SOIL MECHANICS AND BITUMINOUS MATERIALS
RESEARCH LABORATORY

THE TURNAGAIN HEIGHTS LANDSLIDE IN ANCHORAGE, ALASKA

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Description of the Slide

During the Alaskan earthquake of March 27, 1964 a number of major slides occurred in the City of Anchorage.[1,2] The largest of these slides was that along the coastline in the Turnagain Heights area. An aerial view of the slide is shown in Fig. 1a and a plan of the slide area in Fig. 3.

The coast line in this area was marked by bluffs some 70 feet high, sloping at about 1-1/2:1 down to the bay. The slide extended about 8500 feet from west to east along the bluff-line and retrogressed inward from the coast a distance of about 1200 feet at the west end and about 600 feet at the east end. The total area within the slide zone was thus about 130 acres.

Within the slide area the original ground surface was completely devastated by displacements which broke up the ground into a complex system of ridges and depressions, producing an extremely irregular and hummocky surface. A general view of the central part of the slide area is shown in Fig. 1b and a similar view at the east end in Fig. 2a. In the depressed areas between the ridges the ground dropped an average of about 35 feet during the sliding.

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1. Shannon and Wilson, Inc., 1964, Report on Anchorage Area Soil Studies, Alaska, to U. S. Army Engineer District, Anchorage, Alaska: Seattle, Wash.
2. Hansen, W. R., "Effects of the Earthquake of March 27, 1964 at Anchorage, Alaska, Geological Survey Professional Paper 542-A.

Fig. Ia AERIAL VIEW OF TURNAGAIN HEIGHTS LANDSLIDE

Fig. Ib CONDITIONS IN CENTRAL PART OF SLIDE AREA

Fig.2a CONDITIONS IN EAST END OF SLIDE AREA

Fig.2b HOUSES DESTROYED BY SLIDE

Lateral movement of the soil mass during the slide was extensive. The material from the original bluff line moved out into the bay in some places as much as 2000 feet (see Fig. 3), and in general the outward movement beyond the original bluff line tended to mirror the extent of inland regression of the slide behind the bluff-line.

The ground at the west end of the slide area was undeveloped but that at the east end had been developed as a residential area. Thus about 75 houses located in the east end of the slide area were destroyed. A view of some of these houses after the slide is shown in Fig. 2b.

A study conducted by the Engineering Geology Evaluation Group[3] in Anchorage reveals the extent of movements of some of the houses in the slide area. Fig. 4, prepared by this group, shows the initial and final positions of houses as determined by surveys before and after the earthquake. It may be seen that some of the houses moved laterally about 500 or 600 feet, usually, though not always, towards the original coast line. The transverse movement of some of these houses is indicative of the complex mechanism involved in the development of the slide.

Eye-witness accounts indicate that the sliding began about 2 minutes after the start of the earthquake and continued to some extent after the earthquake-induced ground motions had ceased. It appears to have started as two separate slides, one at the east end and one at the west end, which merged together as they developed and enlarged.

Numerous tension cracks developed in the ground behind the slide zone during the earthquake as a result of a general movement of the

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3. Engineering Geology Evaluation Group, 1964, Geologic Report - 27 March 1964 Earthquake in Greater Anchorage Area: Prepared for and published by Alaska State Housing Authority and the City of Anchorage, Anchorage, Alaska.

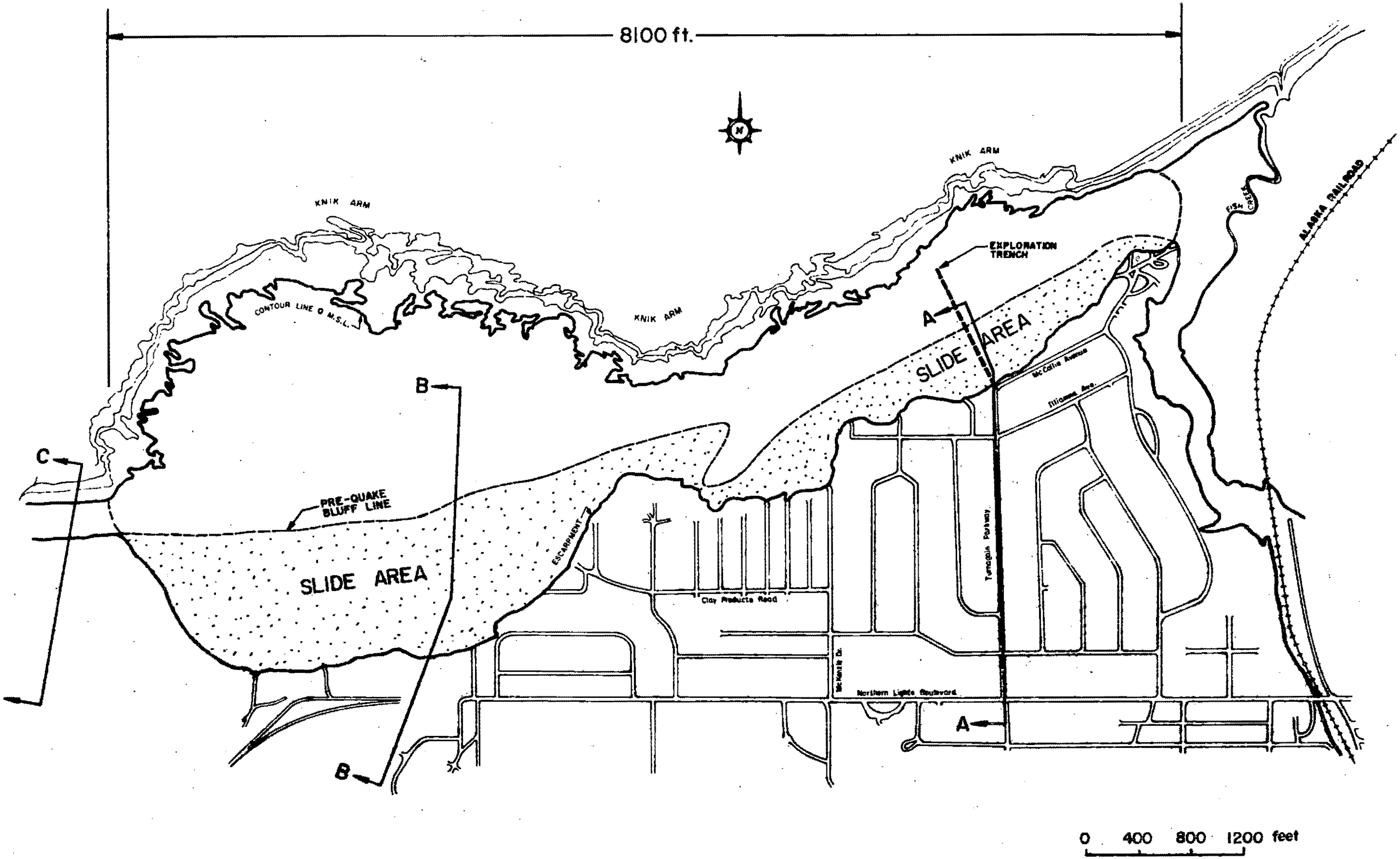

Fig. 3 PLAN OF SLIDE AREA

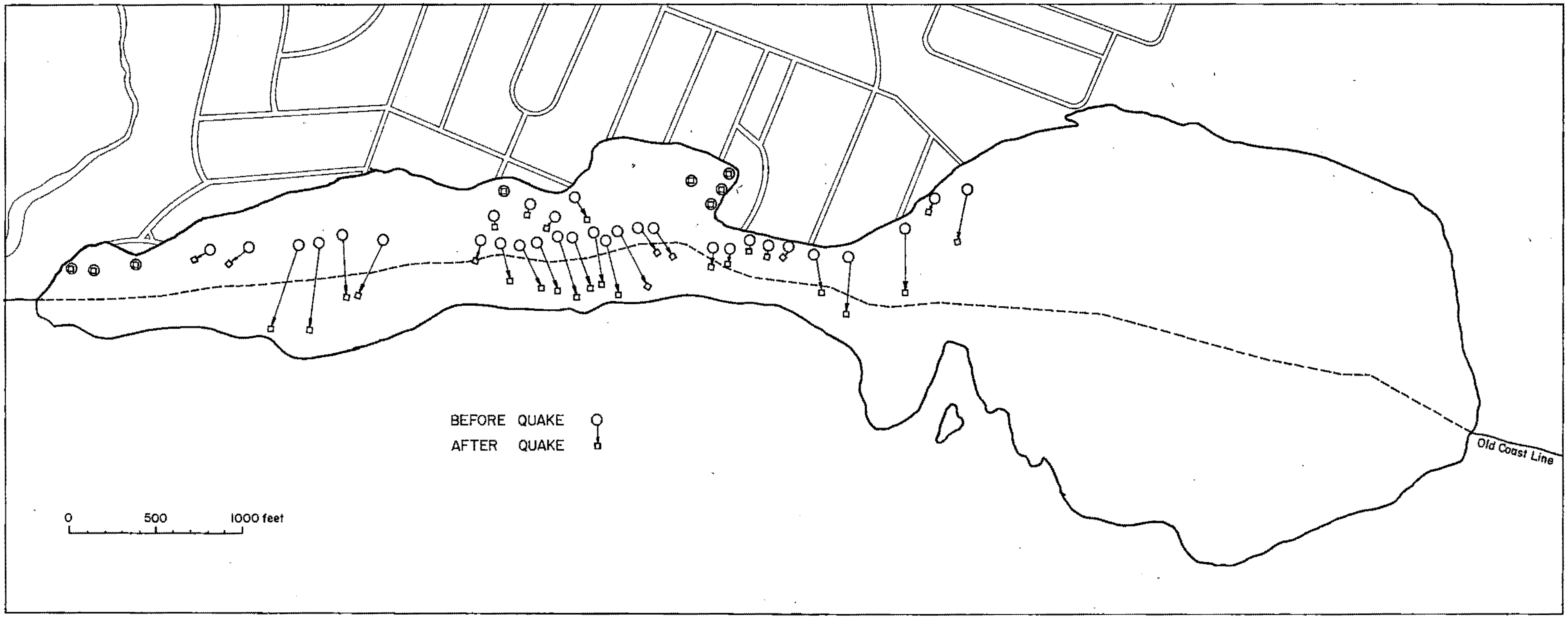

Fig. 4 MOVEMENTS OF HOUSES IN SLIDE AREA

land in the direction of sliding towards the coast-line. Cracking was more intensive behind the east end of the slide and surveys in this area showed that the accumulated extension in a 2000 ft zone extending behind the slide area amounted to about 3 feet. No appreciable movement occurred in this area in a 3-month period following the earthquake.

The Alaska Earthquake

The earthquake which caused the Turnagain Heights landslide has been rated by the U. S. Coast and Geodetic Survey as a shock of magnitude 8.5 on the revised Richter scale with its epicenter about 80 miles east of Anchorage.

Since there were no strong-motion seismograph stations in the area, little is known concerning the magnitude of the ground motions induced by the earthquake. However on the basis of the observed damage and related effects, it is estimated that the intensity in the Anchorage area was approximately VIII on the Modified Mercalli Scale.

One of the main features of the ground motions induced by the earthquake was its unusually long duration. At least 6 people were found from areas in and outside the Anchorage area who had timed the duration of ground shaking by means of watches; the durations observed by these people ranged from about 4.5 to 7.5 minutes. On this basis the duration of strong motions appears to have been at least 4 minutes.

Eye-witness reports seem to be in general agreement that the first ground motions at Anchorage had a strong east-west component. As the shaking continued, however, the main component is said to have shifted first to a north-south direction and later to a more complex motion on which little general agreement can be obtained.

Geology of the Anchorage Area[4]

The Anchorage area, located within a well-known earthquake region, occupies 150 square miles at the head of Cook Inlet in south-central Alaska. The city is built on lowlands, a relatively flat plain of outwash sand and gravel underlain by Bootlegger Cove Clay. Rocks of Mesozoic and Tertiary age are overlain by unconsolidated deposits of glacial drift, mantled with wind-deposited silt. The lowlands range from sea level to 1200 feet with an average elevation less than 200 feet above mean sea level. The highest point in the nearby Chugach Range is at elevation 4301.

The oldest rocks exposed in the Anchorage area are the metamorphics of the Chugach Mountains. These Mesozoic rocks are mainly greenstones, graywackes, slates, argillites and limestones. Tertiary shales of the Matanuska formation locally comprise the bedrock beneath glacial drift. The drift has been deposited during the Pleistocene and consists of successions of unsorted material (till), outwash sand and gravel, estuarine or lacustrine clay and silt beds. Undifferentiated drift and morainal and alluvial fan material have been deposited between other glacial drift and non-glacial materials, resulting in poorly defined boundaries. Non-glacial deposits include loess, alluvium, estuarine silt, dune sand, and swamp deposits.

Ablation till is fragmental and unconsolidated material underlying outwash, lacustrine, and marine deposits. The till is thickest beneath the lowlands away from the Chugach Mountains. Deposits near the mountains

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4. Miller, R. D. and Dobrovolny, Ernest, 1959, Surficial Geology of Anchorage and Vicinity, Alaska: U. S. Geol. Survey Bull. 1093, 128 p.

contain till with subordinate outwash sediments. Till in the Anchorage area has been identified in drill cuttings on the basis of hardness and relative impermeability.

Clay and silt are present as an extensive deposit which has been designated the Bootlegger Cove clay. It overlies some of the ground morainal till and prodelta deposits and is a blue-gray, plastic clay which is relatively impermeable and easily identified. The deposit is relatively continuous but is concealed to the north by an end moraine and pinches out elsewhere. It ranges from zero to more than 300 feet thick, averaging from 100 to 150 feet. Silty and sandy beds are found within the clay. These are slightly to moderately permeable, but for purposes of ground water development have proved unsatisfactory. Naptowne outwash (sand and gravel) overlying the Bootlegger Cove clay is the significant foundation material for Anchorage. This deposit is relatively shallow, so that clay is exposed in the bluff areas and the slopes are susceptible to slumping and sliding. The outwash is cross-laminated and crudely bedded. The ground surface slopes vary gently to the southwest and the immediately underlying materials grade from coarse sand, cobbles, and boulders (toward Eagle River) to stratified sand (Turnagain).

The geomorphic features are primarily glacial: morainal hills, the outwash plain, the meltwater channels and depressions, lakes and swamps, and other glacial features from older ice sheets. Most of the existing land forms have resulted from ice movement, and only minor topographic modification has taken place since.

All of the larger streams, except Chester Creek, rise in the Chugach Mountains and flow across the outwash plain in channels. Fed by precipitation and snowmelt, the streams are not yet fully integrated with swamps, lakes, and ponds of glacial origin. Chester and Ship Creeks and their tributaries dissect the north half of Anchorage, occupying older valleys cut by larger glacial-fed streams. Campbell Creek, flowing in a deeply incised meltwater channel, extends from a glaciated mountain valley to the lowlands where it meanders through swamps. Many swamp lakes are connected by sluggish streams, and lakes found on ground moraines are very poorly drained. Small swamps are even found in higher environments along edges of stream valleys and near hill tops. This is a result of poor subsurface drainage where relatively impervious clayey till lies underneath, restricting the downward movement of water.

Soil Conditions in the Turnagain Heights Slide Area

An extensive program of soil exploration[1] was undertaken to determine the soil conditions in the Turnagain Heights slide area. The general soil conditions determined by these investigations are shown in Figs. 5 and 6. Figs. 5a and 5b show soil profiles along sections through the east and west ends of the slide area (sections AA and BB in Fig. 2). Fig. 6a shows a soil profile through the area just west of the slide area where no sliding occurred (section CC in Fig. 2). Fig. 6b shows a profile along an east-west section through the slide area.

In general the Turnagain area (Surface Elevation $\simeq$ 70) is covered by a surface layer of sand and gravel which varies in thickness from 15 to 20 ft at the east end of the slide area to about 5 to 10 ft at the west end.

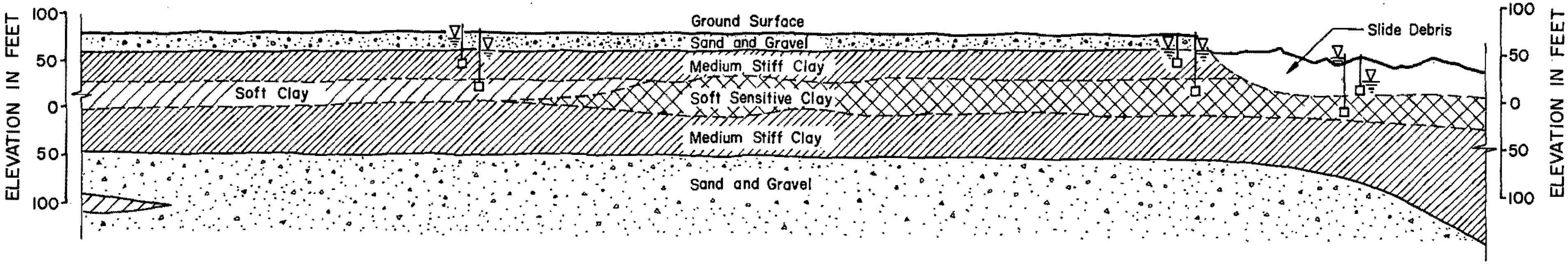

Fig. 5a SOIL PROFILE THROUGH EAST END OF SLIDE AREA (SECTION AA IN FIG. 3)

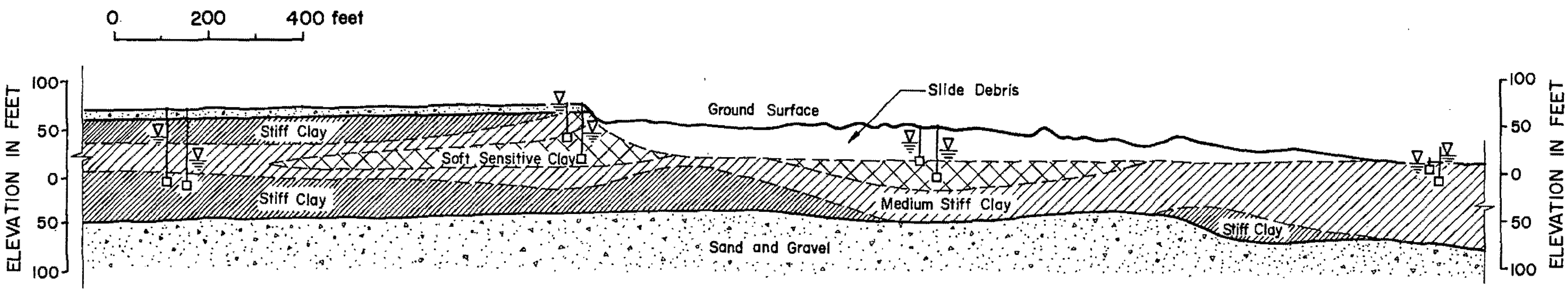

Fig. 5b SOIL PROFILE THROUGH WEST END OF SLIDE AREA (SECTION BB IN FIG. 3)

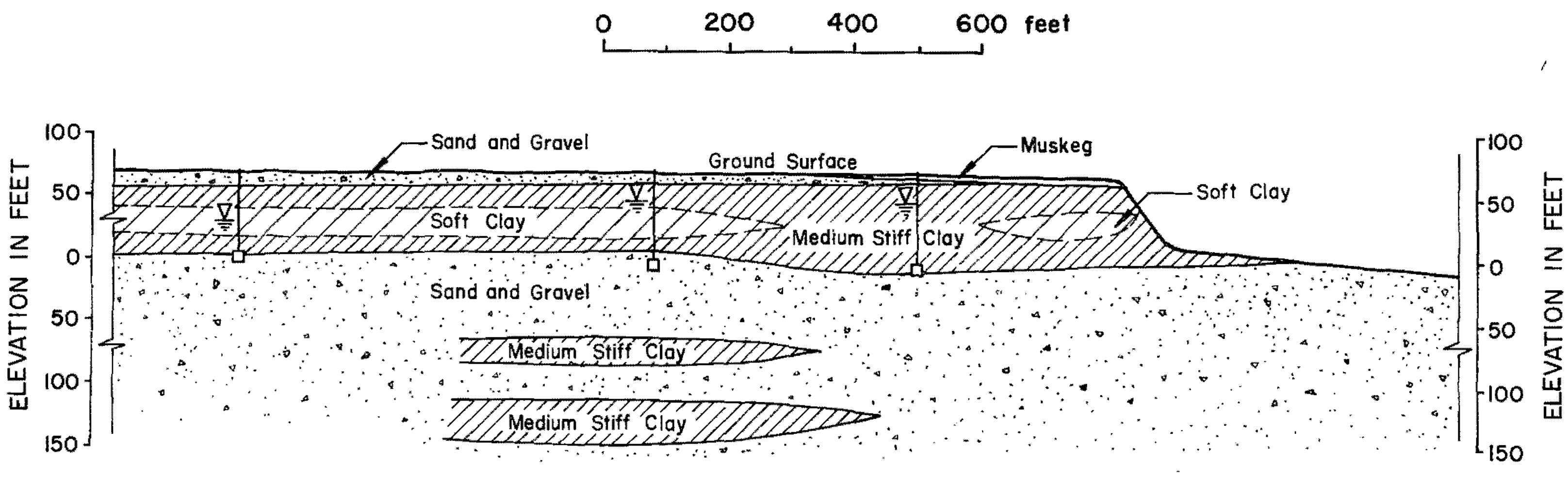

Fig.6a SOIL PROFILE ADJACENT TO SLIDE AREA (SECTION CC IN FIG.3)

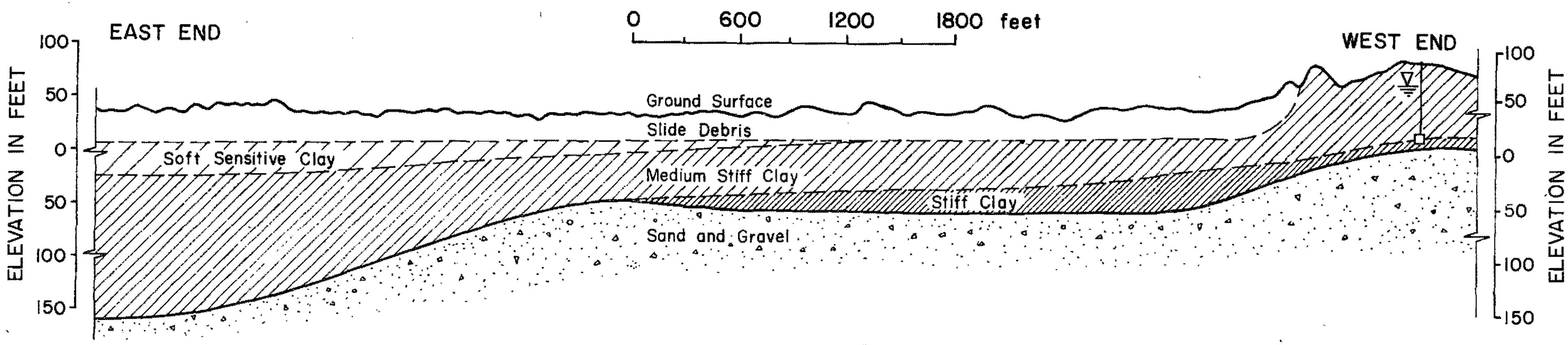

Fig.6b SOIL PROFILE ALONG SLIDE AREA

The sand and gravel is underlain by a deep bed of Bootlegger Cove clay, about 100 to 150 ft in thickness. This soil is a sensitive marine deposit of silty clay with a shear strength decreasing from about 1 ton per sq ft at its surface (El 50 approx.) to about 0.45 ton per sq ft at El. 0 and then increasing again to about 0.6 ton per sq ft at El. -30; its sensitivity varies between about 5 and 30. The more sensitive samples of the clay typically have a liquid limit of about 33 and a plasticity index of 12, plotting just above the A line on a plasticity chart. A typical variation of strength with depth is illustrated by the data presented in Fig. 13.

Between elevations +25 and -10 at the east end of the slide area and between elevations +35 and +10 at the west end of the slide area, the water content of the clay exceeds the liquid limit and it is in these zones that maximum sensitivity is indicated by the shear strength determinations. In these ranges the strength of the remolded soil is of the order of 0.02 ton per sq ft. It is an unfortunate fact that the zone of maximum sensitivity generally coincides with the zone of lowest strength for this deposit. In general the sensitivity of the clay appears to decrease with increasing distance behind the original coast-line (see profiles AA and BB in Fig. 5) and the extremely sensitive clay was not found in borings made about 2000 ft behind the original bluff-line.

The clay deposit contains numerous thin strata and seams of silt and fine sand which are apparently not continuous throughout the deposit. At some points the sand strata are up to 3 ft in thickness but, more typically, they vary from a fraction of an inch to several inches in depth. At the east end of the slide areas, frequent thin seams of fine sand were found between elevations 30 and 37, a sand layer several feet thick was encountered near elevation 20 and a number of thinner lenses were found below this, their thickness and frequency deminishing with depth; below elevation 10, sand lenses were thin and were only found at occasional intervals.

At the west end of the slide area sand lenses were thinner and less frequently encountered than at the east end. However the sand lenses in this part of the slide zone were commonly associated with silt lenses. Such lenses were found at frequent intervals near and above elevation +20 but below this they were only occasionally encountered.

Seaward of the bluff-line which existed prior to the slide (see Figs. 6a and 9) the clay was overlain by a deposit of estuary silt which sloped gently downwards away from the shoreline. This material, which would tend to liquefy during the earthquake, probably played a significant role in the development of the slide by providing a lubricated surface on which blocks of soil from the sliding mass could move outwards with little resistance to motion.

A soil profile along a section CC just west of the slide area is shown in Fig. 5a. It may be noted that the extent of the zone of very sensitive clay is much reduced in this area. In addition the clay is somewhat stiffer than that at corresponding depths in the adjacent slide area, no sand lenses were observed in the borings above elevation 15 and the total depth of clay is some 30 ft less than in the adjacent slide area. These facts possibly explain why sliding did not extend further to the west in spite of the generally similar topographic and geologic conditions in this area.

Strength of Soils Under Cyclic Loading Conditions

Laboratory studies[5,6] were conducted to investigate the strengths of the soils in the Turnagain Heights slide area under conditions simulating those developed during the earthquake. For soil elements in the embankment some distance behind the bluffs, these conditions can be simulated approximately by simple shear tests conducted under cyclic loading conditions.

The results of such tests performed on undisturbed samples of the sensitive silty-clay are shown in Fig. 7. Some of these tests were performed using the simple-shear equipment developed by the Norwegian Geotechnical Institute,[7] modified to permit the conduct of cyclic stress or cyclic strain tests. Other tests were performed using cyclic loading triaxial compression procedures.[8] Both types of test gave essentially the same results, showing that failure of this clay could be induced by cyclic stresses substantially less than the static strength of the clay under undrained loading conditions. For 30 significant stress cycles, such as might have been developed during the earthquake, failure was induced by cyclic stresses equal to about 55 percent of the static strength; for the soft sensitive clay which might be considered to have an in-situ shear strength of about 850 psf, this corresponds to a cyclic shear strength of about 470 psf.

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5. Seed. H. Bolton and K. L. Lee, "Pulsating Load Tests on Samples of Fine Silty Sand From Anchorage, Alaska," Appendix C of Reference 1.

6. Seed, H. Bolton and C. K. Chan, "Pulsating Load Tests on Samples of Clay and Silt from Anchorage, Alaska," Appendix D of Reference 1.

7. Bjerrum, L. and Landva, A., "Direct Simple Shear Tests on a Norwegian Quick Clay," Geotechnique, Vol. XVI, No. 1, March, 1966.

8. Seed, H. Bolton and C. K. Chan, "Clay Strength Under Earthquake Loading Conditions," Journal, Soil Mechanics and Foundations Division, American Society of Civil Engineers, March, 1966.

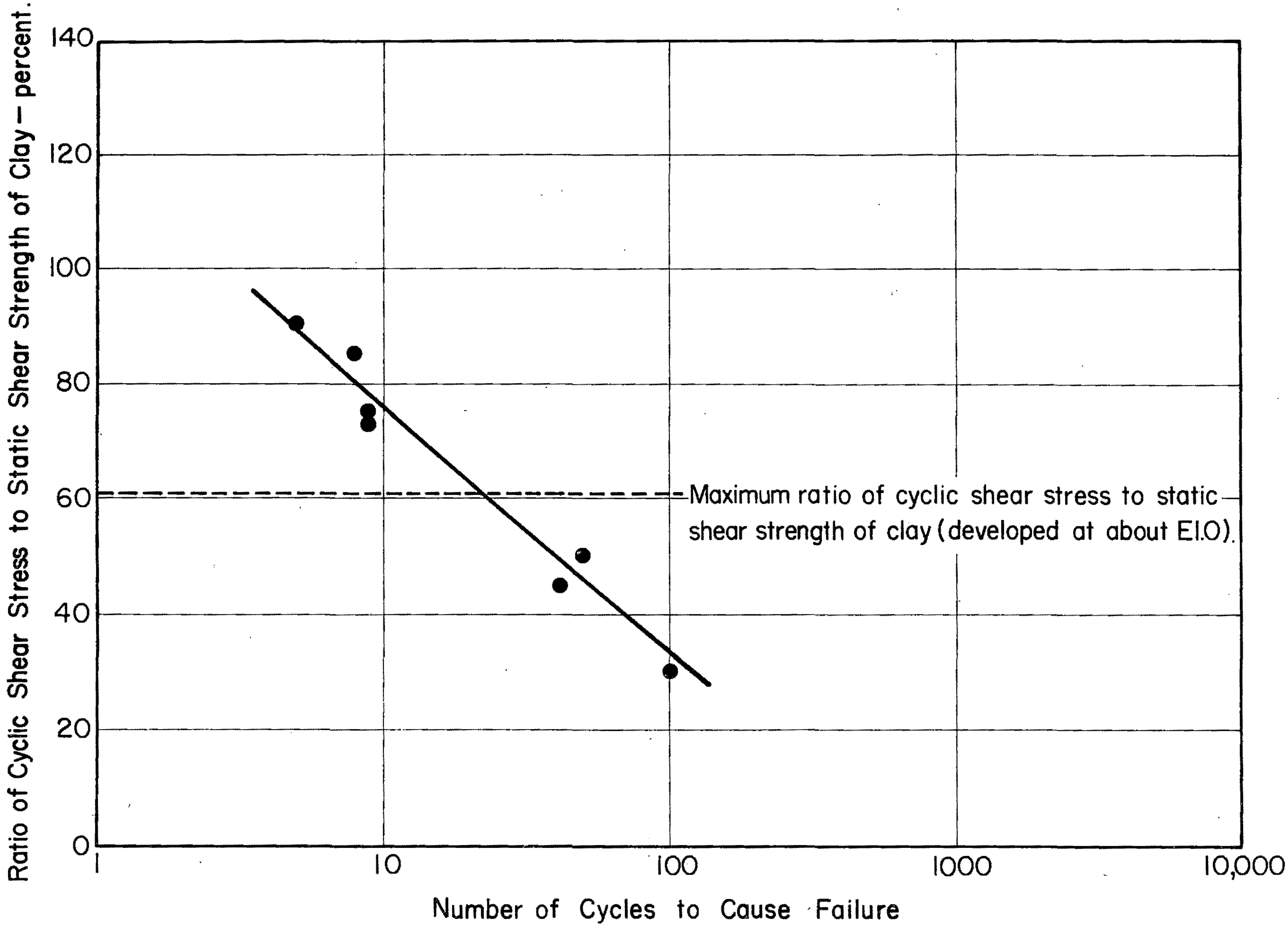

Fig. 7 STRENGTH OF SAMPLES OF SILTY CLAY UNDER CYCLIC LOADING CONDITIONS.

Under cyclic loading conditions, loose and medium dense saturated sands may fail by liquefaction. The cyclic shear stresses required to induce liquefaction of the sand seams were investigated by tests on a typical sample taken at about El. 20 at the east end of the slide area. For this purpose, the cyclic loading triaxial test procedure used previously by Seed and Lee[9] was adopted. The results of such tests may be corrected to other void ratios and confining pressures on the basis of previous data for similar materials.[10] It is also necessary to apply a correction to the data to convert the results to those corresponding to simple shear conditions. For samples of similar sand at about elevation 12.5 and for the approximate in-situ void ratio conditions as measured by field density determinations, the test data indicated the relationship between cyclic shear stress and number of stress cycles required to cause liquefaction shown in Fig. 8. It will be seen that liquefaction of the sand seams may be expected to occur in 30 cycles at an cyclic shear stress of about 420 psf, a value somewhat less than that required to induce failure of the soft clay in the same number of cycles. Thus for comparable shear stresses in the zone behind the bluffs, liquefaction might be expected in the sand before failure would develop in the clay.

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9. H. Bolton Seed, and K. L. Lee, "Liquefaction of Sands During Cyclic Loading," Journal, Soil Mechanics and Foundations Division, American Society of Civil Engineers, October, 1966.

10. Seed, H. Bolton and K. L. Lee, "Studies of the Liquefaction of Sands Under Cyclic Loading Conditions," Soil Mechanics and Bituminous Materials Laboratory, Report No. TE 65-5, University of California, Berkeley, December, 1965.

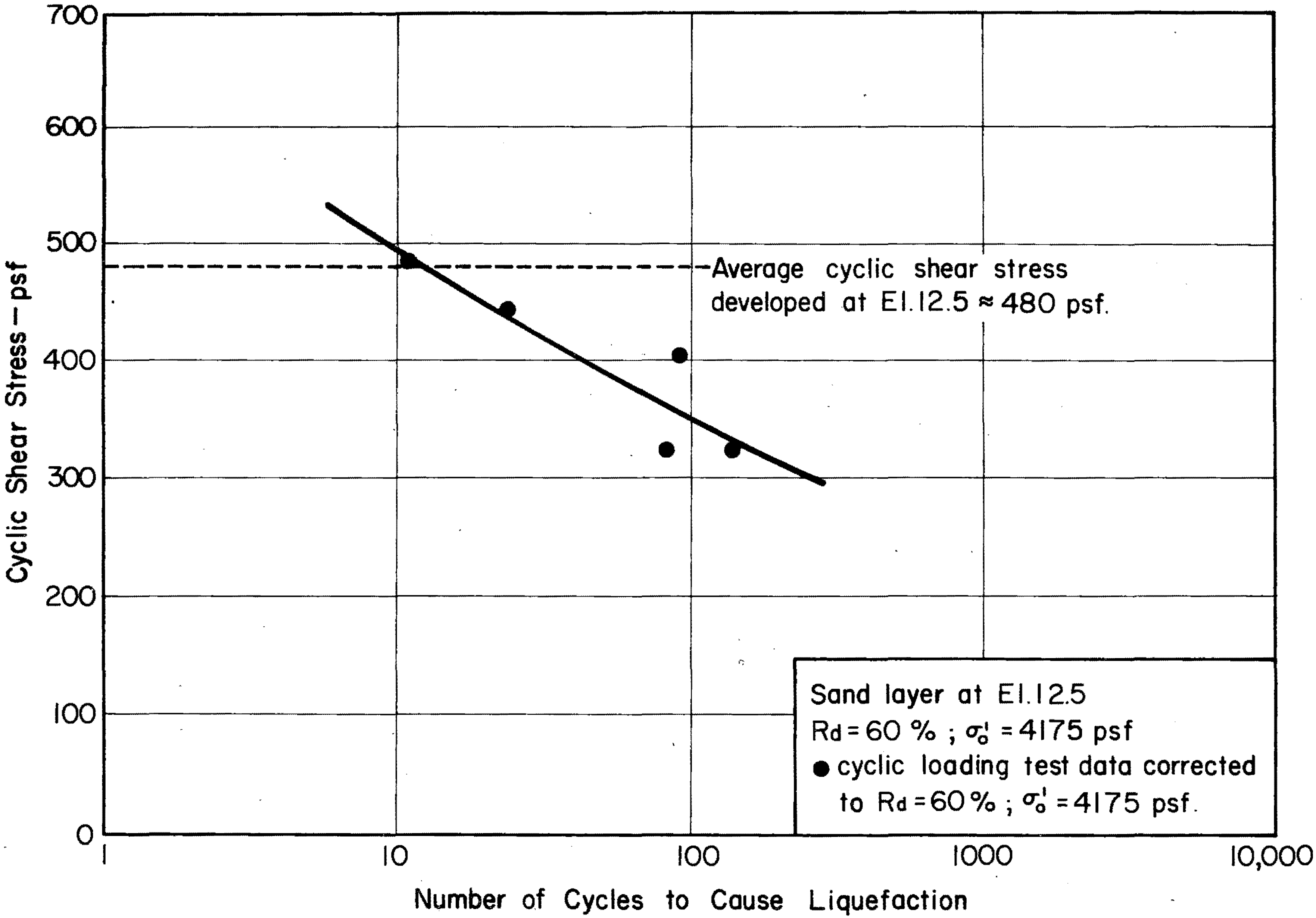


Fig.8 STRESSES INDUCING LIQUEFACTION OF SAND UNDER CYCLIC LOADING CONDITIONS

Investigation of Soil Displacement in Slide Zone

Several methods were used to investigate the nature of the soil displacements in the slide zone and the possible locations of slip surfaces. These included:

(a) Continuous sample borings

A number of borings were made from which continuous undisturbed samples were recovered to a depth of 100 ft or more. In cases of sample loss, a supplementary boring was made adjacent to the first and overlapping samples were extracted.

In some cases the samples were closely examined in the field to detect possible zones of disturbance. Penetrometer tests and miniature vane tests were conducted at frequent intervals along a freshly exposed section of the sample to aid in this investigation, and the location of disturbed zones was facilitated by the presence of the thin horizontal silt and sand seams in the clay. Several samples were recovered in this way which showed considerable intermixing of the sand and clay, together with a much reduced strength in the clay portion of the sample. The positions of these samples on a soil profile through the east end of the slide zone are shown in Fig. 9.

For the majority of the borings, samples were shipped to the laboratory where they were subjected to a detailed examination involving a description of the soil, stratification and any evidence of disturbance, augmented by determinations of Atterberg limits, natural water content, shear strength and sensitivity at about 1 ft intervals. Shear strength determinations were made by a torsional vane shear device specially developed for rapid testing in this investigation.[1]

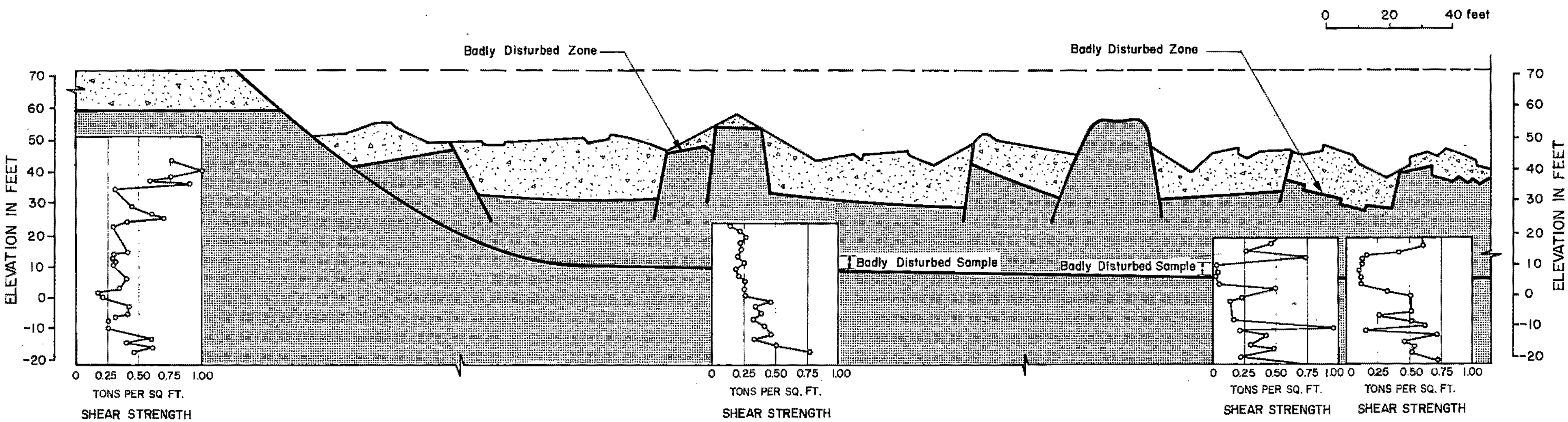


Fig. 9 DETAILED SOIL PROFILE THROUGH EAST END OF SLIDE AREA

Strength values varied considerably in some borings and numerous zones of extremely low strength were encountered. These zones were attributed to sample disturbance during sliding and their positions were noted in relation to other observations. The variation of shear strength with depth in several borings made along a soil profile at the east end of the slide area is shown in Fig. 9. It may be seen that just behind the slide area shear strengths rarely drop below 0.3 kg per sq cm while in borings made in the slide area there are many values less than 0.25 kg per sq cm and some as low as 0.03 kg per sq cm. It is interesting to note that the zones of very low shear strength agree well with the locations of very badly disturbed samples in the soil profile.

In the early stages of the investigation considerable confusion was caused by the observation that borings made as little as 10 to 20 ft apart showed radical differences in the variation of shear strength with depth. However subsequent interpretation of the mechanism of slide movement led to the conclusion that such variations are to be expected in slide zones of this type.

(b) Trench section

To throw further light on the distribution of materials in the slide area, a 1000 ft trench extending from the back side of the slide zone to some distance beyond the toe of the original bluff-line was constructed through the east end of the slide zone. The trenching was accomplished in two phases; a shallow trench for access and deeper trenching to expose the surface of the clay throughout the slide profile for mapping. This trench was deepened to as much as 30 ft in some places to uncover the clay which was buried under substantial depths of sand and gravel in depressions in the slide area. Details of the stratigraphy revealed by the walls of the trench were mapped as the excavating progressed to take advantage of the fresh exposures.

The distribution of the sand and gravel and the clay within the slide area, as revealed by this trench section, is shown in Fig. 10. The ridges of clay separated by depressed zones are readily apparent, as is the erratic distribution of the sand and gravel which originally overlaid the clay uniformly to a depth of about 15 ft. In many places the stratification of the exposed clay was relatively undisturbed but in other zones the clay was apparently severely disturbed and disoriented. The locations of the badly disturbed zones are shown on the profile. Unless otherwise noted, the stratification was essentially parallel to the upper surface of the clay.

(c) <u>Survey of clay ridges</u>

The clay ridges, illustrated in Fig. 1b, were a characteristic feature of the slide area. Hansen[2] has described them as follows: "Hundreds of sharp crested clay ridges alternating with collapsed troughs, and oriented normal to the direction of slippage distinguished the disruption pattern of the Turnagain Heights slide....Most of the clay ridges ranged in height from about 10 to 15 ft but a few were more than 20 ft high. They were as much as 300 ft long and were spread 50 to 150 ft apart. Their steep sides, which sloped 60° to 70°, were furrowed and grooved by slippage of one surface against another. On the average the ridges were sharper crested and more closely spaced"... in the west end of the slide area than in the east end.

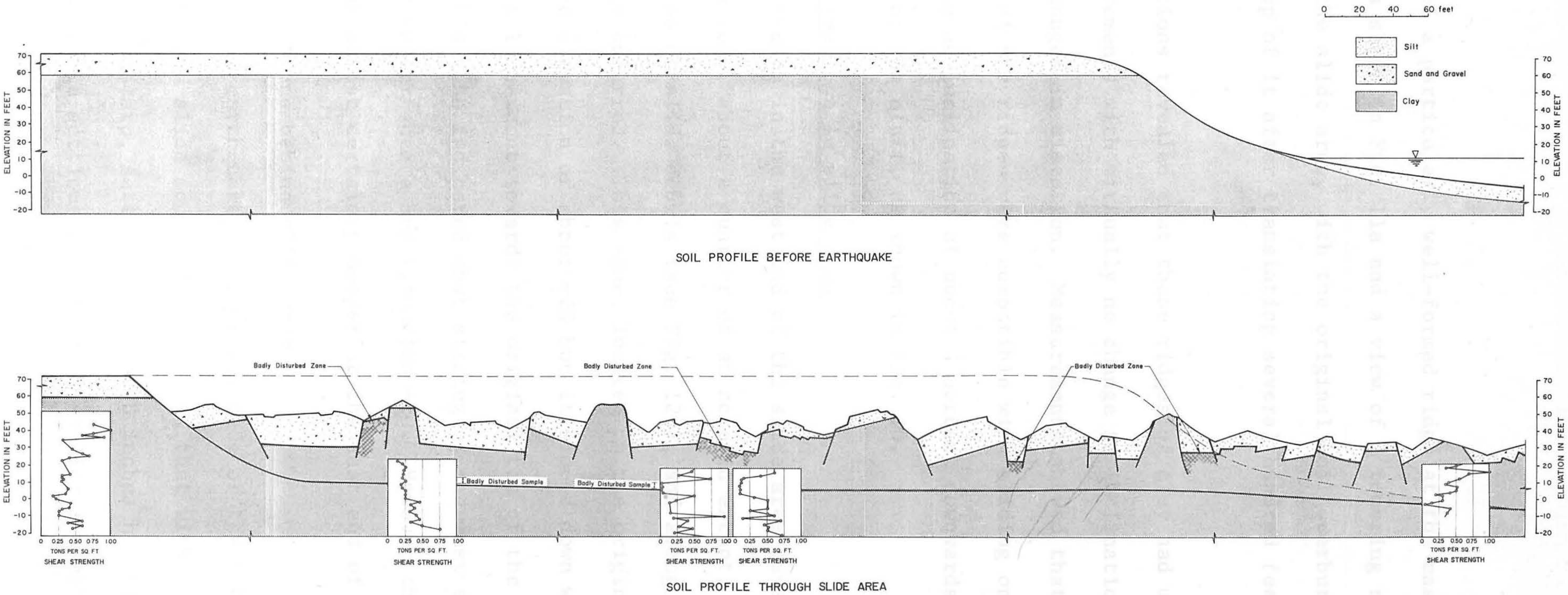


Fig. 10 SOIL PROFILE IN TRENCH SECTION THROUGH EAST END OF SLIDE AREA

A view of a particularly well-formed ridge at the east end of the slide area is shown in Fig. 11a and a view of a towering ridge in the west end of the slide area, with the original soil overburden still perched on top of it after translating several hundred feet, is shown in Fig. 11b.

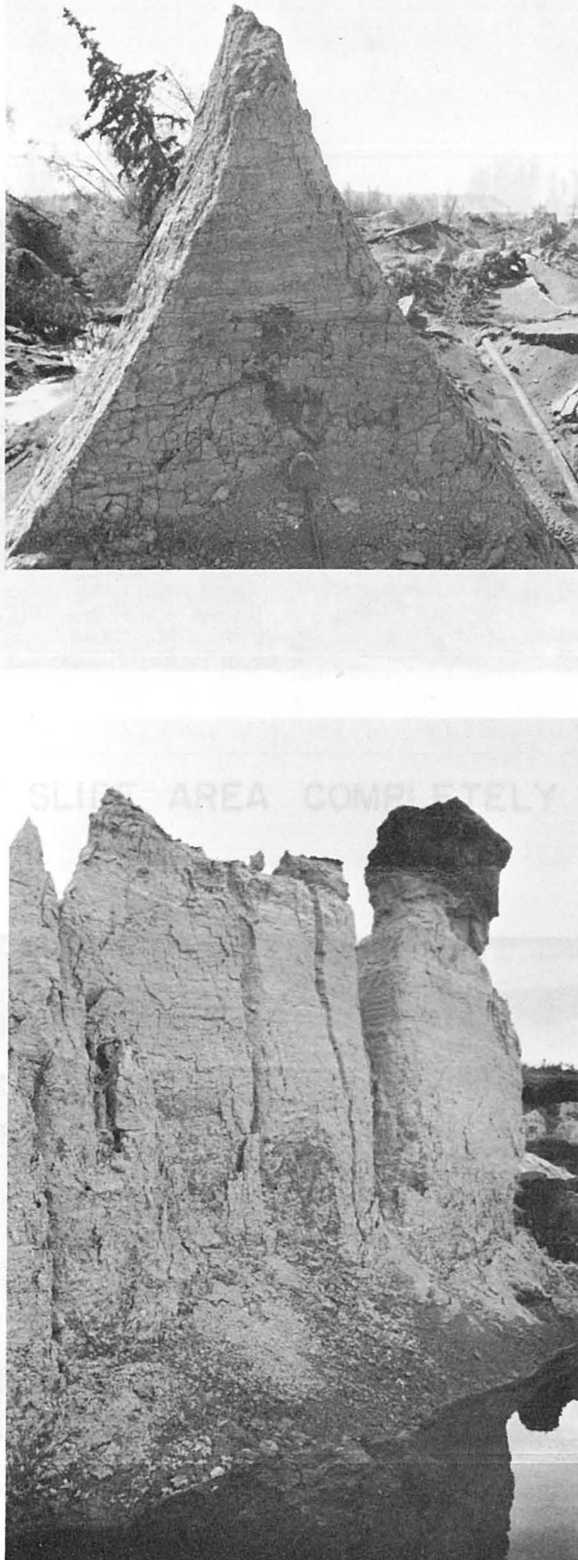
Observations revealed that these ridges of clay had undergone large displacements with virtually no change in inclination and with only small changes in elevation. Measurements showed that the changes in elevation of the ridges were compatible with sliding on a slip surface having an inclination of about 4 percent downwards toward the original toe of the bluff, as shown in Fig. 10.

(d) Observations in slide area

Reconnaissance in the west end of the slide area revealed the presence of a zone, about a quarter of an acre in extent, which was completely free of slide debris (see Fig. 12a), and along which sliding had apparently occurred. This zone, located on the original bluff line, had an average elevation of about +20 but it sloped down with an average grade of about 15 percent towards the original toe of the bluff. The presence of this zone indicated that sliding at the west end of the slide zone occurred on surfaces above elevation +20. However the lower surface of sliding was almost certainly deeper at the east end of the slide area.

(e) Observations behind slide area

Following the earthquake a survey was made of the settlements of the area behind the slide zone. It was found that within several hundred feet of the slide scarp, settlements up to 8 inches had occurred. Beyond this distance settlements were much smaller in magnitude. Such



(a)

(b)

Fig.11 RIDGES OF CLAY IN SLIDE AREA

Fig.12a ZONE OF SLIDE AREA COMPLETELY FREE OF DEBRIS

Fig.12b OFF-SHORE SILT DEPOSIT

observations are indicative of a movement of soil towards the main slide area.

Analysis of Turnagain Heights Slide

Depth of Slide Zone

A detailed section through the east end of the slide zone, as revealed by the exploration trench and the results of tests on samples from several borings made along this section is shown in Fig. 9. Samples taken from borings within the slide area show unusually low strengths over certain depth ranges followed by a sudden marked increase in strength. It was considered that the low strength values probably indicate material disturbed by sliding and the higher values reflect the presence of undisturbed soil. Thus the boundaries at which marked increases in strength occur may be indicative of the position of the main sliding surface.

Furthermore in two of the borings shown in Fig. 9, continuous samples of soil from the slide area were critically examined in the field for evidence of severe disturbance. The presence of thin seams in the clay greatly facilitated this examination. The positions of samples evidencing severe disturbance in each of these holes are noted on Fig. 9.

Based on the positions of the severely disturbed samples, the boundaries between low and high strengths of the clay and the positions of clay zones exhibiting lower strengths than similar material at the same elevation outside the slide area, it was concluded that the main surface of sliding was probably at about El. 8 near the back end of the section, sloping slightly towards the toe of the original bluff line.

The logs of borings made through the center of the west end of the slide area gave some indication of a similar sliding surface at about El. 10. However the debris-free zones somewhat further west indicated the slide surface to be at about El. 20. Thus the main slide surface at the west end of the slide area may have varied between El. 10 and El. 20, reaching the higher elevations with increasing distance in the westerly direction. In the direction of sliding, the main slide surface probably had an average slope of about 4 percent behind the original bluff line, steepening to about 10 or 15 percent in this location.

Stability Before Earthquake and Sliding Potential

Analyses of the stability of the bluffs along the coast line prior to the earthquake using undrained shear strengths measured on samples obtained from adjacent areas indicate a factor of safety of about 0.85. However this method of analysis has been shown previously[11] to give too low values for the factor of safety of natural slopes consisting mostly of normally consolidated soft clay and the low computed value is not necessarily indicative of marginal stability.

On the other hand, some support for the belief that the bluffs had only marginal stability before the earthquake is provided by the fact that they have a long history of sliding induced by undercutting of the toe of the slopes as a result of coastal erosion. However such slides are likely to be shallow and their occurrence is not necessarily indicative of potential instability along a deep-seated sliding surface.

Whatever the margin of stability may have been it was clearly sufficient to prevent failure of the bluff slopes as a result of

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[11] Bjerrum, L. and B. Kjaernsli, "Analysis of the Stability of Some Norwegian Natural Clay Slopes," Geotechnique, Vol. 7, No. 1, 1957.

accelerations induced by[12]:

(a) an earthquake of magnitude 7.3 at an epicentral distance of about 35 miles (1943)

(b) an earthquake of magnitude 6.3 at an epicentral distance of about 50 miles (1951)

(c) an earthquake of magnitude 6.5 to 7 at an epicentral distance of about 60 miles (1954); this earthquake triggered a substantial slide along the Alaska Railroad at Potter Hill, about 10 miles south of Anchorage

and (d) the first 1.5 minutes of the Alaska Earthquake of 1964.

Thus it might well be considered adequate for many purposes. It is an extremely significant fact that if the duration of ground shaking in the 1964 quake had been no longer than that associated with many previous strong motions records, the slide would not have occurred at all and the bluffs would have been considered to have an adequate margin of stability to withstand an extremely strong ground motion.

In the light of these facts it is difficult to know whether detailed studies made before the earthquake would have predicted the probability of major sliding induced by even a major earthquake at an epicentral distance of 75 miles, or not. However it is extremely doubtful that any analyses would have anticipated the extent of inland regression of the slide (1200 feet behind a 70 ft high slope at the west end of the slide area). It is of special interest therefore to examine not only the cause of the slide occurring at all, but also the probable reasons for its large inland regression.

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12. Preliminary Report Prince William Sound, Alaskan Earthquakes March-April 1964, U. S. Coast and Geodetic Survey, Washington 25, D. C.

Causes of Slide Development

During a major earthquake, the maximum ground accelerations and the corresponding inertia forces which they induce in slopes, invariably develop during the first 30 seconds of ground shaking. In the case of the Turnagain slide, numerous eye-witnesses report no evidence of an impending slope failure during the first 1.5 minutes of the ground motions. This would seem to indicate that deformations began to develop, not during the period of application of large inertia forces at the beginning of the earthquake, but during a period of smaller inertia forces mid-way through the 'quake. The only explanation for the fact that smaller inertia forces could induce displacements not previously induced by larger inertia forces, is that the soil was in a substantially weaker condition during their period of application than it was at the time the larger forces were developed. Thus it seems reasonable to conclude that the primary causes of the landslide in the Turnagain area was the loss of strength of the soil as a result of the ground displacements induced by the earthquake; and further that if no loss of strength had developed, the stresses induced by the earthquake would have been insufficient to cause failure to occur. This conclusion dramatically illustrates the importance of considering the possible loss of strength of soils as a result of cyclic loading in assessing the stability of embankments during earthquakes.

Reasons for inland regression of slide zone

The large extent of inland regression of the slide zone may be attributed to several major factors:

(a) <u>Presence of shore-line silt deposits</u>

One of the prime factors leading to the large inland regression of the slide was undoubtedly the presence of a sloping layer of estuary silt at the toe of the bluff. The nature of this material may be seen in the photograph, Fig. 12b, which was taken at an adjacent area of the coast line. During the earthquake this silt would liquefy and thereby provide a semifluid sloping surface to facilitate the translation of soil masses falling onto it. Thus failure of the bluff by any type of slide would cause the sliding mass to be deposited on the silt and thereby continue to slide outwards into the water. This would leave a new exposed face which might well fail by a repetition of the same mechanism, this process continuing as long as the earthquake continued. In fact the slide would be able to retrogress because there was no opportunity for a build-up of slide debris at the toe of the slope to buttress the unstable condition.

(b) <u>Loss of strength in soil behind bluff-line</u>

As previously noted, the slide must have been induced by a loss of strength in the soil comprising the bluff. In fact, the configuration of the slide area following the earthquake is indicative of sliding induced by the presence of an extremely weak zone underlying the slide area and extending back well behind the original bluff-line. The probable presence of such a zone is also indicated by

(1) the lateral extension of the zone behind the slide area in the direction of the coast-line, indicating movement of level ground towards the slide area. This could only occur by movement along an extremely weak layer and indicates that the

severely weakened zone extended beyond the boundaries of the slide area, at least at the east end of the slide.

(2) Settlement of up to 8 inches in the zone just behind the slide area, probably due to movement along the weakened zone towards the slide area itself.

(3) The relatively horizontal nature of the lower boundary of the slide area, indicating that some characteristics of a depositional layer may have been responsible for the slide movements.

Possible causes of the development of such a weakened zone are

(1) a loss of strength in the clay deposits due to the cyclic shear stresses and strains induced in them by the earthquake;

and (2) liquefaction of sand lenses and seams, within the clay deposit, due to the cyclic loading induced by the earthquake.

The presence of a weak zone due to either of these causes would lead to a rapid development of the slide once it had started. It is of interest therefore to look into the possibility of such effects developing in the soil behind the bluff-line.

For this purpose an analysis has been made to assess the likely magnitude of the stresses and strains developed in the soil during the first 1-1/2 minutes of ground shaking and their effects on the sand and clay deposits. The entire deposit of soil overlying the firm rock base at a depth of about 470 ft was considered to respond as a damped elastic system to horizontal base motions caused by the earthquake and the resulting soil response was computed following the procedure described by

Seed and Idriss[13] (1966).

The soil conditions used in the analysis are shown in Fig. 13. They are considered representative of those existing in and behind the east end of the slide area before the earthquake. Shear moduli for use in the analysis were selected on the basis of vibration tests on undisturbed samples[1] giving due consideration to the effects of sample disturbance, amplitude of shear strains developed in the field and the stiffening effect of sand lenses in the soft clay. A damping factor of 20 percent was chosen on the basis of previous test data and the value found to give a reasonable assessment of soil response in the Niigata earthquake.[13] For analysis purposes the entire soil deposit was treated as a series of 27 different layers. A layer thickness of 5 ft was used in the soft and medium clays, which are of primary interest, and appropriately thicker layers elsewhere. Previous studies have shown that this number of layers is sufficient to give an accurate representation of the response of the deposit.

It was assumed that the base motions would be similar in form to those previously recorded on firm ground at a distance of 75 miles from a major earthquake. Accordingly they were considered to have the same form as the accelerations recorded at Pasadena as a result of the California Kern County earthquake of 1952, shown in Fig. 14. However since this earthquake had a magnitude of only 7.7 (compared with the Alaskan earthquake magnitude of 8.5) and a duration of strong motion of only about 35 seconds, it was considered appropriate to increase the

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[13] Seed, H. Bolton and I. M. Idriss, "An Analysis of Soil Liquefaction in The Niigata Earthquake," Soil Mechanics and Bituminous Materials Laboratory, Institute of Transportation and Traffic Engineering, University of California, April, 1966.

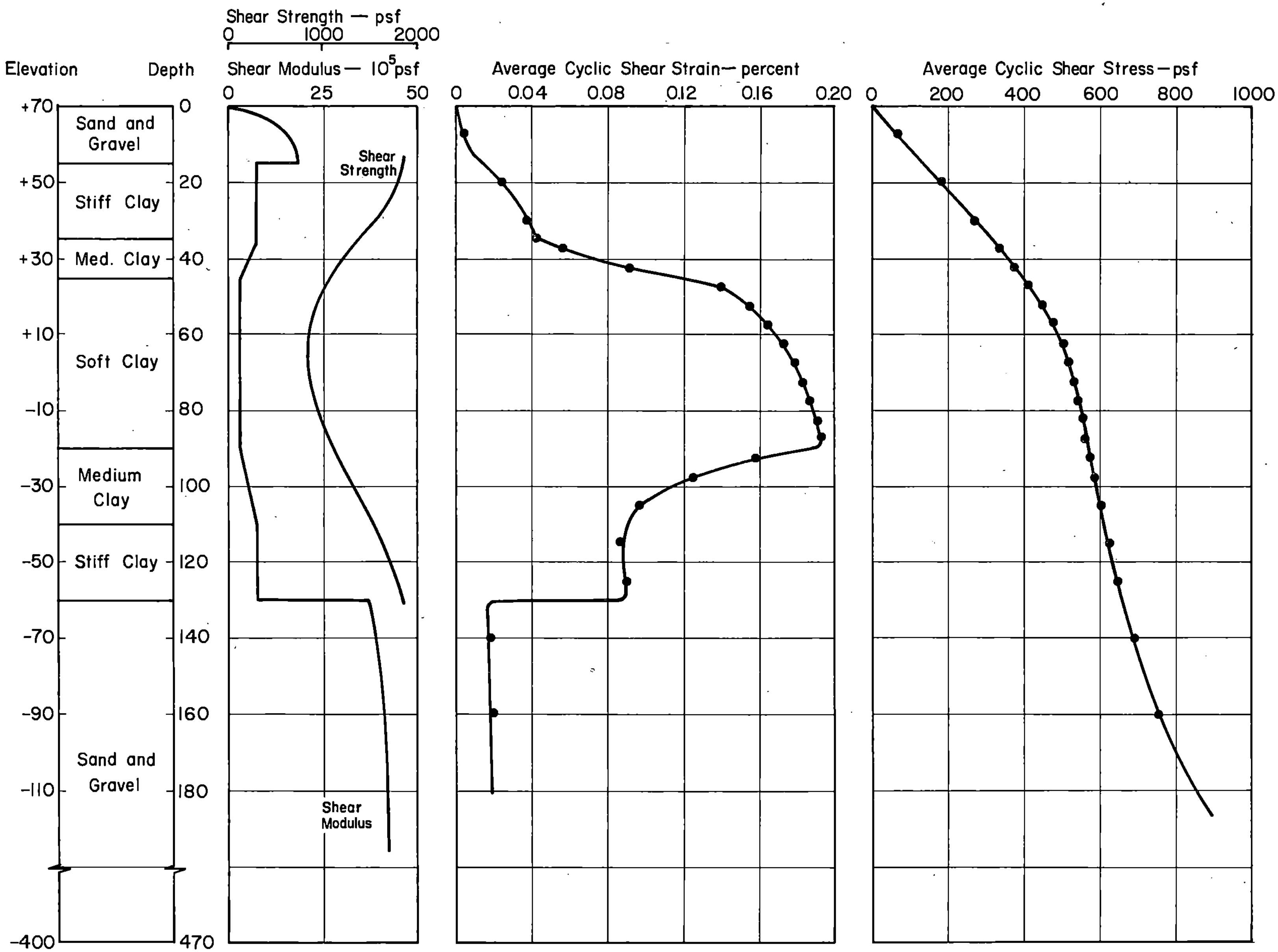

Fig. 13 RESPONSE OF SOIL DEPOSIT TO HORIZONTAL BASE MOTIONS

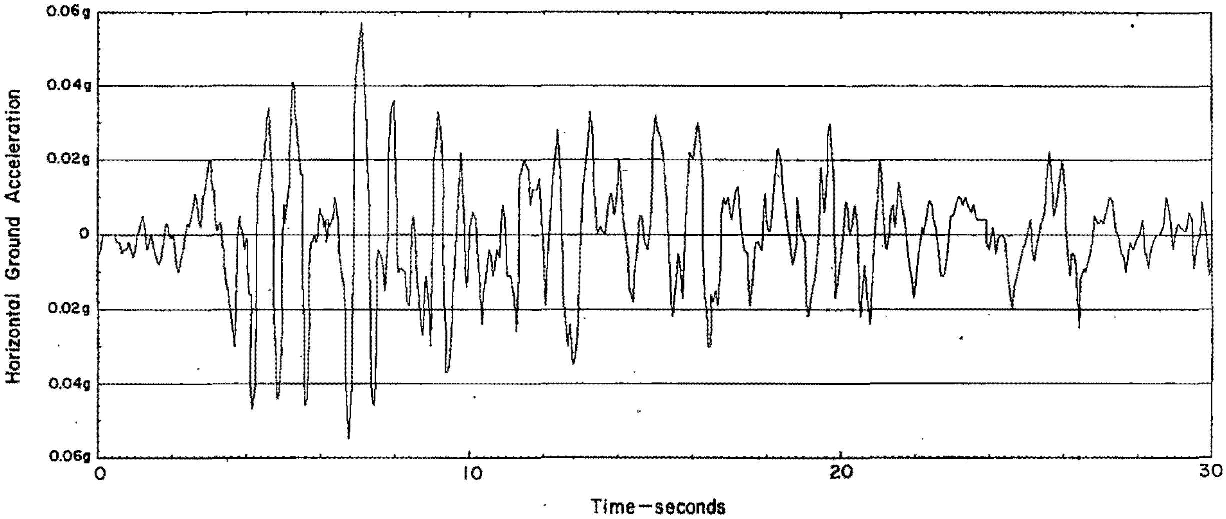

Fig.14 HORIZONTAL GROUND ACCELERATIONS AT PASADENA DUE TO KERN COUNTY, CALIFORNIA EARTHQUAKE, 1952.

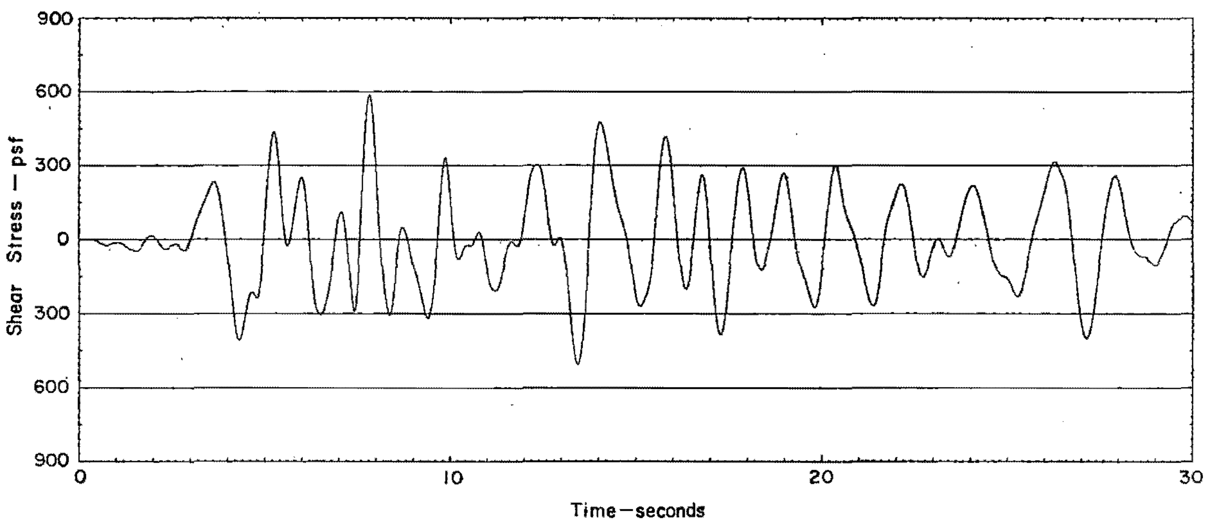

Fig.15a SHEAR STRESS VARIATION NEAR TOP OF SOFT CLAY LAYER (EL. 22.5)

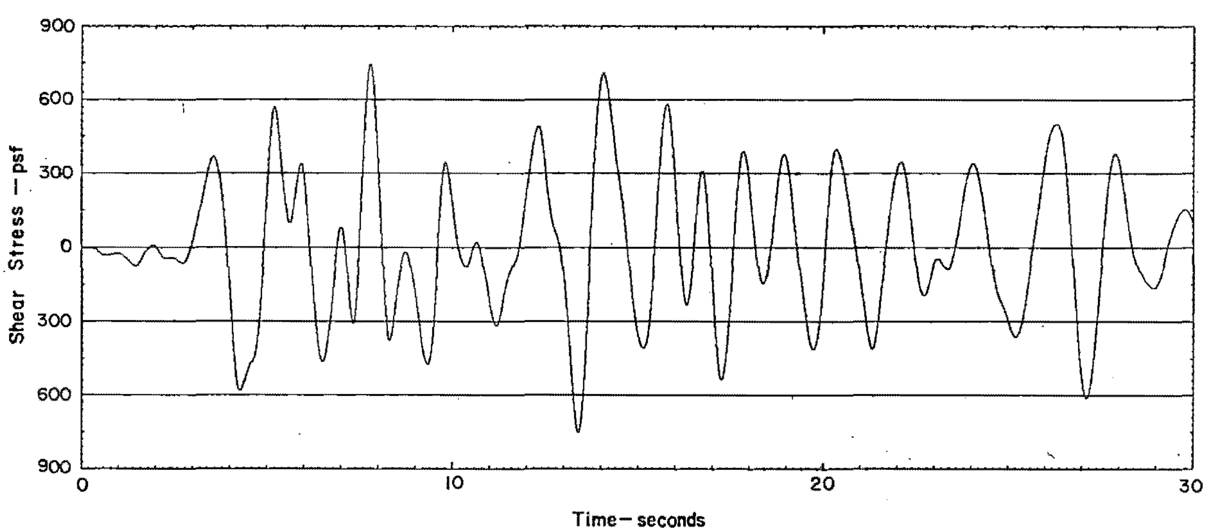

Fig.15b SHEAR STRESS VARIATION NEAR BOTTOM OF SOFT CLAY LAYER (EL.-17.5)

amplitude of the motions by a scaling factor of 1.5 and to simulate the first 2 minutes of the longer duration of the Alaskan earthquake by repeating the Pasadena record three times in sequence. Thus the record shown in Fig. 14, with the ordinates multiplied by 1.5, was considered to represent 30 to 40 seconds of base rock shaking in the Anchorage area during the first two minutes of the Alaskan earthquake.

The computed response of the soil deposits to this base motion are presented in Table 1 and Fig. 13. The maximum ground surface acceleration was determined to be 0.11 g and the fundamental period of the deposit was 2.01 seconds. These values are in good agreement with corresponding values estimated by residents and observers of the Anchorage ground shaking.

The computed variations of shear stresses developed near the top and bottom of the soft clay layer are shown in Fig. 15. The stresses near the bottom of the layer have the same form as those near the top of the layer, but they are significantly larger in magnitude. At both levels the amplitude of the stress cycles varies considerably. However a careful study of these response patterns shows that for analysis purposes, it is reasonable to represent the stress history near the top of the soft clay by 10 significant cycles with an average amplitude of 415 psf and that near the bottom of the layer by the same number of stress cycles with an average amplitude of 560 psf. These average stress cycles have a period of about 1.2 seconds and occur at a rate of 10 cycles per 30 to 40 seconds of ground shaking.

Following this procedure, the average cyclic shear stress and shear strain were determined at all levels in the soil deposit and the resulting values are presented in Table 1 and Fig. 13. It is readily

Table 1

Response Analysis of Soil Deposits

Elev.	Depth ft.	Material	Shear Strength s_u-psf	Shear Modulus $x10^5$-psf	Average Cyclic Strain, percent	Average Cyclic Stress, τ_c-psf	Effective Overburden Pressure, σ_o' -psf	Ratio τ_c/σ_o'	Ratio τ_c/s_u
62.5	7.5	Sand and gravel	-	15.1	.0045	69			
50	20	Stiff clay	1800	7.2	.0255	184	2100	0.87	
40	30	Stiff clay	1600	7.2	.0376	271	2700	1.00	
32.5	37.5	Med. clay	1300	5.9	.057	335	3140	1.07	
27.5	42.5	Med. clay	1100	4.1	.091	375	3415	1.10	
22.5	47.5	Soft clay	1000	2.9	.142	414	3675	1.12	0.41
17.5	52.5	Soft clay	900	2.9	.155	450	3925	1.14	0.50
12.5	57.5	Soft clay	870	2.9	.165	480	4175	1.15	0.55
7.5	62.5	Soft clay	850	2.9	.173	501	4425	1.13	0.59
2.5	67.5	Soft clay	850	2.9	.179	519	4675	1.11	0.61
-2.5	72.5	Soft clay	860	2.9	.183	530	4925	1.08	0.61
-7.5	77.5	Soft clay	890	2.9	.186	540	5175	1.04	0.60
-12.5	82.5	Soft clay	950	2.9	.191	555	5425	1.02	0.58
-17.5	87.5	Soft clay	1000	2.9	.193	560	5675	0.99	0.56
-22.5	92.5	Med. clay	1150	3.6	.159	572			
-27.5	97.5	Med. clay	1250	4.7	.125	588			
-35	105	Med. clay	1400	6.3	.096	605			
-45	115	Stiff clay	1600	7.2	.087	625			
-55	125	Stiff clay	1800	7.2	.090	646			
-70	140	Sand and gravel	-	38.1	.018	690			
-135	205	Sand and gravel	-	42.9	.022	930			
-275	345	Sand and gravel	-	50.2	.032	1620			

apparent that the maximum shear strains occur in the soft clay layer, with the magnitude increasing progressively from about 0.12 percent to about 0.19 percent with increasing depth in this layer. Because of the varying shear moduli, the average cyclic shear stress increases progressively with increasing depth in the soil deposit as indicated in Fig. 13.

The values of cyclic shear stress shown in Fig. 13 together with the cyclic loading test data presented previously can be used to assess the possibility of strength loss in the soil deposits behind the bluff. However in making this assessment it is important to recognize that the analysis was made using average shear moduli for clay layers containing sand lenses. Since the sand in such deposits (unless it liquefies) will be significantly stiffer than the surrounding clay, the stresses in the sand lenses would actually be higher than those shown on the figure while the stresses on the adjacent clay would be correspondingly lower.

A loss in strength of sand during cyclic loading may develop as a result of liquefaction. Details of a method for analyzing the possibility of liquefaction developing have been presented by Seed and Idriss[13] and Seed and Lee[10]. It has been shown that in a deposit of sand of uniform relative density, the danger of liquefaction is determined by the ratio of the cyclic shear stress to the initial effective overburden pressure, τ_c/σ_o'; the larger the ratio, the smaller is the number of stress cycles required to induce liquefaction. Values of this ratio for the upper 90 ft of the soil deposit are listed in Table 1. It will be seen that the ratio is a maximum at a depth of 57.5 ft, indicating that liquefaction, if it occurred at all, would be likely to develop first at about El. 12.5. However this is not a strong maximum and initial liquefaction might be expected to occur anywhere in the range El. 0 to El. 20. If liquefaction occurred, the pattern of cyclic shear stresses would be changed and assessments of the possibility of further liquefaction could not be made from the data in Fig. 13.

The possibility of liquefaction occurring at El. 12.5 may be assessed by comparing the magnitude of the cyclic shear stress developed at this level with the cyclic stresses required to induce liquefaction of the sand in cyclic loading tests for comparable initial conditions. This comparison is shown in Fig. 8, from which it may be seen that the computed average value of the shear stress developed at this elevation would induce liquefaction in about 12 stress cycles; that is, in about 45 seconds of ground shaking. Since the actual stresses probably exceed the computed stresses as noted above, liquefaction of sand lenses or layers in the zone around El. 12.5 might reasonably be expected to occur within the first 45 seconds of the earthquake.

A similar analysis can be made to determine the possibility of loss of strength in the clay. For a saturated clay, the possibility of failure occurring is indicated by the ratio of the average cyclic shear stress to the undrained shear strength, τ_c/s_u, with higher ratios being more critical. The computed values of this ratio are listed in Table 1, from which it may be seen that the ratio is a maximum in the zone from El. +3 to El. -3. Again, however, there is no strong maximum and the most dangerous condition could occur in the range from El. +10 to El. -15.

The number of stress cycles required to induce failure at the most critical elevation can be assessed by comparing the computed ratio of

τ_c/s_u with the values of this ratio required to cause failure of the clay as determined by cyclic loading tests. This comparison is shown in Fig. 7, from which it may be seen that for constant stress conditions, failure of the clay would require about 22 stress cycles, or about 1.5 minutes of ground shaking.

On the basis of this analysis it would appear that liquefaction of the sand between El. 0 to 20 would be likely to occur before any significant strength loss developed in the clay. In fact such liquefaction might be expected to have occurred in the first 45 seconds of the earthquake, so that the soft clay behind the bluff contained numerous lenses of liquefied sand; at a somewhat later stage strength loss of the clay in some zones would also develop. This sequence of events is also indicated by the results of cyclic strain tests conducted on samples of the sand and clay under conditions representative of those developed in-situ.

Thus it would appear that before failure occurred near the bluff line, the soil behind the bluff had developed a severely weakened and largely liquefied zone somewhere between El. 0 and El. 20; once failure occurred at the bluff line permitting movement along this weakened zone, sliding would progress rapidly leading to the large inland regression of the slide area. However because of the lenticular nature of the sand seams, the weakened zone or main slide surface would necessarily pass through clay in some section of its length, producing a composite surface on which sliding would develop. It is perhaps worthy of note that the most probable zone of sand liquefaction indicated by the analysis (El. 0 to 20) is in somewhat better agreement with the probable position of the base of the main slide surface (El. +10 to El. +5) than is the computed position of the zone of maximum weakening in the clay (El. +10 to El. -15).

Evidence supporting the concept that sand lenses or seams in and behind the slide area played an important role in the development of the slide is provided by:

(1) several samples recovered from the slide area showed sand and clay intermixed in a form which could only have occurred as a result of the sand possessing fluid characteristics.

(2) ridges of sand, 2 to 3 ft high, 3 to 6 ft wide and about 100 ft long were formed by sand boils within the slide area[2] as shown in Fig. 16a, (A large boil, spread over an area of 3200 sq ft., was also noted near but outside the slide area by the Engineering Geology Evaluation Group[3]).

(3) A description of the slide movements by residents of a house in the slide area during the earthquake: "The floor ripped and sand came up from below into the living room." It is difficult to imagine such an inflow of sand except by liquefaction.

(4) Although there is considerable evidence (lateral extension, cracking, settlement) that the area behind the slide area was underlain by a severely weakened layer during the earthquake, there was no evidence of a weakened clay zone underlying this area revealed by borings made several weeks after the earthquake.

(5) Although sand lenses were encountered in many borings made in the slide area, very few were noted in borings made

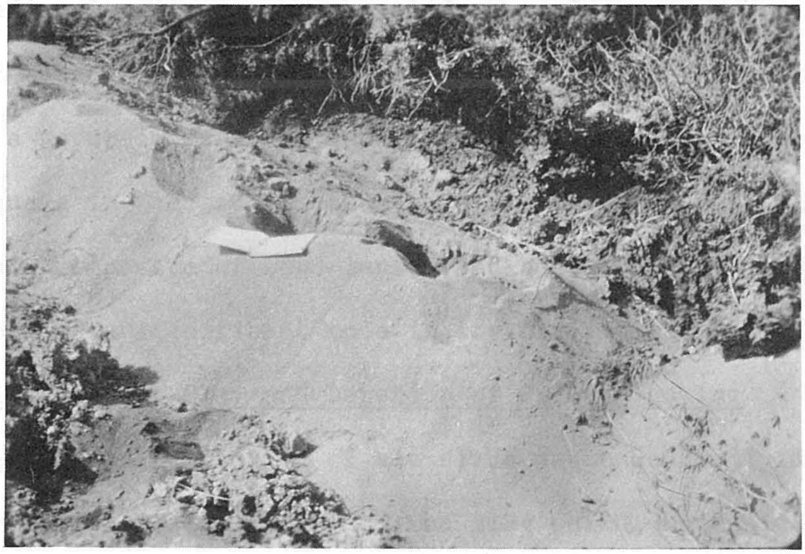

Fig.16a RIDGE OF SAND FORMED BY SAND BOIL IN SLIDE AREA

Direction of Sliding

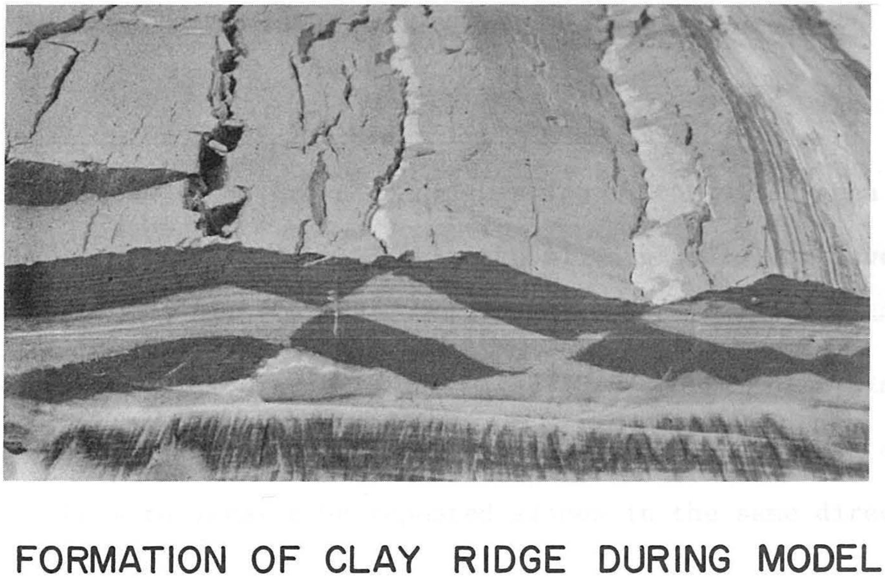

Fig.16b FORMATION OF CLAY RIDGE DURING MODEL TEST

immediately adjacent to the slide area.[14]

While some of this evidence is circumstantial, it lends support to the concept that a severely weakened zone developed in the early stages of the earthquake in the area behind the bluff-line and that although this zone was a composite of sand lenses and adjacent clay, liquefaction of the sand played a significant role in its development.

(c) Sensitivity of Soft Clay and Duration of Ground Shaking

The large inland regression of the slide was undoubtedly also influenced by the high sensitivity of the clay which, once sliding of a mass was initiated, would result in a further drastic loss of shear strength due to remolding in the shear zones thereby facilitating lateral displacement. Furthermore, the extremely long duration of the earthquake (more than 4 minutes) was a prime factor in determining the extent of the movements. Sliding stopped for the most part once the ground motions stopped and there can be little doubt that the extent of the slide zone, especially at the east end of the slide area, would have been substantially increased in the earthquake had continued a little longer.

Mechanics of slide movement as indicated by model tests

An examination of the ground configuration in the slide area (see Figs. 1 and 2) immediately reveals that the sliding could not have developed simply as a result of a progressive series of conventional slides in clay along circular arcs directed towards the coast-line. Trees and poles could be noted inclined in opposite directions - a pattern not likely to develop by repeated slides in the same direction.

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[14] U. S. Army Engineer District, Anchorage, Alaska, Internal Report on Turnagain Earthquake Slide Studies.

Furthermore the vertical ridges of clay, previously described, which moved through large distances with little or no change in elevation were a characteristic feature of the slide area and the mechanism leading to their formation required clarification. Finally, it was important to obtain some concept of the probable mechanism of sliding in order to obtain a better concept of the nature of the material in the slide zone and thereby assess its vulnerability to further sliding if a similar earthquake should occur.

To throw some light on the mechanics of slide development when clay soils are underlain by a layer of extremely weak material, a series of model tests were conducted at the University of California. A bank of clay, about 4 inches high, composed of an extremely weak layer overlain by layers of stronger clay was constructed to the approximate configuration of the bluffs in the Turnagain Heights area. The layers of clay comprising the model were given different colors to facilitate observations of the slip surfaces and the model was maintained in a stable condition during construction by supporting the face and toe of the slope with a retractable bulkhead. Thus the model might be considered to represent the conditions in the Turnagain area after a severely weakened zone had developed behind the bluff-line after 30 to 60 seconds of the earthquake.

After construction, the retractable bulkhead supporting the model embankment was removed and sliding was allowed to develop. In a number of tests sliding was facilitated by vibrating the model on a shaking table after the bulkhead was withdrawn. It was found that the general pattern of the slide area in the model bore a close resemblance to that in the

Turnagain area, with ridges of clay developing and translating through substantial distances without change in elevation.

The models provided an excellent means for observing the mechanics of slide development. It was found that failure usually involved the following sequence of movements, illustrated in Fig. 17:

(1) A series of several rotational slides, starting just behind the crest of the slope and retrogressing backward from the slope, causing outwards and downwards movement of the ground surface; these slides are accompanied by extrusion of the weak layer from the toe of the slide (Figs. 17a to 17d).

(2) After several such slides, the upper layers of stronger clay, which have moved downwards, cut through the weak layer so that no further extrusion from the toe can occur, (Fig. 17d). Beyond this point continued sliding results in the outward movement of a prismatic ridge of soil, essentially without change in elevation (Fig. 17e).

(3) Following the lateral translation of the ridge, extensive tension cracking develops in the clay behind the ridge giving the impression that a wide zone of soil is affected by the movements (Fig. 17f).

(4) Settlement of the area behind the ridge forming a sloping depressed zone in which the tension cracks may separate the soil into a series of blocks. This is well illustrated by the photograph in Fig. 16b which was taken during the conduct of a model test and by Fig. 17f.

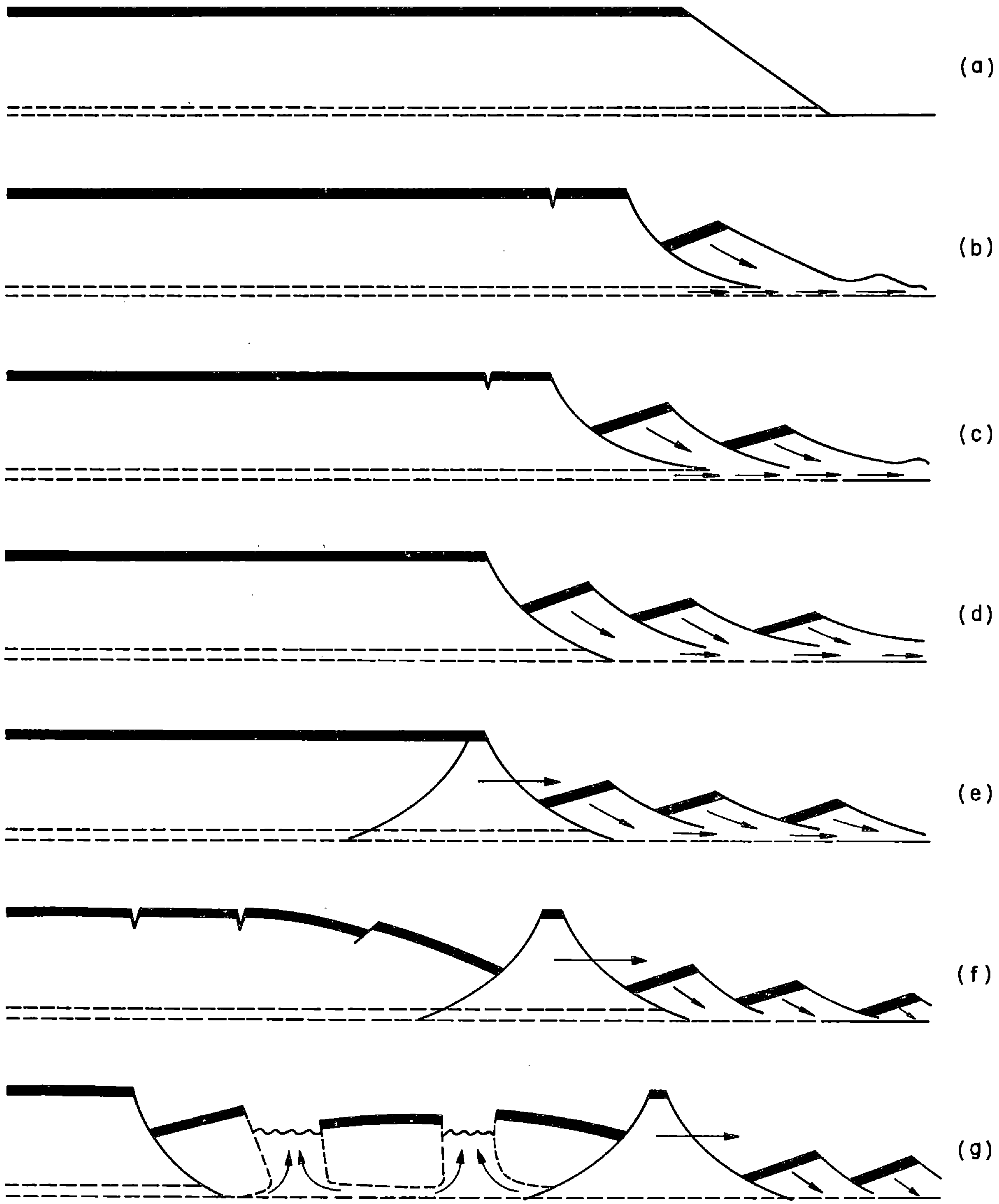


Fig.17 FAILURE MECHANISM OBSERVED IN MODEL TESTS

(5) When the outer end of the depressed zone has subsided sufficiently a conventional type of slide movement occurs along a surface consisting of a short curved portion in the stiff clays and a substantial horizontal section along the weak layer at the base. This movement causes lateral translation and subsidence of the soil behind the ridge, forming a new scarp at the back of the slide area and a graben-type of depression. It is usually accompanied by a slight heaving of the previously subsided toe of the graben area, and sometimes by a separation of the slide mass into several blocks by weak material squeezed to the sides of the blocks as they subside (see Fig. 17g).

(6) During the rotation and subsidence of the stiff clay to form the new scarp line, the downward movement of the stiff clay again cuts through the weak layer so that its extrusion from the base of the yet-unaffected area is prevented. Thus continuation of the slide, either due to the fact that the base is slightly inclined or to inertia forces induced by shaking, causes a repetition of the operations described in (2) to (5) above and a progressive retrogression of the slide area.

It is important to recognize that:

(1) The prismatic ridges were only observed to form when the weak layer was unable to extrude through the base of the slide area. Thus, apart from the possible slope of the main sliding surface the ridges maintain their original elevations, no material moves through them and all the material which was originally behind a ridge remains behind it as the entire mass translates laterally.

This latter fact provides a basis for determining the nature of the movements in the slide area.

and (2) The subsidence of material to form a gráben type depression was accompanied by some sideways extrusion of underlying weak material. It is impossible for subsidence and spreading of the original ground to occur without some extrusion of material in this way.

The concepts of slide movements developed from the model tests provided a valuable key to understanding the mechanism of slide development and to analyzing the soil conditions in the slide area.

Mechanics of Slide Development

Based on the foregoing concepts and analyses, it is possible to develop a hypothesis for the mechanics of the Turnagain Heights landslide as follows:

(1) Possibly due to the greater depth of clay near the bluff-line, which would give the material in this area a longer period of vibration and a reduced response, the soil comprising the bluffs was sufficiently strong to withstand about 2 minutes of ground shaking before failure developed.

(2) As long as the bluffs remained stable, the soil behind the bluffs was buttressed against lateral movement. However during the first minute or so of the earthquake, liquefaction of sand lenses occurred at about El. 5 to 20 at the east end of the slide area and El. 15 to 25 at the west end of the slide area so that a severely weakened zone extended backward from the temporarily stable bluffs to some considerable distance inland.

(3) Finally, as a result of inertia forces and loss of strength in the soils, failure developed near the bluffs by a conventional type of slide mechanism. The slide mass slipped outwards into the water by sliding on the surface of the sloping silt deposit, leaving the slide surface exposed and permitting the development of a second slide of the same type.

(4) After one or more such slides had occurred, the sliding surface merged into the previously weakened zone behind the original bluff-line. At this stage the mode of failure changed to a form similar to that shown in Fig. 17, and failure progressed rapidly resulting in the development of ridges of clay throughout the slide area and badly disintegrated depressed zones between the ridges. At this stage the main surface of sliding was the zone severely weakened by liquefaction of sand lenses, but the sliding also involved extensive shear zones in the clay. Remolding of the clay between lenses and elsewhere during sliding resulted in a substantial quantity of very weak clay in the slide zone.

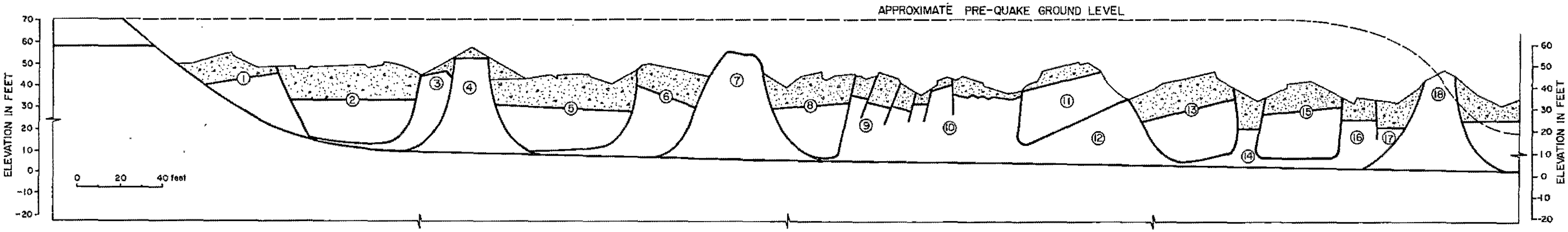
(5) Translation of soil in the slide zone was facilitated by the severely weakened zone and once slide debris passed beyond the bluff-line, it slipped rapidly outwards along the sloping surface of the off-shore silt deposit.

(6) Sliding continued as long as the earthquake continued but movements were rapidly arrested once the ground motions stopped, partly because the off-shore silt deposit was stabilized, sand deposits in the slide zone were stabilized and inertia

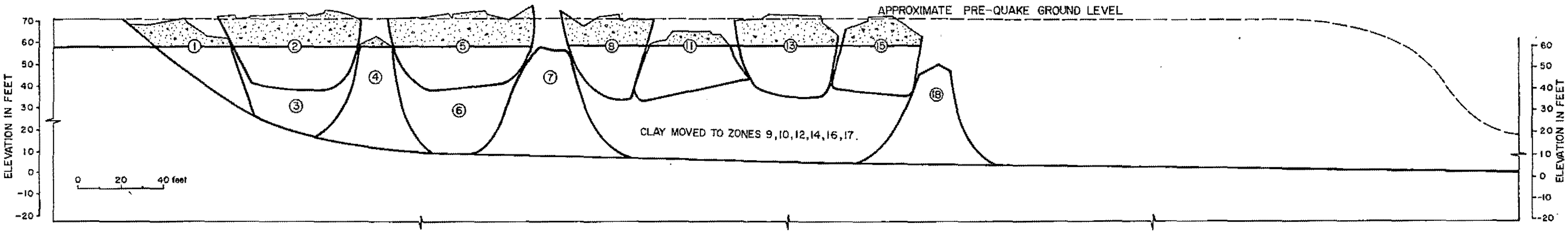
forces were eliminated. However the substantial volume of remolded clay in and near the slide zone permitted a continuation of sliding to some extent after the ground motions had stopped.

In keeping with this hypothesis and the pattern of slide development indicated by the model tests, it is possible to reconstruct the mechanics of failure leading to the configuration of material in the trench section through the east end of the slide area, shown in Fig. 18. Since no material moves through the ridges of clay, the volume of soil between ridges after sliding must be the same as the volume between ridges before sliding. Thus the initial positions of the ridges can be determined by finding the lateral spacing required to recompress the soil into its original thickness. The probable vertical positions of the intervening soil blocks can then be determined from a comparison of their dimensions, the available space between the initial positions of the clay ridges and the initial position of the well-defined boundary between the clay and the overlying sand and gravel outwash.

By means of this procedure, the probable initial positions of blocks of soil in the slide zone has been reconstructed as shown in Fig. 18. In the space between the slide scarp and the first clay ridge (block No. 4), the volume of material indicates that this clay ridge must have had the initial position shown in the reconstructed section. The probable position of block No. 1 can be determined by sliding it back along the slide scarp until the outwash/clay boundary is in the correct initial position. It is then found that soil block No. 2 fits into the space between block No. 1 and the clay ridge. Thus it appears that the soil occupying space No. 3 in the slide section must have been extruded



SOIL PROFILE AFTER EARTHQUAKE



RECONSTRUCTED SOIL PROFILE

Fig.18 RECONSTRUCTION OF SOIL DISPLACEMENT AT EAST END OF SLIDE ZONE laterally from the space below block No. 2 in the reconstructed section. This is in agreement with the fact that the clay in space No. 3 was observed to be very badly disturbed during construction of the trench (see Fig. 10).

Proceeding in this way leads to the full reconstructed section shown in Fig. 18. It will be seen that the procedure indicates that the clay ridges underwent large lateral translations during the sliding (ridge No. 18 moved a distance of 240 feet). These displacements are in good agreement with the observed movements of houses in the slide area shown in Fig. 4. The slide reconstruction also indicates that failure involved the displacement of large zones of clay in the depth range from El. +5 to +30. Thus the condition of the clay within the slide area would be expected to vary enormously. At some sections it would be relatively undisturbed except for a small depth near the main failure zone, while in others it would consist of a thick layer, down to El. +5 approximately, of displaced and badly disturbed material. This would serve to explain why two borings in the slide area, made only 10 to 20 ft apart but with the one passing through block No. 2 in Fig. 18 and the other passing through section No. 3, might show radically different strength variations in the clay.

Conclusion

The preceeding pages have presented the results of a study of the Turnagain Heights Landslide which occurred during the Alaska Earthquake of March 27, 1964. The main conclusions resulting from the study are:

(1) The nature of the Bootlegger Cove clay formation is very complex, involving sand and silt lenses and seams distributed throughout

a silty clay varying in sensitivity from 5 to 35. It is extremely difficult to analyse the behavior of such deposits even under static loading and still more difficult under earthquake loading conditions.

(2) Since no measurements are available concerning the ground motions developed at Anchorage during the earthquake, analyses of the soil behavior must necessarily be based on estimates of motions provided by observers in the area. It is extremely desirable that efforts be made to establish strong-motion accelerographs in areas likely to suffer major damage during earthquakes in order to provide a basis for analyses of soil behavior.

(3) The landslide at Turnagain Heights did not begin until about 1-1/2 to 2 minutes after the earthquake started. It is reasonable to believe therefore that it did not develop during the period of maximum ground motions, but developed as a result of loss of strength in the underlying soils as a result of the sequence of earthquake motions.

(4) The extensive inland regression of the slide was probably due in large measure to liquefaction of sand lenses and weakening of clay soil over a zone extending well behind the original bluff-line, the presence of an off-shore deposit of silt which facilitated removal of slide debris from the toe of the slide area, the high sensitivity of the clay, and the extremely long duration of the earthquake.

(5) Sliding was accompanied by large lateral translations of material and the formation of numerous characteristic ridges of clay which translated with little change in elevation. The main surface of sliding was a weakened zone varying in elevation from about El. 8 at the east end of the slide area to about El. 20 at the west end. However the mechanics of soil movement above this zone were complex, involving the subsidence of large blocks of soil and the lateral displacement of clay in a layer of clay about 25 ft in thickness.

(6) The material in the slide area following the earthquake varied widely from relatively undisturbed material in some sections to extremely disturbed material in others. Reconsolidation of the remolded material under the new overburden pressures prevailing in the slide area would lead to considerable strengthening of the slide area with time and to marked differential settlements.

(7) Assessment of the vulnerability of slopes to failures during earthquakes is a complex problem involving considerations of the intensity and duration of ground shaking, the effect of the shaking on soil properties, the forces induced by the shaking, the nature of the soil deposits involved and the magnitude of displacements which may occur. In many cases the margin between severe destruction of an area of sliding and completely adequate performance may be very slight. As events developed, the Turnagain Heights landslide may now be cited as a classic case of a major slide disaster induced by an earthquake

On the other hand, if the duration of ground shaking in the Alaska earthquake had been at all comparable to that recorded in other large earthquakes (say 1 to 1-1/2 minutes) the slide would probably not have developed and the area could have been cited as a classic case illustrating the safety of slopes underlain by clay soils during even major earthquakes. In the light of such possibilities, experiences of slope failures or non-failures during earthquakes require careful appraisal before they can be translated to other areas and conditions.

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