joints, and as a result the concrete was not monolithic.

In one building, the concrete aggregate in the construction joint probably acted something like ball bearings during the earthquake. Undoubtedly, poor construction supervision was a major fault in the failures. The extensive damage to multistory reinforced concrete and structural steel buildings is in contrast with the relative lack of damage to low rigid buildings having small floor area.

The earthquake damage in Anchorage was selective, generally damaging the tall buildings more severely than the low buildings. This damage pattern is attributable to the distance that Anchorage was from the epicenter, with the longer period ground motion having a dominant effect at this distance.

The effects of soils under earthquake loading need much more study, particularly with respect to their relation to structures.

The need for experienced competent engineering judgment on general design and details of earthquake resistive construction is apparent. The earthquake provisions of a building code can be nothing more than a guide to minimum standards, and these standards may be quite inadequate for special situations.

Precast reinforced concrete performed rather poorly in too many instances. However, failures were almost inevitably associated with the connections to the precast elements. It would appear obvious that the material was usually not at fault, rather a lack of sound engineering judgment was the principal cause. There was no evidence to indicate that one construction material was superior to another when given comparable design and construction attention.

Fig. 1. Downtown Anchorage from the air after the earthquake.

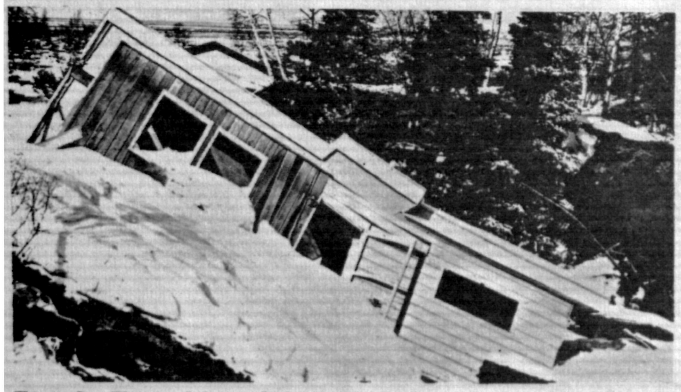


Fig. 2. Typical landslide damage to a wood frame dwelling in the Turnagain section of Anchorage.

Fig. 3. Fourth Avenue landslide damage. Anchorage-Westward Hotel in the background.

Fig. 4. Photograph taken in a store across the street from the damage shown in Fig. 3. Items on the shelves have not been rearranged since the earthquake, indicating low earthquake intensity if long period effects are neglected.

Fig. 5. Typical undamaged wood frames dwellings. This type of structure was rarely damaged except for chimneys. Landslide was also an obvious exception.

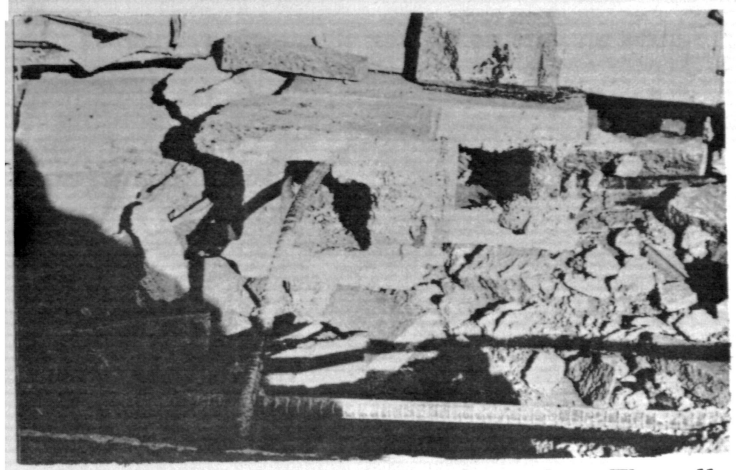


Fig. 6. Example of defective workmanship. The cell containing the steel had never been filled with grout, and therefore was not earthquake resistive.

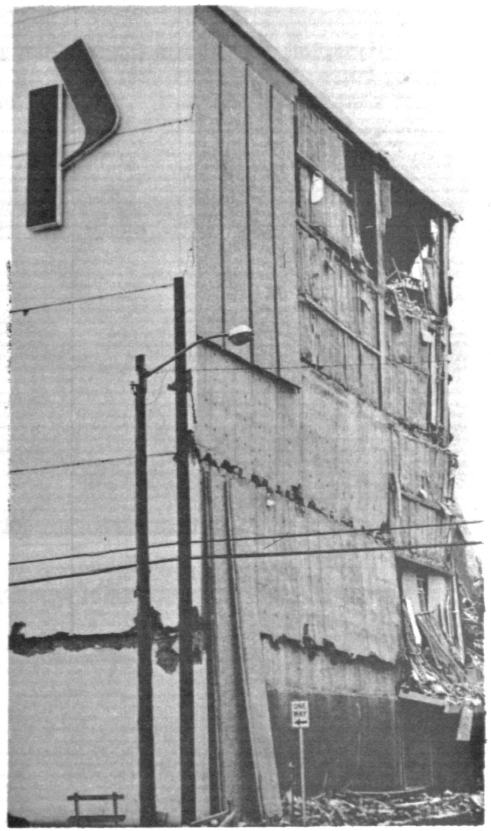


Fig. 7. Penney Store, southeast corner. Wall on the right had been faced with 5" precast concrete panels which failed during the earthquake. Note wall offset at 2nd floor level.

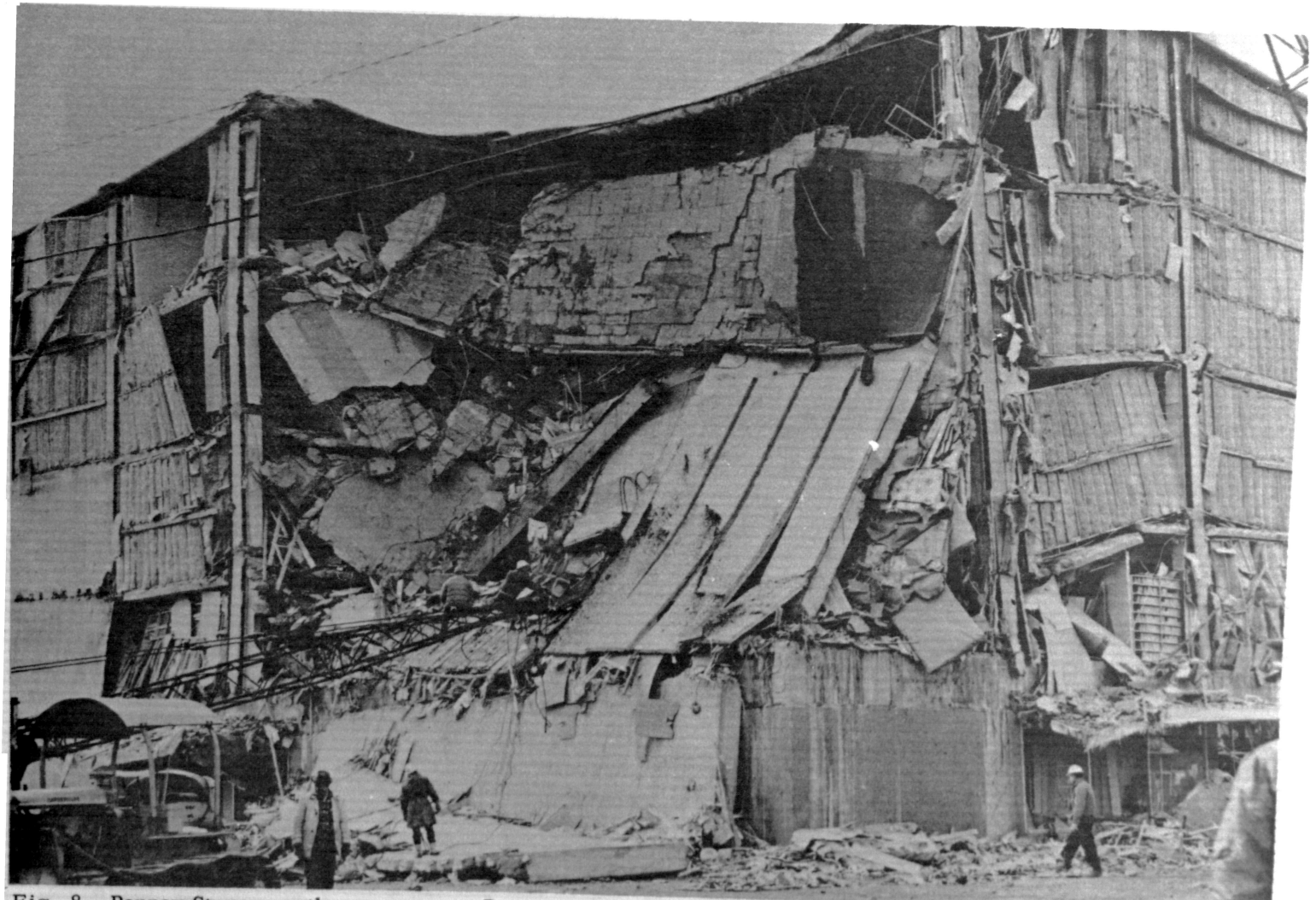


Fig. 8. Penney Store, northeast corner. Precast concrete facing panels failed. Building does not have a complete frame.

Fig. 9. Hill Building. Insulated metal panel "skin" not damaged. Damage occurred to the rigid reinforced concrete central core walls which are located directly below the damaged penthouse.

Fig. 11. One of two toppled core towers at the Four Seasons Apartment Building.

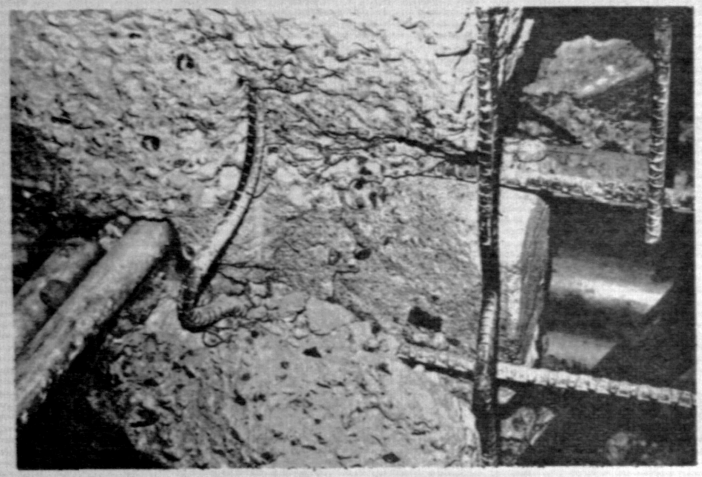


Fig. 10. Hill Building. Concrete in core wall below first floor failed due to its exceptionally poor quality, causing core walls to rotate and damage partitions.

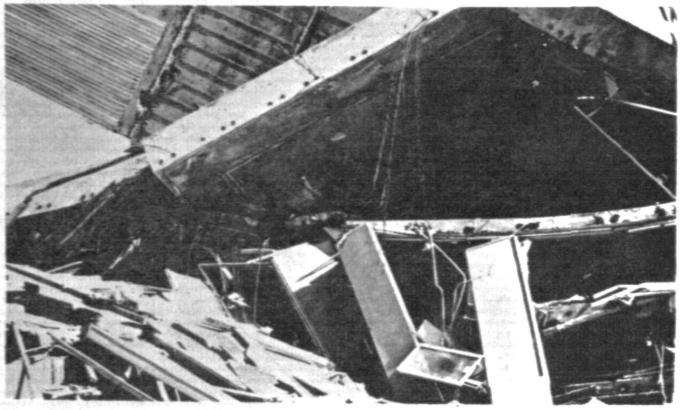


Fig. 12. Stacked floor slabs at Four Seasons Apartment Building. Note failed prestressing rods.

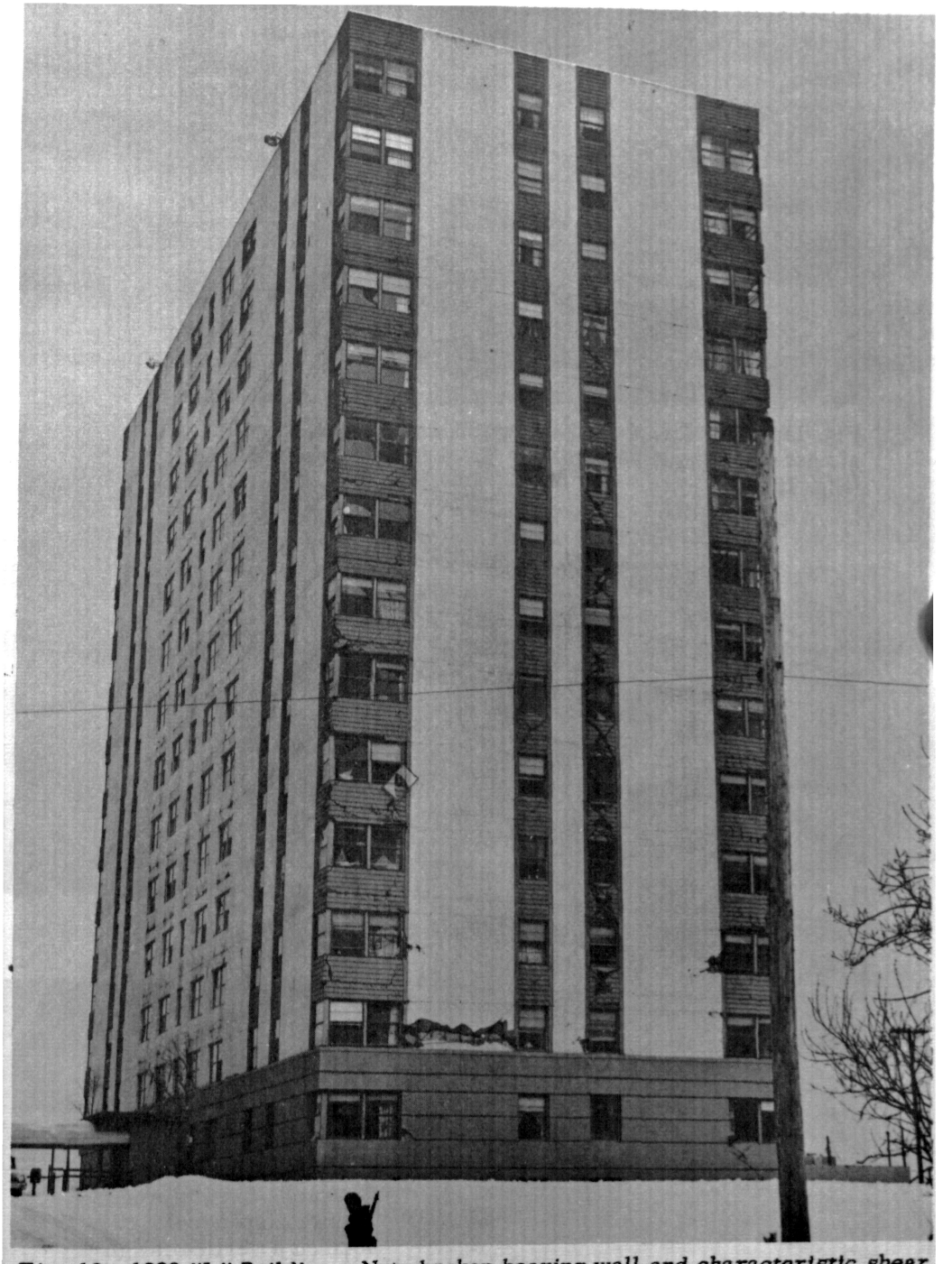


Fig. 13. 1200 "L" Building. Note broken bearing wall and characteristic shear X-cracks in the walls. --Bureau of Land Management photo.

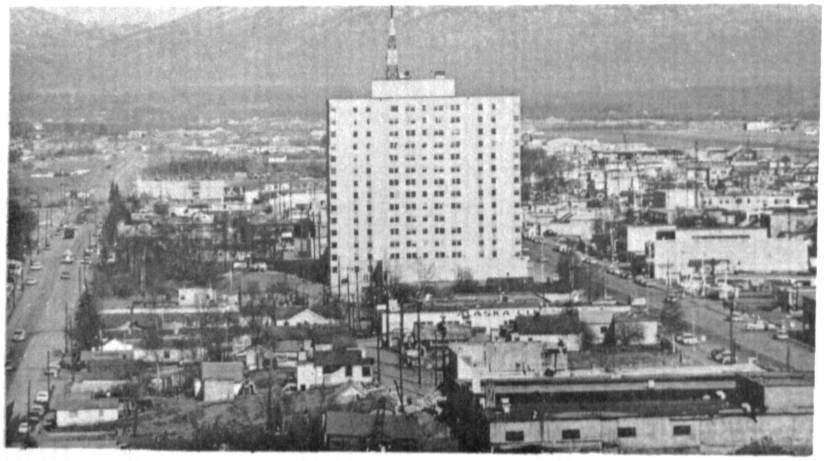


Fig. 14. Mt. McKinley Apartment Building. Structurally almost identical damage to 1200 "L" Apartment Building (Fig. 13).

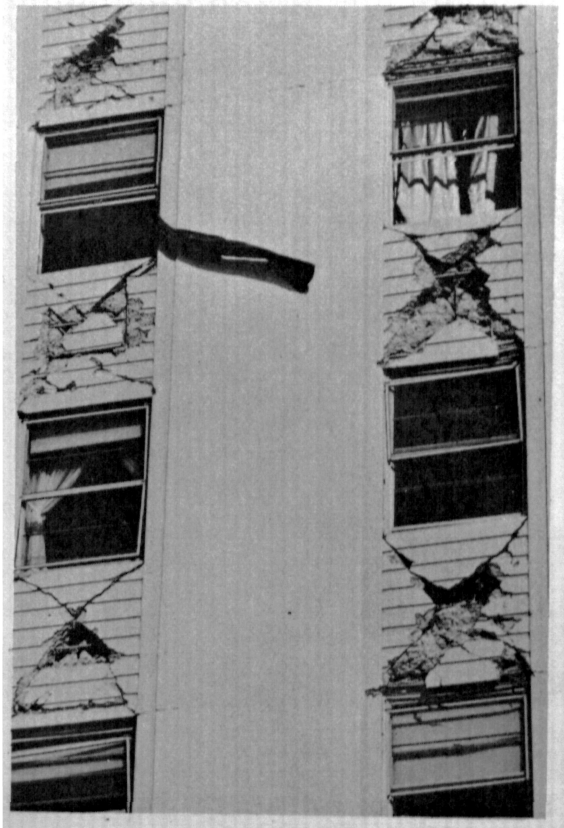


Fig. 15. Characteristic X-cracks in reinforced concrete spandrel walls formed a vertical alignment of damage.

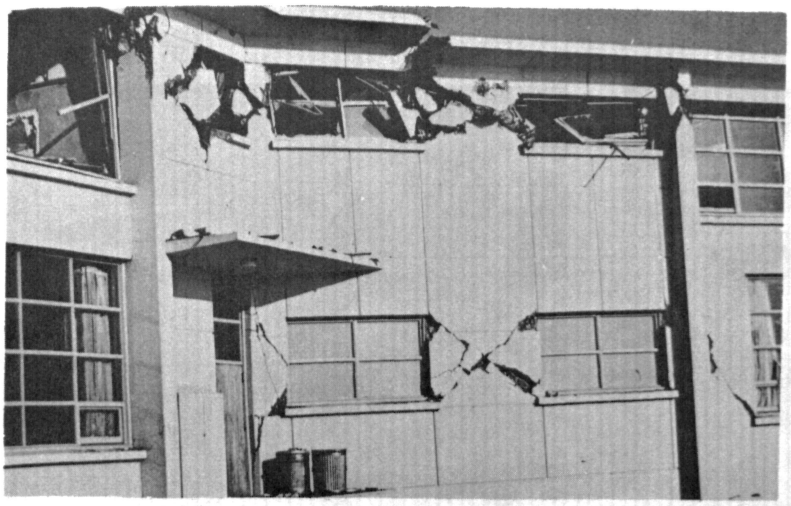


Fig. 16. Portion of the severely damaged West High School, Anchorage.

Fig. 17. Typical column damage to West High School, Anchorage.

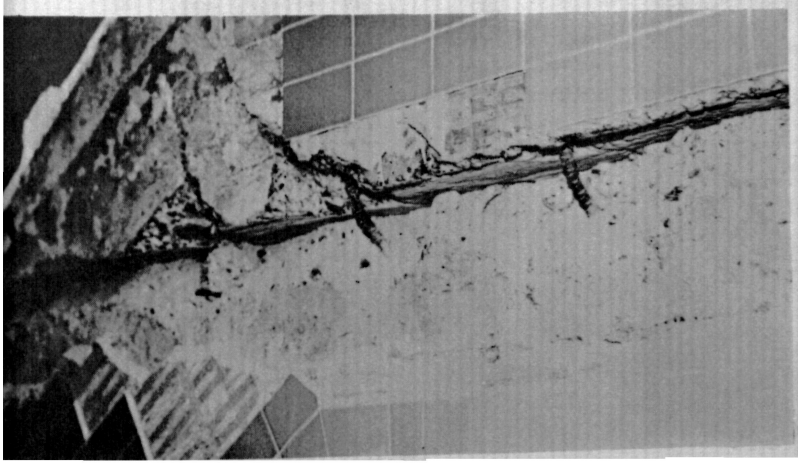
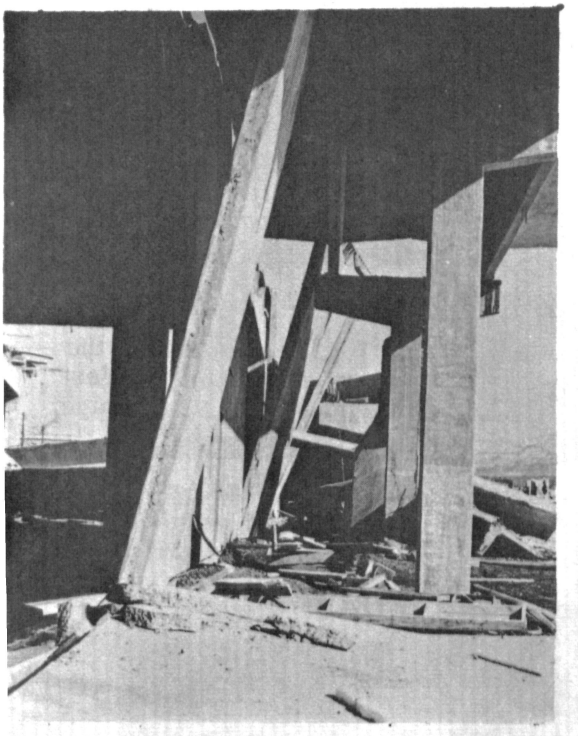
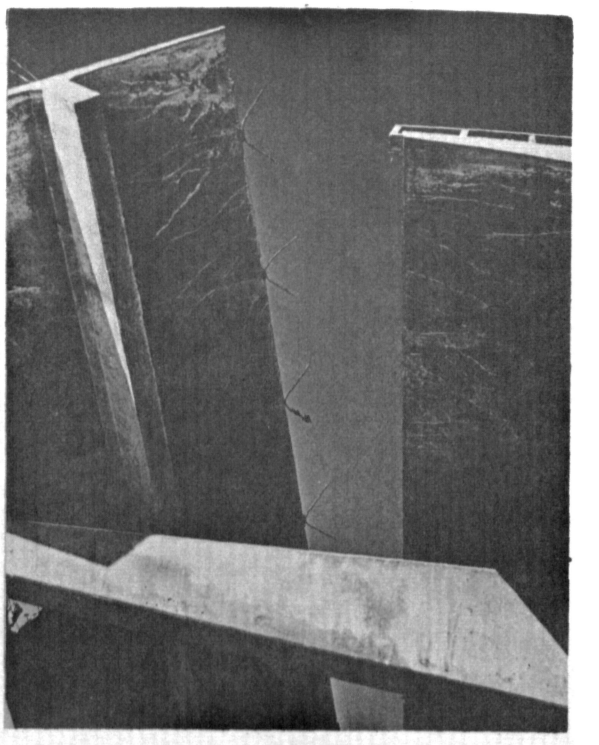


Fig. 18. Movement along a reinforced concrete construction joint, West High School. Laitenance in the joint has not been removed.





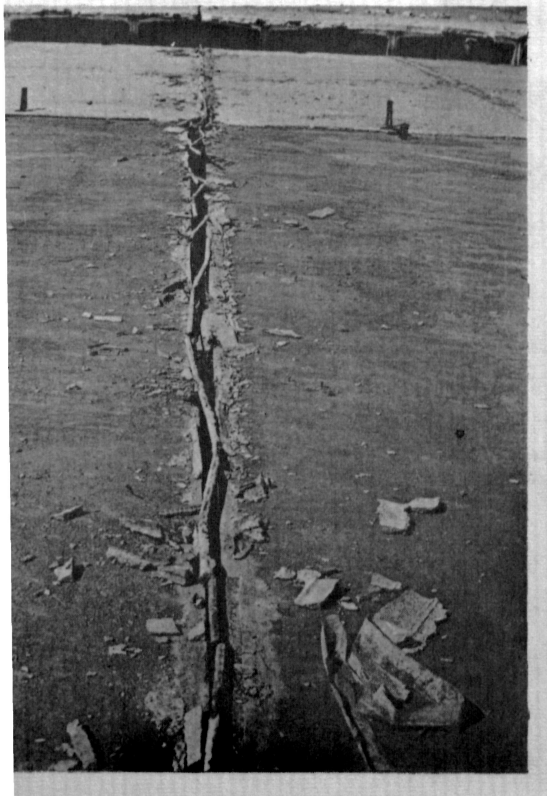


Fig. 19 (upper left). Failure of precast concrete connections, Alaska Sales and Service.

Fig. 20 (upper right). Failure of precast concrete connections, Alaska Sales and Service.

Fig. 21 (left). Failure of connections between precast roof elements, Alaska Sales and Service.

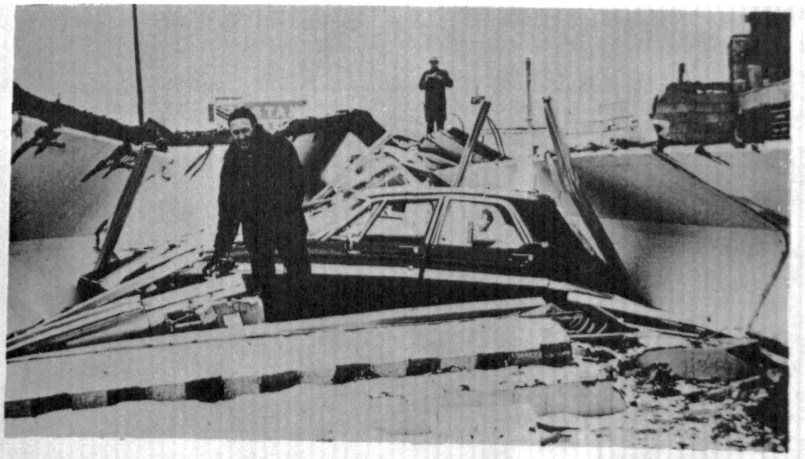


Fig. 22. Chrysler Sales. Failure of precast roof was probably caused by the collapse of the hollow concrete block supporting structure.

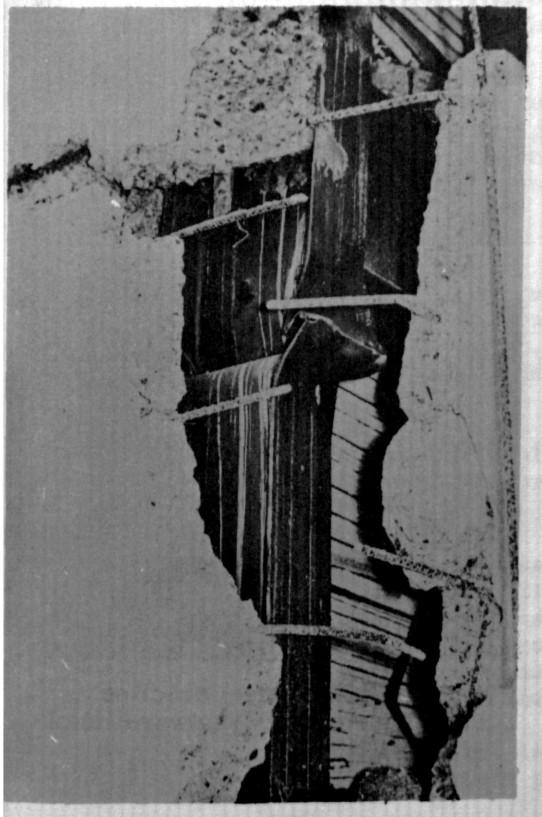


Fig. 23. Cordova Building. Steel column failed due to stair framing into this column which great ly increased column stiffness, thereby bringing most of the lateral force to this column.

Fig. 24. Anchorage-Westward Hotel, East elevation.

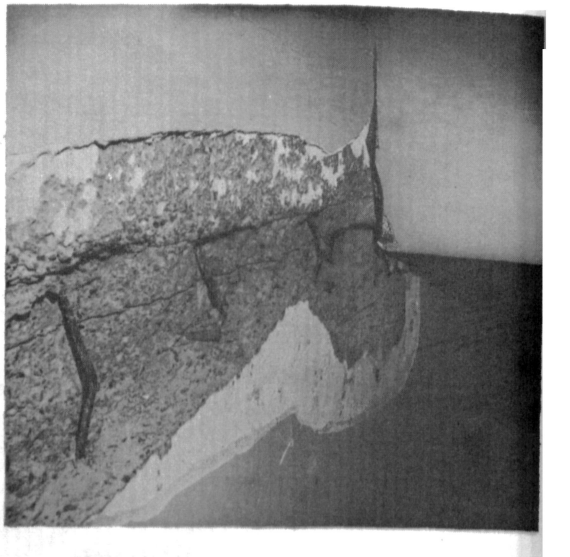


Fig. 25. Movement along construction joint of interior shear wall, Anchorage-Westward Hotel.

Fig. 26. Typical damage to openings piercing interior shear walls, Anchorage-Westward Hotel.

Fig. 27. Buckled steel erection column, Anchorage-Westward Hotel.

STRUCTURAL ENGINEERING ASPECTS OF THE ALASKAN EARTHQUAKE OF MARCH 27, 1964

BY KARL V. STEINBRUGGE

QUESTION BY: I.L. HOLMES - NEW ZEALAND

I must state that as a designer I find the information given this morning on Skopje and Anchorage somewhat disappointing. I don't feel that I have much more information than I was able to derive from photographs, for instance, in the National Geographic magazine. We have been shown many colourful slides but they are similar to pictures already published and from which engineers have been able to derive similar conclusions as we have heard today as to the physical nature of the failures. Is it not possible to obtain the design calculations for the buildings damaged and relate the designer's intentions to what in fact happened in the earthquake? This is the most important information to obtain. It is no use examining a badly damaged building which has been badly designed. It is of great use though, to examine a building which was well designed and for which the designer's intentions are known, and to find out what happened to his calculations and assumptions. I presume these calculations can in many cases be obtained from the local authorities. If not, strong efforts should be made to get them from the designers concerned.

I would like to comment further on the matter of obtaining calculations where the designer is reluctant to make them available. For other types of national disaster there is machinery in most countries for a Commission of Enquiry. This operates as a court and can get any information it wants on demand. I fail to see why such machinery should not be available in the case of an earthquake where the question of public safety is equally important.

AUTHOR'S REPLY: There are certain practical problems in discussing 15 or 20 major buildings within a 30 minute period. Comments must be extremely brief. The coloured slides which were carefully selected by a group of Structural Engineers to point out lessons in Anchorage, may have been too subtle for some in the audience.

Every effort is being made within ethical and legal limits to obtain and publish all relevant information.

QUESTION BY: G.R. WALKER - NEW ZEALAND

In present analysis and design it is very common to use the ductility factor concept. Has any attempt been made to use this concept in analysing the damage caused by earthquakes?

AUTHOR'S REPLY: In general, the structures in Anchorage were braced by rigid elements such as shear walls. Ductility, such as might be encountered in a moment resisting frame, did not enter into the damage patterns created by shear walls.

QUESTION BY: A.W. SMITH - NEW ZEALAND

Several speakers have mentioned the effect on people. Would the panel care to comment upon the reaction of stairways and elevators to the earthquake and how people were able to get out of the buildings.

AUTHOR'S REPLY: Remarkably little injury and life loss occurred. This may be partly explained by the fact that relatively few buildings collapsed. Those that did collapse usually did not do so immediately, and persons were able to leave these structures before collapse. It must be remembered that the duration was long, perhaps 4 minutes. The type of ground motion was such that persons could walk with difficulty, usually using whatever supports were available.

QUESTION BY: J. DESPEYROUX - FRANCE

Regarding intensity scales and UNESCO's work in April. At Anchorage - tall buildings suffered much, small rigid ones suffered less; therefore there is a difference between looking at each.

At Skopje, smaller buildings suffered much and tall buildings little. Thus there is a difference in earthquakes and this shows an inadequacy of the idea of intensity. Any revision by UNESCO besides including structural difference in the buildings should also include difference in shape of response spectrum.

Damage due to failure of columns with inadequate ties is shown in many examples in Anchorage and Skopje; in cantilever beams, buckling of longitudinal bars caused bending of the cantilever. It is necessary to have special provision in all cases against longitudinal buckling.

AUTHOR'S REPLY: Considerable thought has been given to intensity ratings of different types of ground motions. Richter's recent book "Elementary Seismology" should be referred to. The effects of long period motions, as contrasted to short

period effects, may also be found in detail in the studies of the 1952 Kern County, California, earthquake; the 1954 Dixie Valley, Nevada, earthquake; and the preliminary reports on the 1964 Alaskan earthquake.

QUESTION BY: M. IFRIM - RUMANIA

The pictures that have been shown, show how great damage can be. Very few papers are about the explanation of Anchorage failures. It would be of very great use if the I.A.E.E. and U.N.E.S.C.O. could publish a book of photographs of failures with the explanation of failure. It would be of great interest to show for these seismic regions what measures one could take to prevent the large damage. It is not too difficult to find a cheap system for the construction and connections. I have designed several churches in Bucharest which survived the earthquake whereas others were damaged. The computations were easy and they did not cost much. Other engineers probably have similar experiences and all this information should be exchanged or pooled to prevent further large damage.

AUTHOR'S REPLY: Detailed studies are in progress which will describe the mode of failure of most of the major structures in Anchorage. These studies are expected to be published in a year or two by the U.S. Government.

THE GREAT ALASKAN EARTHQUAKE OF MARCH 27, 1964

BY R.W. BINDER

D I S C U S S I O N

Pictures which I have seen, in my opinion, do not fully portray the very spectacular and dramatic situation in the slide and subsidence areas of Anchorage.

Due to absence of instrumental accelograph records, the big debate seems to center on the ground motion; length of time of the strong shaking, the severity (acceleration etc.) and type of ground motion (horizontal vertical) at Anchorage.

My own reaction to damaged structures which did not collapse (reason for collapse were generally obvious) is as follows.

Inasmuch as many of the damaged structures had been (intended to be) designed for seismic motion, the failures were in some respects different from those I have seen in the many areas where I had observed earthquake effects:

1. The vibrated structures seemed to understand the (fair or good) design and resulted in damage taking place in poor or inadequate details with subsequent damage to the particular structure. I believe I saw more specific damage to details than I had seen in any other earthquake. It was again evident that when a structure is vibrated by a chaotic and random motion that every detail is important and no detail is unimportant; notch effects create problems; anchorage details must be effective and should be designed to do a specific job.

2. If one considers a shear wall design as a shell type structure, then as for shells, the perimeter or boundary conditions are significantly important. Detailed damage indicated failure to deliver the designed forces to shear walls effectively.

3. Numerous observed damage effects have been the same in many earthquakes damaged areas; however, due to the absence of ground motion records, etc., some may be conjectural, inclusive of those attributed to soils, etc.

GENERAL OBSERVATIONS AND NOTES.

a. The contribution to significant damage effects when excessive physical torsion was present is a matter of record. Still undetermined is the contribution due to less evident physical torsion.

b. The contribution to damage due to the "abuse of the use of rigidities" could be seen in many areas. It appeared that in numerous cases "rigidities" were placed on the structure for architectural or some other purpose without taking into account the disastrous consequences of such misused rigidities when subjected to seismic inertia forces.

c. The basic criteria and implementation of such criteria for design remains to be determined (over and beyond reasonable opinions which may have been expressed); one such important item is the design treatment of overturning. Skilful design beyond code minimums, etc., is also subject to careful scrutiny.

d. Mass and mass distribution effects were noted. Unnecessary mass (dead or live) should have been avoided.

e. Hangers for electrical and mechanical installations, etc., should have the same attention as other structural elements.

f. Stiffness should not be considered as a substitute for strength or vice versa.

g. When the vertical resisting elements are concentrated in a small service core, special skilful overall design is mandatory; distortions, diaphragms, deflections, etc., take on special important roles.

h. Structures should be adequately tied together, recognizing at the same time the participation of rigidities, etc.

i. Assuming that the period of ground motion was in the order of 0.7 sec. or greater, it should be recognized if the period of ground motion had been shorter (i.e. epicenter closer to Anchorage) it may be concluded that some of the present structures with short periods could have been adversely affected.

j. The accumulation of possible damage effects which go unnoticed should be considered; thus period readings before and after earthquakes are of significant importance.

These observations and notes are intended only as the writer's general observations and are not presented as a formal report on The Great Alaskan Earthquake of March 27, 1964.

Much work remains to be accomplished to fully document the damage effects; thus, a challenge to the earthquake engineering organisations. In my opinion, some of the public statements have been given without benefit of factual data, and have been the result of emotional reactions.

Structures should be carefully inspected to determine the type of requisite repair: (a) superficial (b) structural. Structural damage should not be repaired until a thorough review is made by a qualified engineer who should undertake to determine the cause and the probable future effects of the proposed structural repairs.

As to Anchorage, until we know a lot more about the details, as well as details of design criteria, etc., I am inclined to the opinion, if structures are built on sound and not false foundations and soil conditions and designs conform to sound engineering in addition to complying with the letter and spirit of the SEAOC seismic code, that a considerable body of damage could have been avoided. At the same time, it is recognized that improvements through research, etc., rest upon the shoulders of those who can learn from the events of yesterday.