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A GUIDE TO THE GEOLOGY OF ANCHORAGE: A COMMENTARY ON THE GEOTECHNICAL AND HISTORICAL ASPECTS OF SELECTED LOCALITIES IN THE CITY

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Geological and Geophysical Surveys

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enhance future editions of this guidebook.

PREFACE

Anchorage is a dynamic city both in the sense of its rapid and diverse socio-economic growth and with respect to the geologic setting within which it is located. Even with the profound strides the people of Anchorage have made in the realms of culture, technology, industry, and commerce, still the complexity of the earth mechanisms influencing the region make progress here an unrelenting challenge. The following guide is not intended to be an exhaustive study of either the geology or the history of the area. Instead, it is our intent to provide you with a sense of the geologic problems, as well as the successes and the occassional failures, which have characterized the brief but colorful history of Anchorage. We encourage you to pursue individual topics introduced herein; you may find some of the references in the Bibliography to be of help. We welcome you to Anchorage and sincerely hope that you will enjoy your visit.

- R.U.

Introduction

The natural setting of Anchorage has been quite conducive to growth as a population and economic center. Topography in the central Anchorage area is of low relief, gently sloping from the Chugach mountains on the east toward Cook Inlet on the west and south. Much of the central part of Anchorage lies on a broad, gently sloping alluvial plain truncated by bluffs along the Inlet and along major streams flowing from the mountains to the Inlet, the most important of which is Ship Creek.

In addition to favorable topography, the region also offers a moderation of the more typical Alaskan climates, being warmer and wetter than the continental climate of the interior of the state and yet cooler and drier than the maritime climate of the coastal areas. This climatic modification is a result of the surrounding mountain ranges and Cook Inlet. The mean annual temperature is 36°F. Average minimum winter and maximum summer temperatures are 6°F and 66°F respectively. The coldest temperature recorded is -68°F, the warmest being 83°F. The mean annual precipitation is 14.5 inches and the mean annual snowfall is 59 inches. To the east, in the Chugach Mountains, the temperatures are lower and precipitation higher supplying the Anchorage lowland with abundant water resources both through surface runoff and groundwater aquifers. The average wind velocity is 5 mph. The prevailing winds are out of the northeast in winter and northwest in summer. Gusting winds along the Chugach mountain front occasionally cause damage to hillside homes and businesses.

Vegetation in the Anchorage lowland outside the urban areas varies somewhat in relation to water table levels and soil drainage. The well drained podsol soils support coniferous types of vegetation, mainly spruce, alder, and willow, and the deciduous types, primarily birch, cottonwood, and aspen. Mixed coniferous and deciduous trees are common in coastal areas that have not been disturbed. In the lowland areas of poor drainage, spruce bogs consist of stunted and slowly growing black spruce. These spruce bogs are sometimes referred to as "drunken forests" or "elfen forests" where the spruce are extremely stunted and leaning at various angles which may indicate the presence of near-surface permafrost. Treeless bogs occur where the vegetation is primarily mosses and shrubs caused by extremely poor drainage.

General History

On the famous search for the Northwest passage, British explorer Captain James Cook sailed into the waters that bear his name, dropping anchor off Fire Island within site of present-day Anchorage. On June 1, 1778, Captain Cook sent 2 boats ashore and took possession of this part of Alaska in the name of George III. Cook described encounters with natives of the area and surmised the Russians had not had direct contact with the people yet. The natives of the region, the Tanaina people of the Athapaskan tribe, were limited in their travel by the mountains enclosing the area on three sides. Russian fur traders arrived in the early 19th century as did Russian orthodox missionaries and there is much evidence of the Russian influence on the native culture. The native population may have numbered several thousand at this time, however, a smallpox epidemic began in 1839 and killed almost 50% of the population. Later, an influenza epidemic in 1918 struck the population, further decimating their numbers.

Today, the Russian Orthodox church, St. Nicholas, probably built around 1870, and a cemetery at the village of Eklutna, 30 miles north of Anchorage, is the closest remaining evidence in the area of this early Russian and Native history. The people of Eklutna restored the church in 1975 and maintain the colorful spirit houses in the cemetery.

Archaeological research of earlier inhabitants has shown at least seasonal Eskimo habitation in the Cook Inlet area dating from 1000 AD to 1700 AD. Archaeologists conclude that the Tanaina people arrived in the area sometime between 1650 AD and Captain Cook's 1778 visit. The Tanaina people established fish camps along Ship Creek in the summers. The only archaeological site in the Anchorage bowl area is at Point Woronzof, where evidence of summer camps and possibly a permanent winter site have been studied. One group of house pits possibly dates back prior to 1741, the time of Vitus Bering's explorations. There is also an archaeological site at Beluga Point, a rocky point jutting into Turnagain Arm about 1 mile south of McHugh Creek, that has been termed the most important site in the Upper Cook Inlet area in terms of the area's prehistory. Materials from this site have been dated to range in age from 800 years to estimates of 6,500 to 9,000 years ago.

Although Captain Cook claimed much of the territory he visited for the British, it was simply too difficult to develop

without a northwest passage. Subsequent British expeditions to Alaska found Russian trade firmly established. The Russian interest in Alaska's furs from the abundant sea otter dates back to the mid- 1740's. Ruthless bands of professional hunters arrived in the Aleutian Islands. Countless Aleuts unfortunately were slaughtered by these Russians along with the sea otters. Eventually the Russians realized the Aleuts were more valuable alive and the remaining Aleuts were enlisted as hunters along with their well-suited native craft, the kayak.

The Russian Czar granted the fur monopoly with all Russian activities in Alaska consolidated under the Russian-American Company in 1799 with Alexander Baranov as resident manager. Baranov had been busy since 1790 establishing trading posts on Kodiak Island and at Sitka. Russian orthodox missionaries and the Russian-American Company combined to bring some degree of fairness in dealing with the native population, at least when compared to the first group of Russian fur hunters in the Aleutians. Many native people were converted to the Christian faith by the missionaries. Sitka became the central government and grew into quite a civilized community during the next half century and the fur trade flourished.

However, in the mid 1860's the Russians faced many problems and the North American colony was becoming too much of a burden. In 1867, Russia sold Alaska to the U.S for $7,200,000. Much opposition was in evidence as Americans termed the deal "Seward's Folly" and called the territory "Seward's Icebox". The perception of Alaska as one continuous snowfield and little else even persists today, despite the fanfare of statehood, the enormous publicity generated by the building of the Alaska pipeline, and the attention paid to the vast Natural Resources of this state.

The transfer of Alaska to American rule was not without problems. The Russians departed peaceably and the Americans sent Brigadier General Rousseau and 250 soldiers to oversee the transfer. The town of Sitka wished to set up a government to establish order. However, they found that Alaska being neither a state nor a territory was something of a legal non-entity. Congress cared little about supporting the new land and the military was not especially interested in governing the Americans in Alaska, much less their own troops. Reports were not indicative of smooth, well-run society. Alaskans mostly decided to establish their own systems and tried to create some order.

Congress remained unconcerned and, still being displeased with the original purchase, took little action to help the territory, passing the Organic Act in 1884 which gave Alaska a governor and placed it under the civil and criminal laws of Oregon. In 1900, after a slew of lawsuits over gold-mining claims in Nome, along the Yukon, and elsewhere, Congress was forced to establish an act providing for three U.S. District Judges for Alaska. Then in 1906, Congress allowed Alaska to send a non-voting delegate to Washington. Finally, in 1912, 45 years after the purchase and only after numerous problems from the gold rushes of the previous years and the attendant development, Congress passed legislation giving Alaska Territorial status with an elected legislature.

The town of Anchorage actually got off to its start with a survey crew of the Alaska Engineering Commission in 1914. The AEC was the federal agency charged with building the Alaska Railroad. Gold, it seems, was the eye-catcher congress had needed to take note of its northern territory. Once the rush had eased, the desperate need to open up the vast, inaccessible country finally was addressed. Congress had given President Taft authority to establish a commission to develop the country and its resources along with the territorial status in 1912. An initial study was made in the fall of 1912 but no decisions were made as Taft prepared to turn over the government to newly elected President Woodrow Wilson. Wilson was definitely interested and supportive of the Alaska Railroad legislation as was his Interior Secretary. The Alaska delegate, James Wickersham, worked to garner more support and the Alaska Railroad bill was passed in 1913. This bill authorized the president to locate, construct, and operate a railroad by which the Pacific could be united with the navigable waters of the Interior. Two primary routes were studied, one based out of Cordova and using the Copper River as a northwestern route, the other with a basic alignment linking Seward to Fairbanks through the Susitna basin, known as the western route. The survey crew of AEC in 1914 was particulary interested in access to the Matanuska coal fields and argued for the Susitna route with headquarters at Ship Creek.

There was a community in the Cook Inlet area at that time at Knik, located further up the Knik Arm on its northern shore. Unfortunately, this trading community was not located on the direct route for the proposed railroad nor did it have a good harbor. The AEC picked Ship Creek as location for headquarters

because it was on the proposed route and also offered good anchorage for supply ships.

In 1914, the AEC crew found few inhabitants in the Ship Creek area - a homesteader 4 miles upstream on Ship Creek, 2 cabins occupied by foresters nearer the mouth of Ship Creek, a small cabin in the Government Hill area, and a warehouse belonging to G.W. Palmer supplying his trading post at Knik. Ship Creek became AEC headquarters and a few log buildings were built. By March of 1915, "Tent City", as the original camp was called, began to form along the north side of Ship Creek beneath Government Hill on speculation that President Wilson would pick the western route for the Alaska Railroad. On April 9, 1915, Wilson announced his choice of the western route for the railroad. By June, nearly 2,000 people lived in tent city, most of them sourdoughs from all over Alaska and the Northwest Territory.

The tent city location had become overcrowded and was needed for the expansion of the railroad. In July of 1915, the AEC laid out a townsite and held an auction. The townsite was chosen to the south of the tent city and the plan was reminiscent of western railroad towns of the past. Originally, 240 acres were cleared and the boundaries were from present day 1st Avenue to 9th Avenue and from the bluffline to Cordova Street. Areas were reserved for municipal, federal, and school buildings as well as for the railroad and park areas. Elderberry Park, at the end of Fifth Avenue on the inlet is one of the 3 original park reserves from the townsite. A crowd estimated at between 1,000 and 3,000 turned out for the auction and the mood was indeed speculative. Several lots in the central business area sold for 3 and 4 times their appraised values.

There are various stories about how Anchorage got its name. However, the Historic Landmarks Preservation Commission of the Municipality of Anchorage published accounts in its book on Anchorage, "Patterns of the Past" that put the credit to the U.S. Post Office. Evidently, shortly after President Wilson selected the western route for the railroad, the U.S. Post Office Department declared that mail delivered to the Ship Creek settlement should be addressed to Anchorage, Alaska. The area had been primarily referred to as Ship Creek, but was also known as Ship Creek Landing and Knik Anchorage. Although citizens voted on a name in August, 1915 and chose the uninspiring name of

"Alaska City", the Post Office and federal agencies refused to change and the name remained Anchorage.

The AEC managed Anchorage through a townsite manager for its first 5 years. Water lines for fire protection and sanitary reasons were extended to the town and power was provided by the AEC power plant. Anchorage townspeople began a Chamber of Commerce and a Commercial Club which unofficially advised the AEC manager but there were no elected officials. The AEC turned over management to the civilians upon the city's incorporation in 1920 and citizens elected an 8 member council with the highest vote-getter as Mayor.

World War I slowed work on the railroad and once over, many workers who had left to fight returned to the U.S instead of Alaska. The economy did not boom quite as expected. The railroad was completed in 1923 and Anchorage turned out to welcome President Harding on July 12, 1923, who was on his way to Nenana to drive the golden spike marking the opening of the great Alaska Railroad.

The Anchorage economy of the 20's and 30's remained dependent on the railroad. If the Alaska Railroad did well, so did Anchorage. The town was linked to Seward and Fairbanks and the interior by the railroad. Otherwise, one took a boat out, or, after 1922, possibly an airplane. Vehicles other than dog sleds in winter did not get very far from Anchorage. Anchorage began its aviation career in 1922 with the arrival of its first airplane and in a few short years boasted 40 more. Aviation became the 2nd biggest payroll behind the railroad. The airplane continues as an important link to the vast areas of Alaska.

Alaska did not suffer as much as the U.S under the great depression. In 1935, the U.S. government under the New Deal administration selected 204 families from hardy midwestern stock to come north and start an agricultural colony in the Mat-Su Valley, some 45 miles north of Anchorage. Citizens turned out to welcome the first group of 68 families and wish them well in their venture.

The economy experienced a boom due to military spending in the 1940's, and the effect of military spending continues to be important to Alaska today. Over a 6 year period from 1934-1940, numerous congressional authorizations for military development in

Alaska, especially for the Army Air Corps, were made but funds were never appropriated. Finally, with world tensions increasing, Congress approved funds in May, 1940. Construction of Fort Richardson began in June of 1940 at the present site of Elmendorf AFB. The base was named for a former Alaska Road Commission President and its airfield named after Capt. Hugh M. Elmendorf, a noted Army Air Corp test pilot. In 1950, Fort Richardson was moved to its present site and the newly organized Air Force took over the original site calling it Elmendorf Air Base. Most of the land north of Ship Creek to the Eagle River, 13 miles north on the Glenn Highway, is military land.

In November of 1942, the U.S. Army Corps of Engineers opened the Alaska Highway. The construction, through agreements between the U.S. and Canada, was needed to relieve Alaska from total dependence on shipping in wartime and to supply a land route for wartime supplies and equipment. Approximately 1500 miles, involving some hurculean engineering feats, were completed in a record 8 months. The link from Dawson Creek, British Columbia, to Fairbanks finally gave Alaska an overland access to the rest of the country. The following year, 1943, the Glenn Highway from Anchorage to Tok, tied Anchorage into the Alaska Highway.

Anchorage entered the decade of the 50's with a population of 43,000. Homesteading was made easy for G.I.'s and growth followed the war years. Oil was discovered on the Kenai Peninsula in 1957, a road was completed between Anchorage and Seward, and an oil pipeline from Whittier, in Prince William Sound, was built.

Aviation was growing by leaps and bounds. After the war, flying in Alaska was upgraded with hundreds of new airfields and technology left over from the military buildup. Anchorage also needed a new airport to accomodate increased traffic and the new polar flight routes which, for example, cut 2,000 miles off the usual New York-San Francisco-Tokyo trip. The Anchorage International Airport was officially opened in December of 1951.

Alaska was becoming less remote and Anchorage was growing up. Voters had been asked the question of statehood in 1946 and the answer had been nearly 3-2 in favor. However, the battle for statehood was neither smooth nor short. Finally, on June 20, 1958, Congress voted to admit Alaska as the 49th State and President Eisenhower signed the proclamation for Statehood on

January 3, 1959. While Juneau was the state capitol, Anchorage was the largest city, becoming the business center of Alaska. The oil development on the Kenai Peninsula in 1957 represented the first seismic interest in Alaska by the major oil companies. By 1963 the oil business was firmly established with major companies and support firms headquartered in Anchorage. Off-shore drilling in Cook Inlet was also beginning.

The 60's were bringing prosperity and continued growth to Anchorage and Alaska. However, good times were disastrously interrupted by the Good Friday, March 27, 1964 Earthquake. Centered in Prince William Sound, the quake measured 8.6 on the Richter scale and was one of the strongest earthquakes to ever hit the North American Continent. The destruction in Anchorage was severe; many homes were destroyed, an entire block of downtown businesses dropped 10 or more feet, roads twisted and heaved, and communications were cut off. But Alaskans are resourceful people and they rebuilt their damaged city, beginning almost immediately after the ground settled down. Two years later Anchorage was given the All American City Award, largely based on the speedy recovery from the disastrous quake.

The oil boom from the Kenai peninsula was to take a second seat to a new discovery. Early in 1968, Atlantic Richfield and Humble Oil struck oil at Prudhoe Bay, on the North Slope of Alaska's arctic region. This promised to begin a boom of fantastic proportions. Plans were made by major oil companies to build a mind boggling 800 mile pipeline through some of the most rugged country in the world, from the North Slope to Valdez, where they would construct a terminal for oil tankers to carry off the black gold.

Plans for the pipeline were not to move quickly, however, as environmental and Native Land Claims settlements had to first be addressed and settled before construction could begin. The battles over these two issues were often bitter and dragged out over several years with construction finally authorized in January, 1975, and beginning that spring. The rush to the state by construction workers from all over the U.S. was not anticipated and taxed the facilities of the city. Prices escalated and laborers working the pipeline made astounding wages. The line was completed and the first oil gushed forth at Valdez on July 28, 1977. The crush of the boom was over, but the economics of such a construction project gave Anchorage a new

look and new sophistication as oil company corporate headquarters centered here.

The Anchorage of today faces a bright future. Tourism has been growing over the years into a big business. Oil money in the state coffers helps fund large construction projects throughout the state. There are definitely problems with this source of income, but Alaskans realize the importance of broadening the economic base and are continually searching out new ways to do so.

Anchorage continues to grow, with the municipality now including the fast growing suburban communities to the north of Eagle River and Chugiak. Population figures from the 1980 census exceed 200,000. The arts and educational institutions rival those of many cities much greater in size. Alaska offers great potential in many areas, with the natural resources of oil, timber, fish, and minerals heading the list. Development continues and Anchorage seems to dominate as the hub of Alaskan activity.

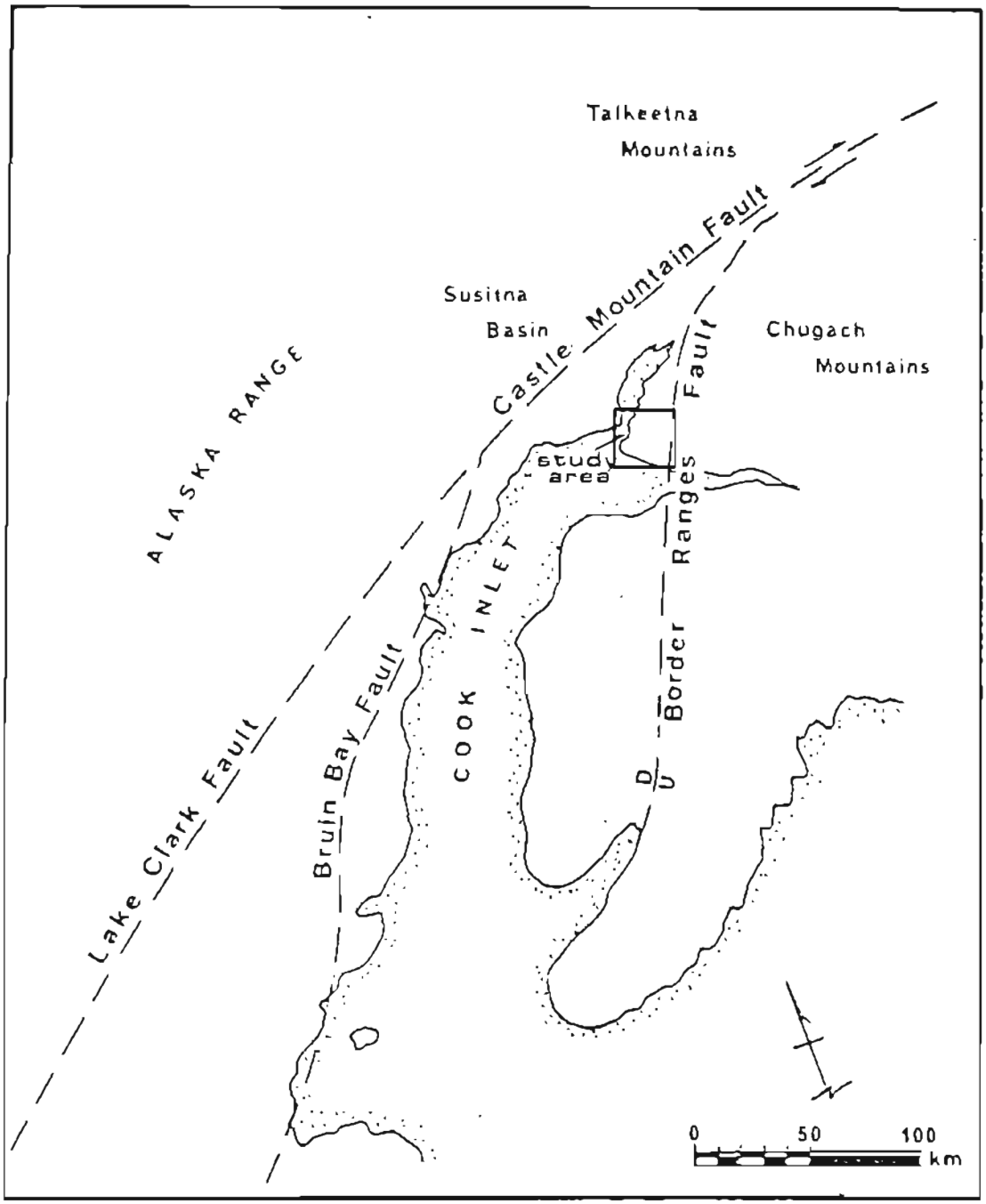


Figure 1. A generalized map showing the regional tectonic setting of the Cook Inlet region.

General Geologic Setting

The active tectonic framework within which the Anchorage area is located is evidenced both by the rather frequent occurrence of felt earthquakes and by the surface expression of active fault traces in nearby Quaternary deposits.

The Greater Anchorage area lies on the eastern margin of the Cook Inlet Forearc Basin. This tectonic basin is bounded to the west by the Bruin Bay-Castle Mountain Fault System and to the east by the Border Ranges Fault System along the front of the Chugach Mountains (fig. 1). This is an active tectonic setting with resultant seismicity along both fault systems as well as the underlying Benioff Zone (Aleutian Megathrust) resultant from the subduction of the Pacific Tectonic Plate beneath the North American Tectonic Plate. Although, during a given time period, most seismic events occur along this zone of underthrusting, the likelihood of near-surface events on, or adjacent to the surface fault systems is significant. Seismic activity related to the Castle Mountain system is generally comprised of low magnitude shallow events (M = 3.0 to 4.5) even though the potential for larger magnitudes (greater than M = 6.0) is quite real. The Border Range System is more of an enigma regarding Holocene seismicity. The unpublished work of investigators has shown evidence for low magnitude seismicity along the front but skepticism seems to prevail regarding the source of this seismicity.

The bedrock beneath Anchorage consists of Tertiary clastic sedimentary rocks which to the east form a wedge lapping up against the Mesozoic metamorphic rocks of the Chugach Mountains. No bedrock is exposed in the Anchorage area, as it is generally 330 to 650 feet (100 to 200 m) below the surface, and has been only rarely encountered in deep boreholes.

The Alaska Earthquake

The Good Friday Earthquake of March 27, 1964 was of magnitude 8.4-8.6 on the Richter scale and perhaps the strongest known to have occurred in North America. It was felt over most of Alaska and its destruction was evident within a radius of 400 miles of the epicenter, located 80 miles east-southeast of Anchorage in Prince William Sound. Seismically, it has been tied to the Benioff Zone associated with the Aleutian Megathrust. Anchorage, because of its population, sustained greater losses caused by landslides than all the rest of Alaska combined. Nine lives were

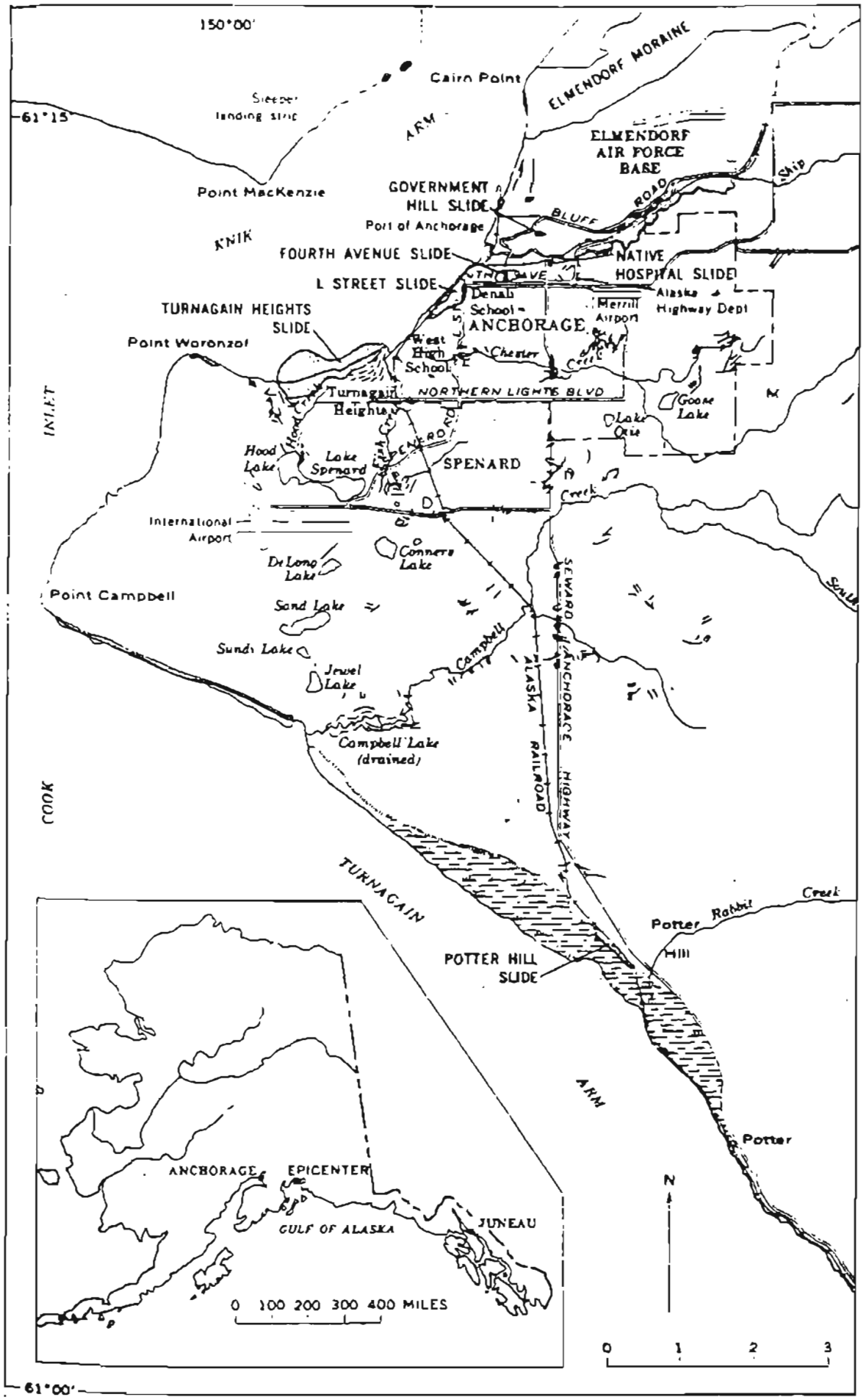


Figure 2. Generalized map of Anchorage, Alaska and vicinity showing locations of major landslides and ground cracks. (From Hansen, 1965.)

lost in Anchorage, and many more were lost in coastal areas due to sea waves (tsunamis).

The triggering of landslides was related to the physical and engineering properties of the finer facies of the Bootlegger Cove Formation, underlying much of the Anchorage area. The Formation contains zones of sensitive clayey silt of low shear strength with high water content and liquefiable sand layers that failed due to increased pore pressure caused by the vibratory stress of the earthquake.

Most of the destructive landslides caused by this event were moved by a combination of translation and rotation, at times sliding on nearly horizontal slip surfaces. Most of the structural damage to buildings occurred in the graben areas at the head of the landslide and within pressure ridges at the toe. Many buildings located within the large slide blocks, away from the head or toe were only slightly damaged despite horizontal translations of several feet. All translatory slides occurred where flat topped bluffs were bounded marginally by steep slopes. These destructive slides occurred in the downtown area at Fourth Avenue and "L" Street, and at Government Hill, and Turnagain Heights. Other less destructive landslides occurred in undeveloped areas. A generalized map of major landslides and ground cracking is shown in Figure 2.

Increased stream discharge, seiche action on lakes and marshes, and fluctuations in ground water levels all occurred temporarily following the earthquake. Residents near Campbell Lake reportedly saw water and sediment ejected over 200 feet from the lake.

Glacial History

The best summaries of the Quaternary history remain those of Karlstrom (1964), Miller and Dobrovolny (1959), Schmoll and Dobrovolny (1972), and Reger and Updike (1983, in press). Karlstrom proposed that at least five major glaciations occurred in Upper Cook Inlet: Mount Susitna, Caribou Hills, Eklutna, Knik, and Naptowne. The dates he established for some of these advances are now seriously questioned, but the geomorphic and stratigraphic evidence still supports their use in the literature. The three earlier glaciations were presumably far more extensive than the later ones. Based upon stratigraphic and geomorphic criteria, Karlstrom (1964) envisions that the ice of

these earlier glaciations coalesced to fill the Cook Inlet trough.

Reconstruction of the earliest glacial and interglacial events has been made cumbersome due to post-glacial topographic modifications, leaving only fragmentary evidence behind. The very earliest of the glaciations recognized is named after Mt. Susitna, whose ice scoured slopes contain erratics to within 100 ft (30 m) of the 4396 ft (1332 m) summit. This altitude cannot be used to calculate the true thickness of the ice at that point in time. Upper Cook Inlet is a seismically active area where post-glacial elevation changes could likely be the result of tectonism. Sea level fluctuations and isostatic changes due to crustal loading and rebound associated with the advance and retreat of glacier ice can also be held responsible for changes in elevation. The Caribou Hills and Eklutna glaciations named elsewhere are also observed at successively lower elevations of Mt. Susitna. The two latest glaciations, Knik and Naptowne, were not extensive enough to fill the entire basin but may have restricted or totally dammed off upper Cook Inlet. This lake could well have contained brackish and/or fresh water during the many years of its existence (or multiple existences). Controversy continues regarding the time frame and extent of glaciers and lakes during middle and late Pleistocene times, and more specifically, the correlations that can be made between glaciofluvial, glaciodeltaic, glaciomarine, and true glacial tills within the basin.

The Bootlegger Cove Formation

A general harmony is believed to exist among researchers that the Bootlegger Cove Formation, as recently defined by Updike (1983, in press) was deposited during late Naptowne times. The Formation was formerly defined primarily on the basis of the cohesive units (Miller and Dobrovolny, 1959) as a light gray, silty clay, locally called the "blue clay". The type locality for the clay is located at Bootlegger Cove, east of the Turnagain Heights Landslide within Westchester Lagoon. The unit has been renamed because clay commonly is a secondary contituent, and the unit has been expanded to encompass a wider variety of sediment textures ranging in size from clay to boulders. The informal nomenclature "Bootlegger Cove Formation" has been chosen (Updike, 1983, in press), which is more consistent with the stratigraphic code where two or more rock types are common. The diversity of textures is resultant from a variety of depositional regimes tied

to a single glaciomarine-glaciodeltaic system. Eight geologic facies are defined based upon their respective engineering and textural parameters, each reflecting the subtle variations characteristic of a late Pleistocene glaciomarine environment.

Facies F.I	CLAY, with very minor silt and sand
Facies F.II	SILTY CLAY AND/OR CLAYEY SILT
Facies F.III	SILTY CLAY AND/OR CLAYEY SILT, SENSITIVE
Facies F.IV	SILTY CLAY AND/OR CLAYEY SILT, with thin silt and sand lenses
Facies F.V	SILTY CLAY AND/OR CLAYEY SILT, with random pebbles, cobbles, and boulders
Facies F.VI	SILTY FINE SAND, with silt and clay layers
Facies F.VII	FINE TO MEDIUM SAND, with traces of silt and gravel
Facies F.VIII	SANDY GRAVEL AND GRAVELLY SAND, with discontinuous layers of silt and fine sand

At the maximum extent of the late Naptowne glacial advance, approximately 14,000-18,000 yrs B.P., an ice front entered the Anchorage basin from the northwest, terminating west of Point Woronzof and Point Campbell. The basin is believed to have also been glacially bounded to the northeast by the Knik-Matanuska lobe and to the south by the Turnagain lobe creating an environment that at times limited the influx of marine water from lower Cook Inlet, resulting in fluctuations between marine and brackish-fresh waters. Ablation of the northwest terminus produced a glacial fan delta prograding eastward into the deeper waters of the basin. The non-cohesive facies F.VI, F.VII, and F.VIII extend eastward from the Point Woronzof and Point Campbell highlands. They are found to grade into and be interbedded with the cohesive facies F.I through F.V representing transitions from the coarser deltaic regime into the finer deep water regime. This gradational transition occurs at the topographic rise to the Point Woronzoff and Point Campbell highlands. Although fluctuations of the fan delta are evident from observed interbedding of the facies, generally this textural sequence reflects the gradational nature of the system from ice contact to glaciodeltaic, to a deep water regime.

During the early phase of the fan building period, F.VIII was beginning to accumulate off of the active ice front, resulting in ice-contact sedimentary features being typically found within this facies. Ice calving from the glacier front caused ice-rafted stones and coarse sand to be randomly interspersed with the silts and clays accumulating in the basin to the east, resulting in F.V being typical of the lower part of the section. The glaciodeltaic deposition continued to be active throughout the period in which an ice front existed to the west. Generally, the non-cohesive facies (F.VI, F.VII, and F.VIII) are the dominant units in the west, and the cohesive facies of the traditional "Bootlegger Cove Clay" (F.I through F.V) are most abundant in the east. The 1964 landslides in the Turnagain Heights area have been attributed to liquefaction and/or sensitive clayey silt failures respectively associated with facies F.VI and F.III.

The formation was deposited on a sequence of indurated till and glaciofluvial deposits which Reger and Updike (1983) believed to be of Knik age. A pronounced unconformity exists between these glacial deposits and the overlying Bootlegger Cove Formation.

Most of the Bootlegger Cove sequence is capped by very fine to coarse, well-sorted sand beds (facies F.VI and F.VII) which represent the waning phase of the Bootlegger Basin when the source-area ice had stagnated, glacial dams had been breached, and the basin was essentially drained. In the Turnagain Heights area and northeastward to Downtown Anchorage, these sands are overlain by glacial outwash sand and gravel representative of a very late Naptowne (Elmendorf) glaciation occurring in the Eagle River area. This outwash plain continuously thins to the southwest, eventually wedging out just east and south of Earthquake Park. The facies F.VI/F.VII sands at the top of the Bootlegger Cove Formation are typical throughout Anchorage, regardless of the overlying stratigraphy, implying that little erosion of the upper surface has occurred since deposition in Late Pleistocene time (ca. 12,5000 y.b.p.).

During the progradation of the delta, grounding and burial of stagnant ice blocks occurred in the delta. The ice could thus persist for a considerable time span after glacial stagnation began. As the ice retreated, an outwash terrace was eroded between the glacier and the abandoned fan delta deposits and can

be observed from Point Woronzof south to Point Campbell. A similar terrace occurs on Fire Island and together they may represent the remnants of an outwash floodplain draining southward from the retreating glacial terminus. At the same time that the ice margin was receding from the vicinity of the map area, the stranded and buried ice blocks in the fan delta deposits were melting, producing kettles and poorly integrated drainage ways and is portrayed as the hummocky terrain of the Point Woronzof and Point Campbell highlands. The surface of the then-recently deposited Bootlegger Cove Formation was exposed as the embayment drained. Materials derived from the irregular dissection of the fan delta were transported eastward onto the cohesive facies surface as channels and fans of medium to coarse sand and gravel. Similarly, retransported delta sediments were deposited on the outwash terrace to the west and to now non-existent areas to the south. Much of the aggregate quarry operation in the Sand Lake area has been carried out in the coarse facies of the fan delta and in the retransported channel deposits in the marginal segments of the delta. Associated with the draining of the embayment and the melting of stranded ice in the fan delta facies was the development of drainage channels which are now abandoned or contain underfit streams, e.g., channels entering the Campbell Creek drainage from the north.

During Holocene (recent) time, the area has been subjected to isostatic rebound coupled with periodic tectonic uplift (Brown and others, 1977) which, combined with fluctuations in sea level, has caused bluff topography to form along Knik Arm, and along streams draining into Cook Inlet. The effect of tidal inlet erosion and slope instability of the Bootlegger Cove Formation has resulted in gradual bluff retreat. Periodic seismic events in the region have further enhanced this retreat by causing massive landslides. The 1964 earthquake landslides are an example.

Soil Testing Procedures

Following the 1964 earthquake, soil studies were performed by private engineering consultant firms and agencies of the federal government (e.g., Shannon and Wilson, 1964; Seed and Wilson, 1967; Hansen, 1965). Little additional research on the engineering geology of Anchorage had been conducted until recently. In 1978, the Alaska Division of Geological and Geophysical Surveys in cooperation with the Office of Earthquake Studies, USGS began new studies.

Because of limited natural exposures within the area, much of the geologic mapping and geotechnical analyses have depended on subsurface exploration. The evaluation of subsurface soil conditions can be accomplished by (1) drilling, sampling, and laboratory testing, and (2) in situ testing. Regardless of the care exercised, the first method has inherent problems with sample disturbance and testing in other than actual conditions. In situ testing is limited both in the variety of techniques available and the interpretation of the derived data based on existing soil behavior theory. Penetration testing is the approach most universally employed and is based on the concept that the force or energy required to push or drive a standardized probe into the soil can be translated into a measure of soil strength or bearing capacity. Two principal penetration test methods are in use, Standard Penetration test (SPT) and Cone Penetration Test (CPT). The SPT method has been in use in Anchorage for many years and remains a standard for local foundation design. Although the CPT method has been in use in Europe for several years, only recently has the method attained the sophistication necessary for acceptance in the United States geotechnical industry and only in the past year has it been used in Alaska. Several CPT sites were selected within the Anchorage area to further enhance our understanding of the Engineering parameters of the Bootlegger Cove Formation. The test consists of pushing an instrumented cone-tipped probe into the soil and continuously recording the resistance of the soil to that penetration. The logs recorded by the CPT system correlate very well with previously obtained geotechnical borehole logs (fig. 3). For example, the stratigraphic cross-section of Figure 13 was derived from both CPT logs and borehole data. Such correlations provide a three dimensional stratigraphic picture consistent with the previously proposed model of an ice-marginal deltaic regime extending into a quiet water depositional basin to the east.

Cores acquired from drilling porgrams have been subjected to a number of static and dynamic testing programs. State-of-the-art testing of the various facies of the Bootlegger Cove Formation using cyclic simple shear, resonant column, and cylcic triaxial techniques have yielded a suite of index properties for the formation which are presently being used in the design of large structures (e.g., Updike and others, 1981).

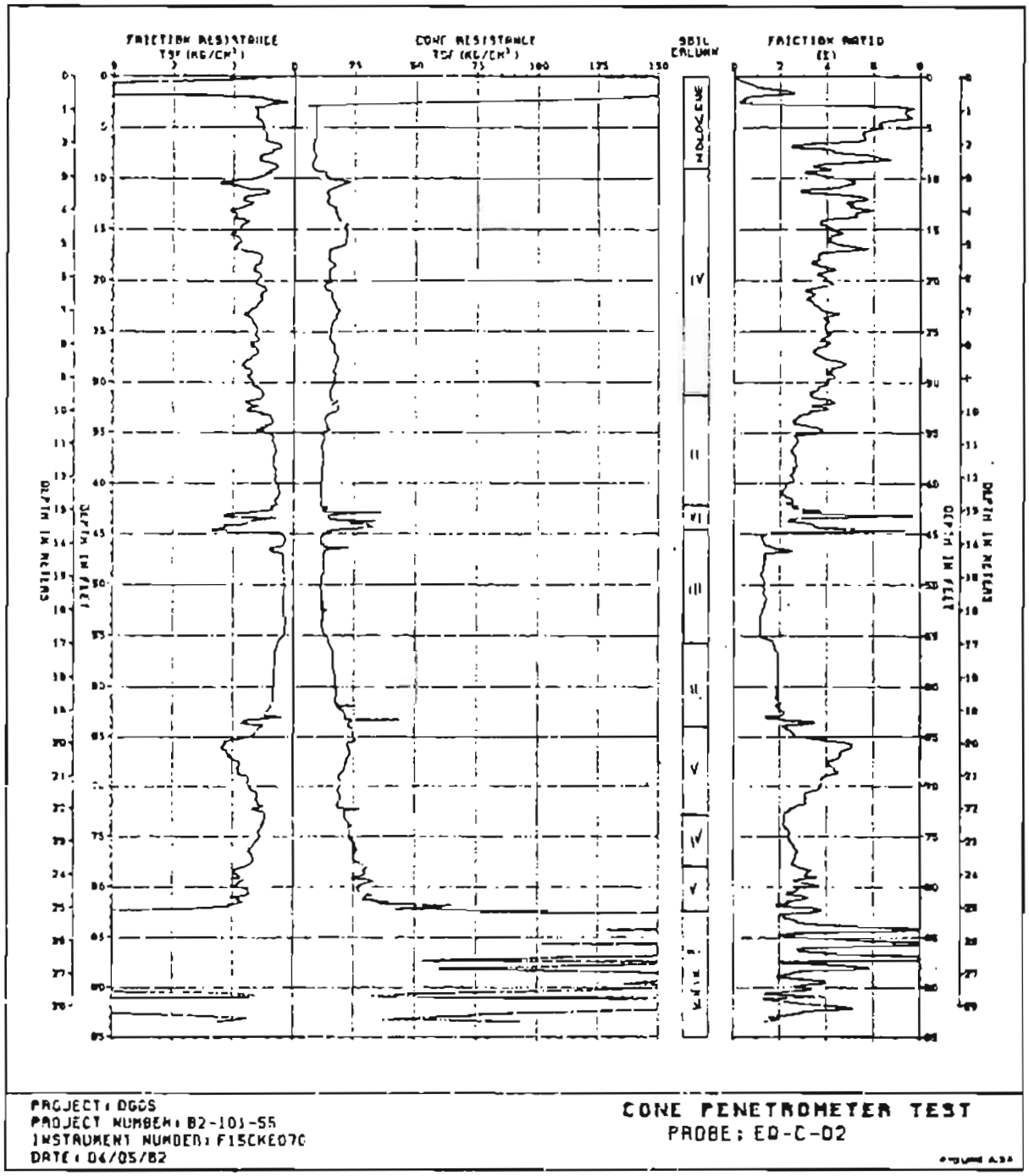


Figure 3. Example of a Cone Penetrometer log with Bootlegger Cove Formation correlations. (From Ertec, 1982.)

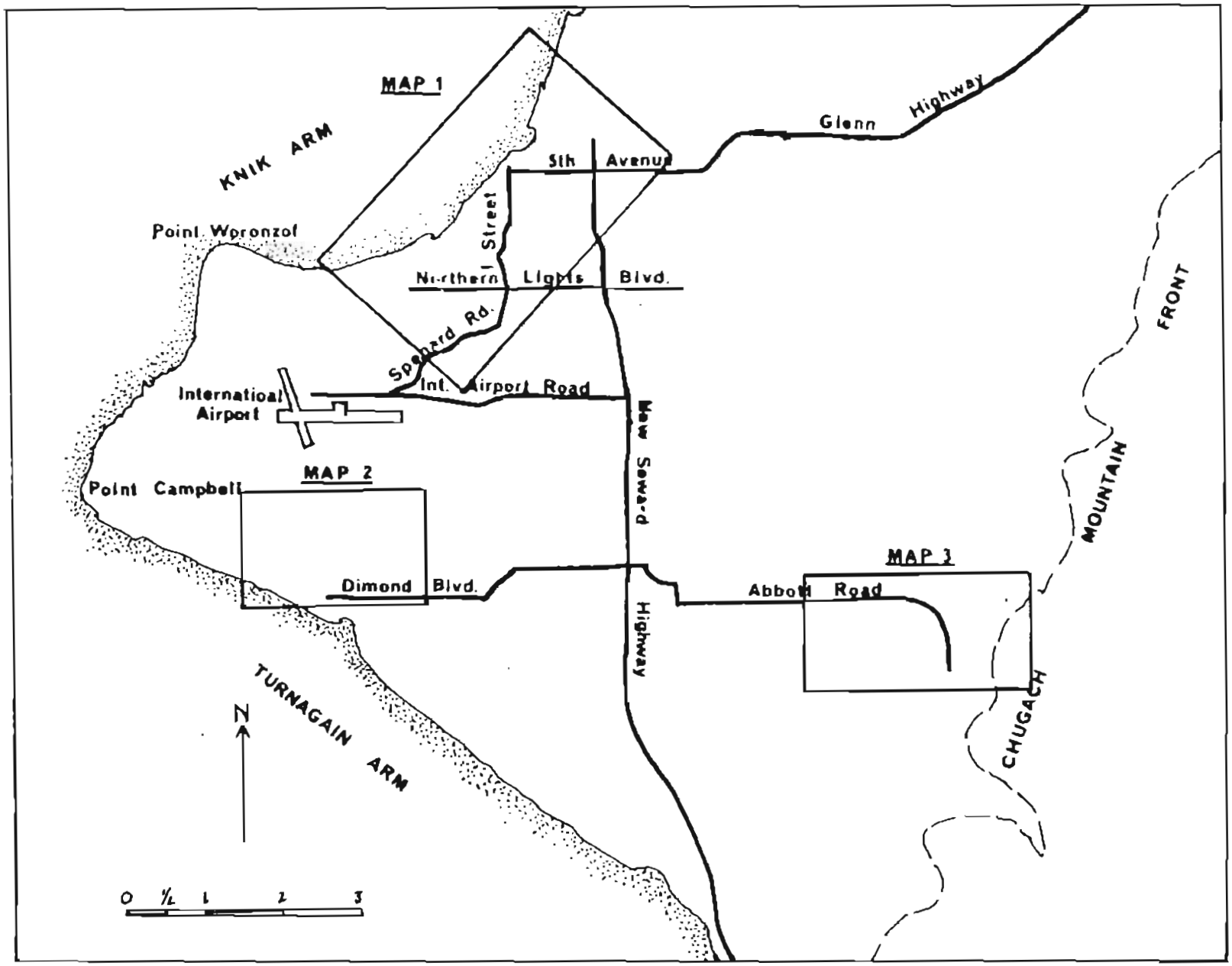


Figure 4. Generalized map of Anchorage, Alaska showing road log map locations.

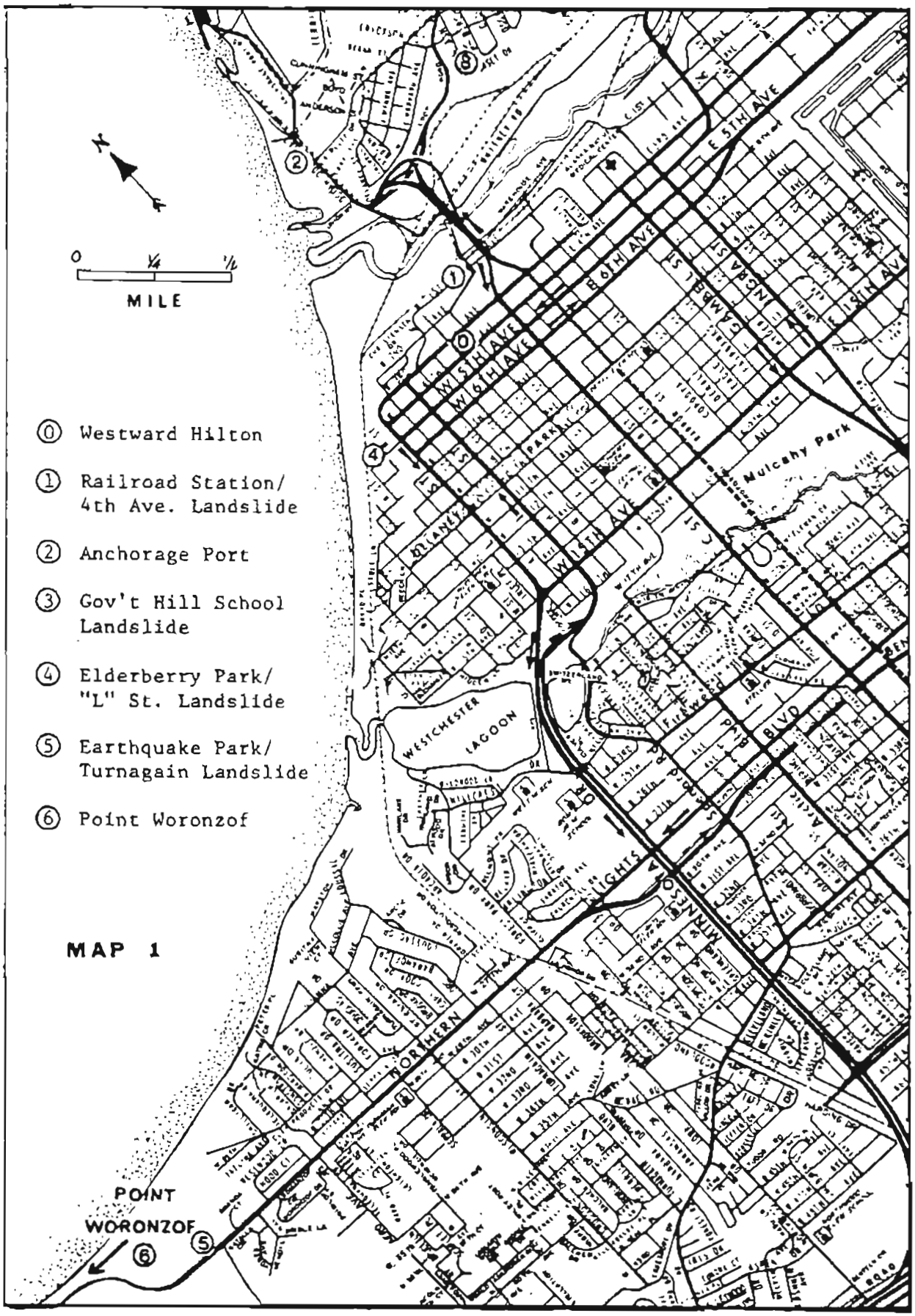
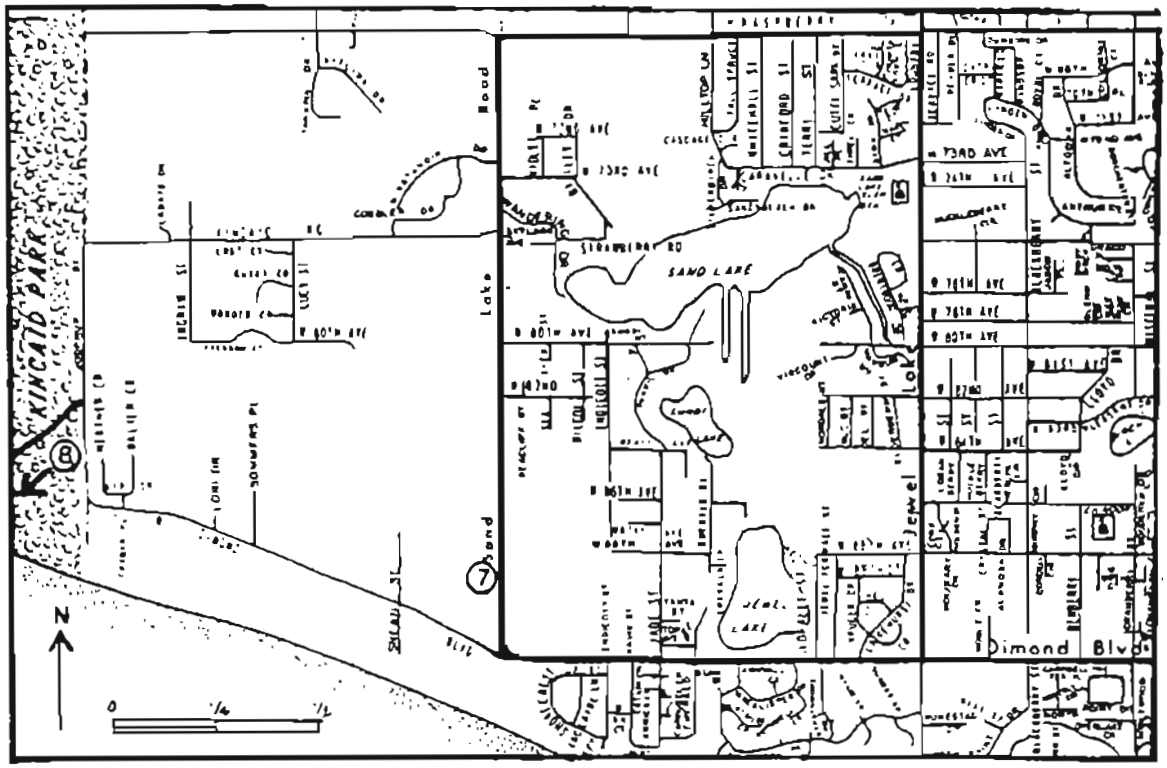
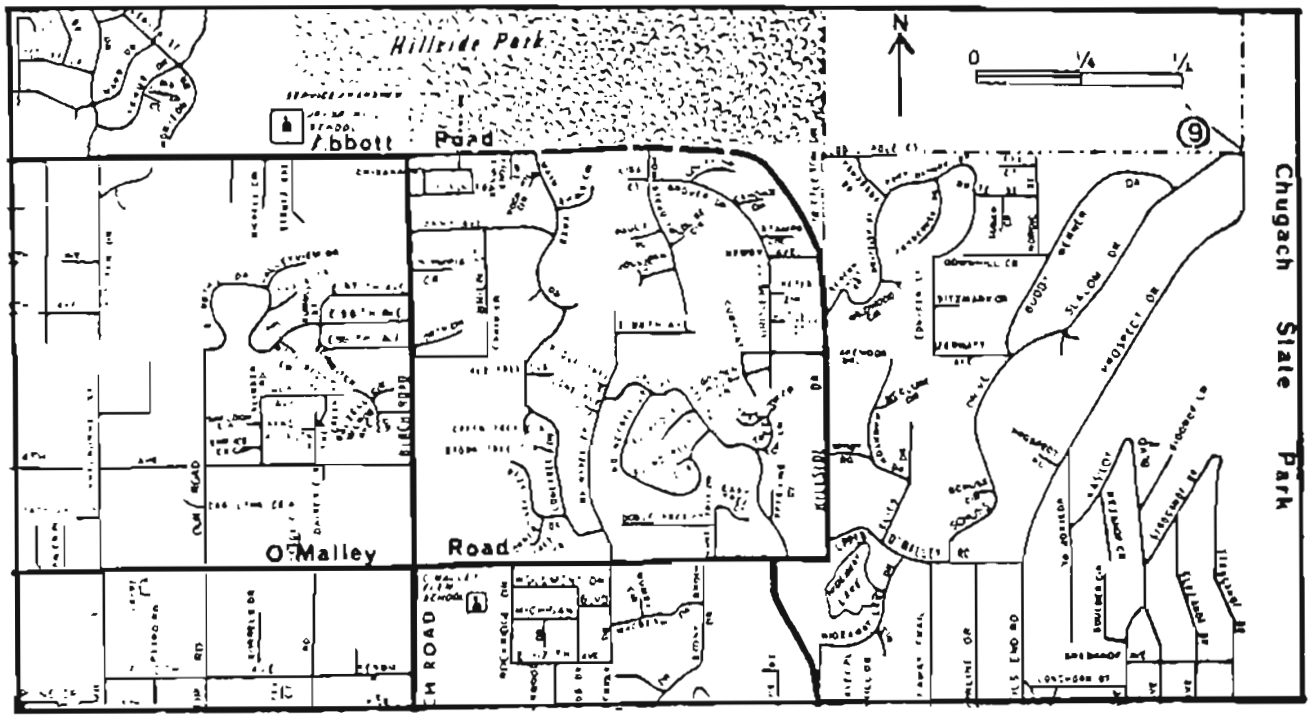


Figure 5. Location map 1.



MAP 2

7. Sand Lake Pits
8. Kincaid Park



MAP 3

9. Hillside

Figure 6. Lacation maps 2 and 3.

ROAD LOG

Assemble at the Westward Hilton Hotel, 3rd and "E" Streets. Drive north on "E" Street to T-intersection and turn right, continue downhill turning left to the front entrance of the Alaska Railroad, Anchorage Passenger Terminal Building. Meet near the steam engine across the street from the terminal.

STOP 1. Base of the 4th Avenue Buttress.

You are now standing at what was the toe of the 4th Avenue Landslide that resulted from soil failure during the 1964 Prince William Sound Earthquake. Several landslides occurred in the Anchorage Area as a result of the earthquake and the 4th Avenue slide was one of the largest of these (fig.2). All of these landslides had in common a loss of integrity within the Bootlegger Cove Formation (see Introduction). Vertical displacement along the 4th Avenue Landslide was as much as 10 ft and horizontal movement was up to 20 ft (fig.7). The result of such displacement was total destruction of buildings between where you are standing and the far side of 4th Avenue which is four blocks south of this stop.

In an effort to lend future stability to the slide area, the U.S. Army Corps of Engineers constructed an earthen buttress between 4th Avenue and where you are standing. This buttress is visible as the uniform grassy slope that extends up (south) to the Post Office Mall, east (to your left) to the Travelodge Motel, and west (right) to the Westward Hilton. Details of the principles and design of the buttress are discussed in Attachment 1. Immediately after the landslides, a number of specifically designed metal slope indicator casings were installed in and around the 4th Avenue Landslide. These casings can be surveyed by means of an electronic probe inserted down the casing which will indicate any gradual or abrubt displacements within the landslide or surrounding area since installation of the casings. Updike (1983) has completed an examination of these casings and has concluded that only minor (a few inches) movement has apparently occurred in this area since 1964.

The Alaska Railroad terminal building was built in 1942 and includes administrative offices. Anchorage owes its very existence to a the campsite selected by a survey crew for the Alaska Railroad dating to 1914. The original site for the railroad camp was north of Ship Creek, however, the creek basin

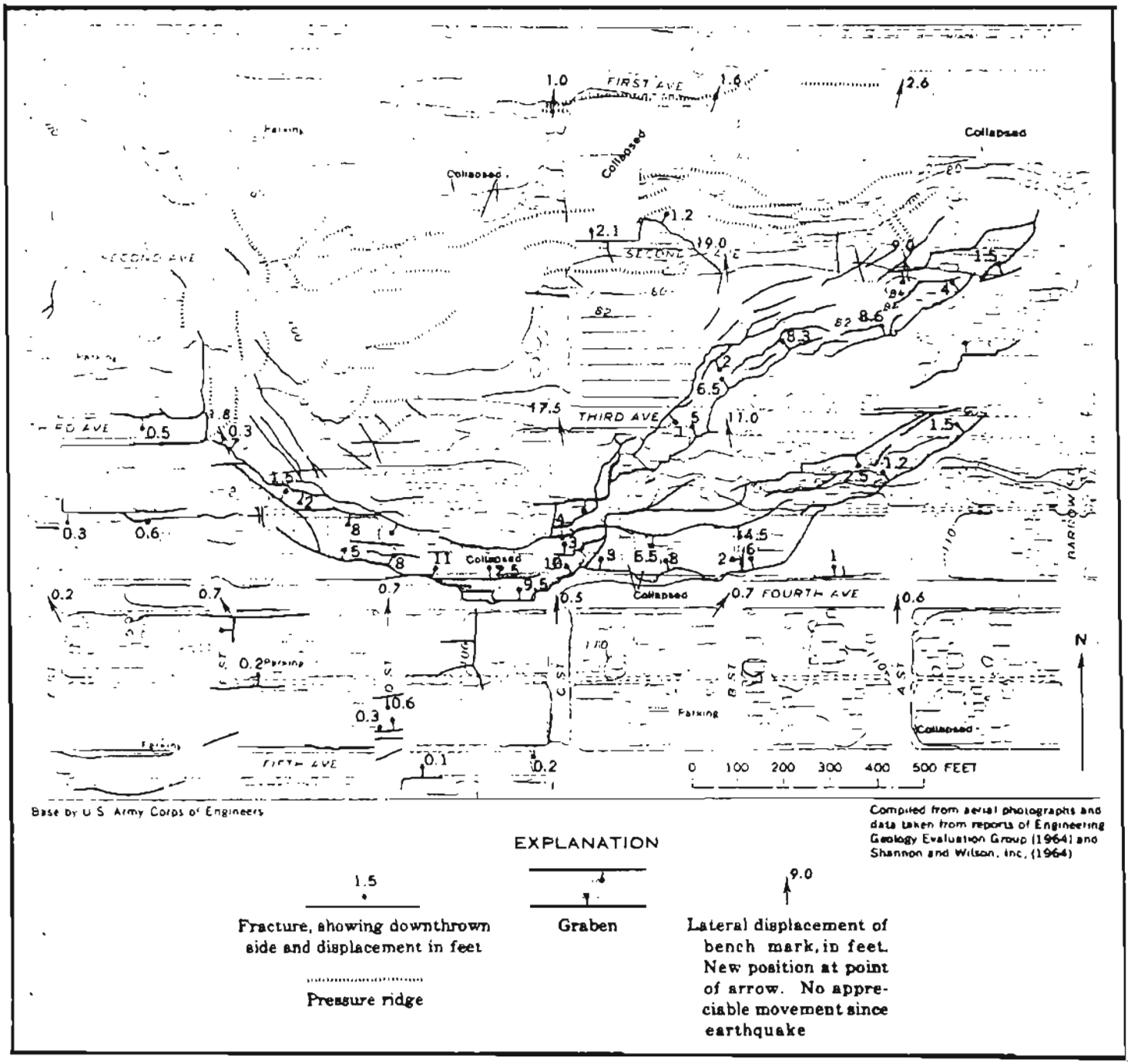


Figure 7. Fourth Ave. landslide area, Anchorage, Alaska. (From Hansen, 1965.)

has been altered over the years beginning with a rechanneling of the creek in the 1920's. Very few of the original buildings remain today; a few have been moved others were lost to fire. Most present-day structures date to a building program in the 1930's and 1940's.

END Stop 1. Depart east from the Railroad Terminal to "C" Street, turn left onto "C" Street and cross the bridge over Ship Creek. This creek drains into Knik Arm from the Chugach Mountains and in mid to late summer supports a substantial run of salmon. In the 1920's the channel was altered here from a meandering to straight configuration for land utilization. At the stop sign after the bridge turn left, drive across the railroad tracks that go to the repair yards on your right, and stop at Ocean Dock Road intersection. Turn left onto Ocean Dock Road, in about 4 blocks the road makes a right curve and becomes Port Road; continue north. You will note petroleum tank farms on both sides of the road as we drive north. This is a major storage and distribution point for petroleum products for Alaska. Extensive ground fissuring and damage to tanks occurred throughout this area in 1964 (fig. 8). Just before reaching the Anchorage Port you will note a municipally-owned section of land to left now being used for temporary parking and storage. Enter this area and stop.

STOP 2. Anchorage Port Area

The area near the mouth of Ship Creek has long been used as a point of disembarcation for Upper Cook Inlet. The original Anchorage townsite, however, was not picked because of its ideal port advantages. The problems of extreme tidal fluctuations, heavy silt load in the waters, and broad mudflats of Knik Arm affect shipping, but because of the access to the railroad the port location was developed. Delivering goods for the nearby military bases gave impetus to the port's early growth. Prior to the 1964 Earthquake, the ports of Whittier and Seward were more preferred for large ships, but the destruction from the earthquake was nearly total for both these ports. In contrast, the Anchorage port suffered only minor damage and shipping activities resumed in a matter of days. Some major companies that sustained serious damage at these ports relocated their centers of operation to Anchorage. As a result, since the earthquake, the Anchorage port has grown very rapidly to assume its present-day dominance. Even so, periodically a channel must be dredged off-shore from where we are standing to accomodate

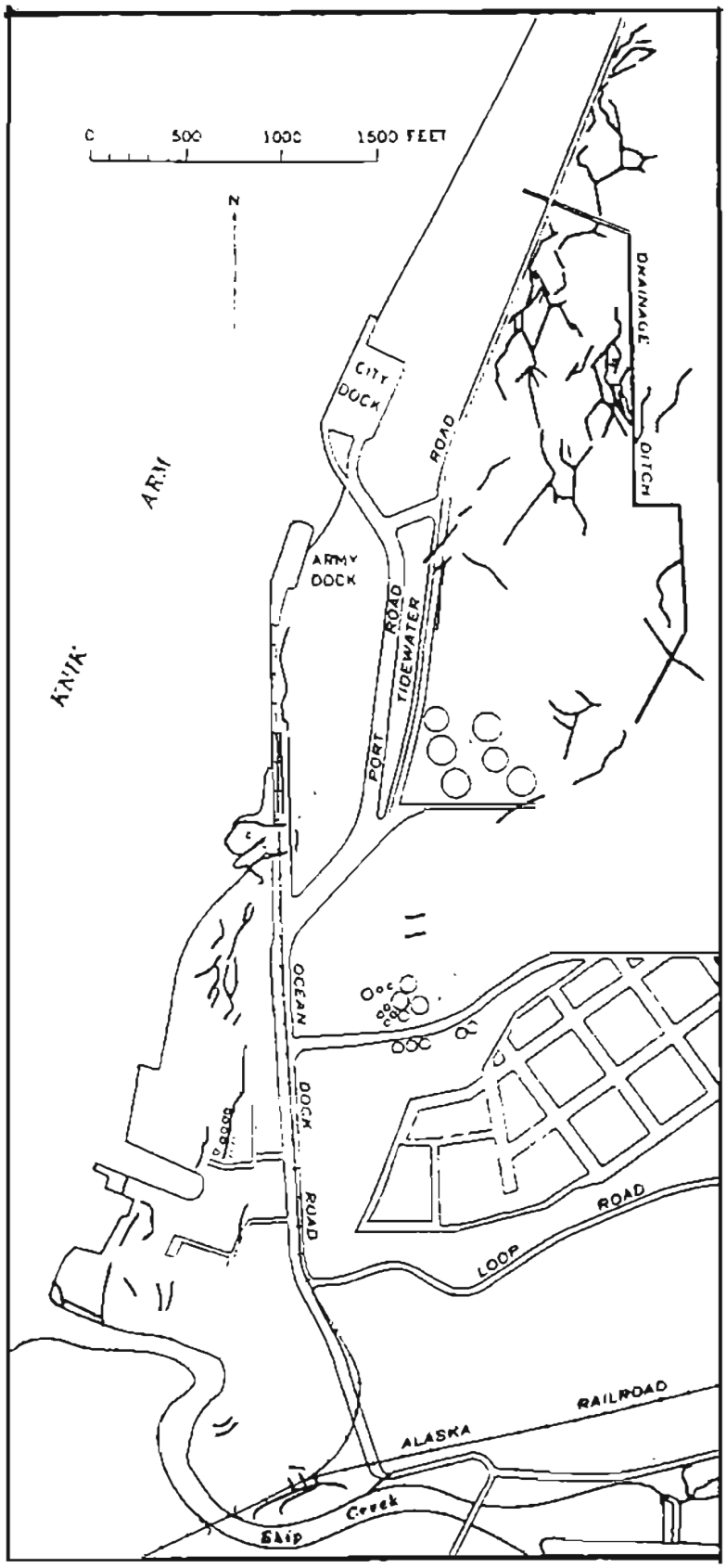


Figure 8. Map of ground cracks at the Port of Anchorage and vicinity, Anchorage, Alaska. (From Hansen, 1965.)

large ships approaching the port. The dredging is required to remove the very large volumes of glacially-derived silts that are transported on the tides. Some people have claimed that certain submarine channels off-shore are filled with gold-bearing sands and gravels and have filed mining claims to support their contentions. Thus far no placer gold claims have been proven.

The wood piles that you see off-shore are all that remains of Ocean Dock, completed in 1919. On the far side of Knik Arm, the point of land you see is Point MacKenzie, approximately 2.5 miles away. This area was occupied by only a few hearty homesteaders until recently. In the past few years several significant thrusts of activity promise to change the character of thousands of acres adjacent to Point MacKenzie. Large parcels of land are being cleared in preparation for use as agricultural land, possibly to include dairy farms, grain, and vegetable production. In support of the agricultural development a good quality, year-round road to the area near the point has been constructed providing access for other purposes (e.g. recreation, home sites, etc). The Matanuska-Susitna Borough, which has jurisdiction over the area, is attempting to attract light industry and even a port facility to be established at Point MacKenzie.

One of the most intriguing prospects for Point MacKenzie is the possible connection with Anchorage by means of a bridge, causeway, or tunnel. This would be a major engineering undertaking because of the problems of poor foundation soils, lack of bedrock to depths of several hundred feet, dramatic tides (30 foot normal range, 5 to 10 knot current velocity), winter ice, and heavy siltation problems. Nevertheless, feasibility and preliminary engineering design studies are underway with a causeway structure most commonly touted. At low tide, large stretches of tidal mud are exposed between here and the far shore. At the same time, however, some channels having depths of more than 180 ft exist directly adjacent to the mudflats. The system of channels and flats can shift with each tide. In winter the system becomes even more complex as sea pack ice mixes with the silty marine waters to form a constantly moving gray and white collage.

If the lighting is good, as you look along the bluff on the far side of the arm from the point you will note the horizontality of both the sediments comprising the bluff and the

upper surface of the bluff. This attests to the glaciomarine and glaciofluvial origin of both the sediments and the surface landform. This later continiuity is, however, punctuated by occasional major declivities which will be emphasized by vegetation descending to high tide line. These are landslides; the one directly across the arm from you is pre-modern times while the one upstream (to the right) occurred in 1964. The Bootlegger Cove Formation, which makes up the lower two-thirds of the bluffs, is again the cause for the ground failures.

END Stop 2. Return on Port Road and Ocean Dock Road about four blocks to West Bluff Road in the vicinity of the Chevron Tank Farm. Turn left onto West Bluff Road. Certain of these tanks have sustained foundation problems due to artificially-induced permafrost resulting in differential heaving and settling of the floors of the tanks. Apparently, the liquids stored in these tanks for long periods maintain temperatures near freezing, the underlying soil freezes in winter but cannot fully thaw during the summer due to the overlying cold liquid.

At the top of the hill turn right onto Cunningham Road, then right onto Cook Road. This road proceeds down through a low area which Updike and Carpenter (1983) believe resulted from a liquefaction-type ground failure in prehistoric time. Cook Road curves left (south) becoming Delaney Road. Continue on Delaney Road to Brown's Point Park. During winter months an excellent view across Cook Inlet is provided here. Note the early American style of architecture of the the two homes adjacent to the park. Turn left onto Harvard Avenue. You will note more homes of early 20th century design, some in various phases of remodeling or total reconstruction. This is one of the oldest neighborhoods in Anchorage. The railroad was authorized to construct housing for permanent employees in 1915. The area above the railroad camp was chosen and soon became known as "Government Hill". There are two stories as to the origin of the name. The first points to the obvious, that the houses were built for government (railroad) employees. The second story indicates that several railroad men were involved with the Panama Canal construction and the housing site reminded them of a similar "Government Hill" in Panama.

You will have a number of glimpses of downtown Anchorage across Ship Creek, to the south, as we continue driving along Harvard Avenue. Wedge-type landslides (pre-1964) occur continuously along (and below) the bluff to your right. Drive

through the traffic light (Loop Road) continuing straight on Hollywood Drive for one block, turn right on Ash Drive and stop at the park at the end of the road.

STOP 3. Government Hill School Landslide

You are now standing at the site of Government Hill School which was severely damaged by vertical displacement along a landslide caused by the 1964 Earthquake (fig. 9). The stepped terrain to the south (baseball field) is the regraded suface of some of the landslide blocks. The toe of this slide impinged on the Alaska Railroad facilities on the Ship Creek floodplain as a soil flow (fig. 10). Both sand facies (F.VI and F.VII) and sensitive silty clay facies (F.III) of the Bootlegger Cove Formation underlie the site. Geotechnical studies immediatley after the earthquake (Shannon and Wilson, 1964) suggested that sand liquefaction was responsible for the slide. However, recent studies by Updike and Carpenter (1983) conclude that liquifaction of F.VI and F.VII is unlikely, even during large magnitude earthquakes, and instead collapse of sensitve silty clay facies was primarily responsible for the landslide.

Fortunately, the earthquake occurred in the late afternoon of Good Friday and as a result the school was unoccupied at the time of the slide. For several years the intact part of the school remained unused, a new school having been constructed several blocks away from the bluff. Recently, the old school was removed and the park constructed, a good example of how potentially hazardous areas can be prudently put to good community use.

If one looks west (to your right as you face Ship Creek), another, much older, landslide of similar dimensions and displacement is now occupied by the Government Hill Curling Club. In fact, most of the bluff line surrounding Government Hill is bordered by old to relatively recent landslides (fig. 11), each being attributed to past failures within the Bootlegger Cove Formation. Some of these slides show evidence of not presently being entirely stabilized and downslope creep continues under aseismic conditions.

From this vantage point, downtown Anchorage and the 4th Avenue buttress are clearly visible. If the weather is good, the front of the Chugach Mountains can be seen to the east.

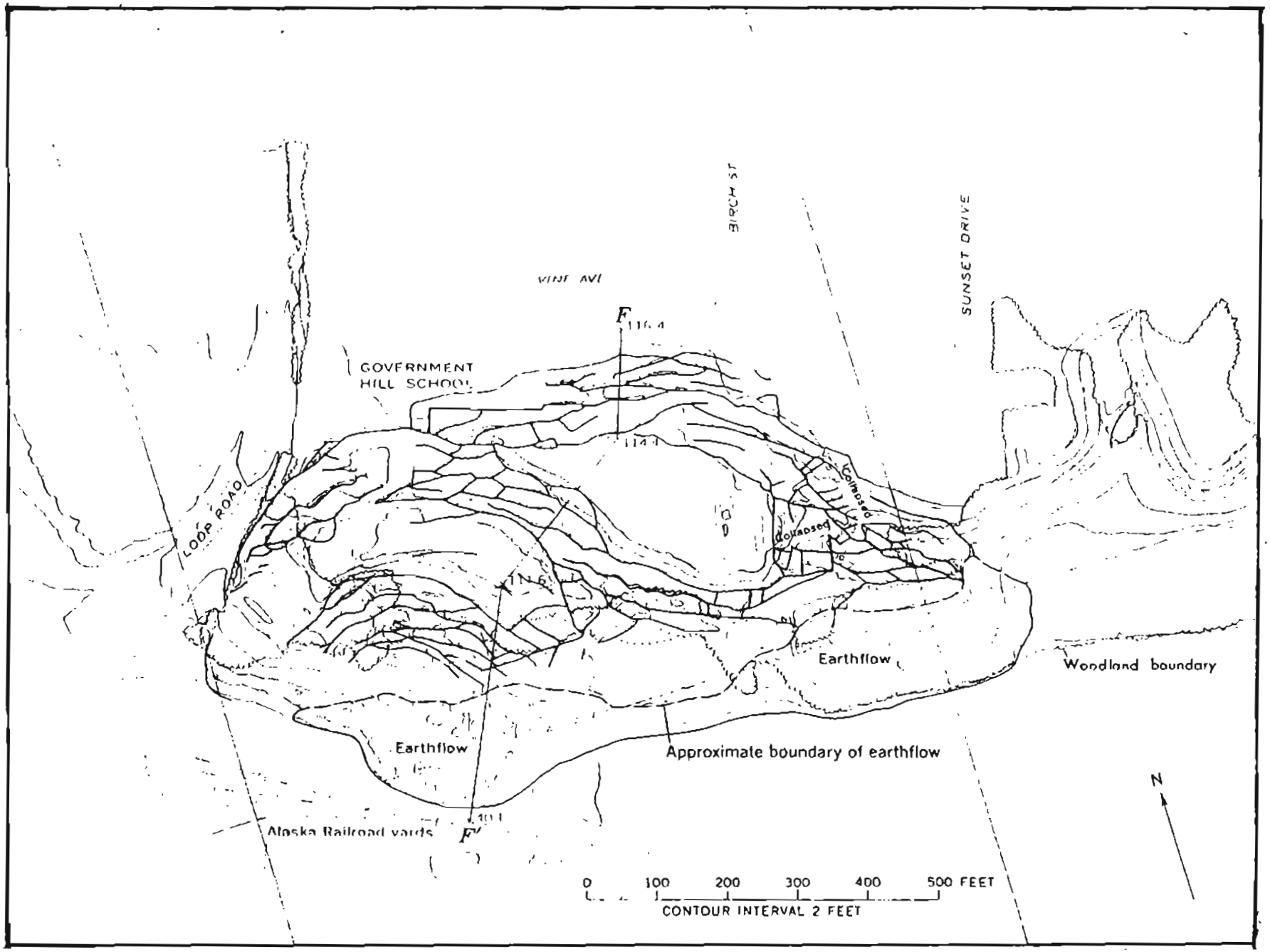


Figure 9. Map of the Government Hill School landslide, Anchorage, Alaska showing major ground cracking. (From Hansen, 1965.)

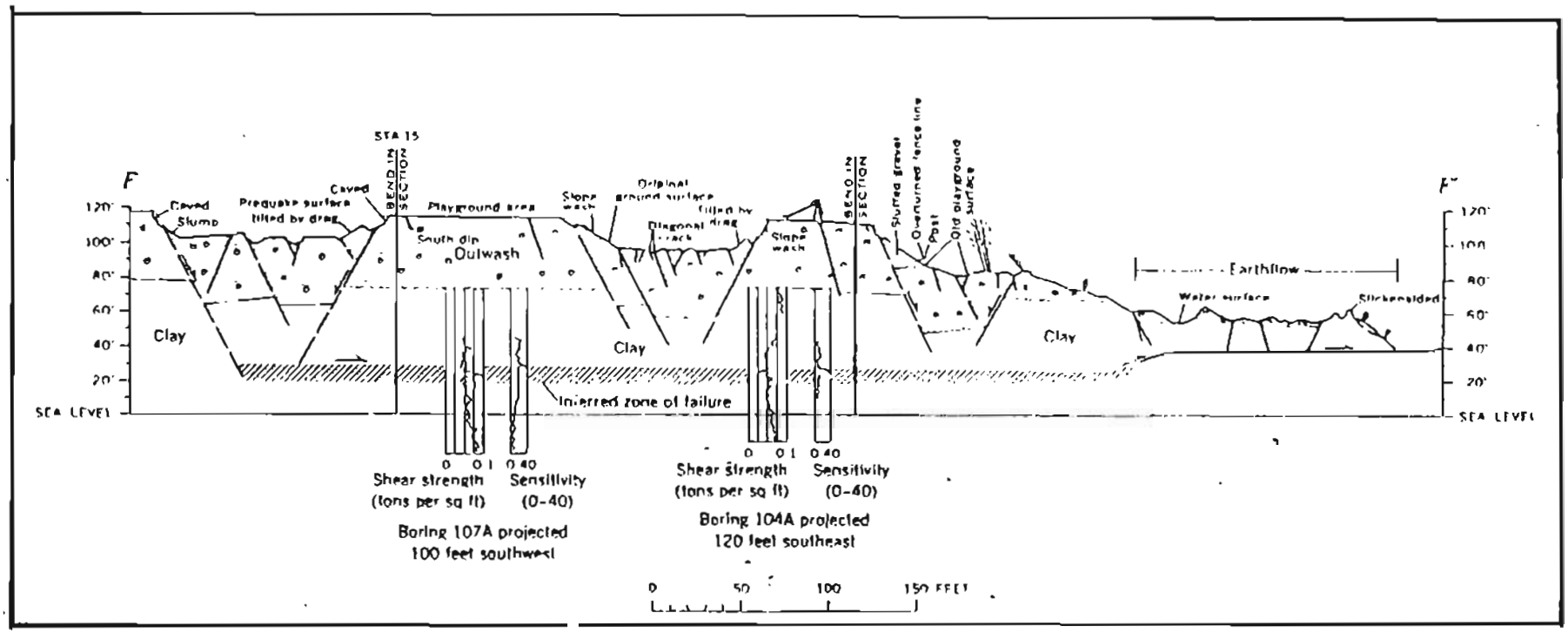


Figure 10. Geologic cross section through the Government Hill School landslide. (From Hansen, 1965.)

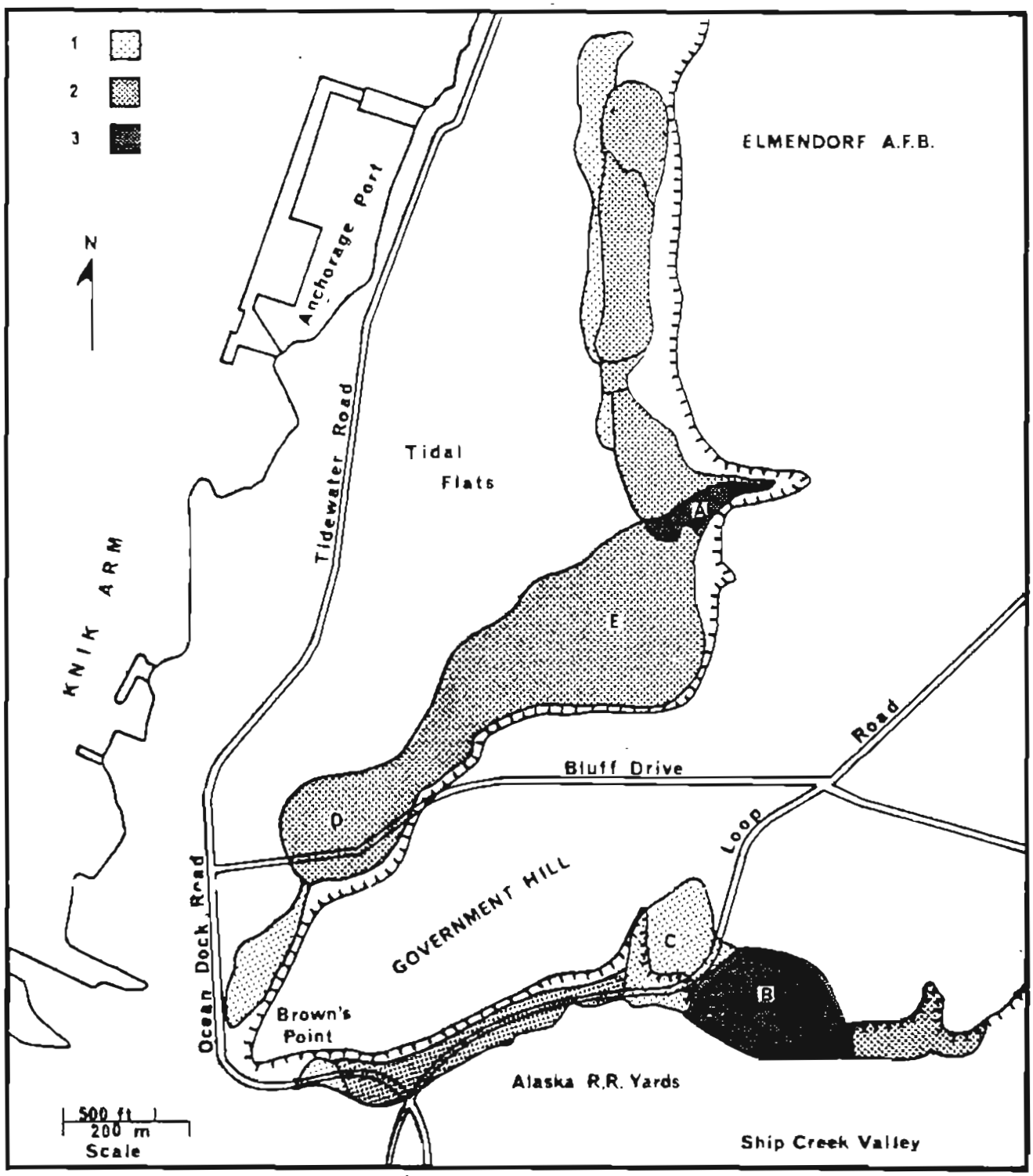


Figure 11. Distribution of landslides in the Government Hill area as discussed in the text. Patterns denote: (1) older landslides, (2) younger landslides, (3) 1964 landslides. Landslides discussed in text: (A) Gaylor's Gulch, (B) Government Hill School, (C) Curling Club, (D) Chevron Tank Farm, (E) P.O.L. Tank Farm.

END Stop 3. Return on Ash Road to Hollywood Drive, turn left, drive to traffic light, turn left onto Loop Road. Descend the Government Hill bluff and cross the Ship Creek floodplain on the "C" Street overpass. Turn right at the first traffic light (3rd Avenue) and drive west to the end of 3rd where it curves into "L" Street. Continue on "L" Street to 5th Avenue, turn right on 5th Avenue and descend the bluff to Elderberry Park.

STOP 4. Elderberry Park.

You are located at the toe of the "L" Street slide of 1964. This slide was somewhat different than the others we have seen in that here essentially no vertical displacement occurred; movement was almost entirely horizontal as a translational gliding block with a graben being produced behind the moving block (fig. 12). This slide involved all or parts of about 30 city blocks, was about 3/4 mile long and over one thousand feet wide. The graben at the back of the slide was up to about 250 feet wide and 10 feet deep; it was here that most building damage occurred. The translational block carried overlying structures as much as 14 feet horizontally with little vertical movement and surprisingly low damage figures. Pressure ridges formed seaward from the slide in the low areas such as where you are standing. These ridges disrupted the Alaska Railroad tracks in front of you. Again, the sensitive clays and silts (F.III, Bootlegger Cove Formation) are believed to have been the source of failure. Some geologists and engineers have speculated that the "L" Street Slide is an example of a slide that did not have sufficient time, shaking, and/or momentum to progress to block failure mode typical of the Anchorage landslides. The logical extension of this line of thinking is to surmise that movement may resume in the next large earthquake. Conversely, one might prefer to believe that F.III is not as extensive or sensitive here as elsewhere so that no further movement should be anticipated, i.e., the 1964 earthquake caused what sensitive soils that were present to fail on a limited basis, in turn inducing present stabilization.

The question of future stability in this area has not been resolved. Soon after the earthquake, the graben was back-filled with sand and gravel, the damaged structures were removed or condemned, and new construction has since followed. One cannot help but be impressed or dismayed by the large high rise structures constructed on the bluff front of the "L" Street slide block and astride the regraded graben. One further note, as you survey the area around Elderberry Park, observe the irregular

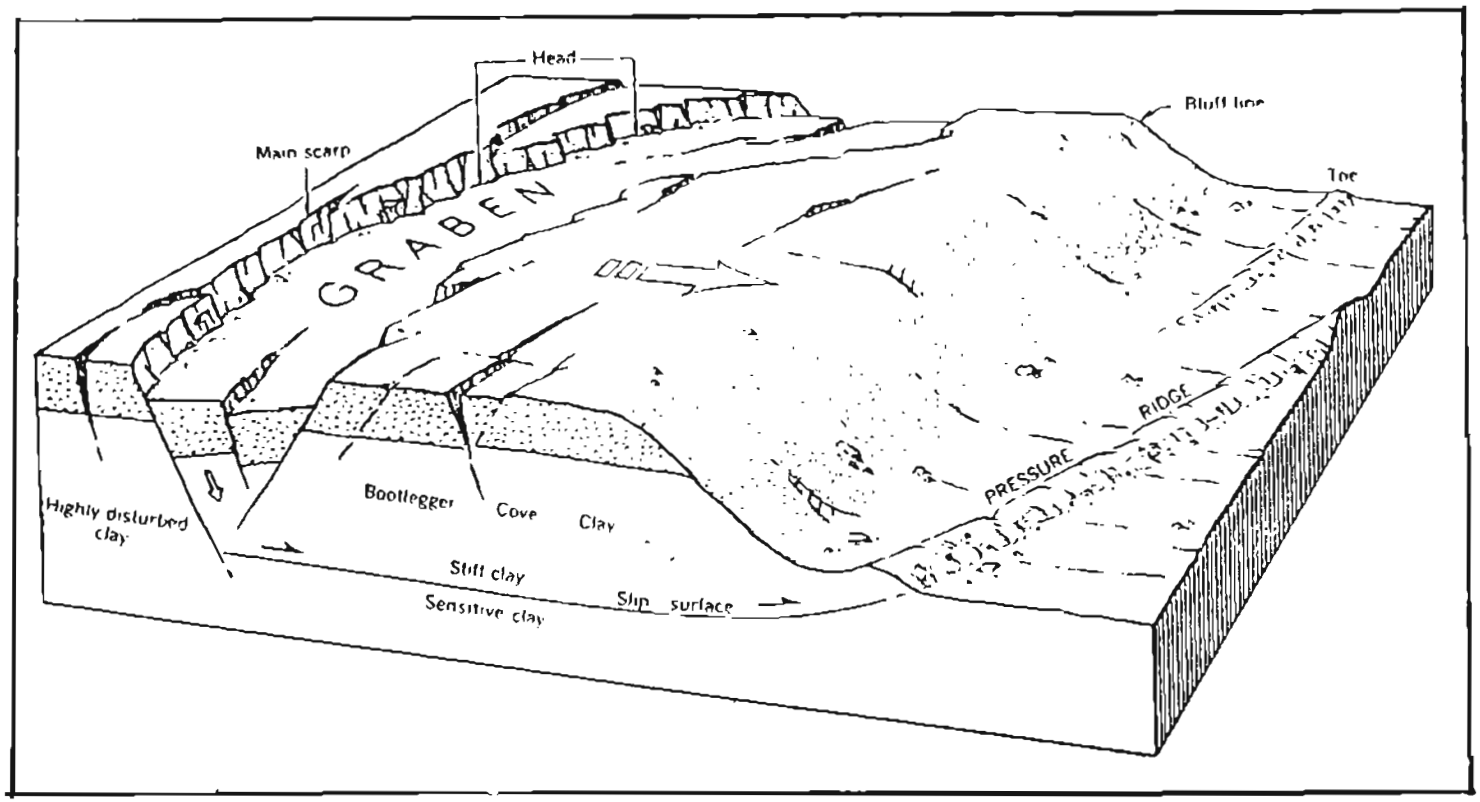


Figure 12. Block diagram of a translatory slide similar to that which occurred at "L" Street. (From Hansen, 1965.)

shape of the terrain. Foundation exploration for some of the luxury condominiums nearby encountered chaotic masses of blue-gray silts and clays mixed with buried trees, all of which leads one to conclude that this entire area below the bluff consists of old landslides which have failed in a manner similar to Government Hill or 4th Avenue.

If you are visiting this stop at low tide you will have a particularly good view of Anchorage's "beaches". The tidal flats visible here are typical of most coastal areas of Cook Inlet. The high mountains surrounding the inlet are presently glaciated so that large quantities of glacial rock flour, silt, and sands are constantly poured into the inlet. These geologically immature sediments are incessantly being transported, deposited, retransported, mixed, and redeposited to form the olive-gray to blue-gray tidal flats. Minor springs near sea level combined with tidally-receding marine waters result in the numerous small meandering channels cutting across the flats. Those of you anxious to dash out on these "beaches" to sunbathe should be forewarned that the waters of the inlet are a chilly 34 to 38°F throughout the year.

In the middle distance to the west across the inlet the prominant mountain you see is Mt. Susitna. In the lowlands on the near side of that mountain is one of the most active faults in this part of the state, the Castle Mountain Fault. This is a strike-slip fault, with a dominant right lateral sense of movement. It extends for several tens of miles in length see fig.) and displaces middle to late Holocene sediments. Recent regional seismicity studies have concluded that this fault (about 20 miles from Anchorage) is capable of generating earthquakes in excess of magnitude 7.0.

Historically, Elderberry Park was one of the original park reserves of Anchorage dating back to 1915. In those early days crime was one of the biggest problems facing the city. Combined with a territorial prohibition law in place since 1916 and the Alaska Engineering Commission's prohibition of the sale and manufacture of alcohol, bootlegging grew into a big business for the young town. Bootleggers built connecting tunnels along the shores of the inlet and the end of 5th Avenue at the bluff was frequently referred to as "Brown Sugar Road" due the wagonloads of sugar hauled to the area. In the 1960's a couple living in a

log cabin at the base of the bluff at 5th Avenue fell into a 10 X 10 foot underground room once used to house a still.

END Stop 4. Return on 5th Avenue to the traffic light, turn right on "L" Street. You are now driving parallel to and crossing the "L" Street Landslide graben. At 9th Avenue we will cross the Delaney Park Strip, a green belt several blocks in length which is the site of numerous athletic, social, and cultural activities throughout the year. Continue south on "L" Street descending to what was once the mouth of the floodplain of Chester Creek. An artificial dam at the mouth of this floodplain has produced Westchester Lagoon, a favorite spot for non-motorized boating in summer and ice-skating in winter. Just south of the lagoon take the exit ramp that leads up to West High School. When this school was constructed isolated remants of discontinuous permafrost were encountered. Turn right onto Hillcrest Drive, continuing to the first stop sign. Turn left onto Forest Park Drive and continue south to Northern Lights Boulevard. Turn right on Northern Lights and within a few blocks you will be driving by the Turanagain Heights subdivision (on your right). The Knik Arm bluff line is about 4 blocks from here. A dense network of open ground fissures extended from the bluff (1964 Turnagain Heights Landslide scarp) to Northern Lights Boulevard. Continue west on Northern Lights to the parking lot at Earthquake Park.

STOP 5. Earthquake Park.

You are now situated at the scarp directly behind the Turnagain Heights Landslide, the single most extensive 1964 landslide in Anchorage (fig. 13). This is a municipal park and a series of interesting displays are provided. Take time to examine these geologic and ecologic pictorials. (see Attachment 2)

END Stop 5. Return from parking area to Northern Lights Boulevard and turn right. Driving west the road becomes Point Woronzof Road and continues approximately 2 miles to Point Woronzof. Turn off (right) into the overlook area at the point.

STOP 6. Point Woronzof.

This area is underlain by stratified silt, sand, and gravel at least to sea level. These deposits are of the late Pleistocene age and were accumulated in an ice-marginal environment. The glacier itself was situated west and north of where you are standing, having entered the Knik Arm-Point

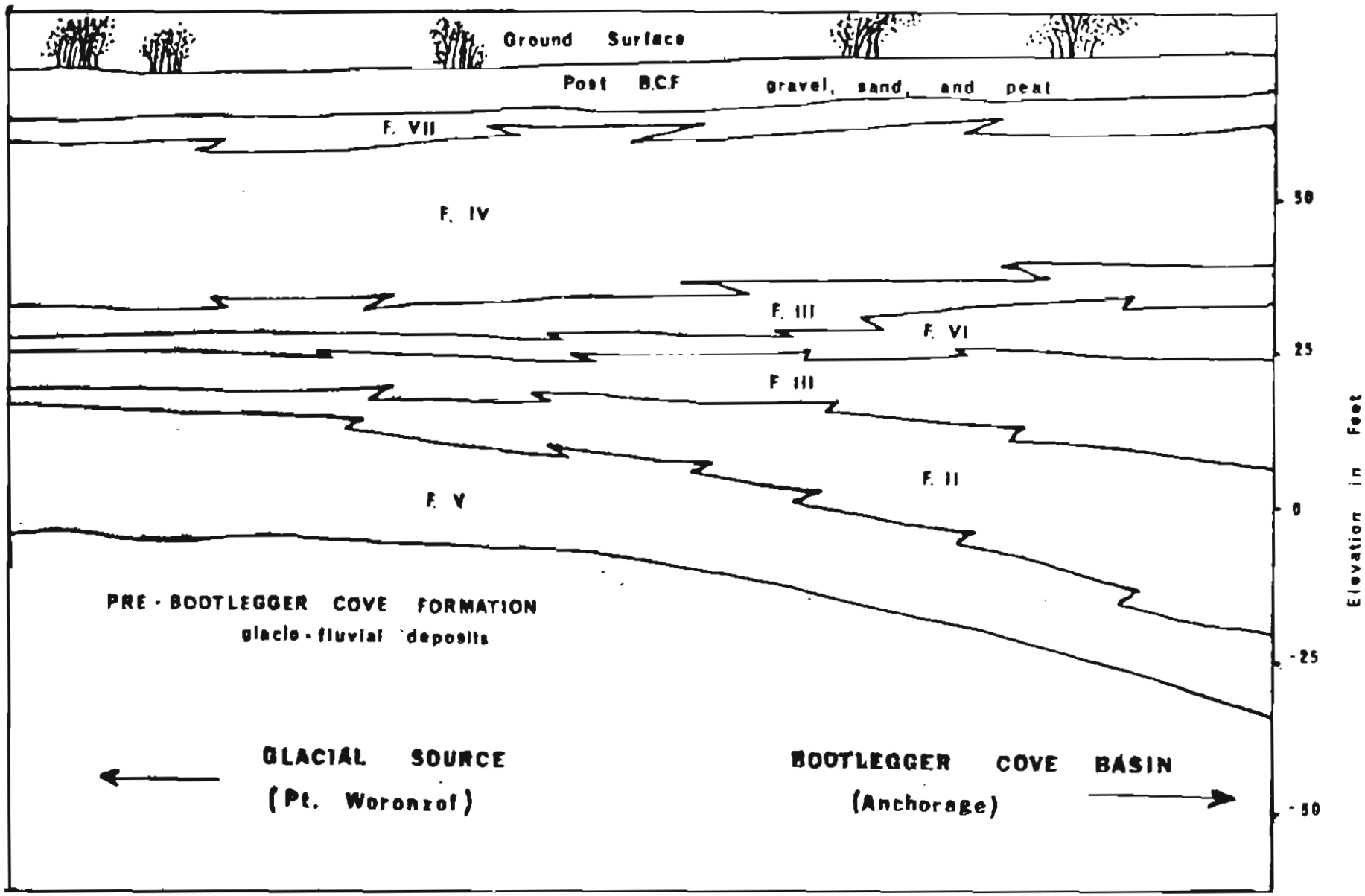


Figure 13. A generalized geologic cross section of the Turnagain Heights area derived from the correlation of geotechnical borehole logs and CPT logs.

MacKenzie area from the north. This glacier was fronting (perhaps floating) in marine waters of the ancestral Upper Cook Inlet. Resultant sedimentation was in the form of glacio-marine fan deltas and ice-contact deposits. The walls of the pit behind the point at this stop show the fan delta sands and gravels; a gully cut into the face of the bluff in front of you shows the highly contorted beds of silt, sand, and gravel typical of ice-contact deposits. Updike and Ulery (1983) have concluded that these deposits are contyemporaneous with the Bootlegger Cove Formation, i.e., the ice marginal deposits grade laterally (eastward) into the quiet basin deposits of the Bootlegger Cove Formation. Fire Island, visible to the west, has been carefully studied by Schmoll and Gardner (1982) and appears to represent a near-ice front complex of silty diamictons and stratified sands and gravels (Attachment 3).

Point Woronzof is a particularly good example of the bluff erosion system typical of Upper Cook Inlet. It is subjected to forces associated with both the incoming and outgoing tides. The point is composed of predominantly sand and gravel which are usually dry, loose, and easily washed away. The bluff line is sparsely vegetated, over 150 feet high, and quite steep. These factors combine for rapid erosion rates. A report by the U.S. Geological Survey (Miller and Dobrovolny, 1959) discusses Point Woronzof erosion rates. According to this publication, the bluff retreated about 30 feet between 1909 and 1918, 26 feet from 1918 to 1941, 13 feet between 1941 and 1947, and 21 feet between 1947 and 1953, for a total of about 90 feet of retreat in 44 years (an average of 2 feet per year). Dr. James Riehle of the USGS conducted some additional measurements utilizing the comparison of several sets of aerial photographs taken between 1960 and 1964, and found another 10 feet of erosion between 1969 and 1978.

One of the primary sources of electrical energy for Anchorage comes from the Beluga River area several tens of miles to the west across Cook Inlet. The transmission lines come to the bluff at Point MacKenzie (across Knik Arm), lie along the floor of the arm, and re-emerge here at Point Woronzof (note the large warning sign regarding the cable's presence).

A pervasive problem in Anchorage construction is the presence of peat deposits up to several feet in thickness at the ground surface. These deposits must be excavated prior to construction because of their poor bearing capacity. The large

volumes of peat must be disposed of and pits like the one behind you have been targeted for such disposal.

On a clear day from this point you will be able to see Mt. McKinley (elevation 20,320 feet) 125 miles to the north, the Talkeetna Mountains 40 miles to the northeast and the Alaska Range 60 miles to the west. The conical peak on the Alaska Range skyline is Mt. Spurr, an active volcano which last erupted in 1953, spreading a thin layer of ash over Anchorage.

About a quarter mile south of Point Woronzof are the remains of what was probably a summer fishing camp for the Tanaina, a nation of the northern Athapaskan Indians. The Tanaina Archaeological Site is the only known archaeological site within the Anchorage bowl. Parts of this site have been dated prior to 1741 (the time of Vitus Bering's explorations) and thus are considered pre-history.

There are three distinct areas of the village site, two of which are considered historic. The "pre-historic" house pits consisted of a semi-subterranean house known by the Russian term "barabara". The eight pits found are located below the bluff and consist of two connecting rectangular rooms covered by a log structure. Located approximately one-eighth of a mile north of the house pits a "banya" or steam house is found and is probably the most recent of the remains. Above, on top of the bluff is the grave site consisting of several pit areas and a grave house roof. The sites are not easily accessible but one house pit has recently been damaged by activities related to gravel extraction.

END Stop 6. Return east on Point Woronzof Road to Postmark Drive, turn right. Drive past the main post office to stop sign and turn left. Anchorage boasts one of the largest concentrations of small, privately owned aircraft in the world. Many of the aircraft are tied here at Anchorage International Airport (on land) and at Lake Hood, the nation's largest floatplane base. Alaska has the largest number of licensed private pilots per capita in the United States primarily because so much of the state is accessible only by airplane. Turn right onto Aircraft Drive and in a few blocks left onto International Airport Road to Jewell Lake Road and turn right. You will note many large commercial transport aircraft used to transport goods internationally to and from the "lower 48 states" as well as

making shipments to and from Alaska bush communities and oil fields. Continue past Delong Lake Park to traffic light (Raspberry Road) and turn right. At next traffic light (Sand Lake Road) turn left and proceed to the large sand and gravel quarries. Park beside the road overlooking these pits.

STOP 7. Sand lake sand and gravel quarries.

Sand and gravel aggregate is the second largest money producing natural resource in the state (after oil). For several years the Sand Lake pits have produced much of the aggregate necessary in Anchorage for construction. These pits are excavated into the stratified glacial fan delta sands and gravels similar to those at Stop 6. Unfortunately, most of good quality gravel has been removed and only sand and gravelly sand remains. No other major source of aggregate is presently being used in Anchorage and as a result much of the present-day gravel is brought into the city by truck and train from as far as 50 miles away.

The dilemma now facing the city is what to do with these soon-to-be-abandoned pits. Alternatives include: 1) regrading slopes, landscaping, and constructing homes, 2) backfilling the pits with waste peat, or 3) using the area for solid waste disposal. The problem has not been resolved (see Attachment 4).

From this point you have a good view of the Chugach Mountains. In the area you can see the mountains are composed of the Valdez Group and McHugh Complex (Jurassic-Cretaceous) including greywacke, phyllite, argillite, metavolcanic, and metasedimentary rocks. These rocks were formed in an island arc-trench environment which has subsequently been folded and thrust onto the continental margin along a suture zone which extends from the Gulf of Alaska along a broad arc to beyond Kodiak Island. Further north from Anchorage recently active faults have been identified along the front of the Chugach Mountains as part of the Border Ranges Fault System (fig.).

Looking south you will see Sand Lake Road ending in a T-intersection. Two to three blocks of houses are visible south of the intersection and beyond that is Turnagain Arm with bluffs similar to Knik Arm. In recent years narrow wedge-type landslides have occurred along these bluffs giving some concern to bluff-front property owners. These slides are due to both slope undercutting at high tide level and to man-made

modifications above or on the slopes affecting natural drainage and vegetation.

END Stop 7. Continue south on Sand Lake Road to Dimond Boulevard intersection. If optional stop 8 is to be made turn right. [If stop 8 is to be skipped turn left onto Dimond and turn forward in the guidebook to "End Stop 8".] Proceeding right on Dimond Boulevard (west) note that some of the abandoned pits are in the process of being backfilled with waste peat. The road curves right, becoming Jophur Road. Continue to the entrance to Kincaid Park; turn left and drive to the end of the dirt road.

STOP 8. Kincaid Park Motorcross Pit.

This area was once an uncontrolled solid waste dump site for Anchorage residents. In the past few years as part of the municipality's efforts in parks and recreation, the sandy basin has been transformed into a motorcycle race course. The sediments underlying the area are eolian silts and sands of middle to late Holocene age. Strong prevailing winds flow down Turnagain Arm (east to west) acting voraciously on the bluff faces here composed of glacial fan delta silts and sands. The sediments are wind-retransported to the crest of the bluffs where longitudinal dunes are constructed inland at an oblique angle to the bluff line. If you have time to hike to some of the natural exposures in or around the pit you will find distinct buried volcanic ash beds (airfall) and peat layers, some of which date back about 3500 years b.p. Dunal morphology is preserved in the adjacent heavily wooded uplands.

END Stop 8. Return to Jophur Road, then east on Dimond Boulevard. Note Jade Park on the left side of road which has been built on backfilled waste peat in one of the abandoned pits. Continue east on Dimond Boulevard about 9 miles up the lower slope of the Chugach Mountains. The road curves right becoming Hillside Drive. Continue about 4 blocks to Upper O'Malley Road to where pavement ends and road forms a "Y". Take the left road (Prospect Road) and continue along this gravel road to the entrance to Chugach State Park. Enter the park and stop at the parking area.

STOP 9. Chugach State Park.

You will note that you are now near timberline on the mountainside. This is the limit of private land in the area. Property in this part of Anchorage is very expensive because of

the spectacular view and alpine environment. Although the land here is in high demand, it is fraught with problems. Beneath a mantle of glacial till is continuous bedrock. No municipal water supply is available up here, so each home must have its own water well. Groundwater resources are at best sporadic and some homes have wells hundreds of feet deep to acquire very modest amounts of water. Because of the shallow bedrock some homes have difficulties in establishing suitable septic systems. Because of the elevation spring comes weeks later and winter begins weeks earlier than at lower elevations. Strong winds are often the case; 50 mph winds occur yearly, occasionally damaging exposed structures. Nevertheless, the Alaskan perseverance prevails and some of the finest homes in Anchorage are located on these sub-alpine lots.

If time permits we recommend that you take a short hike on one of the trails from the parking lot, noting the distinct changes in climate and vegetation from that which you have experienced at previous stops.

END Stop 9. Return to park entrance, turn right onto Slalom Drive, then onto Schuss Drive. Continue south on Schuss Drive to Upper O'Malley and left onto O'Malley Road, driving down from the Chugach Mountains to the New Seward Highway. Turn right onto the New Seward Highway and travel north into Anchorage, to 5th Avenue. Turn left onto 5th Avenue and drive west to downtown Anchorage. At "E" Street turn right and return to the Westward Hilton.

END ROAD LOG

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ATTACHMENT 1.

Fourth Avenue Buttress

BUTTRESS DESIGN EARTHQUAKE-INDUCED SLIDES[a]

By Erwin Long,[1] M. ASCE, and Warren George[2]

INTRODUCTION

The Fourth Avenue slide is one of a number of major landslides that took place as a result of "The Great Alaska Earthquake of 1964." Because this slide affected and endangered the business section of Anchorage, an area of high property value and improvements, it was of special interest to the community. The area of disturbance covers a block of ground approximately 1,800 ft long by 900 ft wide. The stabilization measures for the area were especially significant because of the economic values involved.

CHARACTERISTICS OF THE EARTHQUAKE

The earthquake, which took place in South Central Alaska, with initial shocks timed at 5:36 p. m. on Good Friday, March 27, 1964, had by consideration of all evidence a Richter magnitude variously rated between 8.4 to 8.6 and a duration of 5 min plus. The destruction was estimated at one-half to three-quarter billion dollars. The loss of life was limited to 114, the result solely of many unusually good attendant circumstances.

According to one analysis, this earthquake has been rated to have released at least twice the energy of the San Francisco earthquake.[3] Tectonic uplift or subsidence occurred over an area of 34,000 sq miles, an area four times the size of the State of Massachusetts. Coastal land masses subsided as much as

[a] This paper was part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 93, No. SM4, July, 1967. Manuscript was submitted for review for possible publication on January 25, 1967.

[1] Long, Erwin, Chf., Soil Design Sect., U. S. Army Engr. Dist., Anchorage, Alaska.
[2] George, Warren, Chf., Engrg. Div., U. S. Army Engr. Dist., Anchorage, Alaska.

[3] Grantz, A., Plafker, G., and Kachadoorian, R., "Alaska's Good Friday Earthquake, March 27, 1964," Circular 491 Geological Survey U. S. Department of the Interior, Washington, D. C., 1964.

7-1/2 ft, and a 50-ft rise in land mass has been recorded in other areas.[4] The well-frozen ground and low snow pack limited avalanching; however, large snow slides occurred throughout the area. If such an earthquake had occurred in a more populated area, the loss in property values and lives would probably have been appalling.

The first shocks, arriving from their origin under northern Prince William Sound (a distance of approximately 80 miles east of Anchorage) were followed by a series of shocks occurring along the main fault zone, extending from the origin southwest past Kodiak Island, an estimated distance of 400 miles to 500 miles. It is not entirely clear how much of the intensity of the shocks may have been contributed by the movement that occurred along the secondary fault zones lying north and west of the main fault.[5] The length of the fault-slippage zone, coupled with the unconsolidated deposits on which Anchorage is located, certainly affected the degree of attenuation, magnitude, and variety of arriving shear waves and the locally generated Raleigh waves.

Although estimates of the duration of the earthquake varied considerably, eye-witness observation and timing by watches and clocks indicated a duration of significant magnitude, ranging from 4 min, 25 sec, to 7 min, 20 sec.[6] All timed observations were made at locations situated over relatively deep and saturated alluvium throughout South Central Alaska. It is possible that soil resonance may have increased the apparent duration in some areas as compared to others. Estimates of the period of each shock ranged from 1 sec to nearly 2 sec, with 1.5 sec per shock considered a probable average. A series of four major, documented earthquakes are reported to have occurred near Yakutat, Alaska as recently as 1899, resulting in recorded uplift of 47.5 ft,[7,8,9,10] which indicates that earthquake recurrences are to be expected. The Yakutat area experienced another earthquake in 1958. Although believed to be of less magnitude than that of the 1899 episode, displacements of 21.5 ft laterally and 3.5 ft vertically were recorded.[11] The 1811 series of earthquakes near New Madrid, Missouri resulted in three separate seismic disturbances with intensities of 12 on the Modified Mercalli Scale. The average estimate of duration was 6 min to 7 min, producing widespread landsliding and settlements.[12,13,14]

[4] Plafker, G., Science, Vol. 148, No. 3678, June 25, 1965, pp. 1675-1687.

[5] "Assistance and Recovery, Alaska, 1964," Coast and Geodetic Survey, U. S. Department of Commerce, Washington, D. C., Mar., 1965.

[6] "Earthquake Interviews, Documentary Eye-Witness Accounts," U. S. Army Engineer District, Alaska, 1964 (unpublished).

[7] Heck, N. H., "Earthquake History of the United States," No. 41-1, Part I (revised Edition, 1956, by R. A. Eppley), U. S. Department of Commerce, Washington, D. C., 1956, p. 73.

[8] Tarr, R. S., and Martin, Lawrence, "Earthquakes at Yakutat Bay, Alaska, in September 1899," Professional Paper No. 69, Geological Survey, U. S. Department of the Interior, Washington, D. C., pp. 1-135.

[9] Davidson, Charles, Great Earthquakes, Thomas Murly and Co., London, Chapter XI, 1936, pp. 158-174.

[10] Martin, L., "Alaskan Earthquake of 1899," Bulletin, American Geological Survey, 1910, Vol. 21, pp. 339-406.

[11] Miller, Don J., "Giant Waves in Lituya Bay, Alaska," Professional Paper No. 354-C, Geological Survey, U. S. Department of the Interior, Washington, D. C., 1960, p. 55.

[12] Heck, N. H., op. cit., pp. 33-36.

[13] Fuller, M. L., "The New Madrid Earthquake," Bulletin No. 494, Geological Survey, U. S. Department of the Interior, Washington, D. C., 1912, p. 119.

[14] Davidson, Charles, op. cit.

Strong-motion records of magnitude, frequency, or duration are not available for the 1964 Alaska earthquake. W. K. Cloud, of U. S. Coast and Geodetic Survey, has estimated a maximum acceleration of 0.18g, based on a study of compiled data from other earthquakes. An interesting analysis of tilted tombstones in the Anchorage area indicated a possible maximum acceleration of 0.30g.[15] Because soil failure was gradual and progressive, an average effective acceleration of 0.12g has been selected for analysis purposes.

GEOLOGY AND SOIL CONDITIONS OF THE ANCHORAGE AREA

A fairly detailed geologic knowledge of the Anchorage slide area has been developed as a result of data obtained from 200 exploration borings, varying in depths up to 240 ft,[16] and from the logs of several hundred water wells.[17] The deepest well was 850 ft deep, with tertiary rock encountered at approximately 700 ft.[18] Many of the exploration holes required double drilling to obtain, as closely as possible, continuous samples for correlation and more accurate coverage of the strata. Bucket auger holes were drilled to delineate the zone of failure more accurately, and to observe and study each strata directly.

The terrace on which Anchorage is built is topped by a sand and gravel alluvium formed by meandering streams during the late Wisconsin-Naptowne Glacial period.[19] The present bluff edges of the alluvium terrace were probably formed by stream and sea erosion and from the effects of numerous previous landslides of static and earlier earthquake origin. The underlying Bootlegger-clay formation is believed to have been formed between the Knik and the Naptowne subglacial advances.[19] The Bootlegger formation embraces an upper, stiffer, silty-clay layer strengthened by dessication and possible freezing during the latter stages of its formation. This upper formation contains frequent silt and sand lenses. The intermediate layers have more infrequent sand and silt lenses, and the clays are commonly soft and highly sensitive. The lower clay layers show an increase in strength with depth and sand and silt lenses are almost totally absent. Underlying the Bootlegger-clay formation and continuing to the tertiary rock, are glacial tills of an earlier age and inter-bedded sands, gravels, silts and clays.[18,19] Mineralogical examination of the Bootlegger clay shows predominately quartz with lesser amounts of feld-

[15] "Analysis of Earthquake Damage to Military Construction in Alaska, Mar. 27, 1964," Directorate of Military Construction, Engineering Division, Corps of Engineers, U. S. Department of the Army, Washington, D. C., Aug., 31, 1964, Appendix I, p. 8.

[16] "Anchorage Area Soil Studies, Alaska," report by Shannon & Wilson Inc., for U. S. Army Engineer District, Alaska, Aug., 28, 1964.

[17] Waller, R. M., Cederstrom, D. V., Trainer, F. W., "Data on Wells in the Anchorage Area, Alaska," Report No. 14, Department of Health and Welfare, State of Alaska, in cooperation with the U. S. Geological Survey, 1961.

[18] Waller, R. M., Cederstrom, D. V., Trainer, F. W., op. cit., Well No. 28.

[19] Miller, R. D., and Dobrovolny, Ernest, "Surficial Geology of Anchorage and Vicinity, Alaska," Professional Paper No. 1093, Geological Survey, U. S. Department of the Interior, Washington, D. C., 1959.

spar flour. The dominant "clay" mineral is chlorite and illiteclay. Montmorillonite was not found in any sample.[20]

FOURTH AVENUE SLIDE AREA

The Fourth Avenue slide took place along an east-west trending bluff with variable north-facing slopes averaging 22%. The elevation of the slope toe varied from 20 ft to 26 ft msl. The top of the slope had an elevation of approximately 107 ft msl. The slope failed as a translatory slide with movement to the north and northwest. A graben was formed along the head of the slide by

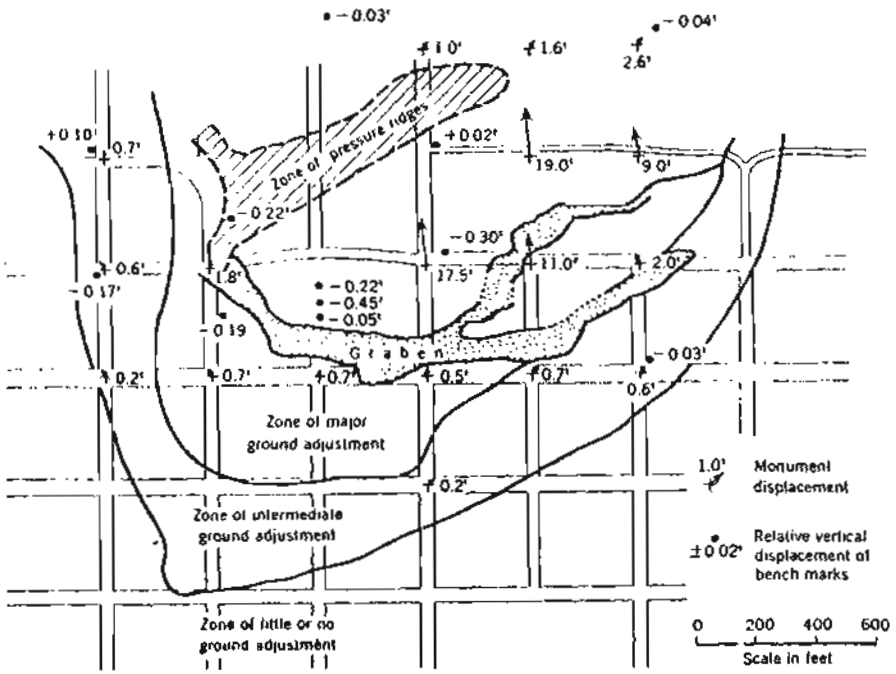

FIG. 1.—GRABEN AND PASSIVE ZONE DISTRIBUTION

the movement of the slide mass toward the bluff and by a soil wedge sinking into the crack formed. The soil wedge descended progressively into the crack that formed. Investigations revealed that the approximate elevation of failure was 45 ft msl with passive failure areas extending down-slope to elevations of 25 ft to 35 ft msl. The extension of the passive zone failure is believed to have been influenced by the effect of the frozen crust. The areas of passive failure appeared to occur in thawed areas, such as along water lines and adjoining basements of heated structures (Fig. 1).

The main graben at the head of the slide dropped 11 ft along the north side of Fourth Avenue. Horizontal ground displacements of 1.8 ft NNE were measured at E Street and Third Avenue (the hinge point) and 19 ft NNE at Second

[20]Shannon & Wilson, Inc., op. cit., Appendix H.

Avenue and B Street. Settlement in ground elevation immediately behind the slide scarp varied from 0.03 ft to 0.2 ft. Indications are that the ground-elevation drop of the slide block between the graben area and the passive toe varied from 0.2 ft to 0.7 ft.

A second graben had started to form 300 ft south of the Fourth Avenue graben across D Street. The graben settled 0.5 ft by the end of the earthquake and caused severe damage to one modern three-story office building. This second wedge is believed to have started toward the end of the earthquake, and in all probability would have gradually enlarged with a longer duration of shaking. The main graben split into two grabens east of C Street. The northern branch may be connected structurally to the extension of an old graben which was located between C Street and E Street on Third Avenue. The suspected old graben lying north of the Fourth Avenue graben was subsequently exposed in a trench excavation.

Many quite comprehensive investigations were accomplished in the slide area to ascertain the mechanism of failure and to guide buttress design. Borings to obtain undisturbed samples were made at 41 locations. Bucket auger holes were also drilled at strategic locations. Slope-indicator casing was installed for observations at ten locations. In-place vane-shear tests were conducted at three locations. Testing and studies included continuous torsional vane and pocket penetrometer shear-strength determinations, unconfined compressive-strength tests, Q and R triaxial compression-strength determinations, consolidation tests, dynamic modulus tests, pulsating-load tests, and mineralogical studies. Physico-chemical analysis and foramanifera determinations were made on samples from other slide areas. These were considered representative of this particular Bootlegger strata.

PHYSICAL PROPERTIES OF SOIL MATERIALS

Although initial failure within a sand layer is indicated, failure was also possible in the adjacent underlying clay layers, and both factors were considered in the stabilization design.

Sand.—Sand, inter-bedded with thin silt and clay layering, was found to exist predominately between el. 40 msl and el. 60 msl, with an average thickness of 15 ft. Standard penetration resistance varied from 50 blows per ft to over 100 blows per ft in this strata, indicating a very dense formation.

Clay.—The shear strength of the clay was more than 0.5 tsf in the upper zone, where more continuous sand layers were present. Below these sand layers, the undisturbed shear strength averaged 0.35 tsf down to el. 20 msl. The strengths then increased with depth. Some highly sensitive samples showed undisturbed strengths as high as 0.66 tsf. Other tests indicated shear strengths as low as 0.2 tsf. Although sampling was carefully performed, some influence of disturbance from handling cannot be ignored.

The remolded strength has special significance, as it indicates the minimum strength resisting sliding with remolding along a potential slide plane. The remolded strength was generally found to be more than 0.1 tsf above the principal sand layer. Shear-strength values as low as 0.006 tsf to 0.05 tsf were encountered between el. 20 msl and el. 40 msl.

In the upper clay zone above el. 45, and in the clays below el. zero, sensitivities were generally found to be below ten. In the zone between el. 0 and el.

45, sensitivities varied in individual layers from less than 10 to values over 50, with many samples showing sensitivities above 30. It is noted, and believed to have significance, that the highest sensitivities and lowest remolded strengths occurred in the area of greatest volume of ground water seepage. As expected, the water-plasticity ratio or liquidity index increased with sensitivity. Remolded strengths were also lowest where the liquidity index was highest.

The majority of the liquid limits of the typical clay range between 24% and 46% for the upper, stiff-clay layer, from 20% to 38% in the sensitive clay zone, and from 32% to 52% in the lower, stiff-clay zone. The liquid limits vary from 22% to 28% for clay with sensitivities over 40.

Dynamic modulus tests conducted by Wilson[16] tend to indicate a modulus of shear of 1,500 psi for soft, sensitive clay, 5,000 psi for the stiff clays, and approximately 20,000 psi for sand at a 60 ft depth. The shearing modulus of sand and clay would approach zero as liquefaction of the layer occurred.

Pulsating-load tests conducted by H. B. Seed on saturated sands have indicated[21,22] that failure caused by dynamic loading will probably occur sooner at shallower depths because of the increased probability of stress reversal coupled with a low confining pressure. A higher degree of compaction would tend to increase the number of cycles required for failure. In the clay the number of cycles required for failure appear to be reduced with an increase in the degree of stress reversal; however, because the shear modulus increases very slowly with depth, clays tend to fail more readily under dynamic loading at greater depths.[23]

The dry density of the clays generally ranged between 90 pcf and 100 lbs pcf, with an average apparent specific gravity of 2.79. The wet density of the clays was about 125 lb pcf.

EARTHQUAKE DYNAMICS LOADING

Dynamic loading of the soil mass during the earthquake would have a varying effect on the soils, depending on their inherent strengths and their deformation characteristics. The more sensitive, high water-content clays would, of course, have a greater deformation for an applied stress than sands or less sensitive clay. The greater strain or deformation in such materials would tend to cause a relatively rapid pore-pressure increase with associated reduction in strength. Where a soft clay adjoins a harder layer, the movement of the more rigid soil would tend to cause a greater increase in pore pressure. The strain of the soft clay adjoining a sand layer would be greater than that of the sand layer up to the point of liquefaction of the sand layer. Depending on the density of the sand, the effective stress level, and the initial strength of the clay, the sliding should begin in the clay if stress applications were insufficient to cause liquefaction of the sand; conversely, the sand might liquify prior to

[21] Seed, H. Bolton, and Lee, Kenneth L., "Liquefaction of Saturated Sands During Cyclic Loading," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 92, No. SM6, Proc. Paper 4972, Nov., 1966, pp. 105-134.

[22] Shannon & Wilson, Inc., op. cit., Appendix C.

[23] Seed, H. Bolton, and Chan, Clarence K., Strength of Clays under Simulated Earthquake Loading Condition, Institute of Transportation and Traffic Engineering, University of California, Berkeley, Calif., 1966.

the failure of the clay. Laboratory tests have indicated that the smaller the surcharge on sand, the more easily it will liquify under dynamic stress of the earthquake.[21] Experience in the 1954 and 1964 earthquakes at Rabbit Creek near Anchorage and at Niigata, Japan, have further supported this theory. The New Madrid earthquake of 1811 also showed evidence of liquefaction at relatively small static stresses.[12,13,14] Tests on clay indicate a pronounced increase in early failure with greater intensity of applied cyclic stress.[23]

Evaluation of all clay-related slide failures in the Anchorage area indicated that they occurred near the surface of the soft, sensitive-clay layers.[18] Investigations by R. B. Peck disclosed that soil resonance did result in large horizontal displacements during small distance earthquakes in Mexico City in May, 1962.[24,25] These phenomena are helpful in understanding the Fourth Avenue slide.

ZONE OF FAILURE

Evidence from exploration data indicates that liquefaction of a sand layer occurred at el. 48 msl immediately behind the slide scarp on Fourth Avenue. It is probable that failure occurred either entirely within the dominant sand layer or in an underlying layer in which both sand and clay were present.

There is strong evidence that the lateral movement of soil outward in a translatory slide relative to the pre-earthquake slope relates to the loss in quantity by settlement in the grabens. On this basis, the volume of soil loss divided by the horizontal displacement of the section should approximate the depth of failure. The values thus obtained indicate a possible depth of failure at the el. 45 msl.[26]

Failure was of a translatory nature in all of the major Anchorage slide areas. The translatory plane in the Fourth Avenue slide may have had a slight slope, and it is possible that failure occurred at several elevations. As previously indicated, the passive toe failure zone of the sliding wedge occurred predominantly along a line of heated structures and utility lines, indicating the greater failure-resistance afforded by frozen ground, which, without thawing influence, probably ranged from 6 ft to 10 ft deep. The break-out elevations of the passive toe failure zones were more predominant at a lower elevation than that of the assumed zone of primary failure. The lower failure plane elevation may also have been caused by horizontal translation of the frozen crust.

Because the opening of cracks must precede the formation of a graben, it is reasonable to assume that well-reinforced foundations could influence the formation of a graben. This possibility is indicated in several locations of the slide areas. For example, the western end of the Fourth Avenue slide terminates at Third Avenue and E Street where the strong foundations of the

[24] Peck, Ralph, B., "Characteristics of Anchorage Earthquake," Letter Report, J781, June 5, 1964 (unpublished).

[25] Zeevaert, Leonardo, "The Effect of Earthquakes in Soft Subsoil Conditions," presented at the Structural Engineers Association of California Convention, held at Yosemite, Calif., Oct. 4, 1963.

[26] Shannon & Wilson, Inc., op. cit., Paragraph 8.3.

Anchorage-Westward Hotel were believed to have had an influence. Several other instances can be cited.

STABILITY EVALUATION

The most direct method for evaluating relative stability of slopes (Stability 1) was developed by Bjerrum[27] (see Fig. 2). The slope-stability charts provide a convenient guide for the rapid evaluation of potentially stable and unstable slopes. Using Bjerrum's chart, a number of potentially unstable slopes that did not fail were observed to be naturally buttressed. The noses or points of many steeper bluffs remained stable, apparently because of better drainage conditions. Other considerations of the effect of drainage are (1) the degree of

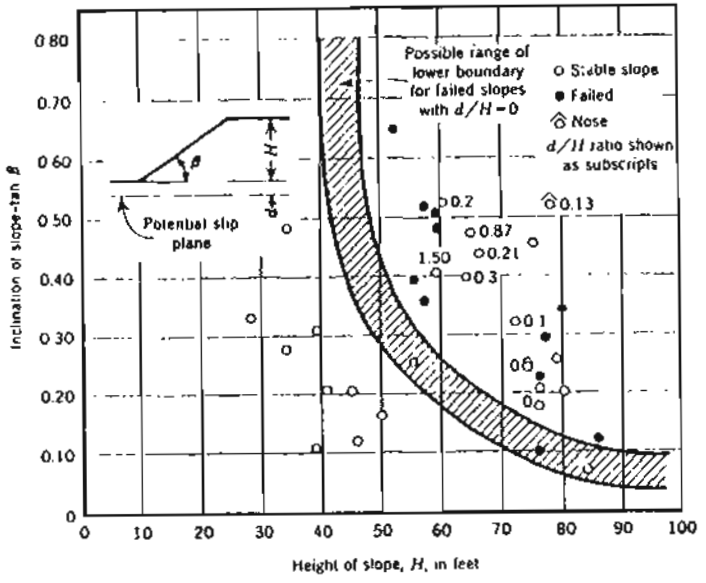

FIG. 2.—SLOPE STABILITY CHART

saturation with its effect on liquefaction; (2) the influence on salt leaching from the clay and its effect on both the undisturbed and remolded strength; and (3) its influence on hydro-static pressure. With the aid of the slope-stability chart, it was determined that a flatter slope would be required to provide stability to the Fourth Avenue area.

An independent problem area, involving a steep slope related to the Fourth Avenue slide and bordered by First and Third Avenues and G and K Streets, was included in the stabilization design. The plane of weakness in this area was found to be at el. 20 msl, which is near the toe of the slope. A large gravel fill had been previously placed along the slopes, which restricted failure at higher elevations. Evaluation of this area indicated the presence of 6 ft of seasonal frost, possibly the major strengthening element.

[27] Shannon & Wilson, Inc., op. cit., Plate 7.1, Fig. 5.

The second method of analysis (S-2) considered in stability evaluation was a static-wedge analysis, using the probable remolded shear strengths along the plane of sliding. Analysis by this method indicated that failure should have occurred at the deepest possible failure planes rather than at the shallowest possible plane, where failure actually occurred.

As a result of the obvious magnitude of resistance required to satisfy a normal wedge analysis, Wilson and Seed conducted laboratory tests to investigate and refine the probable mechanisms of the dynamic translatory failure. When an earthquake causes a soil column to oscillate back and forth,

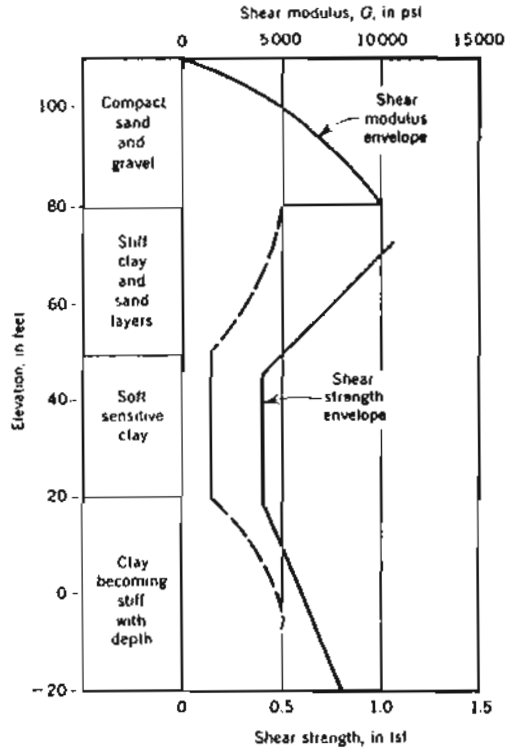

FIG. 3.—GENERALIZED LOG WITH SHEAR MODULUS ENVELOPES

reversing shear strains are developed between adjacent soil layers. The relative magnitude of the shear strains throughout the soil column is not known. However, it is inferred from the fact that failure developed at the top of the soft, sensitive-clay stratum, that the strains were a maximum at this elevation.[28] The change in shear modulus and shear strength with depth is shown in Fig. 3. It was decided that stabilization could be accomplished more effectively within economic limits by providing increased shear resistance at the toe of the failure wedge.

[28] Ibid., Plate 7.1, Figs. 1 & 2.

Evaluation of all slides indicates that the physical dimensions of a slide mass may have some relationship to the half wave length of the shock waves. As the wave lengths and characteristics were not measured, the probable wave lengths and acceleration were determined from the physical dimensions of the slide. In the final evaluation of buttress requirements, four methods of analysis were used. The first method of analysis (Buttress-1)[29] assumed upward transmission of the seismic wave with soil weakening, occurring along the zones previously discussed. From this analysis, it seems likely that the maximum forces acting through underlying strata and tending to accelerate the slide wedge will not exceed the shear resistance along the plane of failure. The force-causing movement after failure would be limited to the active pressure at the head of the slide. A buttress would be required to resist this pressure. It is interesting to note that once liquefaction of a clay layer occurred, sliding could only progress as the result of active pressure at the head of the slide zone. Since sliding ceased immediately on the termination of strong seismic motion, failure as a result of sand liquefaction is indicated. Even with liquefaction of clays, however, a finite resistance approaching the remolded strength could be assumed.

The remaining methods of buttress analysis (B-2, B-3, B-4)[30] assume that seismic forces were translated through the head of the slide mass which forms

TABLE 1.—FORCES OBTAINED BY BUTTRESS DESIGN METHODS

Method number	Static pressure	Dynamic pressure
B-1	132 kips per foot	-0-
B-2	80 kips per foot	120 kips per foot
B-3	80 kips per foot	75 kips per foot
B-4	80 kips per foot	145 kips per foot

the main force after the shear motions through the base have attenuated as failure is approached. B-2 assumes that the average acceleration of the mass would be approximately equal to 0.64 times the magnitude of the peak acceleration. The driving force would be this acceleration acting in a mass of soil 600 ft long plus the active pressure at the head of the slide. The restraining force to be considered would be a buttress capable of resisting the total force, assuming the resistance along the shear plane equals zero.

B-3 assumes that the sliding mass approximates a half wave length (600 ft) and that a 2-in. compression displacement within the slide mass occurs at each end, for a total compression of 4 in. in the 600-ft length. Assuming an average compression modulus, the force necessary to cause the 4-in. compression plus the active pressure at the head of the slide mass would be the maximum force transmitted to a restraining buttress.

B-4 assumes that the liquified layer extends a considerable distance behind

[29] Ibid., paragraphs 7.2 and 7.3, Figs. 3 and 4.

[30] Seed, H. Bolton and Martin, G. R., "Analysis of the Fourth Avenue and 'L' Street Slide Areas," Remolding of Bootlegger Cove Clay with Explosives, Preliminary Stabilization Studies for the Turnagain Buttress, Shannon & Wilson, Inc., for U. S. Army Engineer District, Anchorage, Alaska, Feb., 1965, Appendix D.

the head of the slide and that failure could occur from harmonic motion of reflected waves transmitted through the stiffer soils. Assuming the buttress to be a rigid structure on a nonmoving base and using a 2-in. horizontal amplitude of wave motion, the movement was assumed to be related to the displacement under harmonic conditions. In a nonrigid buttress, the stresses would be somewhat less than assumed. Since failure actually occurred on an oscillating base, the forces (stress) could be higher than assumed. The net result of the two assumptions would make the harmonic analysis a reasonable approximation of the dynamic force. The total force that the buttress must resist would be the active pressure at the head of the slide plus the maximum dynamic force and the inertia force on the buttress itself. Forces obtained by the various methods are shown in Table 1, in which the calculations are based

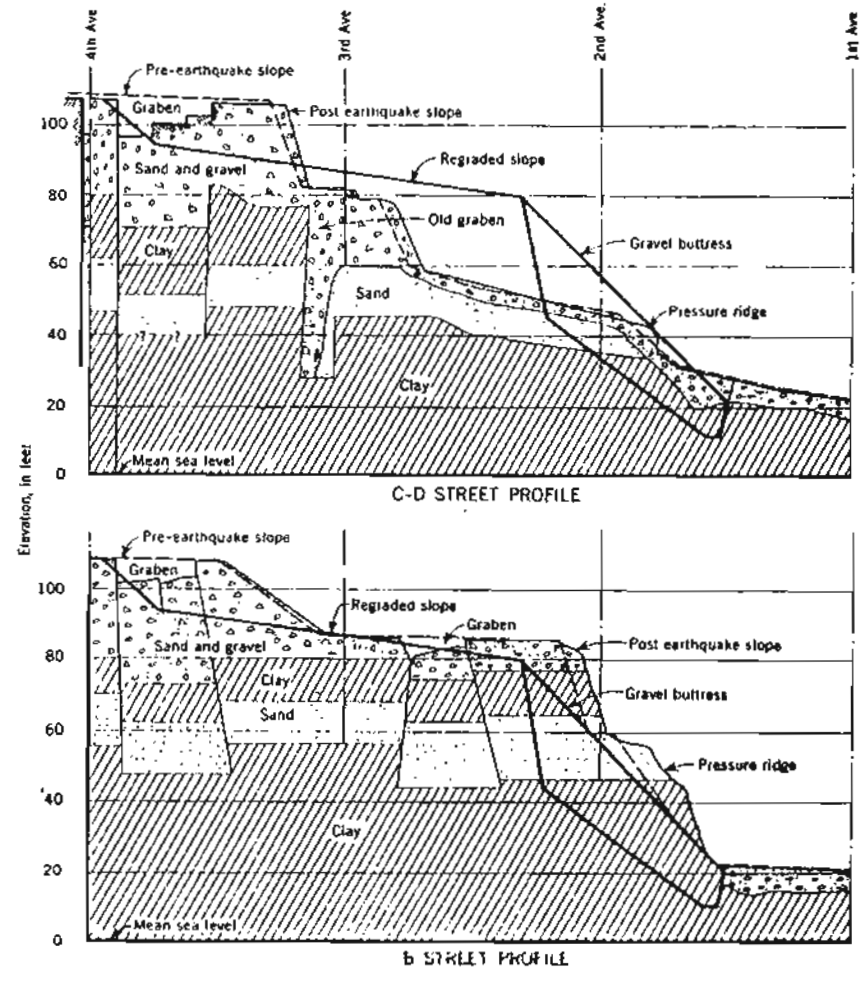

FIG. 4.—FOURTH AVENUE SLIDE AND BUTTRESS PROFILES

on zero resistance along the failure plane. If we assume a portion of the shear zone is liquified sand and a portion is remolded clay, the remolded clay can support an additional resistance of 20 kips per foot along the slide plane.

BUTTRESS DESIGN

The buttress design selected was based on a thorough analysis of all data and possible solutions. The basic criterion for the design was to provide a buttress to resist the forces of an equivalent earthquake. Based on the evaluation of the various stability analyses, the buttress was designed to resist 180 kips per ft. This force is resisted by a buttress using gravel having a 40°-angle of internal friction at the designed placement density of 140 lb pcf—98% of Modified Maximum Providence Vibrated Standard (Fig. 4). The support of the gravel buttress was obtained through the shear of its base.

LIMITATIONS

Limitations on future construction in the slide area have been recommended[16,31] to minimize the risk of damage to structures in the event of future earthquakes. All references to grades hereinafter not otherwise indicated are to the grades established in the Fourth Avenue Stabilization Design.[30]

It is recommended that the area south of the toe of the upper buttress 20% slopes and north of the north side of Fourth Avenue within the Urban Renewal Area be limited in accordance with the following:

1. Buildings should be limited to two-story structures of light construction such that the net increase in soil loading over the limits of the building area will not exceed an average of 500 psf. Above-grade structural floors should not exceed 125 psf live load.
2. Buildings should be limited to a height not exceeding two stories above the highest grade elevations adjacent to the structure.
3. The amount of fill should not exceed the quantity removed first by excavation within that area.
4. The maximum depth of excavation should not exceed 10 ft.

The following criteria are recommended for construction and site grading in the area bounded by the lower elevation of the upper 20% slope buttress and the upper elevation of the lower 20% slope buttress on the south and north sides respectively and by Barrow and E Street on the east and west sides respectively:

1. The maximum depth of cut and the maximum height of fill should not exceed 5 ft except for temporary excavations for placement of footings and footing subgrade. The temporary excavations should be backfilled and compacted immediately following completion of footing construction.
2. The average depth of cut and height of fill should not exceed three feet.
3. Buildings should be limited to two-story structures of light construction

[31] "Final Recommendations on Risk Classifications, Anchorage and Vicinity," Joint release of the U. S. Army Engineer District, Alaska, and Task Force 9, Alaska Reconstruction Commission, Sept. 8, 1964.

so that the net increase in soil loading within the building area will not exceed an average of 400 psf. Above-grade structural floors should not exceed 125 psf live load. No basement should be allowed in this area.
4. All excess excavation should be removed from the site.
5. Utilities may be constructed throughout the area, but in each case proper backfilling and compaction should be rigidly enforced.

The following criteria apply to the lower buttress slopes. They include the 20% slope located from 170 ft to 470 ft south of First Avenue between Barrow Street and D Street and extending north-west west of D Street, and the 20% slope bounded by First Street, Christensen Road, Second Street, and K Street.

1. No cut should be permitted on the slope or within the limits of the lower buttress construction except on a temporary basis to permit construction of footings or utilities as limited above. No basement is permitted in this area.
2. No fill should be permitted on the buttress slope unless the fill slopes are at a 20% or flatter slope. Construction of fills with 20% or flatter side slopes over and north of the toe of the buttress increases the effectiveness of the buttress and is both permissible and desirable.
3. Buildings should be limited to two-story structures of light construction such that the net increase in loading within the building area shall not exceed 400 psf. A two-story structure will be interpreted as two floors. Columns required to support the first floor shall not be considered in limiting the height of the structure.

Building heights in the area south of the G to K buttress should be restricted only by individual stability analysis.

The following additional criteria apply to all construction within the Urban Renewal Development Area:

1. There should be mandatory requirements for builders to furnish all structural and foundation seismic-design calculations for buildings over one-story in height and for any structure with a design live load in excess of 100 psf above the ground floor.
2. All seismic design calculations, drawings, and specifications should be made by registered professional structural and foundation engineers. This is probably the most important consideration given herein for the safety of any individual structure.
3. All construction should have qualified inspection to assure that it is within the intent of approved drawings and specifications.
4. There should be continuous review and updating of seismic-code provisions as found necessary for structures and foundations. If a higher-classification seismic zone is ever established, this area should be included.
5. Adequate horizontal and vertical reinforcement should be provided to resist stresses imposed on the building and foundation by the effect of soil movement on the structure.
6. Basement walls should be designed as retaining walls to withstand statically and dynamically induced earth forces.
7. Special consideration should be given to harmonic motion developed in buildings of extreme length and width during an earthquake.

The construction of the buttress fill and improvement of drainage will increase the vertical load on the slide-disturbed materials. This will result in erratic settlements of the regraded slope for a period of several years or more. In addition, a future earthquake of an intensity and duration comparable to that of March 27, 1964 may result in permanent lateral movement up to approximately 1 ft. Because of these anticipated lateral movements, pile or pier foundations should be considered unsuitable unless carefully tied together at the base and intermediate points by approved ties. Floor slabs should be structurally tied to the walls.

To provide further guidance, it has been recommended that the joint release of Alaska Task Force 9, Alaska Reconstruction Commission, and the U. S. Army Engineer District, Alaska, be quoted:

> "When the stabilization work is completed, all of that area will be returned to Nominal Risk. In the slide area below Fourth Avenue and between Barrow and E Streets, however, construction should be limited to parks, parking areas, and light-occupancy structures not over two stories in height. Even for such structures, certain restrictions must be imposed on depths of excavations or fills and on weights of buildings to prevent an unbalance of the buttress which could impair or destroy its effectiveness."
>
> "In all of the Fourth Avenue slide area between Barrow and I Streets, bounded on the south by a line running from Barrow Street to F Street midway between Fifth and Sixth Avenues and along Fourth Avenue between F Street and I Street, it is anticipated that normal consolidation of the underlying soils will result in some vertical and horizontal movement. Because this condition can be expected to result in localized differential movement, both horizontal and vertical, particular attention must be given to the design of structures and their foundations so that such movements may be accommodated without undue damage to the building."[31]

In other evaluations of the above paragraphs, the calculations should be submitted to the agency responsible for review and control of the seismic-code provisions. Ordinarily the city would undertake this function. The writers assume that the city intends to supplement the Building Code, 1964 edition, with the inclosed recommendations. Alternatively, the city could develop a completely new code. It is recommended that the city employ a well-recognized consulting firm if a new code is to be drafted.

SUMMARY AND CONCLUSIONS

A more accurate analysis of earthquake stability is difficult to obtain unless the basic knowledge of earthquakes and soil dynamics is broadened. Greater knowledge of (1) the nature and properties of seismic waves in soft, saturated and multi-layered soils prior to soil failure and after it, and (2) the complete static and dynamic properties of soil subject to seismic waves is required.

Further delineation and solution of such problems would permit savings in seismic design and a more rational evaluation of slope stability.

In conclusion, the writers have made the following observations:

1. Tests confirm the observed condition of liquefaction at the shallowest possible depth.
2. The most probable depth of failure was el. 45 msl.
3. A number of different theoretical approaches to stability analysis can be separately analyzed to arrive at a reasonable stability solution.
4. Stability against future failure from an equivalent earthquake can be provided by a buttress capable of resisting 180 kips per ft.
5. Limitations on future construction must be provided to guarantee the integrity of the buttress and to protect structures constructed within or directly adjacent to the area of failure.

ACKNOWLEDGMENTS

Among the many contributors to the investigations and conclusions, special acknowledgment is due R. B. Peck, L. Bjerrum, and T. Thompson as Consultants to the Corps of Engineers, and H. Bolton Seed as Consultant to the firm of Shannon & Wilson. Of the many who contributed to the reports and recommendations by Shannon & Wilson, special mention is due W. L. Shannon, S. D. Wilson, and Neal Twelker. Within the Corps of Engineers, special acknowledgment is due G. Bertram, OCE; M. Bubenik, NPD; and J. C. Ireton of the Alaska District. For assistance provided the Corps of Engineers by Metcalf and Eddy, special acknowledgment is due Don Ball.

ATTACHMENT 2.

Turnagain Heights Landslide

can be placed in random failure surface stability analysis which considers: (1) dynamic forces obtained from finite element analysis; (2) variations in soil shear strength; and (3) pore pressures.

In considering the analysis of embankment deformation due to earthquakes, the author has described the good progress being made in elastic response analysis of earth dams. Most of these analysis studies consider that the base is rigid and moves as a unit. However, shear wave lengths for major earthquakes are such that for very large dams the direction of base movement may be in different directions at the same time. It is hoped that future studies may lead to consideration of the effects of exciting forces acting in different directions on the base of the same structure.

Seismic stability analysis is, however, only a part of seismic resistant design of embankments. Important factors such as selection of construction materials, zoning, and provision of drainage must be fully considered. Such matters have been further described elsewhere for earth dams.[58]

[58] Sherard, J. C., "Earthquake Considerations in Earth Dam Design," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 93, No. SM4, Proc. Paper 5322, July, 1967, pp. 377-401.

THE TURNAGAIN HEIGHTS LANDSLIDE, ANCHORAGE, ALASKA[a]

By H. Bolton Seed,[1] M. ASCE, and Stanley D. Wilson,[2] F. ASCE

INTRODUCTION

During the Alaskan earthquake of March 27, 1964, a number of major slides occurred in the City of Anchorage.[3,4] 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. 1 and a plan of the slide area in Fig. 5.

The coastline in this area was marked by bluffs some 70 ft high, sloping at about 1-1/2:1 down to the bay. The slide extended about 8,500 ft from west to east along the bluff line, and retrogressed inward from the coast a distance of about 1,200 ft at the west end and about 600 ft 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. 2, and a similar view at the east end in Fig. 3. In the depressed areas between the ridges the ground dropped an average of about 35 ft during the sliding.

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 2,000 ft (see Fig. 5), and in general the outward movement beyond

[a] This paper was part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 93, No. SM4, July, 1967. Manuscript was submitted for review for possible publication on January 30, 1967.

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[2] Vice-Pres., Shannon and Wilson, Inc., Seattle, Wash.

[3] Shannon and Wilson, Inc., Report on Anchorage Area Soil Studies, Alaska, to U. S. Army Engineer District, Anchorage, Alaska, Seattle, Wash., 1964.

[4] Hansen, W. R., "Effects of the Earthquake of March 27, 1964 at Anchorage, Alaska," Geological Survey Professional Paper 542-A, U. S. Dept. of the Interior, Washington, D. C., 1965.

the original bluff line tended to mirror the extent of inland regression of the slide behind the bluff line.

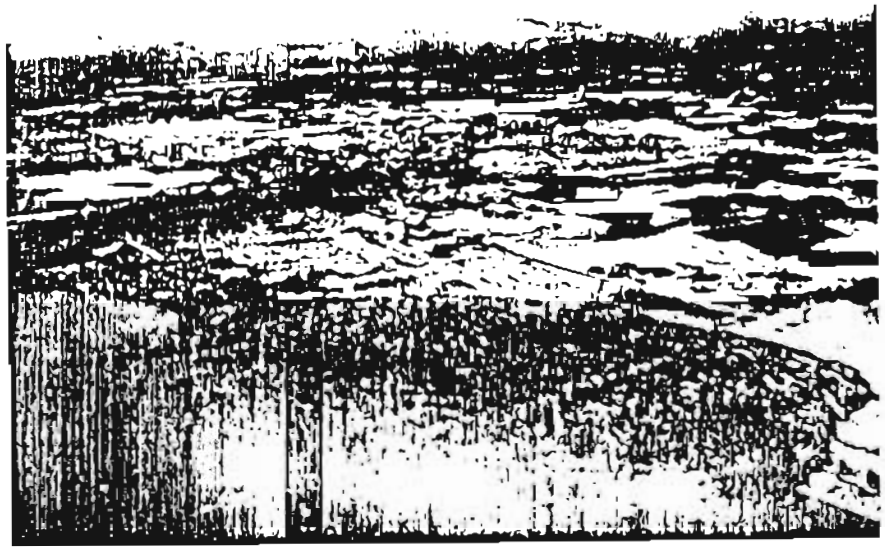

FIG. 1.—AERIAL VIEW OF TURNAGAIN HEIGHTS LANDSLIDE

FIG. 2.—CONDITIONS IN CENTRAL PART OF SLIDE AREA

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. 3.

A study conducted by the Engineering Geology Evaluation Group in Anchorage [5] reveals the extent of movements of some of the houses in the slide area. Fig. 6, 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 ft or 600 ft, usually, but not always, towards the original coastline. The transverse movement of

FIG. 3.—CONDITIONS IN EAST END OF SLIDE AREA

FIG. 4.—HOUSES DESTROYED BY SLIDE

some of these houses is indicative of the complex mechanism involved in the development of the slide.

Eyewitness accounts indicate that the sliding began about 2 min 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

[5] Engineering Geology Evaluation Group, Geologic Report: 27 March 1964 Earthquake in Greater Anchorage Area: Alaska State Housing Authority and the City of Anchorage, Anchorage, Alaska, 1964.

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 land in the direction of sliding towards the coastline. Cracking was more intensive behind the east end of the slide and surveys in this area showed that the accumulated extension in a 2,000 ft zone extending behind

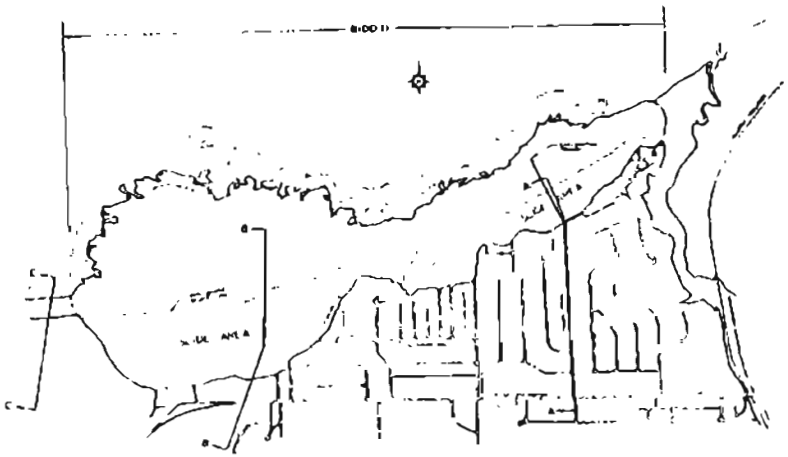

FIG. 5.—PLAN OF SLIDE AREA

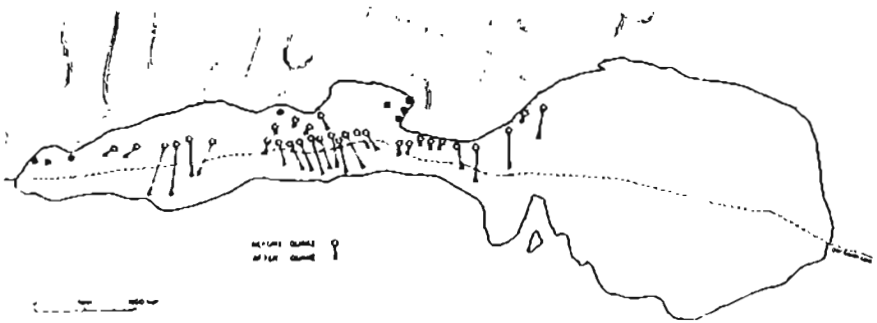

FIG. 6.—MOVEMENTS OF HOUSES IN SLIDE AREA

the slide area amounted to about 3 ft. 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 United States 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. Because 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 min. On this basis the duration of strong motions appears to have been at least 4 min.

Eyewitness 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

The Anchorage area,[6] located within a well-known earthquake region, occupies 150 sq 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 underlaid by Bootlegger Cove clay. Rocks of Mesozoic and Tertiary age are overlaid by unconsolidated deposits of glacial drift, mantled with wind-deposited silt. The lowlands range from sea level to 1,200 ft with an average elevation less than 200 ft above mean sea level. The highest point in the nearby Chugach Range is at El. 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 nonglacial materials, resulting in poorly defined boundaries. Nonglacial 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 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, relatively impermeable, and easily identified. The deposit is more or less continuous but is concealed to the north by an end moraine and pinches out elsewhere. It ranges from zero to more than 300 ft thick, averaging from 100 ft to 150 ft. Silty and

[6] Miller, R. D., and Dobrovolny, Ernest, "Surficial Geology of Anchorage and Vicinity, Alaska," Bulletin 1093, U. S. Geological Survey, Washington, D. C., 1959.

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 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[3] 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. 7 and 8. Figs. 7(*a*) and 7(*b*) show soil profiles along sections through the east and west ends of the slide area (sections AA and BB in Fig. 5). Fig. 8(*a*) shows a soil profile through the area just west of the slide area where no sliding occurred (section CC in Fig. 5). Fig. 8(*b*) shows a profile along an east-west section through the slide area.

In general, the Turnagain area (Surface Elevation ~ 70) is covered by a surface layer of sand and gravel which varies in thickness from 15 ft to 20 ft at the east end of the slide area to about 5 ft to 10 ft at the west end. The sand and gravel is underlaid by a deep bed of Bootlegger Cove clay, about 100 ft 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

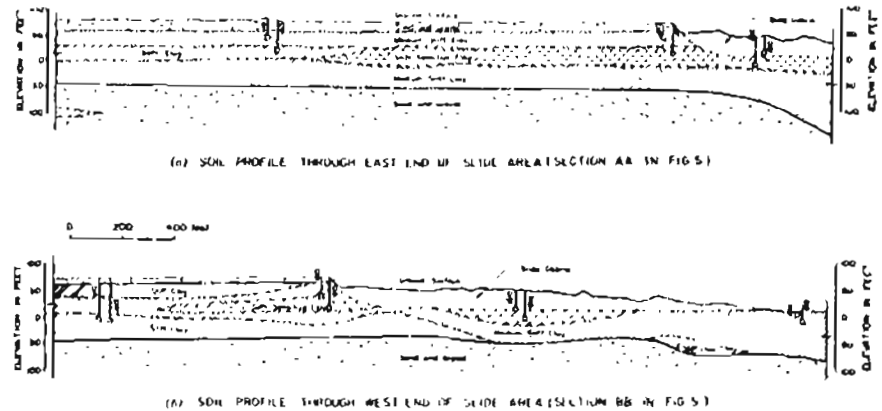

FIG. 7.—SOIL PROFILES THROUGH SLIDE AREA

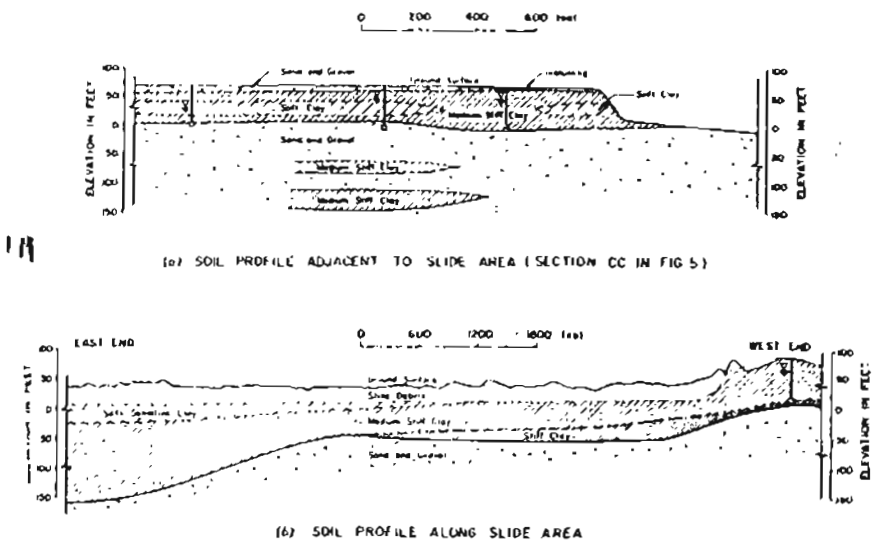

FIG. 8.—SOIL PROFILES ALONG SLIDE AREA AND ADJACENT TO IT

strength of the remolded soil is on 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 coastline (see profiles AA and BB in Fig. 7), and the extremely sensitive clay was not

found in borings made about 2,000 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 El. 30 and El. 37, a sand layer several ft thick were encountered near El. 20, and a number of thinner lenses were found below this, their thickness and frequency diminishing with depth; below El. 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 El. +20, but below this they were only occasionally encountered.

Seaward of the bluff line that existed prior to the slide [see Figs. 8 (*a*) and 11] the clay was overlaid 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. 8(*a*). It may be noted that the extent of the zone of 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 El. 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 despite the generally similar topographic and geologic conditions in this area.

STRENGTH OF SOILS UNDER CYCLIC LOADING CONDITIONS

Laboratory studies[7,8] 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. 9. Some of these tests were performed using the simple-shear equipment developed by the Norwegian Geotechnical Institute,[9] modified to permit cyclic stress or cyclic strain tests. Other tests were per-

[7] Seed, H. Bolton, and Lee, K. L., "Pulsating Load Tests on Samples of Fine Silty Sand From Anchorage, Alaska," Report on Anchorage Area Soil Studies, Alaska, to U. S. Army Engineer District, Anchorage, Alaska, Shannon and Wilson, Inc., Seattle, Wash., 1964, Appendix C.

[8] Seed, H. Bolton, and Chan, C. K., "Pulsating Load Tests on Samples of Clay and Silt from Anchorage, Alaska," Report on Anchorage Area Soil Studies, Alaska, to U. S. Army Engineer District, Anchorage, Alaska, Shannon and Wilson, Inc., Seattle, Wash., 1964, Appendix D.

[9] Bjerrum, L., and Landva, A., "Direct Simple Shear Tests on a Norwegian Quick Clay," Geotechnique, Vol. XVI, No. 1, Mar., 1966.

formed using cyclic loading triaxial compression procedures.[10] 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% 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.

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

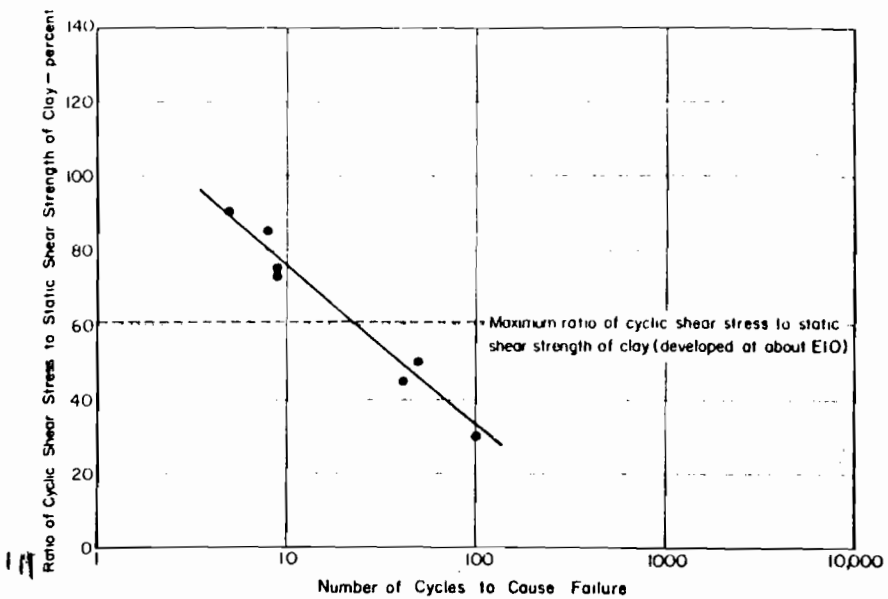

FIG. 9.—STRENGTH OF SAMPLES OF SILTY CLAY UNDER CYCLIC LOADING CONDITIONS

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[11] 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.[12] It is also necessary to apply a correction to the data to convert the results to those

[10] Seed, H. Bolton, and Chan, C. K., "Clay Strength Under Earthquake Loading Conditions," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 92, No. SM2, Proc. Paper 4723, Mar., 1966, pp. 53-78.

[11] Seed, H. Bolton, and Lee, K. L., "Liquefaction of Sands During Cyclic Loading," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 92, No. SM6, Proc. Paper 4972, Nov., 1966, pp. 105-134.

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

corresponding to simple shear conditions. For samples of similar sand at about El. 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. 10. It will be seen that liquefaction of the sand seams may be expected to occur in 30 cycles at a 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

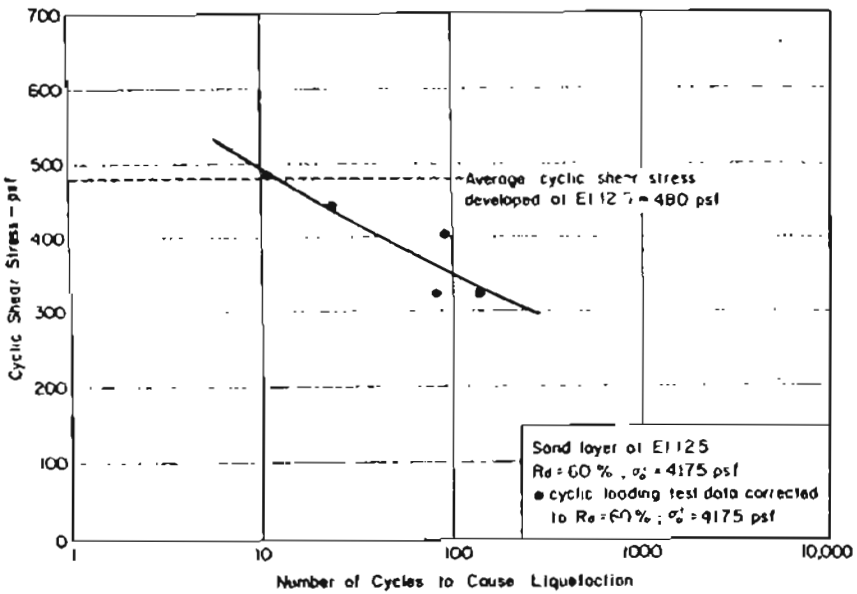

FIG. 10.—STRESSES INDUCING LIQUEFACTION OF SAND UNDER CYCLIC LOADING CONDITIONS

the zone behind the bluffs, liquefaction might be expected in the sand before failure would develop in the clay.

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:

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. 11.

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.[3]

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. 11. 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 low shear strength agree well with the locations of 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 ft 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.

Trench Section.—To throw further light on the distribution of materials in the slide area, a 1,000 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 that 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. 12. 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.

Survey of Clay Ridges.—The clay ridges, illustrated in Fig. 2, were a characteristic feature of the slide area. Hansen[4] has described them as follows:

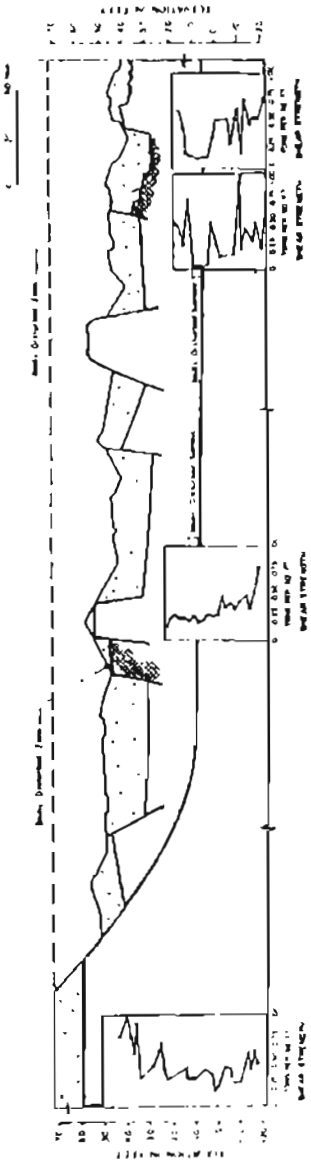

FIG. 11.—DETAILED SOIL PROFILE THROUGH EAST END OF SLIDE AREA

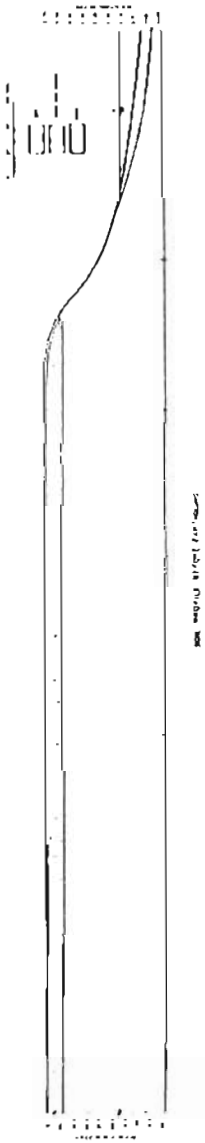

FIG. 12.—SOIL PROFILE IN TRENCH SECTION THROUGH EAST END OF SLIDE AREA

"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.

A view of a particulary well-formed ridge at the east end of the slide area is shown in Fig. 13(*a*) 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. 13(*b*).

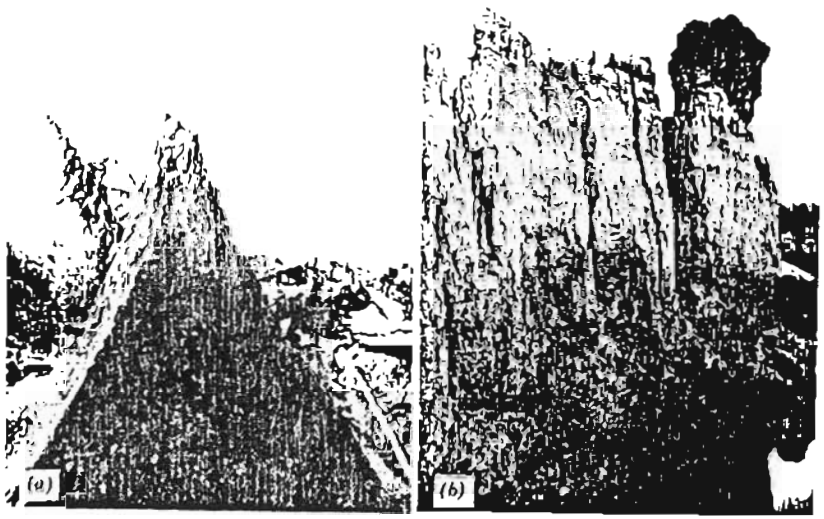

FIG. 13.—RIDGES OF CLAY IN SLIDE AREA

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% downwards toward the original toe of the bluff, as shown in Fig. 12.

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. 14), 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% 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 El. +20. However, the lower surface of sliding was almost certainly deeper at the east end of the slide area.

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 in. had occurred. Beyond this distance settlements were much smaller in magnitude. Such 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. 11. 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 dis-

FIG. 14.—ZONE OF SLIDE AREA COMPLETELY FREE OF DEBRIS

turbed 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. 11, 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. 11.

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% behind the original bluff line, steepening to about 10% or 15% in this location.

Stability Before Earthquake and Sliding Potential.—Analyses of the stability of the bluffs along the coastline 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[13] 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 accelerations induced by:[14] (1) An earthquake of magnitude 7.3 at an epicentral distance of about 35 miles (1943); (2) an earthquake of magnitude 6.3 at an epicentral distance of about 50 miles (1951); (3) 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 (4) the first 1.5 min 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. However it is extremely doubtful that any analyses would have anticipated the extent of inland regression of the slide (1,200 ft 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.

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 sec of ground shaking. In the case of the Turnagain slide, numerous eyewitnesses report no evidence of an impending slope failure during the first 1.5 min 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,

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

[14] Preliminary Report: Prince William Sound, Alaskan Earthquakes, March-April 1964, U. S. Coast and Geodetic Survey, Washington, D. C., Apr., 1964.

but during a period of smaller inertia forces midway 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.

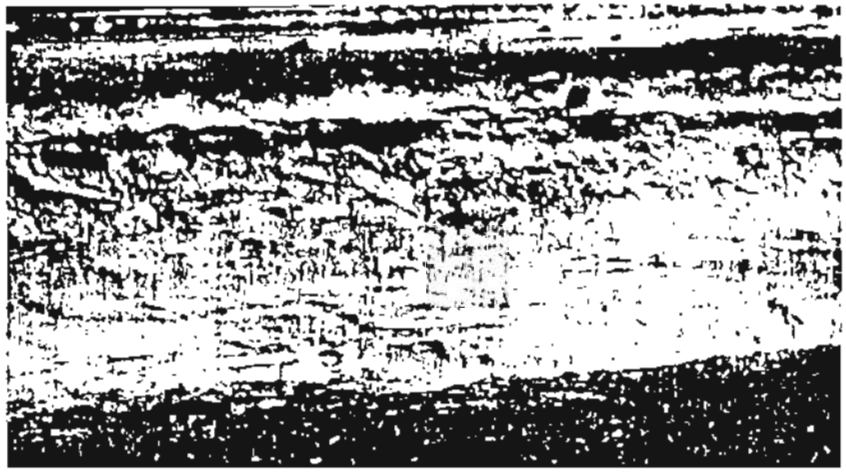

FIG. 15.—OFFSHORE SILT DEPOSIT

Reasons for Inland Regression of Slide Zone.—The large extent of inland regression of the slide zone may be attributed to several major factors:

Presence of Shoreline Silt Deposits.—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. 15, which was taken at an adjacent area of the coastline. 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.

Loss of Strength in Soil Behind Bluff Line.—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 coastline, indicating movement of level ground towards the slide area. This could only occur by movement along an extremely weak layer, and indicates that the severly weakened zone extended beyond the boundaries of the slide area, at least at the east end of the slide.

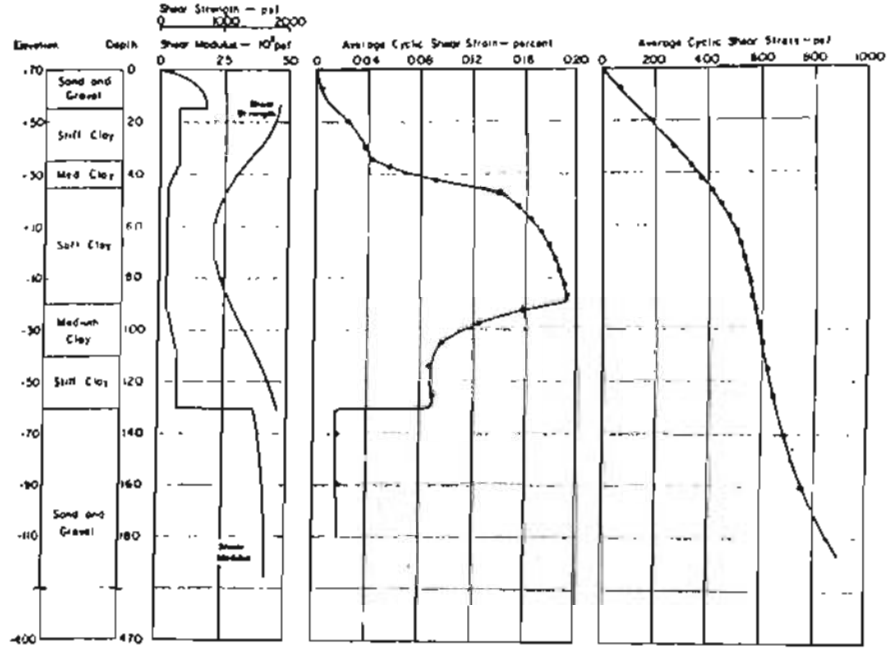

FIG. 16.—RESPONSE OF SOIL DEPOSIT TO HORIZONTAL BASE MOTIONS

2. Settlement of up to 8 in. in the zone just behind the slide area, probably resulting from 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 a loss of strength in the clay deposits caused by the cyclic shear stresses and strains induced in them by the earthquake, and liquefaction of sand lenses and seams, within the clay deposit, caused by the cyclic loading induced by the earthquake. The presence of a weak zone as the result of either of these causes would lead

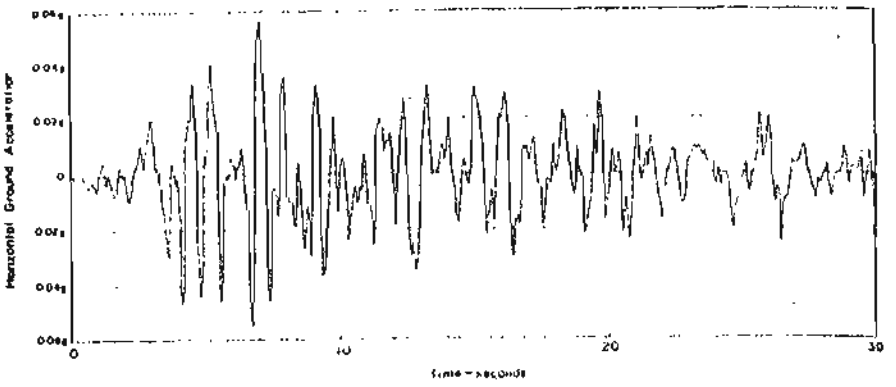

FIG. 17.—HORIZONTAL GROUND ACCELERATIONS AT PASADENA DUE TO KERN COUNTY, CALIFORNIA, EARTHQUAKE OF 1952

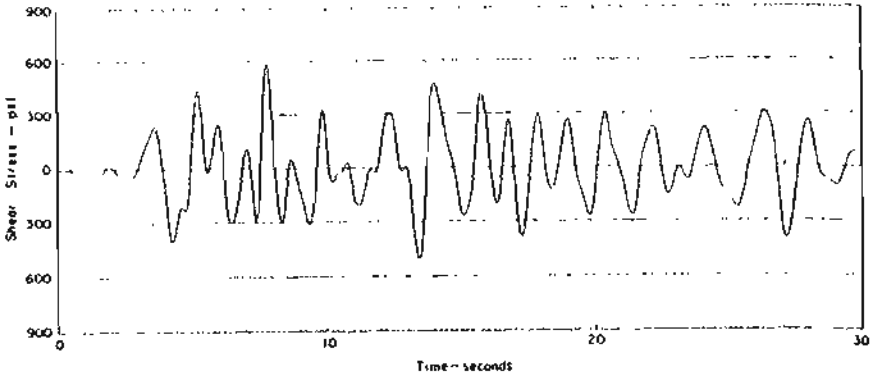

FIG. 18.—SHEAR STRESS VARIATION NEAR TOP OF SOFT CLAY LAYER (EL. 22.5)

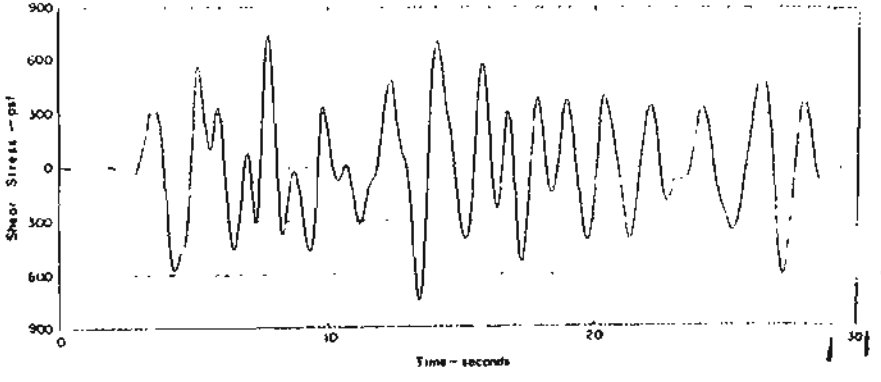

FIG. 19.—SHEAR STRESS VARIATION NEAR BOTTOM OF SOFT CLAY LAYER (EL. -17.5)

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 min 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.[15]

The soil conditions used in the analysis are shown in Fig. 16. 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[3] 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% 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.[16] 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. 17. However, because 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 sec, it was considered appropriate to increase the amplitude of the motions by a scaling factor of 1.5 and to simulate the first 2 min of the longer duration of the Alaskan earthquake by repeating the Pasadena record three times in sequence. Thus the record shown in Fig. 17, with the ordinates multiplied by 1.5, was considered to represent 30 sec to 40 sec of base rock shaking in the Anchorage area during the first 2 min of the Alaskan earthquake.

The computed response of the soil deposits to this base motion are presented in Table 1 and Fig. 16. The maximum ground surface acceleration was determined to be 0.11 g and the fundamental period of the deposit was 2.01 sec. 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 Figs. 18 and 19. The stresses near the bottom of the layer have the same form as those near the top of the layer, but they are significantly greater 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

[15] Seed, H. Bolton, and Idriss, I. M., "Analysis of Soil Liquefaction: Niigata Earthquake," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 93, No. SM3, Proc. Paper 5233, May, 1967, pp. 83-108.

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 sec and occur at a rate of 10 cycles per 30 to 40 sec 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. 16. It is readily apparent that the maximum shear strains occur in the soft clay layer, with the magnitude increasing progressively from about 0.12% to about 0.19% with increasing depth in this layer. Because of the varying shear moduli, the average cyclic shear stress

TABLE 1.—RESPONSE ANALYSIS OF SOIL DEPOSITS

Elevation	Depth, in feet	Material	Shear Strength, s_u, in pounds per square foot	Shear Modulus $\times 10^6$, in pounds per square foot	Average Cyclic Strain, as a percentage	Average Cyclic Stress, τ_c, in pounds per square foot	Effective Overburden Pressure, σ_o', in pounds per square foot	Ratio τ_c/σ_o'	Ratio τ_c/s_u
+ 62.5	7.5	Sand and gravel	—	15.1	0.0045	69			
+ 50	20.0	Stiff clay	1,800	7.2	0.0255	184	2,100	0.87	
+ 40	30.0	Stiff clay	1,600	7.2	0.0376	271	2,700	1.00	
+ 32.5	37.5	Medium clay	1,300	5.9	0.057	335	3,140	1.07	
+ 27.5	42.5	Medium clay	1,100	4.1	0.091	375	3,415	1.10	
+ 22.5	47.5	Soft clay	1,000	2.9	0.142	415	3,675	1.12	0.41
+ 17.5	52.5	Soft clay	900	2.9	0.155	450	3,925	1.14	0.50
+ 12.5	57.5	Soft clay	870	2.9	0.165	480	4,175	1.15	0.55
+ 7.5	62.5	Soft clay	850	2.9	0.173	501	4,425	1.13	0.59
+ 2.5	67.5	Soft clay	850	2.9	0.179	519	4,675	1.11	0.61
− 2.5	72.5	Soft clay	850	2.9	0.183	530	4,925	1.08	0.61
− 7.5	77.5	Soft clay	890	2.9	0.186	540	5,175	1.04	0.60
− 12.5	82.5	Soft clay	950	2.9	0.191	555	5,425	1.02	0.58
− 17.5	87.5	Soft clay	1,000	2.9	0.193	560	5,675	0.99	0.56
− 22.5	92.5	Medium clay	1,150	3.5	0.159	572			
− 27.5	97.5	Medium clay	1,250	4.7	0.125	586			
− 35	105.0	Medium clay	1,400	6.3	0.096	605			
− 45	115.0	Stiff clay	1,600	7.2	0.087	625			
− 55	125.0	Stiff clay	1,800	7.2	0.090	646			
− 70	140.0	Sand and gravel	—	38.1	0.018	690			
−135	205.0	Sand and gravel	—	42.9	0.022	930			
−275	345.0	Sand and gravel	—	50.2	0.032	1,620			

increases progressively with increasing depth in the soil deposit as indicated in Fig. 16.

The values of cyclic shear stress shown in Fig. 16 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. Because 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[15] and Seed and Lee.[12] 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. 16.

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. 10, 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 sec of ground shaking. Because 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 sec 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, 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. 9, 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 min of ground shaking.

On the basis of this analysis it would appear that liquefaction of the sand between El. 0 and El. 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 sec 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 El. 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 the following facts:

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

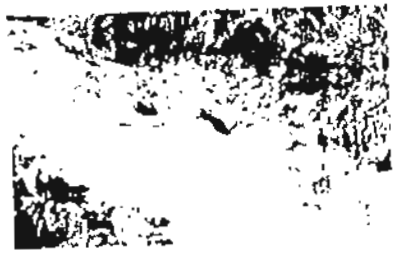

FIG. 20.—RIDGE OF SAND FORMED BY SAND BOIL IN SLIDE AREA

FIG. 21.—FORMATION OF CLAY RIDGE DURING MODEL TEST

2. Ridges of sand, 2 ft to 3 ft high, 3 ft to 6 ft wide, and about 100 ft long, were formed by sand boils within the slide area[4] as shown in Fig. 20. (A large boil, spread over an area of 3,200 sq ft, was also noted near but outside the slide area by the Engineering Geology Evaluation Group.[5])

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 underlaid 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 immediately adjacent to the slide area.[16]

[16] Internal Report on Turnagain Earthquake Slide Studies, U. S. Army Engineer District, Anchorage, Alaska, Apr., 1966.

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.

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 as the result of remolding in the shear zones, thereby facilitating lateral displacement. Furthermore, the extremely long duration of the earthquake (more than 4 min) 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 if 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 through 4) immediately reveals that the sliding could not have developed simply as a result of a progressive series of conventional slides of clay along circular arcs directed towards the coastline. Trees and poles could be noted inclined in opposite directions—a pattern not likely to develop by repeated slides in the same direction. 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 underlaid by a layer of extremely weak material, a series of model tests was conducted at the University of California. A bank of clay, about 4 in. high, composed of an extremely weak layer overlaid 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 sec 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. 22:

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. 22(*a*) to 22 (*d*)].

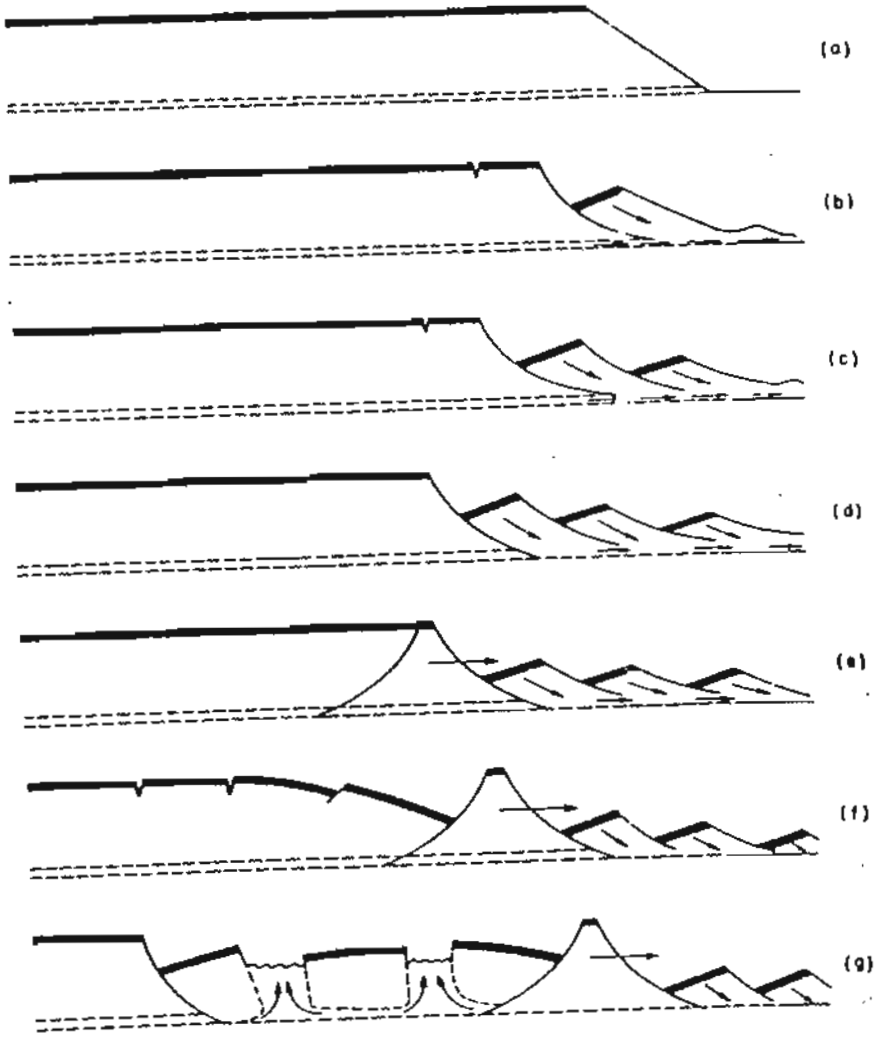

FIG. 22.—FAILURE MECHANISM OBSERVED IN MODEL TESTS

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. 22 (*d*)]. Beyond this point continued sliding results in the outward movement of a prismatic ridge of soil, essentially without change in elevation [Fig. 22 (*e*)].

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. 22(*f*)].

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. 21, which was taken during a model test, and by Fig. 22(*f*).

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. 22(*g*)].

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 because the base is slightly inclined, or as a result of 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, first, that 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 that 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. Secondly, the subsidence of material to form a graben 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 because of 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 min 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 10

20 at the east end of the slide area and El. 15 to El. 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. 22, 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 extremely 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 offshore silt deposit.

6. Sliding continued as long as the earthquake continued but movements were rapidly arrested once the ground motions stopped, partly because the offshore 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. 23. 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. 23. 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 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

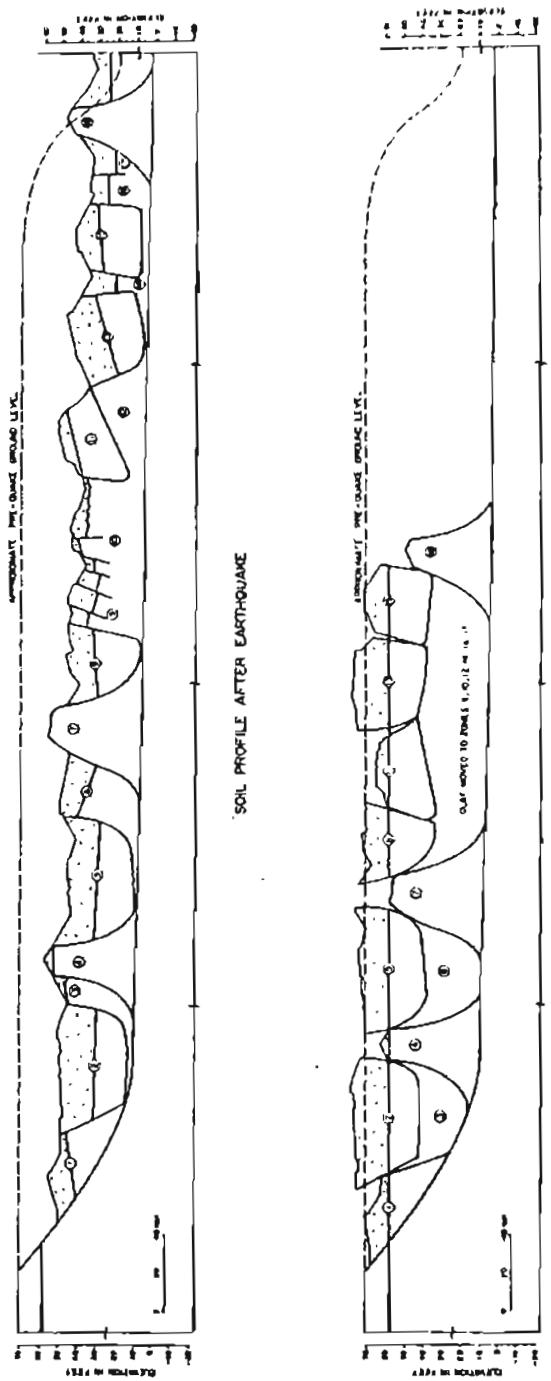

FIG. 23.—RECONSTRUCTION OF SOIL DISPLACEMENT AT EAST END OF SLIDE ZONE

to be badly disturbed during construction of the trench (see Fig. 12).

Proceeding in this way leads to the full reconstructed section shown in Fig. 23. 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 ft). These displacements are in good agreement with the observed movements of houses in the slide area shown in Fig. 6. The slide reconstruction also indicates that failure involved the displacement of large zones of clay in the depth range from El. +5 to El. +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 ft to 20 ft apart but with one passing through block No. 2 in Fig. 23 and the other passing through section No. 3, might show radically different strength variations in the clay.

CONCLUSIONS

The results of a study of the Turnagain Heights Landslide, which occurred during the Alaska Earthquake of March 27, 1964, have been presented herein. 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 analyze the behavior of such deposits even under static loading, and still more difficult under earthquake loading conditions.

2. Because 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 min 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 caused by 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 offshore 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 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. However, 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 min), the slide would probably not have developed and the area could have been cited as a classic case illustrating the safety of slopes underlaid by clay soils during even major earthquakes. In the light of such possibilities, experiences of slope failures or nonfailures during earthquakes require careful appraisal before they can be translated to other areas and conditions.

ACKNOWLEDGMENTS

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The writers extend their appreciation to the many engineers who aided in the investigations. Special acknowledgment is due to Warren George, Erwin Long, and J. Ireton of the U. S. Army Corps of Engineers; W. L. Shannon of Shannon and Wilson, Inc.; J. K. Mitchell, R. J. Woodward, Jr., H. Peacock, and I. M. Idriss, members of the staff of the University of California Soil Mechanics and Bituminous Materials Research Laboratory; L. Bjerrum, R. B. Peck, and T. Thomson, consultants to the U. S. Army Corps of Engineers; G. W. Housner, who provided the earthquake record in Fig. 17; and the United States Geologic Survey, which provided the photograph in Fig. 20.

ATTACHMENT 3.

Fire Island Stratigraphy

I N Q U A

International Union for Quaternary Research

XI Congress

Moscow, August 1982

ABSTRACTS

Volume I

Moscow 1982

DIAMICTON OF SUBGLACIAL OR SUBAQUEOUS ORIGIN, FIRE ISLAND, ANCHORAGE, ALASKA

Henry R. Schmoll and Cynthia A. Gardner (U.S.A.)

Fire Island, approximately 17 km^2 in area, lies 5 km off the west shore of Anchorage in an isolated and seemingly anomalous position at the upper end of the Cook Inlet basin in south central Alaska. The irregular topography, consisting of hills of poorly sorted sandy gravel that rise 10 to 30 m above adjacent abandoned channels, has led to the island being mapped as a moraine. However, linking such a moraine with unequivocal late Pleistocene lateral moraines along the margins of the basin has proved geologically awkward. This problem suggests that either a different depositional environment was involved or that the island has had a more complex history than previously thought.

A nearly continuous exposure along sea bluffs on the east side of the island, about 8 km long and as high as 60 m, reveals diamicton generally overlying a complex of bedded, interfingering sand and gravel beds that locally includes minor lenses of silt and finer grained sand. This stratigraphy is not incompatible with the concept of the island as a moraine formed when glacier ice advanced into the center of upper Cook Inlet and overrode outwash previously deposited there. However, detailed inspection of the stratigraphy, in particular the texture of the diamicton and its relation to the other units, suggests other interpretations. The diamicton is very irregular in thickness, in places thickening downwards from 5 to 30 m and then thinning again within a distance of 100 m. Marginal to these thicker masses of diamicton, the sand and gravel beds are severely contorted, implying rapid emplacement of the diamicton mass by flow or by slumping. Such deformation indicates that the sediments were soft (non-frozen) and probably saturated during the emplacement. In many places the diamicton is intimately interbedded with and grades laterally into the sand and gravel, and thus the diamicton in part occurs as beds at different stratigraphic horizons. Such local diamicton beds appear unlikely to result from separate glacial events. Texturally the diamicton is bimodal, consisting of pebbles and cobbles in a matrix of fine sand, and it conceivably could have originated from a mixing of the other depositional units. These details of the exposed stratigraphy suggest that much if not all of the diamicton could have been deposited subaqueously as part of a deltaic sequence, possibly near one or more lobes of glacier ice that fronted in inlet waters.

ATTACHMENT 4.

Sand Lake Gravel Quarries

Sand Lake pits pose dilemma to builders, city planners

By STAN JONES
Daily News business reporter

Anchorage Daily News
Saturday, December 11, 1982

Mining gravel is a little like eating olives in polite company: when you're done, what do you do with the pits?

That's exactly the question facing municipal planners, gravel companies and residents in that part of southwest Anchorage known as the Sand Lake area.

For decades, a group of gravel pits stretching north and west from the intersection of Dimond Boulevard and Sand Lake Road have provided the foundations on which Anchorage's buildings have stood.

But now the Sand Lake pits are just about mined out. Although some useable sand and gravel are left, there's a growing body of sentiment among the planners and some of the pit operators that it's time to stop digging and start filling the holes back in so that homes can be built, lawns can sprout, and children can romp where the dump trucks used to rumble.

Closing down the pits would also ease the truck traffic on Sand Lake Road and Dimond Boulevard and the attendant clouds of dust that have plagued the area area since the first homes were built there.

The Sand Lake pits currently operate on a variety of grandfather-rights and conditional use permits scheduled to expire at the end of this month. But because no one has decided exactly what should be done with the pits if they are closed, it's uncertain if the deadline will hold.

Already one operator has received a three-month extension from the municipality as he prepares a comprehensive five-year plan for continuing to operate while he restores his pit, and others have contacted the municipality about obtaining similar stays of execution.

The planners' decision is complicated by the fact that Anchorage is facing a general gravel shortage, meaning that a shutdown in Sand Lake could have a major effect on construction costs.

A draft study on the subject prepared for the municipality by JohnsonBraund Design Group-S&S Engineers estimates the shortfall between now and the end of the century may total 12.4 million tons.

Because there are few other major sources of sand or gravel left in the Anchorage bowl, the Sand Lake supply would have to be replaced from sources much farther from town — probably at least as far as the Eagle River-Birchwood area, and possibly as far the Matanuska Valley.

While there are other potential sites — notably in Cook Inlet just off of Point Woronzof, near Eklutna, and near Girdwood — they have yet to be developed, and it's not known if they would yield cheaper gravel than existing out-of-town sites.

At first glance, importing the replacement gravel looks prohibitively expensive. The cheapest material from Sand Lake — a low-quality sand suitable mainly for parking lots and for filling in residential lots after the overburden of peat has been scraped away — sells for about $1.25 per ton at the pit, according to Cliff Tweedy of Earthwork Excavation and Paving Company. But material from the Matanuska Valley, known in the business as "Palmer gold", is hauled into Anchorage by train, and sells at the gravel terminals for prices starting at about $3.90 per ton, Tweedy said.

It doesn't take a computer to figure out that prices could apparently triple if the Matanuska Valley takes up where Sand Lake leaves off.

But a simple comparison of purchase prices leaves out a major, perhaps the major, component of the cost of getting gravel to a construction site: the cost of running a dump truck.

That cost adds about $5 per ton to the expense of getting sand or gravel where it's needed, whether the gravel is from Sand Lake or a gravel dump next to the railroad. That means Sand Lake sand costs about $6.25 per ton, delivered, ver-

sus about $8.90 for Palmer gold. The difference in delivered cost is thus closer to 40 percent than threefold.

Tweedy estimates the typical $100,000 home being built in Anchorage these days includes $5,000 to $8,000 worth of fill material. So a gravel price increase of 40 percent could add $2,000 to $3200 to the cost of the home.

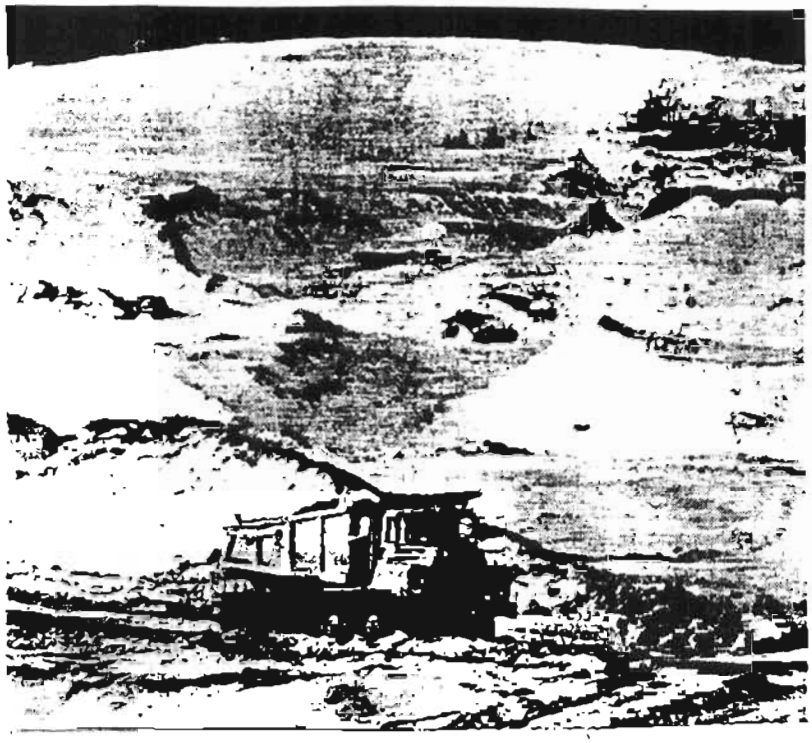
"I know if we can't get it here in town, we'll have to go to Palmer," says Glenn Roberts, a builder who frequently relies on Tweedy for his gravel needs. "There will definitely be an increase."

Because the expense of truck transportation is so critical in determining the final cost of foundation materials, gravel pits tend to serve only construction projects relatively nearby. The Sand Lake pits, for example, tend to supply builders in southwest Anchorage. The Hillside area and South Anchorage generally are supplied either from pits in those areas or from Palmer gold hauled in by railroad.

The northeast quadrant of Anchorage tends to be supplied by truck from pits in the Eagle River-Birchwood area. Although Sand Lake is closer in miles, traffic congestion in Anchorage's core area make Eagle River closer in time.

"I did a lot of talking with contractors this summer, and they told me it's viable to bring in material from Eklutna as far south as the corner of Dimond Boulevard and the Old Seward Highway," says Lori Kincaid, a member of the municipal plannng and zoning commission. "The only place it's cheaper to get it directly from Sand Lake is the Sand Lake area, from the information I've been given."

For municipal planners, who have the power to close the pits, the question is one of a tradeoff. Is it more important to have the gravel the Sand Lake pits supply, or the prime developable land that would be made available if they closed?



If the Sand Lake gravel pits are closed, the price of fill could rise by as much as 40 percent. Anchorage Daily News/Jim Lavrakas

Some think that time may be running out — that if the pits operate much longer, it will be impossible to salvage the land for residential, office, or industrial construction. That's because redeveloping them will require that they be filled back in and if they get much deeper the cost of the fill could become prohibitive.

The fill likely to be used for such projects, ironically, is the peat that has been dug out of other areas and replaced by Sand Lake gravel and sand.

Some of the gravel operators are ready to close their pits now and start redeveloping. But one gravel industry source — who asked not to be named — said competitive pressures would make it impossible for an operator to close on his own, because it would leave him without a supply of cheap Sand Lake fill that his competitors would still have available.

In addition, he said, as long as one of the pits in the area still operates, the others will be difficult or impossible to redevelop because buyers will be reluctant to move in next to an active gravel pit.

Only the municipality can save the situation, the source suggested, by closing all the pits at once.

But Kincaid thinks it would be possible to redevelop the pits in a series of steps, beginning at the southeast corner of the area and moving northwest.

"The idea that people who want to start development now cannot build during extraction is not necessarily true," she says. "If it's phased properly, it can be done."