THE STABILITY ANALYSIS OF A FOUNDATION FAILURE

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At the intersection of Jamaicaway and Huntington Avenue, in Boston, a highway overpass was constructed in 1936. Shortly after the roadways were completed and opened to traffic, a settlement occurred in the approach fill at a point about 500 ft. from the overpass proper.

The overpass and approach fill are shown in plan in Figure 1. The slide is indicated by a broken line and occurred where the southbound roadway, to the south of the overpass, skirts Leverett Pond. For a length of 250 ft. along this roadway and an entering lane, there was

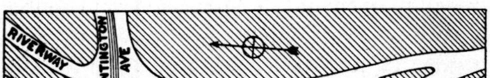

Figure 1. Plan of Overpass and Approach Fill

a settlement which in places amounted to 6 ft. In the pond, a little over 100 ft. to the west of the roadway, a small island was raised and greatly increased in size. Figure 2 is a photograph of the slide on the roadway looking toward the south, while Figure 3, also looking toward the south, shows the island. The immediate area of the slide is shown to larger scale in Figure 4.

Fay, Spofford and Thorndike, Consulting Engineers, were retained by the Massachusetts Department of Public Works to investigate, report and prepare plans for reconstruction. At the request of the consulting engineers, soil tests and stability investigations were made at the Soil Mechanics Laboratory at the Massachusetts Institute of Technology by the writer under the supervision of Dr. Glennon Gilboy.

Borings were made by the department both before and after the settlement. The latter disclosed that at a depth of about

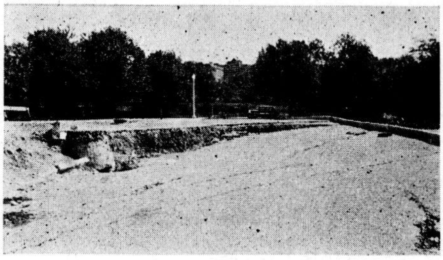

Figure 2. The Slide Looking South

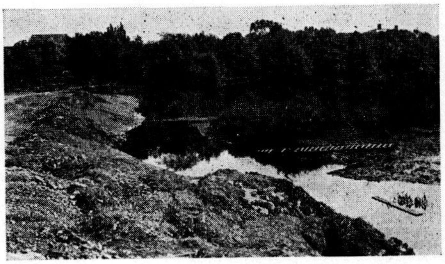

Figure 3. Island in Pond Affected by Slide

20 ft. below the area which settled, but extending only slightly outside the limits of this area, there was a pocket of peaty silt with a thickness of as much as 50 ft. Figure 4 gives the thicknesses of this stratum, the various dotted lines being lines of equal thickness. The numerous wash borings which had previously been made at the site of the overpass proper had given no indications of this dangerous stratum.

Along station 20 + 50 which is normal to the main roadway and approximately through the center of the slide, three undisturbed sample borings were taken, using 6-in. casing. The location of this section is shown in Figure 4 and the section itself in Figure 5. From each of these large borings four samples were

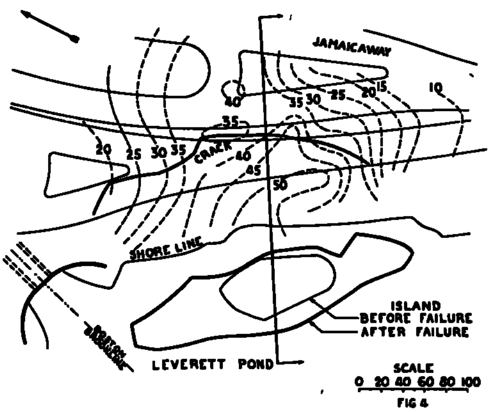

Figure 4. Plan of Slide

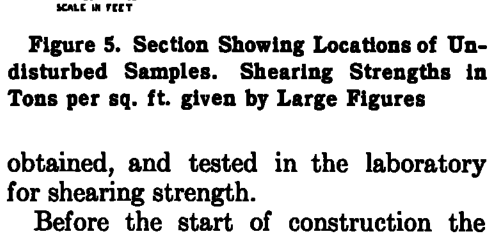

Figure 5. Section Showing Locations of Undisturbed Samples. Shearing Strengths in Tons per sq. ft. given by Large Figures

obtained, and tested in the laboratory for shearing strength.

Before the start of construction the overburden load on the stratum of peaty silt varied from 0.3 to 0.5 ton per sq. ft. The additional load caused by filling for the approach ramp amounted on the average to 0.6 ton per sq. ft. giving an ultimate overburden load of 0.9 to 1.1 ton per sq. ft. Shearing tests were run on all samples at a normal load equal to or slightly less than the overburden load before construction. It was found in this series of tests that there were only minor changes in shearing strength due to small changes in normal load and due to differences in speed of conducting tests. The shearing strengths at these light normal loads varied from 0.05 to 0.22 ton per sq. ft. and are indicated on the section in Figure 5.

A smaller number of shearing tests were run at a normal load of 1 ton per sq. ft., the approximate overburden load for the completed fill. These tests showed shearing strengths which were of the order of two or three times as great as the tests under the lighter normal loads. Consolidation tests were run on two samples and from the results of these tests it was estimated that if no lateral flow were to occur in the buried compressible strata, the settlement due to compression under the added overburden of 0.6 ton per sq. ft. would ultimately amount to about $3\frac{1}{2}$ ft. Of this settlement 20 per cent or 8 in. would take place in three months while more than 18 months would be required for 50 per cent or 21 in. to occur. This slow compression indicated that any increase in shearing strength caused by the added overburden would develop slowly. For this reason the shearing strengths determined under the smaller normal loads and given in Figure 5 were chosen to be used in the stability analyses. It is to be noted that a blind acceptance of the results of the shearing tests at 1 ton per sq. ft. normal load would have led to the use of an effective friction angle of at least 15 deg. which would introduce much higher shearing strengths than those shown on the figure.

The tests showed that the peaty silt had a number of unusual properties. Void-ratios of as high as 10, specific gravities of solid matter of less than 2 and unit weights submerged of as low as 5 lb. per cu. ft. were obtained. Shear test samples cut to a size of 3 in. square shrank after drying to less than 2 in. square.

Level records in the roadways outside of the slide area showed that settlements averaging somewhat over one half foot had taken place at a time shortly after the failure had occurred. While a part of this settlement undoubtedly occurred in the fill itself, this would appear to be in rough agreement with the settlements predicted from the consolidation tests.

The first stability analysis was very simple. The fill at Sta. 20 + 50 amounted to about ten feet of material weighing about 110 lb. per sq. ft., or a load of about 0.55 ton per sq. ft. The cross section of this fill may be represented in simplified form by the heavy dotted cross section shown in Figure 6. Dr. Leo Jürgenson[1] has designated such a section as a "terrace" loading and has given a solution of shearing stresses induced by such a loading on an elastic mass. The curves of Figure 6 represent maximum shearing stresses determined by this method, curves being shown for stresses of 0.10, 0.14 and 0.16 ton per sq. ft. The foundation under discussion was not truly elastic and these computed stresses must be recognized as a rough approximation. However, they are of considerable value as an indication of the conditions which existed. A comparison of these shearing stresses with the shearing strengths obtained by laboratory tests showed that the shaded area of Figure 6 was overstressed in shear and therefore was in a plastic state. Inspection of the figure makes it evident that when the amount of overstress in the shaded zone was thrown to the material outside, the size of the plastic zone must have been greatly increased and a failure was probable.

A more detailed stability analysis and one which has greater quantitative value may be made by the Swedish method of circular sliding surfaces. Referring to Figure 5 it is seen that the settlement at the left and the upheaval on the right must have been accompanied by a movement in the underground from left to right. In other words, there is rotation about a point approximately above the shore line of the pond. In the Swedish method the boundary of this rotating mass which is known as the rupture arc is assumed to be a circle, an approximation which has been justified by many field investigations. Two points on the arc of rupture are immediately evident, the scarp in the roadway and the outer edge of the island. In sections of this

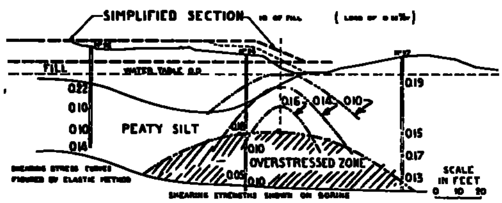

Figure 6. Section Showing Overstressed Zone as Indicated by Jürgenson's Terrace Loading Analysis

type, where the shearing strength does not increase with depth, rupture arcs tend to pass deep into the ground and therefore it follows that the low point of the arc will be near the bottom of the compressible stratum.

The rupture arc is indicated by a light dashed line in Figure 5. The Swedish method of analysis for this case is relatively simple because the location of the rupture arc is known. In contrast, when the stability of a proposed cross-section is being investigated, numerous trial arcs must often be used to determine which is the most dangerous and the procedure may become very tedious.

An average value for the magnitude of the shearing strength along the rupture arc was obtained by study of the data of

[1] L. Jürgenson, "The Application of Theories of Elasticity and Plasticity to Foundation Problems," *Journal* Boston Society of Civil Engineers, July 1934.

Figure 5. This value was chosen before any stability analyses were made and was taken as 0.10 ton per sq. ft. Because of the small number of samples and the variations in their shearing strengths the probable accuracy of this average value was low and it was admitted that it might possibly be in error by 50 per cent or more.

Figure 7 shows the cross-section at Sta. 20 + 50 as used for the first circular arc analysis. From all available level and boring records the cross-sections shown were determined for the situations before and after the failure. The water level in the pond was at Elev. 3 and the water table in the fill was assumed to be

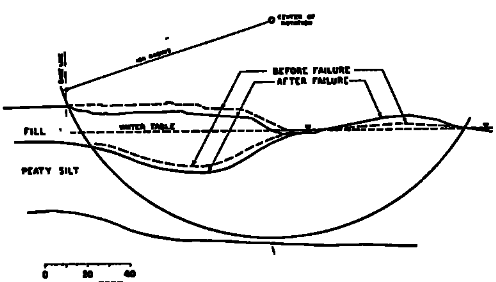

Figure 7. Section for Circular Arc Analyses

constant at this same level. This means that no seepage forces were taken into account but it is likely that in this section such forces would not be large enough to be of major importance. The length of rupture arc within the fill represents such a small portion of the total arc that no special mention will be made of the shearing strength determinations in this material.

The computations for this analysis were simple in principle. The weight of the sliding mass caused a tendency toward counter-clockwise movement about the center of the circle. To determine this moment, cardboard sections of the various portions of the mass were cut. These were weighed to give the areas and balanced on a sharp edge to determine the centers of gravity. The resisting moment was furnished by the forces corresponding to the shearing strength, acting at a moment arm equal to the radius of the rupture arc. The ratio between available resisting moment and counter clockwise dead load moment furnished a direct measure of the safety against failure and was used as a "factor of safety."

For the two sections of Figure 7, the results obtained were as follows:

Before failure,
Factor of Safety = 0.67
After failure,
Factor of Safety = 0.93

Inasmuch as the actual factor of safety must have been unity or slightly greater just after failure, it appears that the value used for average shearing strength must have been on the conservative side by about 10 per cent. This of course does not mean that all data used were accurate within 10 per cent as there may have been numerous compensating errors. However, it was very encouraging to find that the value obtained by laboratory tests was in such good agreement with the full scale failure test in nature.

Other analyses were made at Sta. 20 + 00 and 21 + 00, assuming the same average shearing strength, and slightly higher factors of safety were obtained. Results at Sta. 20 + 50 thus appear to be typical for the slide. Some arching action must have been introduced by the friction on the sides of the bowl-shaped sliding mass, but its effect may be considered to have been accounted for when it was found that the average shearing strength adapted in the computations led to a factor of safety of about unity.

In the redesign, three revisions were introduced which greatly improved the stability conditions. First, a realignment of the southbound roadway was made which moved it 34 ft. further from the pond. Secondly, a large portion of the area between the southbound roadway and the pond was lowered to Elev. 7 where originally it had been at Elev. 15. This change introduced a large relief in load. Thirdly, fill to Elev. 7 was added connecting the island to the shore, thus moving the shoreline to the outer edge of the island. Fill in the vicinity of the island constituted a load which assisted in balancing the load on the roadway side of the failure mass. This balancing load, however, could be used only to a limited degree as a heavy fill in the vicinity of the island would be very likely to cause a failure further out in the pond, on an arc displaced to the right from the one analyzed.

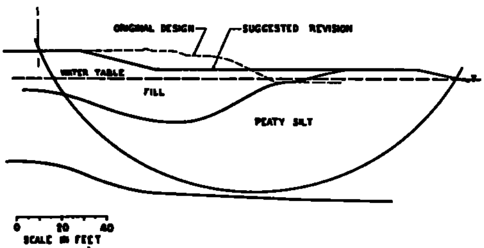

Figure 8. Section Showing a Suggested Design Revision

A design incorporating these three changes is shown in Figure 8. When analyzed on the original surface of rupture this section shows a factor of safety of 1.10. This design, however, was not given serious consideration. Large settlements would be obtained in the roadway due to compression of the peaty stratum. Also a failure along a rupture arc further toward the pond was possible. Most important of all, the publicity which the slide had caused made it imperative that even the most remote possibility of another failure must not be allowed. Thus even though it is probable that the odds were in favor of this section being stable, it was not adopted.

A design introducing a viaduct on steel bearing piles driven through the compressible stratum to the underlying firm stratum was proposed by the consulting engineers and met with favor because of its eliminating any possiblity of further subsidence on the highway. Such a structure, however, did not materially decrease the danger of failure in the area toward the pond from the roadway. In the section of Figure 9, an analysis along the arc shown gives a factor of safety of 1.10. This factor was increased to 1.23 by decreasing the load by the use of cinder fill on the slope adjacent to the roadway. These analyses made use of the previous average

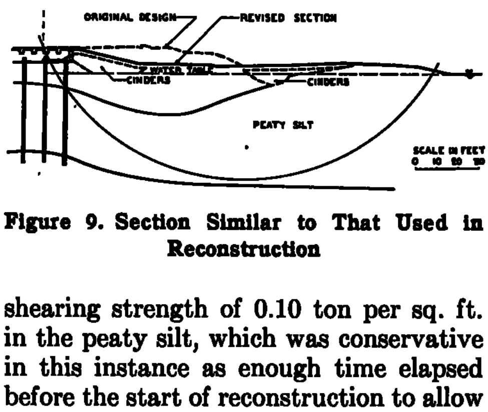

Figure 9. Section Similar to That Used in Reconstruction

shearing strength of 0.10 ton per sq. ft. in the peaty silt, which was conservative in this instance as enough time elapsed before the start of reconstruction to allow some consolidation. The use of cinder fill just under the center of rotation had no effect on the factor of safety along the arc analyzed in Figure 9 but was adopted as it greatly reduced the tendency of failure along arcs with centers of rotation further to the right.

The reconstruction was carried out using a section essentially similar to that of Figure 9. The completed structure was opened to traffic in the Spring of 1938 and there have been no indications of further subsidence.