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Alaska Railroad Terminal Reserve, Anchorage Soil stability study

Stability in the vicinity of Boring Lines 1 and 2

By

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Open-file report

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This report is preliminary and has not been edited or reviewed for conformity with U. S. Geological Survey standards and nomenclature.

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ALASKA RAILROAD TERMINAL RESERVE, ANCHORAGE
SOIL STABILITY STUDY

STABILITY IN THE VICINITY OF BORING LINES 1 AND 2

By David J. Varnes

I. Introduction

This report has been prepared in response to a request dated April 22, 1966, from the General Manager of The Alaska Railroad to the Director, U.S. Geological Survey, for an evaluation of the propriety of continued industrial expansion on land contained within The Alaska Railroad Terminal Reserve and nearby. It is based on field examination June 8-12, 1966, in company with Messrs. E. B. Eckel and E. G. Dobrovolny, field work September 19-October 3, 1966, and May 1-June 4, 1967, on discussions with the staff of The Alaska Railroad, my colleagues, and Professor H. B. Seed of the University of California, and on review of published and unpublished material pertinent to the area and its problems.

During the fall of 1966 a detailed topographic map of the port area was prepared for The Alaska Railroad by Jay Whiteford and Associates. During late 1966 and early 1967 a drilling, sampling and soils testing program in an area of immediate interest along Boring Lines 1 and 2 was made by Adams, Corthell, Lee, Wince, and Associates (ACLW), a consultant engineering firm, under contract to The Alaska Railroad. Most of the basic data used in this report for stability analyses, such as the geometry of the ground surface and the physical properties of the materials, was derived from the Whiteford map and the ACLW investigations.

II. Topography, geology, and development of the port area

The port area as of August 11, 1963, is shown in figures 1 and 2. It consists of a strip of flat ground, from sea level to about 20 feet above high tide, 1-1/2 miles long and as much as 1/2 mile wide, fronting on Knik Arm northward from Ship Creek, and enclosed on the north, east, and southeast by wooded bluffs about 100 feet high.

On the topographic map, figure 3 (in pocket), is shown a part of the boundary of the Alaska Railroad Terminal Reserve, which includes the southern portion of the port area.

The flats in front of the bluff are underlain by estuarine silt, peat, muskeg, and artificial fill. The bluffs are formed of Bootlegger Cove Clay, an estuarine-marine deposit consisting largely of Pleistocene silty clay interbedded with thin lenses of sand, together with an overlying deposit of about 40 feet of glacial outwash composed of sand and gravel.

Bluffs such as these, composed of as much as 50 feet of silt and clay together with a similar thickness of overlying sand and gravel, and exposed to erosive action of strong tidal currents in a region of active freezing and thawing, are almost sure to be progressively modified by slumping of exposed faces. Landsliding has long been the active agent of cliff retreat wherever fine-grained glacial deposits border Knik Arm, and the resulting landforms and deposits are easily seen and interpreted where not obscured by brush and trees or by manmade modifications.

The surficial geology of the Anchorage area has been described by Miller and Dobrovolny (1959), and the relation of the geology to effects of the March 27, 1964, earthquake by Hansen (1965) and by others. The physical properties and seismic response of the Bootlegger Cove Clay and other surficial deposits have been the subject of many studies made following the earthquake and extensive landsliding in 1964. Pertinent to this particular area are the investigations by Shannon and Wilson (1964).

All of the slopes forming the north side of the valley of Ship Creek and the west side of Government Hill, from Loop Road around to the military P.O.L. storage tanks and beyond, appear to have slid at some time in the past (fig. 21). Many of the old scarps are still visible, but construction of roads and houses, and regrading accompanying industrial development have obscured much of the original form of the slopes and of the flats in front of them. It seemed useful, therefore, to attempt to determine how the Terminal Reserve and other parts of the port area looked prior to development and to trace the changes that have occurred.

Figure 4 (in pocket) shows part of a topographic map prepared by the Alaskan Engineering Commission in 1914 superposed on the 1966 topographic map. The irregular hummocky topography produced by old slides is well shown on the earlier map, particularly the flow-type slide where is now the Standard Oil of California Terminal Yard and the smaller slide in the area crossed by Boring Line 1.

Figures 5-10 are reproductions of photographs taken by the Alaskan Engineering Commission of parts of the present Terminal Reserve in the years 1917-21, following the initial settlement in 1915. The approximate positions from which these photographs were taken are indicated on figure 3. The toe of the flow-slide shows on the right in figures 5, 7, and 8. The size of the trees growing on the slide indicates that the flow was probably several decades old in 1918, yet trees at the toe are leaning and fallen indicating, perhaps, some continued movement and tidal erosion. The slide in the area of Boring Line 1 is shown on the left in figure 6 and in the foregrounds of figures 7 and 9. It, too, looks old, judging from the upright mature conifers shown in figure 7. For reference, a few trees are identified by number on figures 7 and 9. Figures 6 and 10 show that almost all the trees were removed from the nose of Government Hill at an early date. Figure 10 shows the discharge pipe of the dredge operating at the site of Army Dock (then called Ocean Dock); it is emptying landward from the railroad trestle and illustrates the beginning of the process of artificial filling that has continued along the shore ever since.

Figures 11-19 (in pocket) are vertical aerial views of the mouth of Ship Creek and the port showing the same area to the same scale (1 inch equals 1,000 feet) at succeeding intervals, beginning with the earliest available photography taken in August 1942 and ending in May 1967. Some landmarks and areas of interest are identified on the overlay for figure 19.

In figure 11, the elongated triangular flat of the port area is modified only by the white trace of the Elmendorf sewer outfall and by the developed areas near the southwest corner of Government Hill. The west slope of Government Hill is clearly broken into long furrows by slide blocks and interrupted by the nearly circular outline of the flow slide. North from the outfall, the crest of the slope consists of a half dozen or more shallow arcs marking the heads of old slides. Faint remnants of the removed material show as lobes of somewhat higher ground supporting a growth of bushes on the flat.

Figure 12 shows development of the tank farm on the flow slide, Bluff Road crossing the slide, and some cuts being made in the slide blocks south of Bluff Road preparatory to placing the five Standard Oil asphalt tanks shown on figure 13. Figure 12 also shows cuts have been made into the slope east and southeast of the Independent Company Warehouse. Figures 13, 14, and 15 show that, as embankments were built on to the flat, particularly north of the Standard Oil development on the flow slide, drainage to the sea was impeded and ponds developed between the embankments or between embankments and the slope. At the same time, runoff from the expanded development at Elmendorf Air Base was apparently greatly increased, cutting a sharp-walled ravine as shown in the right central parts of figures 14 and 15, and no doubt contributing to the formation of ponds in the flat area.

Hydraulically placed fill derived from dredging for the new City Dock, built in 1961, occupies much of the light-colored area southeast of City Dock in figure 15. This fill also apparently created an impervious barrier to waters that formerly flowed from the slopes to the sea. A drainage ditch constructed to divert runoff to the north around the growing fills east of City Dock appears in figure 16. Nevertheless, in the summer of 1966 much of the area between the slopes and the tank farms and fills in the port area was covered by a nearly continuous body of standing water.

III. Flow slide and closed depression

The Standard Oil of California tank farm is built on the disk-shaped flow slide discussed previously. This area should be carefully watched for signs of movement. One of the apparently most active areas within the Terminal Reserve is south of Bluff Road at the foot of the slope back of the line of asphalt tanks. A wooden retaining wall has been displaced here and was observed in September 1966 to be crowding adjacent steam lines.

There appears to be a relation between the slide and the closed depression directly to the south on Government Hill. Although the depression could be a kettle that resulted from the melting of ice within the glacial outwash gravels, it might instead be a much-modified remnant of a grabenlike depression associated with the landslide. The ground between the depression and the crest of the hill to the north above the slide is inclined toward the depression in a way suggesting that the depression is over an old slide block. To test this hypothesis an auger hole was drilled in May 1967 by the U.S. Geological Survey close to the crest of the hill. The top of the clay was found to be significantly higher here than as encountered in Shannon-Wilson boring F-110 closer to center of the depression. The writer now favors the hypothesis that the depression is a graben rather than a kettle and interprets the available information as shown in section A-A', figure 20 (in pocket).

The strong possibility that the depression is an old graben requires that the potential effects of grading around or filling in the depression be closely evaluated so as to avoid reactivating movement.

IV. Response of the port area to the March 27, 1964 earthquake

The effects of the March 27, 1964, earthquake in the port area have been reported by Engineering Geology Evaluation Group (1964, p. 22, fig. 12), Hansen (1965, p. A27-A29, fig. 14), Fisher (1965, p. 81-82, figs. 227-238, 243), Berg and Stratta (1964, p. 44-47), Brevdy (1964, p. 15), National Board of Fire Underwriters and Pacific Fire Rating Bureau (1964, p. 29-30), Stephenson (1964, p. 11-12, figs. 31-34, 37-39), and Steinbrugge, Manning, and Degenkolb (1967, p. 122-124).

Within the area of the port of the ARR Terminal Reserve lying west of Government Hill there was reported damage by the earthquake as follows: Army Dock rendered unusable; cement storage bin overturned at the Permanente Cement Co.; three storage tanks in the Standard Oil Co. tank farm bulged around the base and leaked, one horizontal tank thrown from its supports, empty tanks rocked and tore catwalks; ground cracked in vicinity of Union Oil Co. asphalt plant, and to the south in the filled land to Ship Creek; a crane was thrown down and cement storage bin toppled at the Alaska Aggregate Corp. dock facilities. The whole port area subsided, probably not uniformly, 1 to 2-1/2 feet.

Studies made by the writer in the field in 1966 and 1967 and of aerial photographs taken in the period shortly after the earthquake added a few details to the pattern of ground cracking presented by Hansen; his map with additions is shown in figure 21.

Two items of present interest are shown by the numbers on figure 21. Number 1 refers to a crack in the fill in the area of the asphalt tanks of Union Oil Company. Figure 4 shows a line of hachures that passes under these same tanks. This line, transferred from the 1914 topographic map, indicates the break in slope between grass-covered marsh, to landward, and the bare inclined slippery clay surface, to seaward, that extends to the surface of the water. This boundary is well shown in the left center parts of figures 5 and 9. It seems likely that the underlying shore topography influenced cracking in the fill during the 1964 earthquake, for most of the cracks west of Ocean Dock Road shown in figure 21 are at or below the break in slope of the old shore.

Number 2 on figure 21 refers to a crack east of the Independent Lumber Company warehouse at the base of the slope. It may be seen in the upper right in figure 22, and is of interest because it crosses Boring Line 1, and gives some direct evidence for the potential instability under seismic shaking that is inferred in analyses to follow. The crack has since been obscured by regrading.

Figure 23 (in pocket) shows section B-B' along Boring Line 1 prolonged westward with a slight change in direction at boring ARR-1D, across Ocean Dock Road and through the Union Oil Company tank farm. The large tanks were erected after the 1964 earthquake. The depth to which the sheet pile bulkhead extends at the west edge of the fill is not known by the writer.

V. Stability analyses

A word of caution is necessary regarding interpretation of "safety factors" that are derived in the analyses to follow. These factors generally represent some ratio of resisting forces or moments to driving forces or moments, or of available strength to required strength. A factor of 1.0 implies impending failure. However, the safety factor applies only to the whole suite of conditions assumed or specified in the analysis, including the details of the geometry of the potential failure surface and all the pertinent physical properties of the materials. Different shapes and orientations of the potential failure surface are tried; that yielding the lowest safety factor may be regarded as most likely or most dangerous. The search for the very lowest factor may consume more time than it is worth. Moreover, the in-place physical properties assumed for large masses based on tests of minute fractions of that mass in the laboratory can be regarded only as approximate. The properties may vary significantly from place to place. If water content

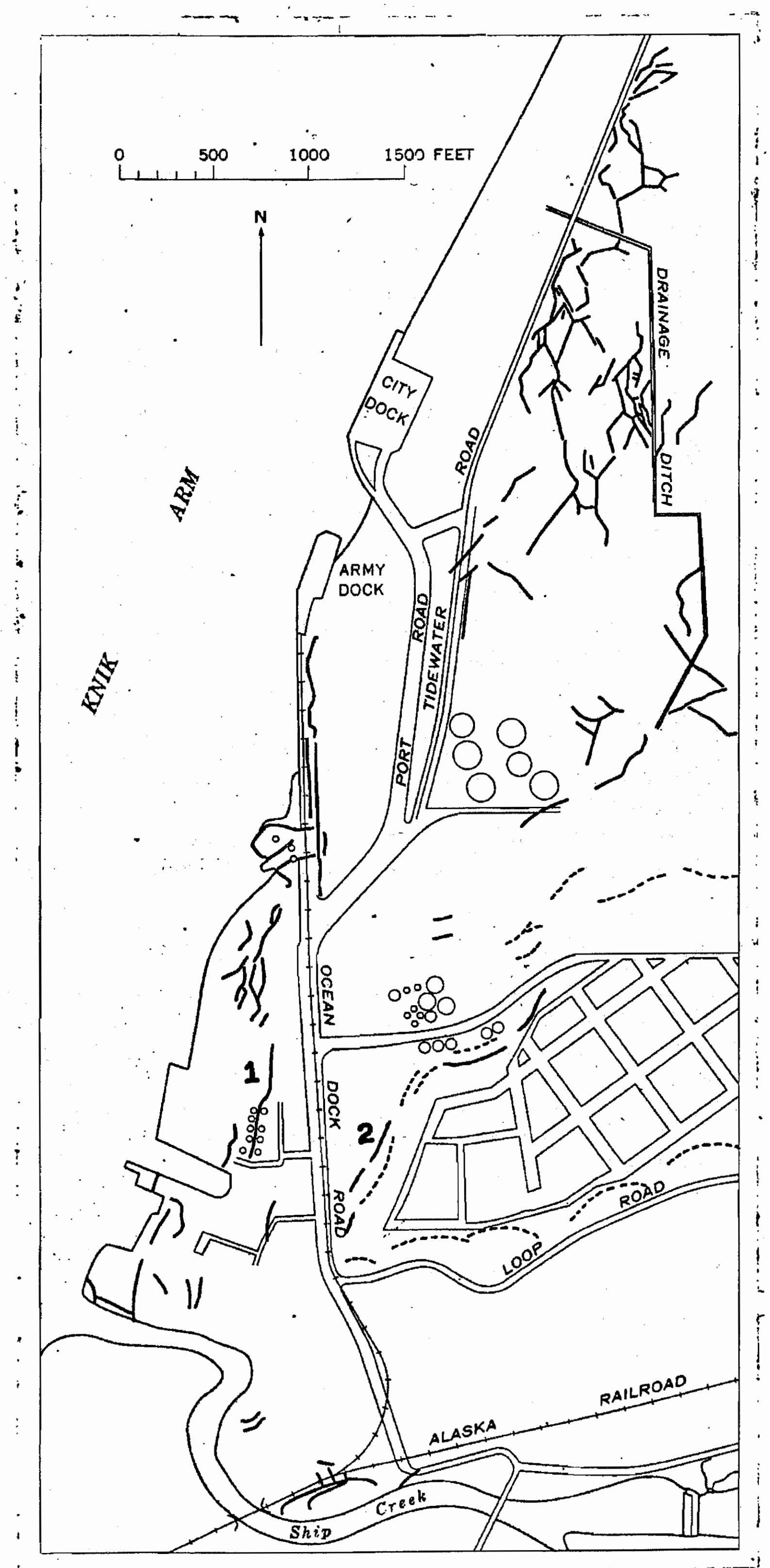


Figure 21
Ground cracks following the March 27, 1964, earthquake (solid) and scarps of old landslides (dashed), Port of Anchorage and vicinity. From Hansen (1965, fig. 14), with additions. The number cracks are discussed in the text.

or pore pressures change, the properties may also vary with time. Hence, it is possible to derive a safety factor of 0.8 or lower for a slope that is standing; conversely a safety factor of 1.2 is not to be regarded as reassuring. Safety factors are simply aids in the making of engineering decisions that are based on judgment of many aspects.

A. Boring Line 1

The presence of old slides in the lower part of the slope crossed by Boring Line 1, indicated by the early topographic map and photographs (figs. 4, 6, and 7), is confirmed also by the logs of borings ARR 1-A, B, and C, made late in 1966. Summary logs and results of tests on samples from the borings are attached to the vertical section figure 24 (in pocket). The difference in elevation of the top of the Bootlegger Cove Clay in borings 1-A and 1-B indicates offset by sliding and positions of the same contact in boring 1-C and in a nearby earlier boring, DM 64-13, suggest a slight backward tilting of the slide block or blocks. No doubt the distribution of the silt and clay beneath the slumped surface of granular materials is more complicated than can be shown on the basis of available information. Regrading has obscured the topographic form of the slides and much material has been removed from the present location of the lumber warehouse. The ground surface as of about 1916 is shown based on a map prepared by the Alaskan Engineering Commission about 1916, similar to but more detailed than that shown in figure 4.

The old slides may have moved along surfaces such as ABCFDE or ABCFG in figure 24 or ABOP in figure 25 (in pocket). The position of the crest of the slope prior to the last period of major sliding is inferred by the construction in figure 26. It is assumed that the last major movement was bounded at the head by surface ABB'. Owing to removal of material from the old slide, particularly at the toe, and reconsolidation of the soft sediments beneath the coal stockpile placed in 1918-21, I believe that the present most dangerous potential failure surfaces would crop out east of the warehouse.

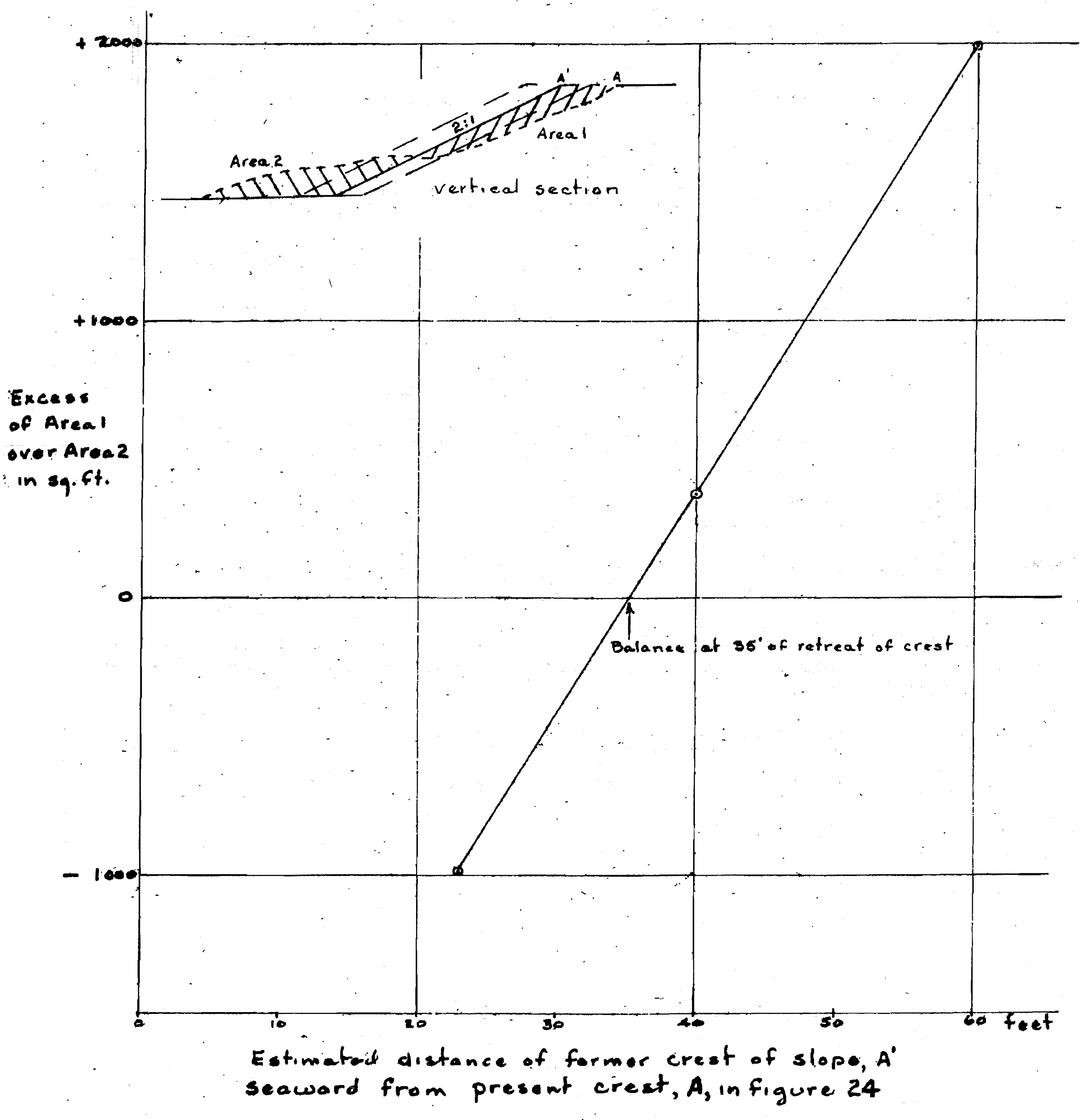


Figure 26

Determination of former crest of slope, Boring Line 1, by balancing, in a cross section through A, the area slid against the area deposited.

1. Analysis of potential failure along circular arc ABCJ, static conditions.

Basic data for the analysis is shown in table 1.

a. Method 1--Base forces

$$\text{Safety factor} = \frac{\text{(Friction at base of slice 11)+(cohesion on BCJ)}}{\text{total W sin } \alpha}$$

(1) S.F. = (10.2 + 209)/157.7 = 1.39 if wedge JNHI is in place.

(2) S.F. = (10.2 + 191.5)/(185.9) = 1.08 if wedge JNHI is removed.

--Moments about center of arc BCJ

$$\text{Safety factor} = \frac{\text{Total resisting moments}}{\text{Total driving moments}}$$

(1) S.F. = [7030 + (10.2 + 209) 100.6]/21967 = 1.32 if wedge JNHI is in place.

(2) S.F. = [3127 + (10.2 + 191.5) 100.6]/21967 = 1.06 if wedge JNHI is removed.

The two ways of calculating by method 1 are mathematically equivalent and agree within the accuracy of estimating locations of centers of gravity of slices.

b. Method 2--Polygons of forces

For explanation of method see Seed (1966, p. 23-25).

(1) Figure 27

S.F. = 1.44 if wedge JNHI is in place.

(2) Figure 28

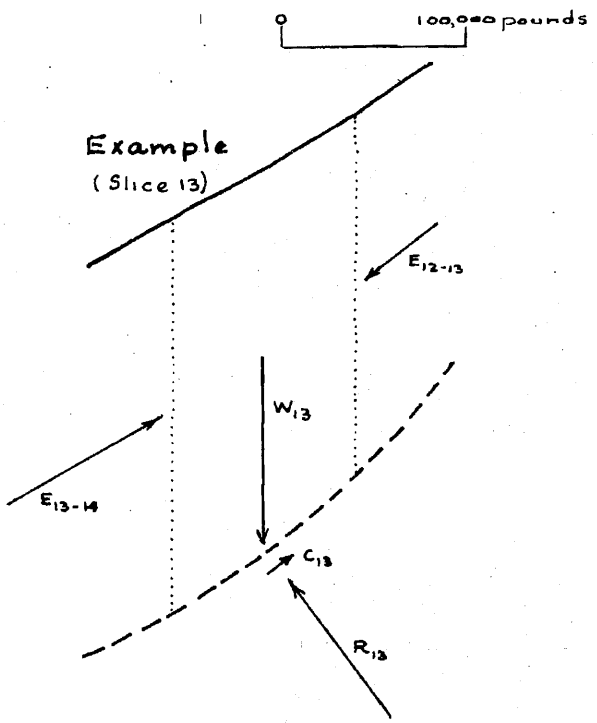
S.F. = 1.15 if wedge JNHI is removed.

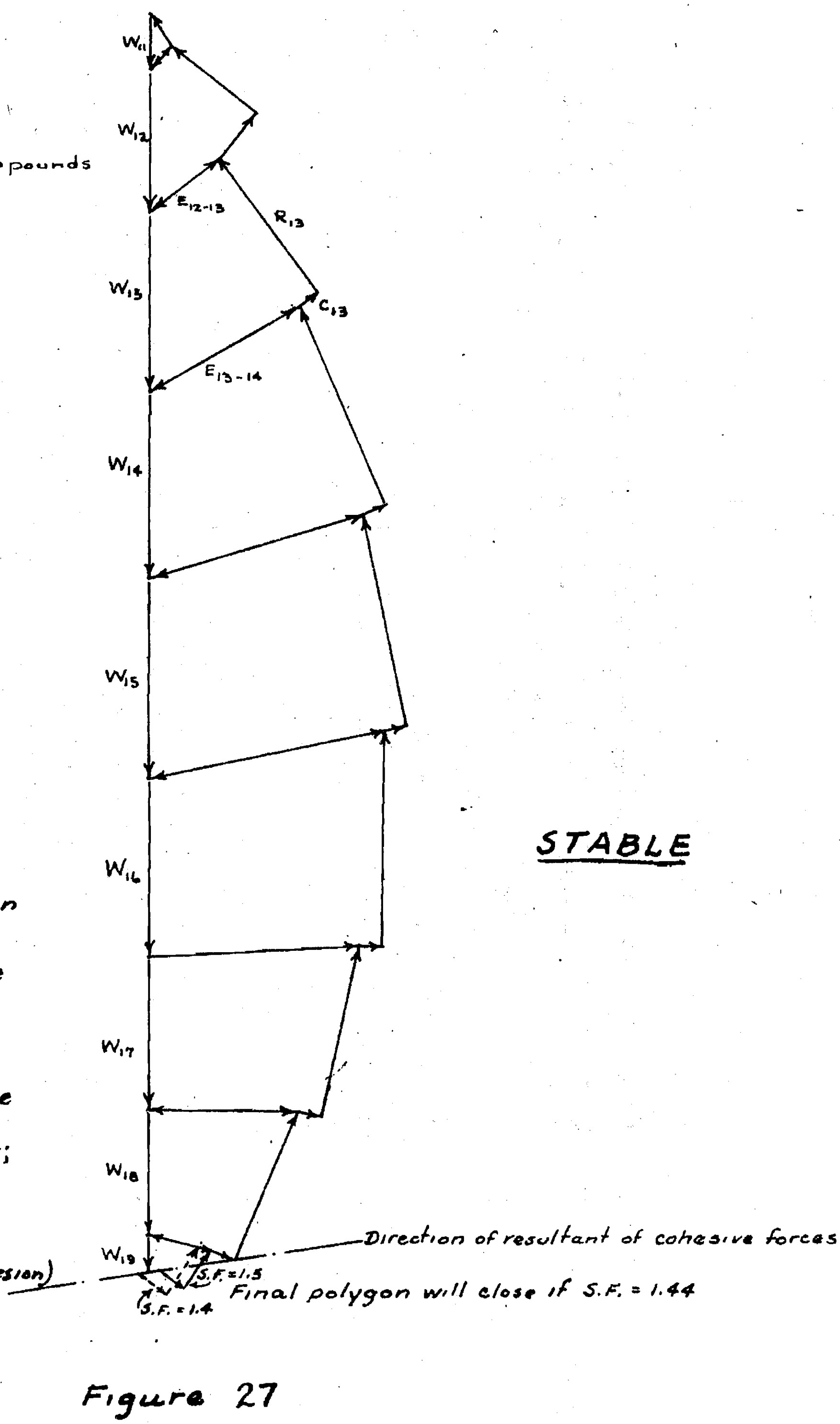
The two methods agree in indicating that, under static conditions, failure along arc ABCJ is unlikely unless the toe of the slope is removed. If the ground was removed to the level of GH (base of warehouse) the stability along arc ABCJ (through the removal of wedge JNHI) would be reduced markedly.

Table 1.--Basic data for analysis of potential failure along arc ABCJ, Boring Line 1

Slice	ϕ	Weight (kips)	$\alpha°$	sin α	cos α	W sin α Tangential component		N component	Friction N tan ϕ	Length contact	Cohesion (kips)	Lever arm	Moments (kips-ft)	
						Drive	Resist						Driving	Resisting
11	35	31.6	62.5	0.887	0.462	28.0	------	14.6	10.2	-------	--------	95.8	3,027	---------
12	0	82.5	52.5	.793	-----	65.4	------	------	--------	34	51	78.3	6,460	---------
13	0	102.4	36	.588	-----	60.2	------	------	--------	25	25	59.3	6,070	---------
14	0	106.7	23	.391	-----	41.7	------	------	--------	22	22	39.3	4,190	---------
15	0	115	11.3	.195	-----	22.4	------	------	--------	21	21	19.3	2,220	---------
16	0	102	0	-----	-----	------	------	------	--------	20	20	0	0	---------
(16)	0	(100)	0	-----	-----	------	------	------	--------	(20)	(20)	0	0	---------
17	0	87.5	-12	.208	-----	------	18.2	------	--------	21	21	21	-----	1,840
(17)	(0)	(67.5)	(-11)	(.191)	-----	------	(12.9)	------	--------	(21)	(21)	(19)	-----	(1,280)
18	0	72.6	-23.5	.399	-----	------	28.8	------	--------	22	22	40	-----	3,900
(18)	(0)	(45.1)	(-23)	(.391)	-----	------	(17.6)	------	--------	(22)	(22)	(38)	-----	(1,715)
19	0	22.3	-35.8	.584	-----	------	13.0	------	--------	27	.27	58	-----	1,290
(19)	0	(2.5)	(-32.5)	(.537)	-----	------	(1.3)	------	--------	(9.5)	(9.5)	(53)	-----	(132)
						217.7	60.0 (31.8)				209.0 (191.5)		21,967	7,030 (3,127)

α = angle between normal to slip surface and the vertical.
( ) = values after removal of prism JNHI are shown in parentheses.





1. Direction of E forces between slices is average of inclination of top and bottom surfaces of slices at boundary, or of line joining centers of gravity of adjacent slices
2. Reaction R is normal to base
3. Cohesion is parallel to base; amount is regulated by Safety factor

   $C_{13} = (S.F.)(\text{length of base})(\text{Unit cohesion})$
4. Safety factor is varied until final polygon closes.

Figure 27

Force polygon stability analysis of potential failure along circular arc ABCJ, line 1, under static conditions, wedge JNHI in place.

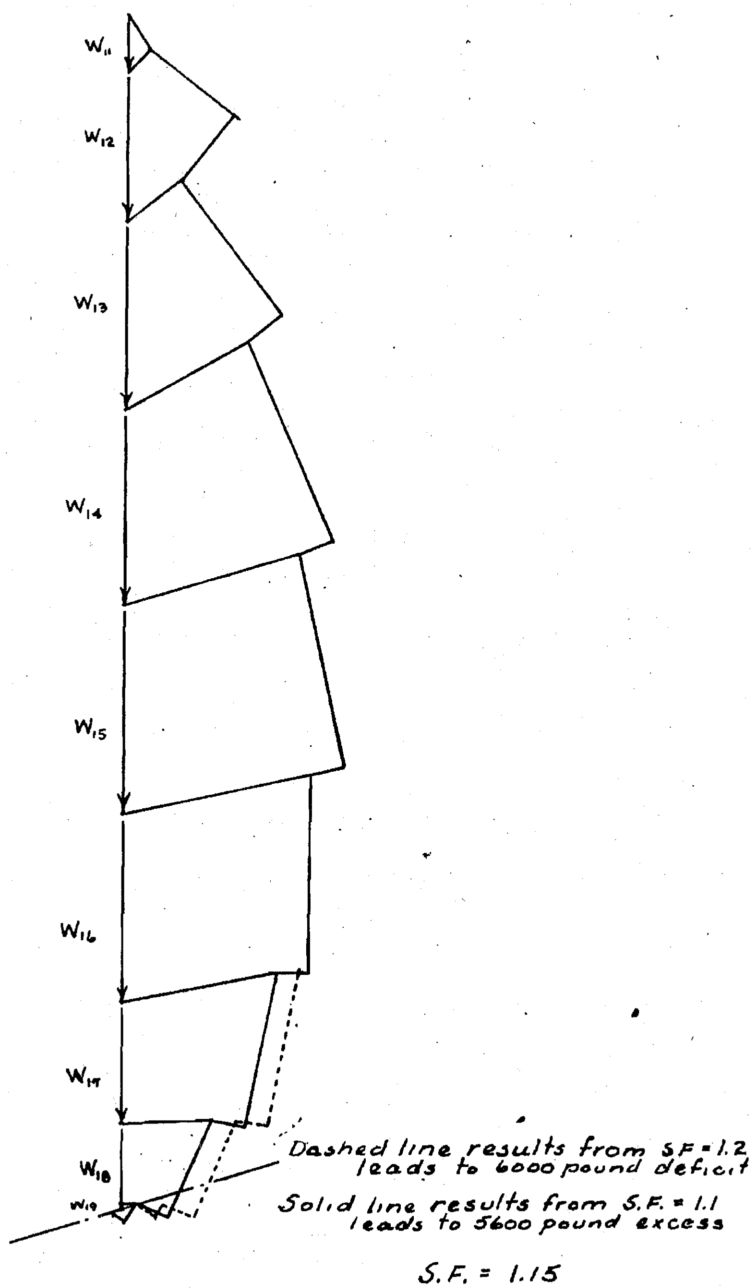


Figure 28

Force polygon stability analysis of potential failure along circular arc ABCN, line 1, under static conditions, wedge JNHI removed.

2. Analysis of failure along circular arc ABCJ, seismic conditions.

According to the stability computations that follow, failure along a circular arc under seismic conditions appears less likely than failure along a more or less horizontal surface. Nevertheless, figure 29 shows that the mass above circular arc ABCJ has a safety factor of only 0.82 with respect to available cohesion if seismic coefficients of 0.15 horizontal and 0.15 vertical combined are used in stability computations. It is estimated that, if the available cohesion were completely used, a seismic coefficient of the order of 0.06-0.065 would reduce the safety factor to 1.0 from its static value of 1.44.

It should be noted that a seismic coefficient of, say, 0.1 (used in the sense of Seed and Martin, 1966) represents the damaging effects of an earthquake with maximum ground motion accelerations considerably exceeding 0.1 gravity.

It is possible that arc ABCJ is not the critical arc for the profile on Line 1 and that some other arc may have a lower factor of safety.

3. Analysis of potential failure along a flat surface KLM, static conditions.

a. Factors governing location of surface, see figure 24.

(1) Will be restricted to zone with lowest value of average cohesion (elev. +33 to -8).

(2) Should stay out of outwash sand and gravel, hence below inferred position of base of sand in pre-1916 slide, that is, below elev. +28.

(3) Should stay below probable position of water table.

(4) Emergence of slip surface at toe will depend on where passive resistance is least.

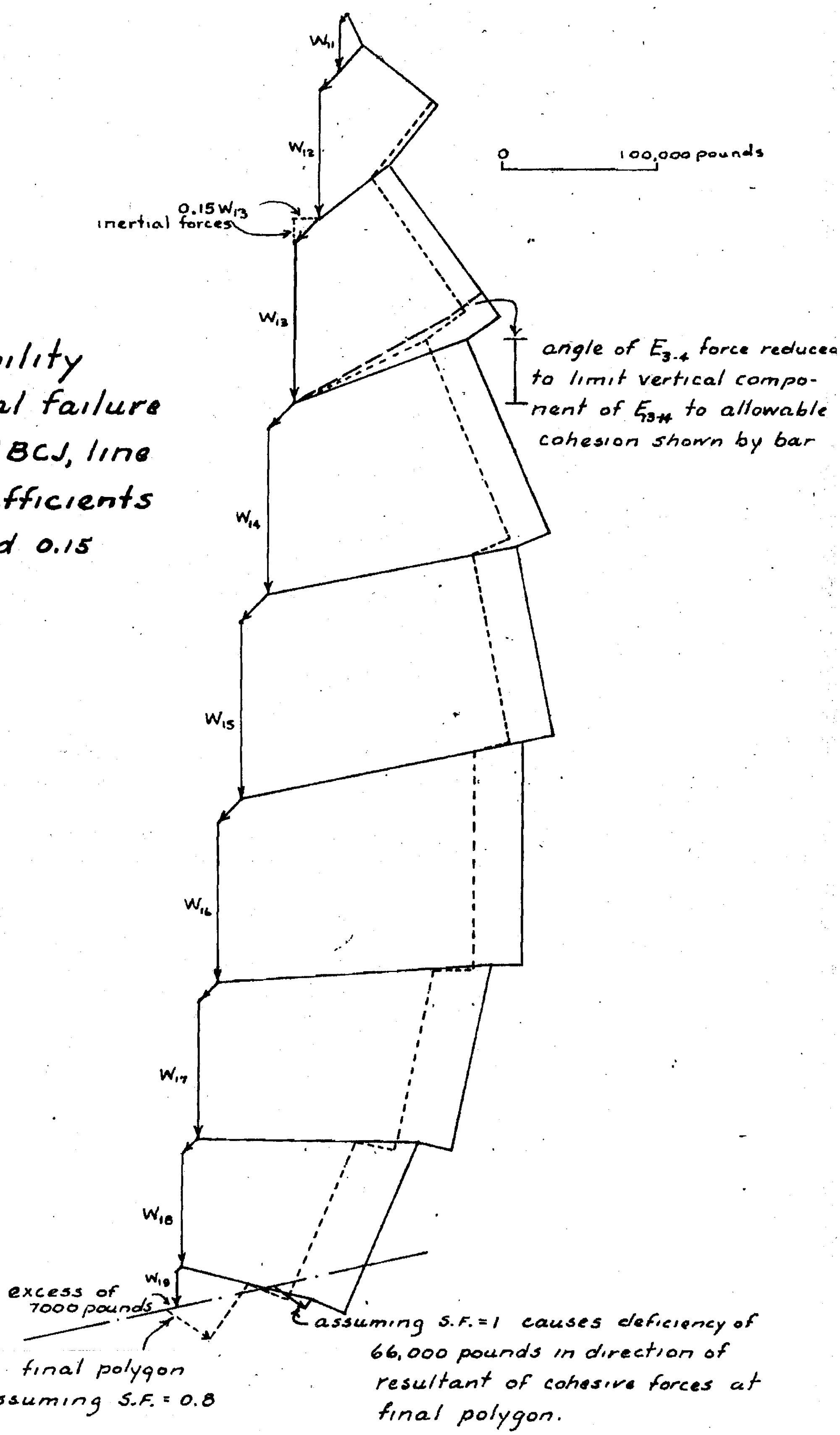
b. Location of least passive resistance.

(1) For failures along a relatively high surface, the point of upturning L will be located vertically beneath Q, the base of a graded slope.

(2) Unless cohesion drops to a low value, a location for L farther toward the northwest is more safe than beneath Q, as more cohesive resistance is mobilized along the longer path.

Figure 29

Force polygon stability analysis of potential failure along circular arc ABCJ, line 1, under seismic coefficients of 0.15 horizontal and 0.15 vertical.



S.F. = 0.82 with respect to available cohesion

(3) The depth of L below Q will be determined by the relation between passive resistance on QL, cohesion available on surface southeastward from L, and active pressure at head of slide.

c. Location of most unsafe active head.

(1) Point of upturning of failure surface will probably be at point M, vertically below crest of slope.

(2) Selection of point M farther to the southeast would lead to no greater active pressure (under static conditions) and would allow more cohesion to be mobilized on slip surface.

(3) If M were farther northwest, the active pressure would be less because of the decrease in height of slope.

d. Elevation of zone of probable failure.

(1) Surface of potential failure will be taken to be at the shallowest location of LM at which active pressure on MA (E_a) requiring mobilization of block MSRA, exceeds passive resistance on LQ (E_p) plus cohesion on LM, see figure 30 and Appendix A.

(2) Driving component of weight due to seaward dip of bedding of about 1° is neglected.

e. Depth of LM that was selected to show on figure 24 was chosen so that safety factor is 1.0 for assumed conditions.

4. Analysis of potential failure along planar surface K'L'M', seismic coefficients of 0.15 horizontal and vertical.

a. A surface of failure was selected that is shallower than the surface of potential static failure because experience has shown that seismic failure will occur as high as possible in the weak part of the Bootlegger Cove Clay. Moreover, liquefaction of thin sand layers is more easily induced the lower the confining pressure (Seed and Lee, 1966). The segment L'M' along the surface K'L'M'S'R' selected for analysis passes through a wet sandy zone encountered in boring ARR 1-B, at depth L' below Q of 17 feet.

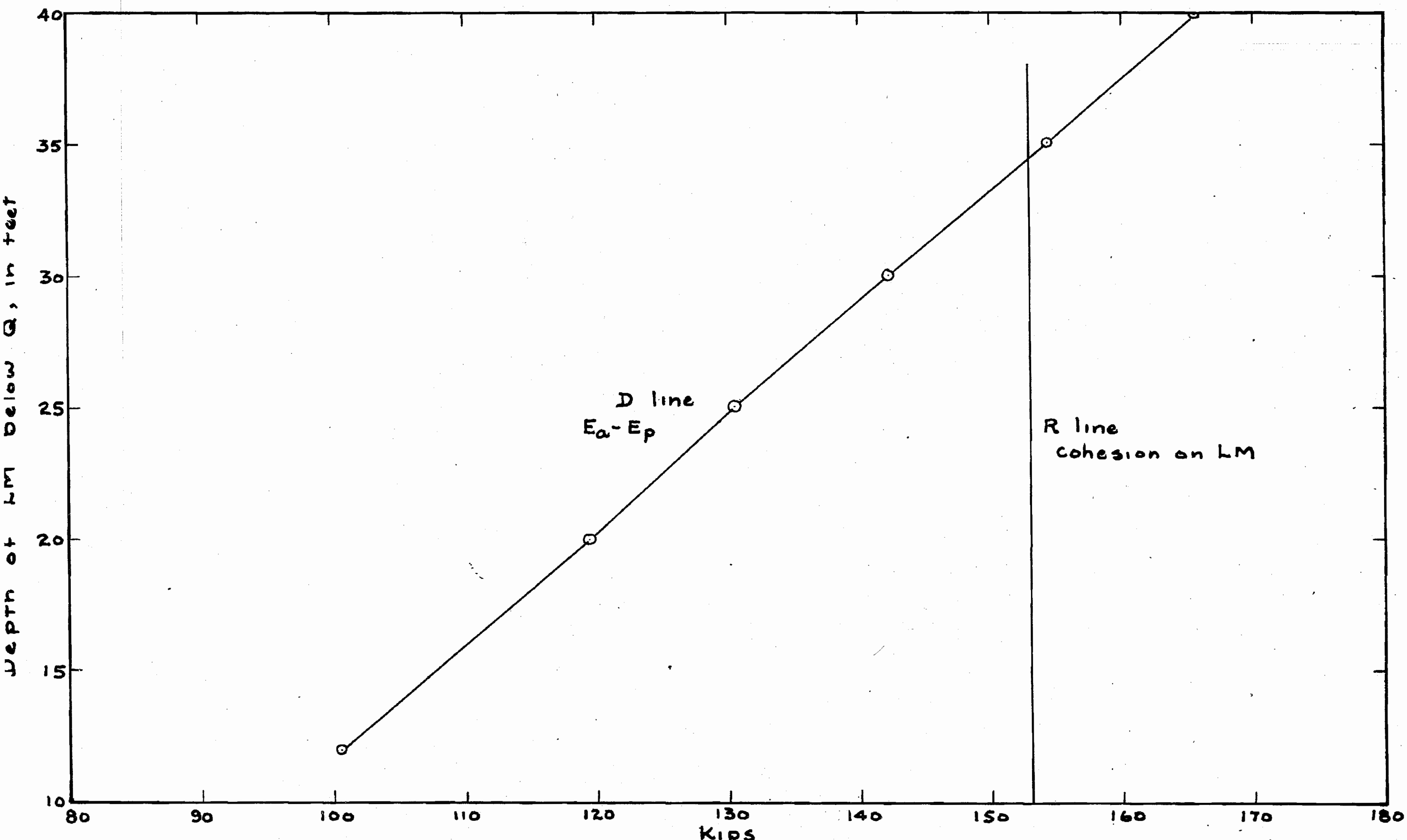


FIGURE 30

Determination of depth of potential failure surface LM, Boring Line 1, for static conditions. Depth must be such that driving force $E_a - E_p$ shown by D line exceeds cohesion along LM shown by R line.

b. Weights

Slice	Weight above K'L'M'S'R' in kips
6	2.3
7	48.8
8	119.0
9	155.0
10	178.8
11	176.0
12	144.0
13	114.0
14	90.2
15	83.8
16	67.1
17	57.5
18	42.5
19	8.8
	1287.8

c. Active pressure at head if active wedge is AM'S'R' (see Appendix A)

E_a = 190.0 kips

d. Resistance

Passive pressure of toe QL'K' E_p = 63.2 kips (see Appendix A)

Cohesion c = 153 times cohesion per foot in kips

e. Stability analyses

(1) S.F. = (63.2 + 153)/190.0 + 1288 x 0.15

= 216.2/383.2

= 0.56

(2) Calculate cohesion required for S.F. = 1

$$1.0 = (20{,}770 + 42.4c + 153c)/314{,}075 - 2\sqrt{2} \times 39 \times (42{,}000 + 11c)/39$$

$$1.0 = (20{,}770 + 42.4c + 153c)/314{,}075 - 2\sqrt{2} \times 39 \times (42{,}000 + 11c)/39 + 193{,}200$$

c = 2240 which appears to be much more than is available

(3) Assume unit cohesion of 1.0 kips (1000 psf) and safety factor of 1.0 and calculate seismic coefficient k that can be tolerated.

$$1.0 = (125/2)\ (1 + k)\ 17^2 + 195{,}420/(125/2)\ (1 + k)$$

$$39^2 + 54{,}650 + 1288\ k$$

$$k = 0.047$$

(4) The computations indicate that the seismic coefficient that can be tolerated for planar failure is less than that for failure along a circular arc.

(5) Potential failures extending farther southeast and involving masses within about 1/2 wavelength of earthquake waves from the base of the slope would require even higher values of cohesion to achieve stability.

B. Boring Line 2

A vertical section along Boring Line 2 is shown in figure 31 (in pocket), together with summary logs of the borings and test results. The boring logs, observed scarps at points L and P, the 1914 and 1916(?) topographic maps, and information from aerial photographs taken before the surface was altered have been used to arrive at the geological interpretation shown on the section. At least two landslide slip surfaces, one cropping out at L and one at P, are inferred. A disturbed zone at point C, depth 71 feet in boring ARR-2B, marked by high water content and soft consistency, is assumed to be on an actual or potential locus of horizontal slippage ABCJ.

1. Analysis of potential failure along circular arc RSP, under static conditions and using the method of moments, is shown in figure 32 (in pocket). The safety factor is 1.06.

2. Analysis of potential failure along a somewhat higher circular arc, AXYZ, under static conditions and using the method of force polygons, is shown in figure 33 (in pocket). The safety factor is 1.09. Analysis by the method of moments yields a safety factor of 0.91, and by the method of forces on the base of slices a safety factor of 0.94. All analyses indicate marginal stability for the assumed conditions. No attempt was made to find the potentially most dangerous circular arc.

3. Analysis of potential failure along circular arc RSP, under seismic conditions assuming seismic coefficients of 0.15 horizontal and 0.15 vertical and using the method of moments, is shown in Appendix B. The safety factor is 0.7.

Seismic coefficients of only 0.018 can be tolerated if a safety factor of 1.0 is assumed and shear strength is not diminished by shaking.

4. Analysis of potential failure along a horizontal or nearly horizontal plane ABC prolonged, under static conditions.

The calculations shown in Appendix C and figures 34 and 35 result in the following safety factors:

For failure along	Safety factor
ABCDE	1.68
ABCJK	1.45
ABC'L	1.40
ABTW	2.75

Under static conditions, a circular arc failure near the toe of the slope appears more likely than a horizontal failure.

5. Analysis of potential failure along surface ABC'L, assuming seismic coefficients of 0.15 horizontal and vertical. The safety factor is 0.52. The calculations are shown in Appendix C.

6. Analysis of potential failure along surface ABCDE assuming seismic coefficients of 0.15 horizontal and vertical and no decrease in shear strength by shaking. The calculations are shown in Appendix C. The safety factor is 0.34.

7. Seismic coefficients of about 0.03 can be tolerated by the slope if a safety factor of 1.0 is assumed and shear strength is not diminished by shaking.

VI. Summary of stability

A. Boring Line 1

1. The hillside has been involved previously in major sliding. These slides greatly complicate correlation of borings and make analysis of the present stability subject to large uncertainties.

2. The slope appears to be stable under static conditions in its present configuration except that shallow sloughing may be expected near the crest.

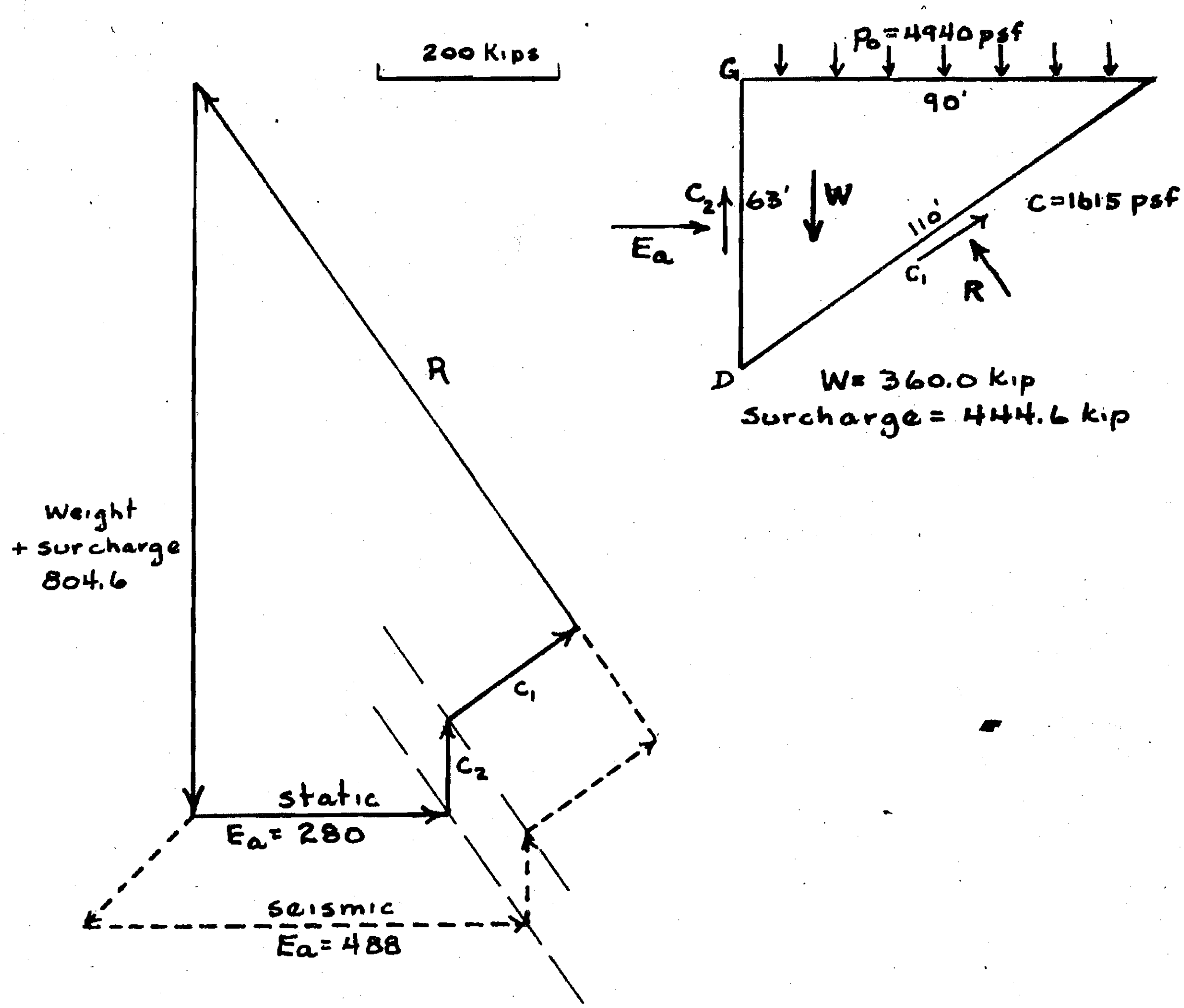


FIGURE 34

Graphical determination of active pressure E_a on GD, Boring Line 2.

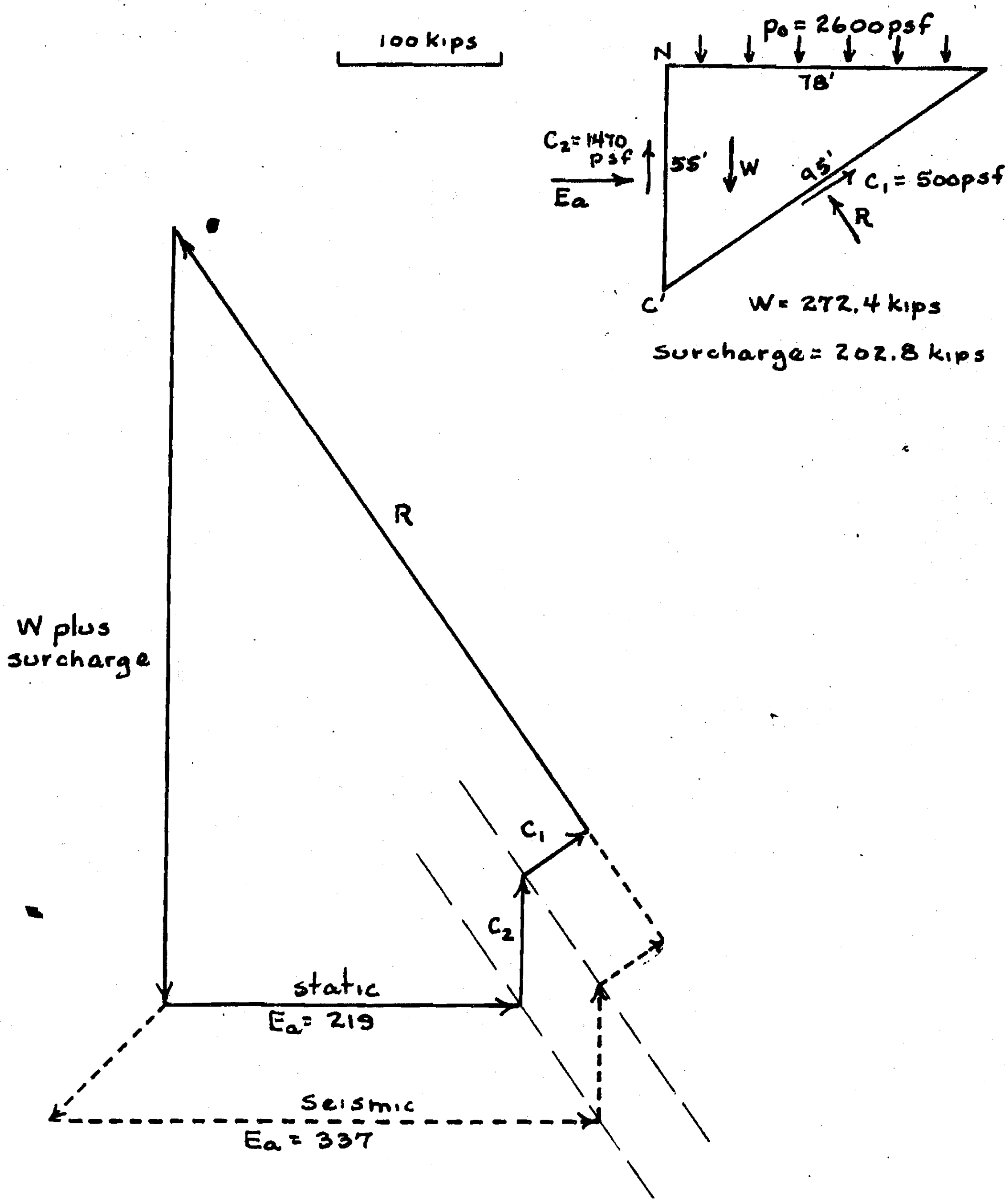


FIGURE 35
Graphical determination of active pressure E_a on NC', Boring Line 2.

3. Removal of material between the Independent Lumber Company warehouse and the slope will greatly decrease the resistance to sliding and lower the safety factor to near 1.

4. The hillside is found to be unstable for circular arc failure if seismic coefficients of the order of 0.06 horizontal and 0.06 vertical combined are used in stability computations, and unstable for failure along a flat surface at smaller seismic coefficients.

5. Failure under strong motion probably would not be confined to the slope but would involve also part of the level surface back of the crest.

B. Boring Line 2

1. Surface expressions of scarps, topographic form, and boring correlations indicate that there are at least two old slides on this slope.

2. The lower part of the slope, from Army Road to the pond, is on the verge of instability under static conditions, with slip on a circular arc the most probable mode of failure near the toe.

3. Possible failure involving the rest of the slope to and beyond boring 2-A has been investigated for a nearly horizontal plane at elevation about +10, which appears to be one of the more hazardous potential failure surfaces.

The slope is stable along this plane under static conditions.

The slope is found to be unstable along this plane if seismic coefficients of about 0.035 horizontal and 0.035 vertical, combined, for assumed failure extending to the south edge of Bluff Road, are used in stability computations.

C. General remarks

1. Not only the areas crossed by lines 1 and 2, but also the entire stretch of slopes along the west side of Government Hill from Ship Creek around to the military P.O.L. storage tanks has slid in the past.

2. Stability analyses indicate that many of these failures may well have been triggered by earthquakes, although slope failure without quakes may be common.

3. In the event of an earthquake, a slope will stand or fall according to its physical condition and to the forces imposed on it. The variation in effects of the March 27, 1964, earthquake can be regarded as a result of differing physical conditions of the slopes. Of equal importance, however, is the possibility that similar slopes were subjected to widely different accelerations depending on their position relative to complexly reflected and interfering surface and body waves.

4. The seismic forces on an earthbank depend not only on the relatively unchanging influence of topography and geology but also on factors that vary from one earthquake to another, such as distance, direction of transmission of energy, intensity, period, and type of wave.

5. Whatever the computations in this report may indicate, the slopes clearly have enough margin of stability to withstand some additional (but unknown) loading corresponding to that resulting from the 1964 earthquake, if conditions affecting stability have not changed since then.

6. That a slope did not fail in the severe quake of March 27, 1964, is no guarantee that it is seismically stable. It might fail under locally higher or more unfavorably directed forces generated by a somewhat smaller quake with a different epicenter.

7. It is also an observable fact that a previous failure does not guarantee stability. The L Street, 4th Avenue, Government Hill School, and Native Hospital slides each occupied, or partly occupied, sites of previous slides. Many other existing slides were not reactivated. Previous sliding may interrupt the continuity of soft zones along bedding and help promote stability. It also may disrupt internal drainage and promote high pore water pressures and instability.

8. The effects of a previous earthquake, even though they did not result in failure, may reduce resistance to future failure.

9. Instability, as defined by a safety factor of less than 1, does not necessarily imply sudden and catastrophic failure. It all depends whether continued movement leads to a decrease in driving force (as in a rotational slide), no essential change in driving force, or changes in soil strength along the sliding surface.

In particular, cyclic stresses produced by earthquakes may exceed resistance for only a small fraction of the cycle, as they build up, decay, and reverse. Hence movement will generally occur in a jigging fashion. Reversing stresses are particularly effective, as Seed and Chan (1966) have shown, in decreasing the effective strength of soils, and reversing stresses are the kind usually produced by earthquake waves.

10. Loading the toe of a slope may not increase stability as much as expected; the improvement depends very much on the characteristics of the material forming the slope, the degree of saturation, and the shapes of potentially dangerous sliding surfaces. For instance, adding weight to the lower upwardly rotating part of a circular arc slide clearly adds to the resisting force and helps stability. If, however, failure is likely to occur along a weak horizontal layer that passes beneath the area being loaded, then adding more load does no good. It is necessary to intercept the weak layer and to support the material overlying it by means of a buttress having sufficient shear strength. Should there be also a higher weak zone, the possibility of failure along a surface that emerges above the buttress must also be considered.

VII. Other engineering geologic problems

The immediate need for information on the stability of slopes in the vicinity of Boring Lines 1 and 2 has required that attention be focused on the actual and potential landslides of this area. It is appropriate, however, to briefly mention other engineering geologic problems that may affect use of the land.

A. Slides in the shore area

The possibility of slides along the shore needs to be considered, particularly where slopes below high water level are naturally steep or have been dredged, and where the shore is loaded with a thick fill or heavy structures.

B. Drainage

The progressive derangement of the natural drainage in recent years is evident from the aerial photographs reproduced as figures 11-19. Part of this is due to new fills, dikes, and roads. It is possible that much of the standing water could be removed, and that ponds could be at least partly drained prior to filling them in. Improved drainage could decrease hazards of instability.

C. Tectonic warping

The whole waterfront area is now about 2 feet lower relative to the sea than it was prior to the 1964 earthquake. The possible effect of this change in level on the soil stability and the relation of changing ground-water levels to the geologic setting need to be determined by continued observation.

VIII. Recommendations

Although this investigation has been directed particularly at the Terminal Reserve lands held by the Alaska Railroad, the following remarks are believed to be applicable to much of the waterfront area, that is, to lands held also by the City of Anchorage and the Department of Defense. Certainly, any action taken by one party in this area, either beneficial or otherwise, can affect the others, and all face common problems that are often best solved by common action.

The geologic setting, the topographic cul-de-sac form of the port area, the already large concentration of petroleum handling and storage installations, and the economic importance of the docks, railroad, and transshipment facilities dictate that more than ordinary prudence be used in planning and carrying out industrial development.

For ease of reference the area can be divided roughly into three zones, each of which presents somewhat different problems:

Zone 1.--From 200 yards back of the top of the bluff to 100 yards in front of the bluff toe. Consists of outwash gravel underlain by Bootlegger Cove Clay. Highly modified by landsliding and artificial fill.

Zone 2.--From 100 yards in front of the bluff toe to mean higher high water (MHHW). Consists of fill, marsh, peat, and estuarine sediments underlain by Bootlegger Cove Clay.

Zone 3.--Seaward of MHHW. Consists of fill, estuarine sediments, underlain by Bootlegger Cove Clay, which is underlain by glacial till.

A. Construction

1. Petroleum storage tanks, other heavy structures, or thick fills should not be placed in zones 1, 2, or 3 unless detailed geologic and engineering studies confirm that the potential problems have been evaluated, that the proposed construction is safe, and that it will cause no unfavorable effects elsewhere.

2. There should be no cutting into the toe of the slope in zone 1, unless measures are taken to fully compensate for the increased landslide hazard caused therefrom.

3. The surface and shallow subsurface waters in zones 2 and 3 and along the bluff face in zone 1, especially at the top of the Bootlegger Cove Clay, should be intercepted and led off without allowing them to reenter the ground. Particular attention should be given to effective disposal of water from the Elmendorf storm drain.

4. A sewerline runs through the slide area crossed by Boring Line 1, near boring ARR-1B. The Elmendorf sewerline passes under the large fill southeast of City Dock. It is suggested that these and other drains and sewerlines that are in or enter the waterfront area be checked periodically for possible distress due to ground creep or consolidation of the surrounding soil.

5. The location and discharge of springs in the tidal zone should be determined so that provision may be made for relief of pressure and removal of the water before they are covered by impervious fills.

B. Future studies

Only a part of the investigations planned or proposed in 1966 could be carried out in the available time. An excellent topographic base map has been prepared and the boring, sampling and testing program along Boring Lines 1 and 2 was concluded and the results analyzed. It was found that geologic mapping in the conventional sense is probably impractical on the brush-covered landslide slopes of Government Hill and of questionable value on the flats, where large changes in the pattern of fills are occurring rapidly.

Periodic vertical large-scale aerial photography would be helpful in many investigations related to land use in the port area.

A plan should be made for relating water level fluctuations in holes in zone 2, and perhaps part of zone 3, among themselves and with tide levels. This should give some information on in-place permeability, will help in laying out and determining the effectiveness of drainage, will yield information that is helpful to stability analyses, and will make it possible to determine how placing fills on the shore affects the water table higher up on the flats. It would be advantageous to have piezometers installed in some of the holes. The safety of hydraulic fills against liquefaction during earthquakes should be checked.

A scheme should be laid out for detecting horizontal and vertical movements in some of the major structures of the waterfront area, including points on the principal docks and on tanks within each of the larger tank farms. A good horizontal and vertical control net should extend eastward beyond the top of the bluffs and tie to stable points well outside of the waterfront and to the city's survey system.

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Note: The formulas used for calculations in Appendix A and Appendix C are explained briefly in Appendix D.

Appendix A

I. Calculation of forces plotted in figure 30. For example: At depth of L of 12 feet below Q.

a. Passive resistance at toe.

Cohesion of sand and gravel in upper 2 feet is neglected.

$E_p = 1/2\ \gamma\ H_1^2\ K_{p1} + cH_2K_{p2}$ $\qquad \phi = 0$

$\qquad K_{p1} = 1,\ K_{p2} = 2\sqrt{2}$

$= 9000 + 28{,}300$ $\qquad H_1 = 12$ feet, $H_2 = 10$ feet

$\qquad c = 1000$ psf

$= 37.3$ kips $\qquad \gamma = 125$ pcf

b. Active pressure at head.

Depth is 76 feet of which 42 feet is in gravel and sand, 34 feet is in clay.

E_{ag} (gravel and sand) $= 1/2\ \gamma\ H^2K_{a_1}$ $\qquad \phi = 35°$

$\qquad K_{a_1} = 0.25$

$E_{ag} = 1/2 \times 125 \times 42^2 \times 0.25$

$= 27.6$ kips; horizontal component 22.6 kips

E_a (clay) $= (1/2\ \gamma\ H^2 + p_oH)\ (K_{a_1}) - cH\ (K_{a_2})$

$\phi = 0, \qquad K_{a_1} = 1, \qquad K_{a_2} = 2\sqrt{2}$

$H = 34$ feet

p_o = surcharge of gravel and sand on clay

$= 42 \times 125 = 5250$ psf

$$\text{avg } c = \frac{(1500 \times 28) + (6 \times 1000)}{34} = 1410 \text{ psf}$$

E_{ac} = (1/2 x 125 x 34^2 + 5250 x 34) x 1 - 2.83 x 1410 x 34

= 250.8 - 135.7 = 115.1

ΣE_a = 22.6 + 115.1 = 137.7

c. Cohesion along LM

Resistance = 1000 x 153 = 153 kips

d. $E_a - E_p$ = 137.7 - 37.3 = 100.4 kips

Similarly, $E_a - E_p$ for greater depths of L below Q are also shown below and plotted on figure 30. Driving forces exceed cohesive resistance on LM at a depth of 34-35 feet.

Depth of L below Q (feet)	Forces in kips		
	E_a	E_p	E_a-E_p
12	137.7	37.3	100.4
20	195.3	75.9	119.4
25	234.5	104.2	130.3
30	277.7	135.5	142.2
35	324.1	170.0	154.1
40	373.2	207.5	165.7

II. Active pressure on AM'S'R', Boring Line 1, seismic coefficients of 0.15 horizontal and vertical.

a. Due to gravity and vertical acceleration.

E_{ag} = 22.6 x 1.15 = 26.0

E_{ac} = (1/2 x 125 x 1.15 x 39^2 + 5250 x 39) - 2.83 x 1360 x 39

= 109,325 + 204,750 - 150,103

= 164.0

$$\Sigma E_a = 164.0 + 26.0 = 190.0 \text{ kips}$$

b. That due to horizontal acceleration is included in inertia of total mass above K'L'M'S'R'

III. Passive resistance of QL'K'

$$E_p = 1/2 \times 125 \times 1.15 \times 17^2 + 1000 \times 15 \times 2\sqrt{2}$$

$$= 20{,}770 + 42{,}420$$

$$= 63.2 \text{ kips}$$

Appendix B

Calculation of stability along arc RSP, Boring Line 2, assuming seismic coefficients of 0.15 horizontal and vertical. See figure 32.

Slice	Weight (kips)	Vertical force (kips)	Lever arm (ft)	Horizontal force (kips)	Lever arm (ft)	Additional moments in kips-ft due to seismic forces: Driving	Additional moments in kips-ft due to seismic forces: Resisting
1	253.5	38.0	94	38.0	49.5	5,450	---------
2	239.0	35.8	62	35.8	61.5	4,420	---------
3	129.3	19.4	22	19.4	66	1,706	---------
4a	16	2.4	22	not counted		-------	50
4b	14.4	2.2	29	2.2	75	165	64
4c	30.5	4.6	13	4.6	75	345	60
5a	17.5	2.6	59	not counted		-------	153
5b	72	10.8	62	10.8	78.5	848	669
6a	11	1.7	93	not counted		-------	158
6b	37.4	5.6	88	5.6	79.5	445	487
Shaded area fig. 32	679	balances		101.8	100	10,180	---------
						23,559	1,641
				Static moments		42,250	44,870
						65,809	46,521

$$\text{Safety factor} = \frac{46,521}{65,809} = 0.7$$

Safety factor becomes 1.0 if seismic coefficients are reduced to 0.018

Appendix C

Calculation of stability for potential failure on horizontal surface ABC prolonged, Boring Line 2.

A. Static failure

1. Failure along ABCDE (fig. 31).

   a. ABCD assumed horizontal, which it may not be, quite.

   b. Active pressure on DF

      On FG

$$\phi = 35°$$

$$E_a = 1/2\ \gamma\ H^2 K_{a_1} \qquad K_{a_1} = 0.25$$

$$= 1/2 \cdot 130 \cdot 38^2 \cdot 0.250$$

$$= 23{,}400 \text{ lb; horizontal component} = 19.2 \text{ kips}$$

      On GD

$$E_a = 1/2\ \left(\gamma + \frac{2p_o}{H}\right) H^2 - 2\sqrt{2}\ cH$$

$$\phi = 0 \qquad \text{where } \gamma = 127 \text{ pcf} \qquad H = 63 \text{ feet}$$

$$E_a = 564{,}000 - 288{,}000 \qquad p_o = \text{surcharge}$$

$$= 276{,}000 \text{ lb} \qquad = 38 \cdot 130 = 4940 \text{ psf}$$

(for graphical solution see also fig. 34)

$$\text{across beds } c = \frac{(26 \times 1000) + (37 \times 2000)}{62} = 1615 \text{ psf}$$

      Total driving force = 295,200 lb

   c. Resisting forces

      Shear strength on AH = 0.72·2000·114.5 = 165,000

      HI = 0.47·2000·264.5 = 248,600

      ID = 0.36·2000·113 = 81,400

      495,000 lb

d. Static safety factor

$$S.F._{ADE} = \frac{495}{295} = 1.68$$

2. Failure along ABCJK

a. Driving forces same as above

b. Resisting forces

On AH	165,000
On HI	248,600
On IJ = 0.36·2000·21	15,100
	428,700

c. Static safety factor

$$S.F._{AJK} = \frac{428.7}{295.2} = 1.45$$

3. Failure along ABC'L

a. Assume failure along old slip plane that is believed to crop out at L

b. Active pressure on MN (approx.)

$E_a = 1/2 \cdot 130 \cdot 15^2 \cdot 0.271 = 3960$ lb; horizontal component 3250 lb

c. Active pressure on NC'

In this case, failure is assumed along an old and presumably softened slip plane. The shear resistance on this surface is estimated at 0.25 tons per sq ft and, after initial failure, to be independent of mean stress ($\phi = 0$ on slip surface).

See figure 35.

$E_a = 219,000$ lb

d. Total driving force, horizontal = 3300 + 219,000 =

222,300 lb

e. Resisting forces

On AH	165,000
HC'	145,800
	310,800 lb

f. Static factor of safety

$$S.F._{ABC'L} = \frac{310.8}{222.3} = 1.40$$

4. Failure on ATW

a. Active pressure on UV

$E_a = 1/2 \cdot 130 \cdot 14^2 \cdot 0.25 = 3180$; horizontal component = 2680 lb

b. Active pressure on TU

$$E_a = 1/2\ (\gamma + \frac{2p_o}{H})H^2 - 2\sqrt{2}\ cH \qquad H = 59 \text{ feet}$$

$$= 329{,}000 - 245{,}000 \qquad p_o = 14 \cdot 130 = 1820 \text{ psf}$$

$$= 84{,}000 \qquad c = 1470 \text{ psf}$$

c. Total driving force = 86,700 lb

d. Resisting forces

Along AH	165,000	
HT	73,400	(0.47·2000·78)
	238,400	total resisting

e. Static safety factor

$$S.F._{ABTW} = \frac{238.4}{86.7} = 2.75$$

B. Seismic failure

1. Failure along ABC'L

a. Weight above slip plane (excluding active wedge).

Block	Area (sq ft)
MC'TV	5,580
VTHO	4,860
OHA	2,910
	3,350 sq ft x 128 (approx avg γ) = 1,710,000 lb

seismic coef. 0.15

inertial force 256,500

b. Active force on MNC'

On MN

E_{ag} = 3250 x 1.15 + 0.15 weight = 5.1 kips

On NC'

E_{ac} = 337,000 by graphical solution shown on figure 35

c. Total horizontal driving force

256.5

5.1

337.0

598.6 kips

d. Resisting force

Assume cohesion not diminished by shaking

On AH	165,000
HC'	145,800
	310,800

e. Seismic safety factor

$$S.F._{ABC'L} = \frac{310.8}{598.6} = 0.52$$

f. Seismic coefficient, k, that can be tolerated if cohesion remains unaffected, and S.F. = 1.0

$$1710\ k + 3.3\ (1 + k) + 9.1\ k + 219 + 475.2\ k\ (1 + \tan 35°16')$$

$$= 310.8$$

$$k = 0.035$$

2. Failure along ABCDE

a. Weight above slip plane (excluding active wedge)

W = weight above AC'M + weight of C'MFD

= 1,710,000 + 128 x 222 x (171 + 101)/2

= 5,575,000

$\underline{\qquad .15}$

936,250 lb horizontal inertial force

b. Active force on FGD

On FG

E_a = 19.2 x 1.15 + weight x 0.15 = 41.1 kips

On GD

E_a = 488,000 lb (see fig. 34)

c. Total horizontal driving force

936.3

41.1

$\underline{488.0}$

1465.4 kips

d. Resisting force

Assume cohesion not diminished by shaking

On AH	114.5 x 0.72 x 2000 =	165,000
HI	264.5 x 0.47 x 2000 =	248,600
ID	113 x 0.36 x 2000 =	81,400
		495,000

e. Seismic safety factor

$$S.F._{ADE} = \frac{495}{1465} = 0.34$$

f. Seismic coefficient, k, that can be tolerated if cohesion remains unaffected and S.F. = 1.0

$$5575\ k + 19.2\ (k + 1) + 126.8\ k + 280 + 804.6\ k\ (1 + \tan 35°16') = 495$$

$$k = 0.028$$

Appendix D

The method used in this report for computing active and passive pressures of frictional or cohesive material is based upon the calculation of pressures exerted on or by a wall if the soil is assumed to adhere to the wall with its full value of cohesion or to have an angle of friction with the wall equal to its own angle of internal friction.

This method appears more logical than the use of the usual Rankine expressions for active and passive pressures, although it may be less conservative and result in higher safety factors.

The general expressions for active pressure E_a and passive pressure E_p are:

$$E_a = [1/2\ \gamma\ H^2 + p_o\ H]\ [K_{a_1}] - cH\ [K_{a_2}]$$

$$E_p = [1/2\ \gamma\ H^2 + p_o\ H]\ [K_{p_1}] + cH\ [K_{p_2}]$$

where γ = density

H = height of wall

p_o = surcharge per unit area

c = cohesion

K_{a_1}, etc. are coefficients that depend on the assumed angle of internal friction, ϕ.

The values of factors pertinent to the present problems are:

	$\phi = 0$	$\phi = 35°$
K_{a_1}	1.0	0.25
K_{a_2}	$2\sqrt{2}$	
K_{p_1}	1.0	
K_{p_2}	$2\sqrt{2}$	

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No 68-318 pl.

DEPARTMENT OF THE INTERIOR
U.S. GEOLOGICAL SURVEY

OPEN-FILE REPORT
1968

Photographs

Figure 1.--Looking northward toward the port area from over the main part of the city of Anchorage, Alaska. Business district is in right foreground. Ship Creek flows to the left into Knik Arm of Cook Inlet. Government Hill rises beyond Ship Creek in right center. In the middle distance is Army Dock, and beyond is the newly constructed City Dock. Photograph taken August 11, 1963, by Steve McCutcheon, Anchorage.

Figure 2.--Looking eastward toward the port area of Anchorage, Alaska, from over Knik Arm. Photograph taken August 11, 1963, by Steve McCutcheon, Anchorage.

Figure 5.--Coal bunker site from marine ways, Anchorage, Alaska, July 1, 1917. Photograph by Alaskan Engineering Commission. Toe of flow-slide is in the woods at the right.

Figure 6.--Ships ways, Anchorage, Alaska. Photograph by Alaskan Engineering Commission, March 1, 1917. Nose at southwest end of Government Hill appears at the left, with landslide deposits at the toe of the slope.

Figure 7.--Looking north from Government Hill. Photograph by Alaskan Engineering Commission, probably in early 1918. Future site of Independent Lumber Company warehouse is about at the foot of the landslide deposits beyond trees marked 1 and 2. Toe of flow-slide is in middle distance at right. Numbers refer to the same trees shown also in figure 9.

Figure 8.--Coal dock site, Anchorage, Alaska, June 27, 1918. Photograph by Alaskan Engineering Commission. Note fallen trees at toe of flow-slide in the middle distance at right.

Figure 9.--View of Anchorage harbor from Government Hill. Photograph by Alaskan Engineering Commission, Nov. 7, 1921. Note that the coal storage bunker has largely been removed. The numbers refer to the same trees as are shown in figure 7.

Figure 10.--Looking towards the terminal yards from end of temporary trestle at high tide, Aug. 20, 1918. Photograph by Alaskan Engineering Commission.

Figures 11-19.--Vertical aerial views of the port area, Anchorage, Alaska, 1942-67. Photographs from various sources have been brought to a common scale of about 1,000 feet to an inch by enlargement of parts of single prints or reduction of uncontrolled mosaics. Distance between ticks on bar is about 5,000 feet; bar is oriented about N-S. Date of photography is shown on print.

Figure 22.--Looking northeastward over the south end of the port area from above Ship Creek, after the earthquake of March 27, 1964. Cracks in the filled areas are mostly parallel with the shoreline as it was before filling. Note cracks at the base of slope, to the right of the power poles near the warehouse in the upper right corner of the photograph. Boring Line 1 passes down the slope and about midway through the warehouse. U.S. Army photograph.

FIG. 1

FIG. 2

FIG. 5

Fig. 6

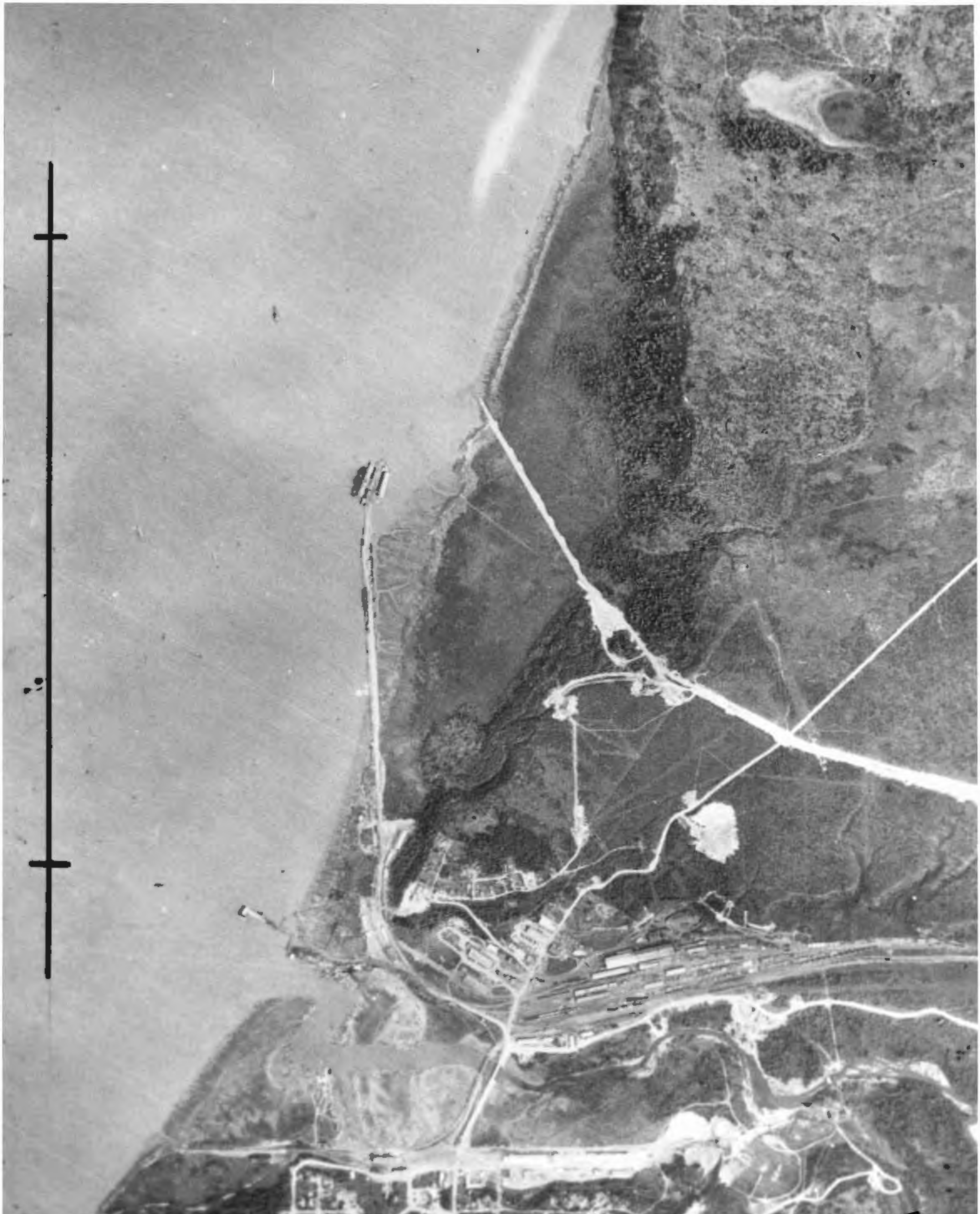
Fig. 7

FIG. 8

FIG. 9


LOOKING TOWARDS THE TERMINAL YARDS FROM END OF TEMPORARY
TRESTLE AT HIGH TIDE.
A.E.C.
G-903

FIG. 10



7 AUG 1942

FIG. 11

25 March 1948

FIG. 12

8 Aug 1950 FIG. 13

18 June 1954

FIG. 14

17 MAY 1962

FIG. 15

30 March 1964

FIG. 16

1 June 1964

FIG. 17

Aug 29 + Sept 2 1966

FIG. 18

2 MAY 1967

FIG. 19

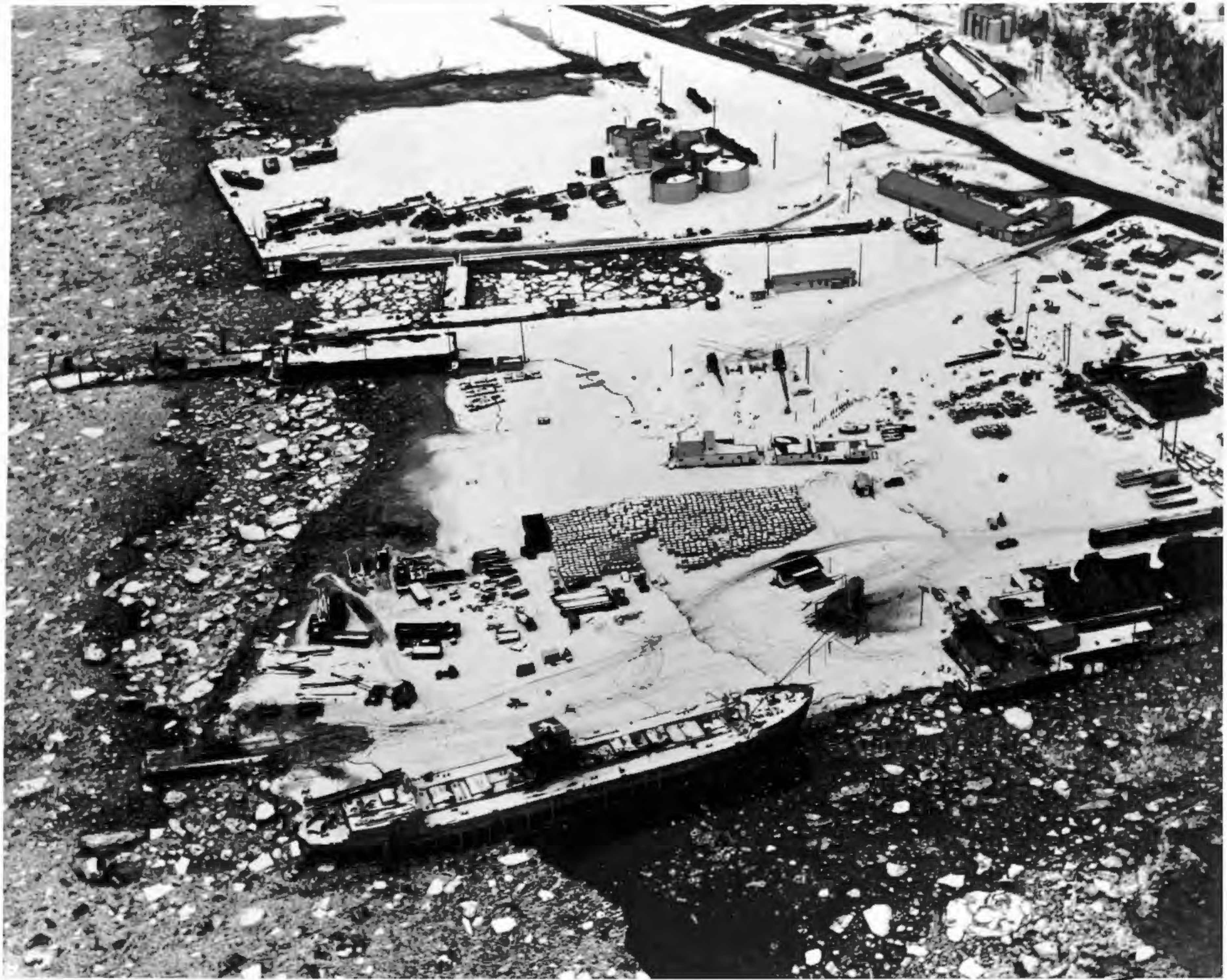


FIG. 22