

59-F-262C

AZATIKA CREEK

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DIVISION OF BUILDING RESEARCH

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SPECIAL REPORT NO. 81

TITLE Failure of the Approach to the Assatika Creek Bridge

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DATE REQUESTED

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APPROVED

DIRECTOR

SUMMARY

This report presents the test results from three Shelby tube samples and from two field vane borings at the site of a bridge abutment failure on Assatika Creek, 5 miles west of 1st Orignal, Ontario. Based on these tests the stability of the approach fill has been analysed and certain recommendations concerning reconstruction are made.

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One of the best means of evaluating design procedures is by the study of full-scale failures in the field. It was of considerable interest therefore to the Soil Mechanics Section of the Division of Building Research, National Research Council, to hear of the failure of an approach fill and bridge abutment over Asatika Creek, five miles west of 1st Orignal, Ontario. The authors visited the site of the failure on 3 September 1958, in the company of Mr. J.D. Paterson, Consulting Engineer, Ottawa and Mr. Jean Morin, County Engineer, Plantagenet, Ontario. On 7 and 8 October 1958, a brief field study was made by the DNR Soil Mechanics Section.

Failure Conditions

The bridge in question is a standard timber truss type of 60-foot span resting on U-shaped reinforced concrete abutments. Each abutment is founded on twenty-eight timber piles, each 35 feet long and driven to refusal. The only soil investigation carried out before the driving of piles was the driving of two probes to determine the length of piles required.

After the abutments were in place, the bridge was assembled and the construction of the south approach fill was begun in early June 1958. The west side was brought up to grade and the east side was being widened when it sank suddenly about 7 feet. A second failure of lesser magnitude occurred when further filling was carried out.

The north approach fill was then constructed. After three days of traffic, and while the west side of the north approach fill was being widened, the west side sank suddenly about 7 feet at a point about fifty feet north of the bridge abutment. The fill adjacent to the abutment sank about 3 feet. The failure lifted the old pile-bent bridge, immediately to the west of the new structure, about 3 or 4 feet. The failure cracked the west wing wall of the abutment.

Following this failure, the soil was excavated from behind the north abutment and tie rods installed between the two parallel wing walls. The approach fill was again brought up to grade but a further failure towards the old bridge occurred. This failure shifted the north abutment, indicating that the piles had been broken. After some further filling in the vicinity of the abutment, it rotated about the bridge seat and the outside ends of the wing walls settled about 6 feet.

The abutment failure is shown in Fig. 1(a). The distortion of the old bridge is shown in Fig. 1(b). Figure 2, a contour plan of the north abutment, shows the shift of

the abutment off-centre and the elevations of the area after failure. These failures had occurred before the site was inspected by R.C. Gauthier and J.D. Paterson, Consulting Engineers, on 10 June 1958. Early in July, the earth against the south abutment was removed as a precaution against further failure.

Site Conditions

a) General

The geology of the area consists of a faulted limestone bedrock plain with a mantle of glacial till and Leda clay and with a shallow surface layer of sand in some areas. The Leda clay is the major soil deposit and is featured by terraces. A study of the surface topography in the vicinity of Asatika Creek and Asatika Bay and of aerial photographs of the region illustrates fairly clearly that the entire area has been involved in a huge landslide during geological history. The slide area covers about 15 square miles and has affected the south shore of the Ottawa River over a length of about 8 miles. The lower portion of Asatika Creek, including the bridge site, is probably the drainage system which developed immediately following the slide. The creek is reported to experience severe flood conditions during spring runoff. The adjacent soils would be expected to be alluvium over Leda clay partly remoulded by the landslide.

b) Borings and Tests

Two vane borings were carried out at the north approach as shown on Fig. 2, (V3 and V4). Three soil samples were obtained using a thin-walled piston-type sampler in boring S-2 as located on Fig. 2. One sampling boring and two vane borings were attempted adjacent to the south approach fill but, with the hand equipment available, it was impossible to penetrate a fairly shallow sand layer.

Results of in situ vane shear tests and of unconfined compression and classification tests on the tube samples are shown on Fig. 3. The results of vane borings V3 and V4 and of samples from boring S-2 show a fairly strong layer from the surface at elevation 68 down to elevation 81. An organic silty-clay layer occurs from elevation 81 to 77. It has a rather high water content, a high liquid limit and an undrained shear strength of 400 to 800 pcf. From elevation 77 to 74 there exists a very soft grey clay, with natural water content 70 to 90 per cent, liquid limit 98 per cent, plastic limit 32 per cent and shear strength of 150 to 300 pcf. Below elevation 74 is a hard sand layer through which sampling and vane testing could not be carried out with available equipment. All elevations are relative to the top of the

south abutment assumed to be at elevation 100 feet.

Vane test V4 was carried out through the existing fill. The results of this vane test would suggest that the very soft clay layer from elevation 74 to 77 had been removed by the failure.

Stability Analysis

The section shown on Fig. 2, just north of the north abutment, was chosen for stability analysis since the first bank failure on the north side occurred at this approximate location and soil test results were available. It was thought that the analysis of a section including the abutment would be too complicated owing to assumptions regarding the restraining influence of the bridge and the piles. The cross-section under study is shown in Fig. 4. Fill dimensions are assumed to be according to design drawings, the original ground surface was selected on the basis of field observations, and the ground water table after construction was assumed to be along the original ground surface. The stability of several potential sliding arcs was analysed using average shear strength values as shown on Fig. 4 and using a fill density of 100 pounds per cubic foot and a submerged density of the natural soil of 55 pounds per cubic foot.

In the analysis, the fill material was assumed to contribute to the driving moment but its strength was not considered to contribute to the resisting moment. This is a common assumption where fresh clay fills are involved in sliding failures. A minimum factor of safety of 1.03 resulted from this analysis. Since the actual factor of safety at failure is 1, this is a satisfactory evaluation of failure conditions.

Because of the extremely soft layer of soil beneath the fill, a composite sliding surface, involving three circular arcs, was analysed as shown in Fig. 5. This analysis gave a factor of safety of 1.14. An infinite number of composite sliding surfaces could be analysed but it was not thought worthwhile to pursue this course.

Considering the various assumptions of fill dimensions, original contours, soil densities, and average shear strengths in the natural ground, the establishment of a minimum factor of safety of 1.03 is fortuitous. Nevertheless, it confirms the usefulness of analysing "end of construction" safety using a $\phi = 0$ method of analysis in these materials.

Recommendations

The failure of this bridge structure was obviously due to a failure in the approach fill. To avoid failures of this type, it is desirable to carry out stage construction in which the fill is raised in level lifts and a period allowed for the soft subsoil to consolidate and increase in shear strength under each lift. In this way, the fill can be built up safely over a period of time to the final grade. Alternatively, the fill adjacent to the bridge abutment may be replaced by a series of pile bents and timber decking, thus eliminating the weight of the fill as a cause of failure.

In this particular case, there appears to be no problem with the south abutment but it would be a worthwhile precaution to avoid backfilling around the abutment at this time. The span from abutment to fill is so short that a timber decking from the abutment to a prepared sill on the embankment would be satisfactory. If no further movements in the south approach fill occur during the year after construction, it may be considered safe.

At the north end, the abutment will probably have to be removed since it has obviously been moved off its piles and may undergo a considerable amount of settlement. This could be checked, since elevations were taken on the abutment in October 1958. Assuming that a new abutment can be built on piles, there is still the question of whether to use an approach fill. If further borings were made, it would be possible to establish with some assurance whether the soft clay was removed over an appreciable area by the failure. If this is the case, a new approach fill could be considered. Otherwise, it would be advisable to carry an extended decking on pile bents from the bridge to the existing stable fill.

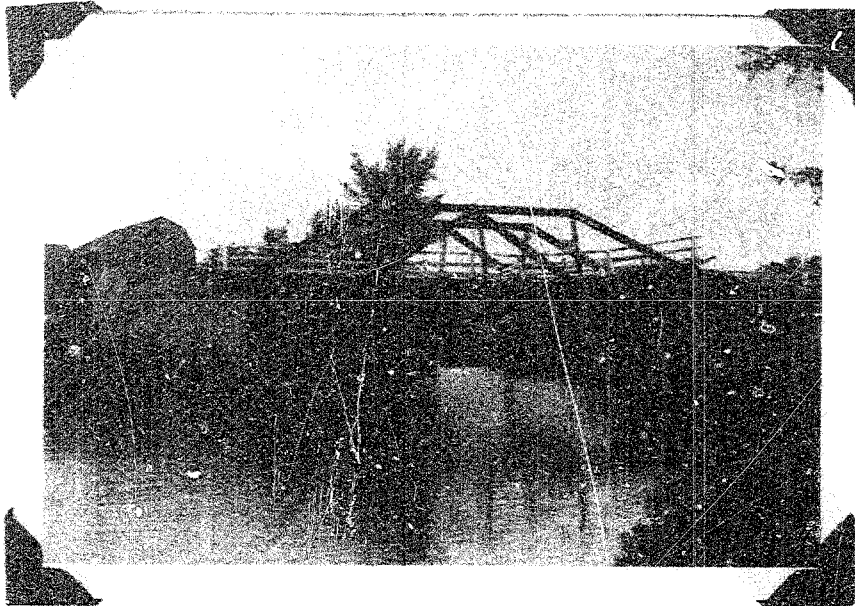


Fig. 1(a) Abutment failure

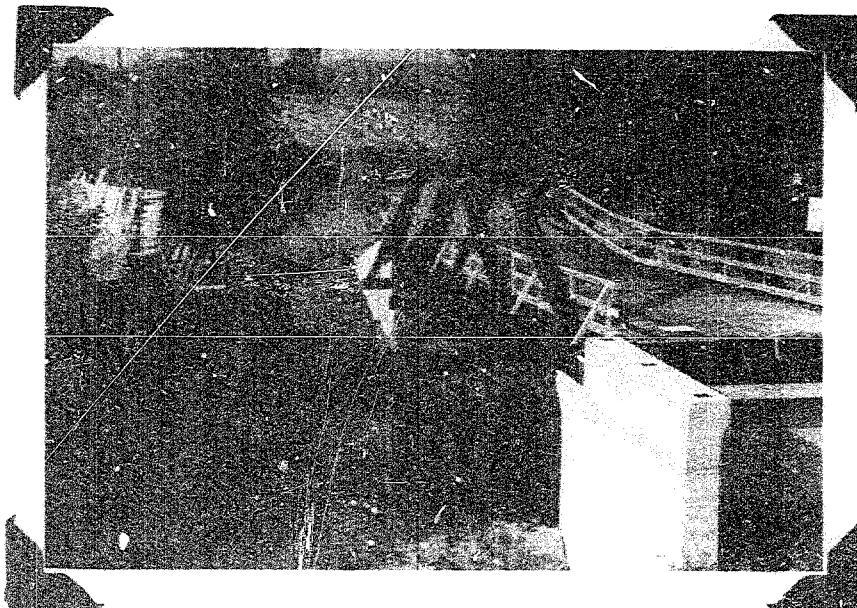
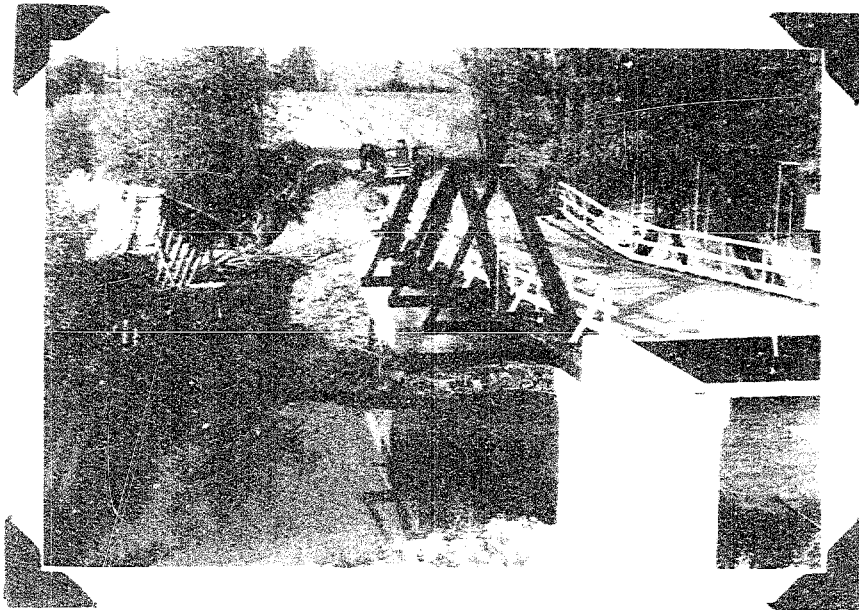
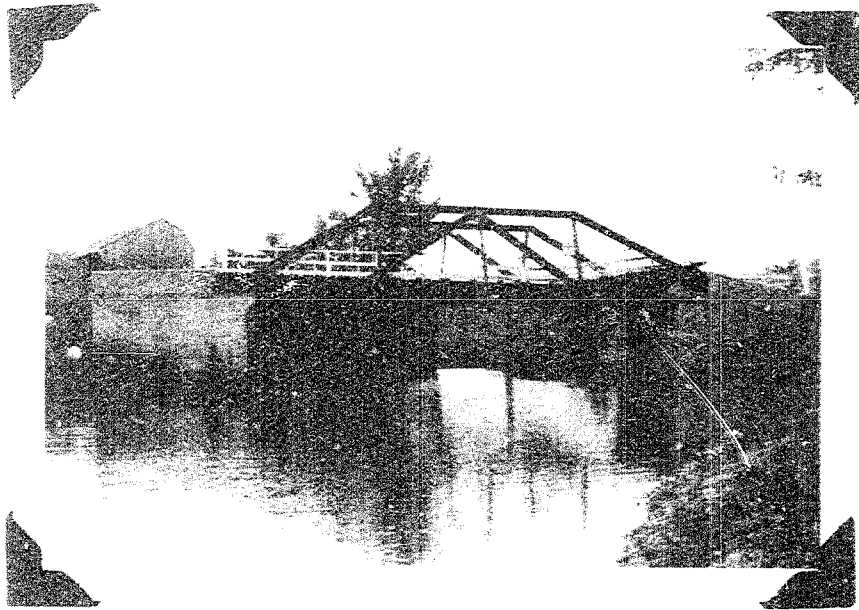


Fig. 1(b) Distortion of old bridge



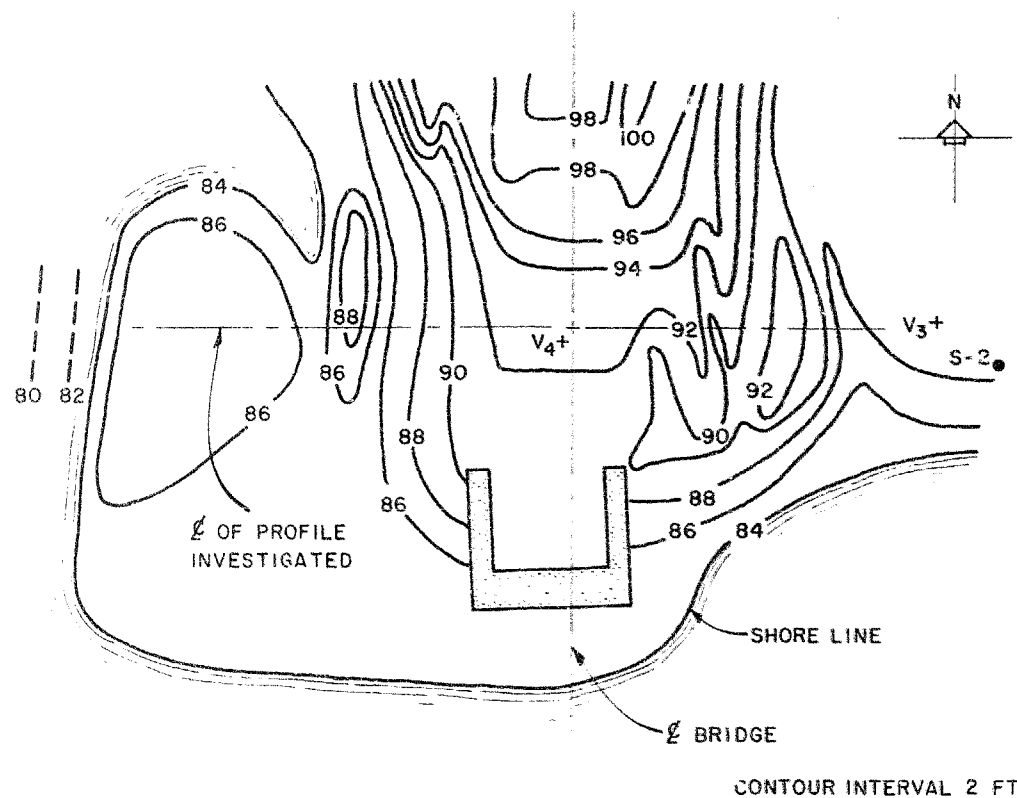


FIGURE 2 PLAN VIEW OF NORTH ABUTMENT

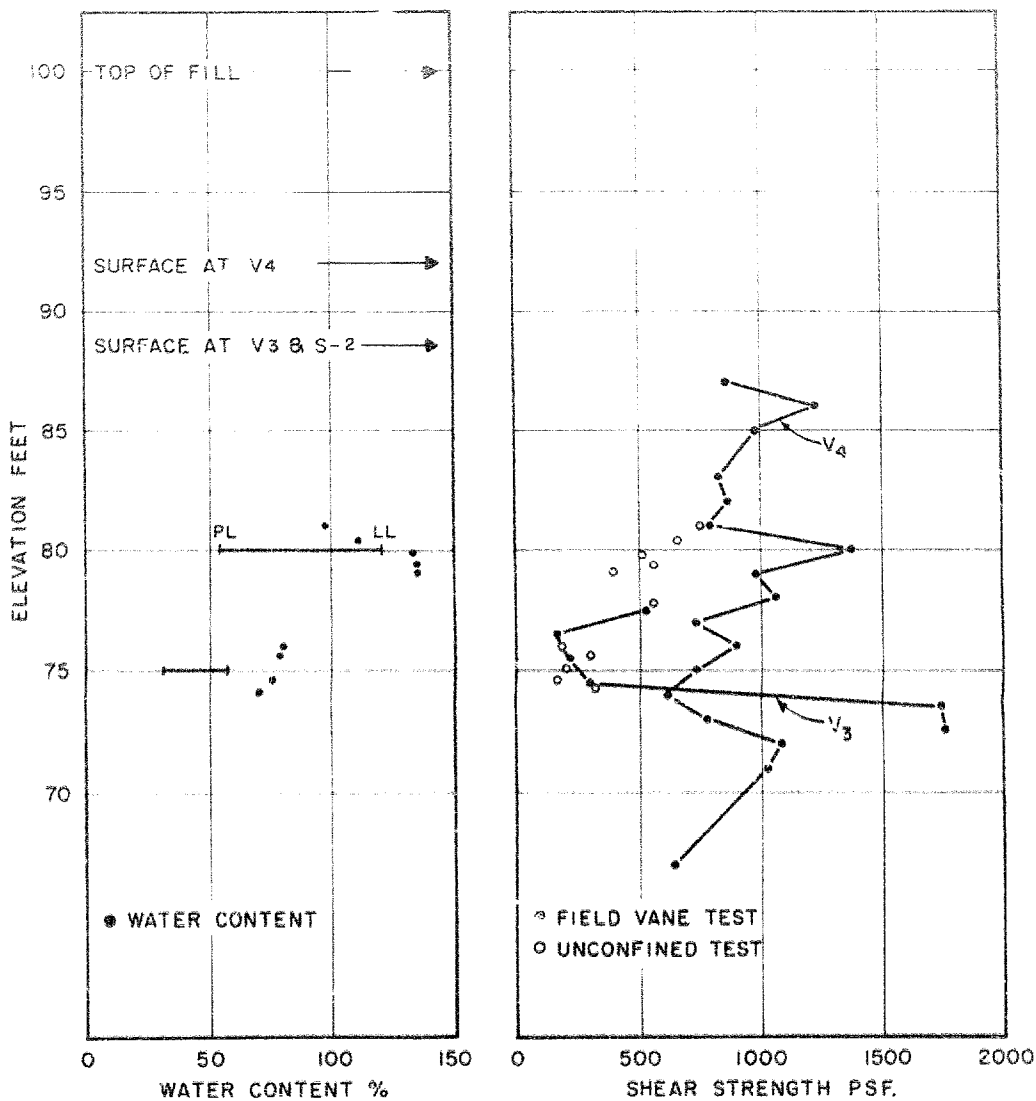


FIGURE 3
AZATIKA CREEK BRIDGE SOIL PROPERTIES
BORINGS V₃, V₄, & S-2

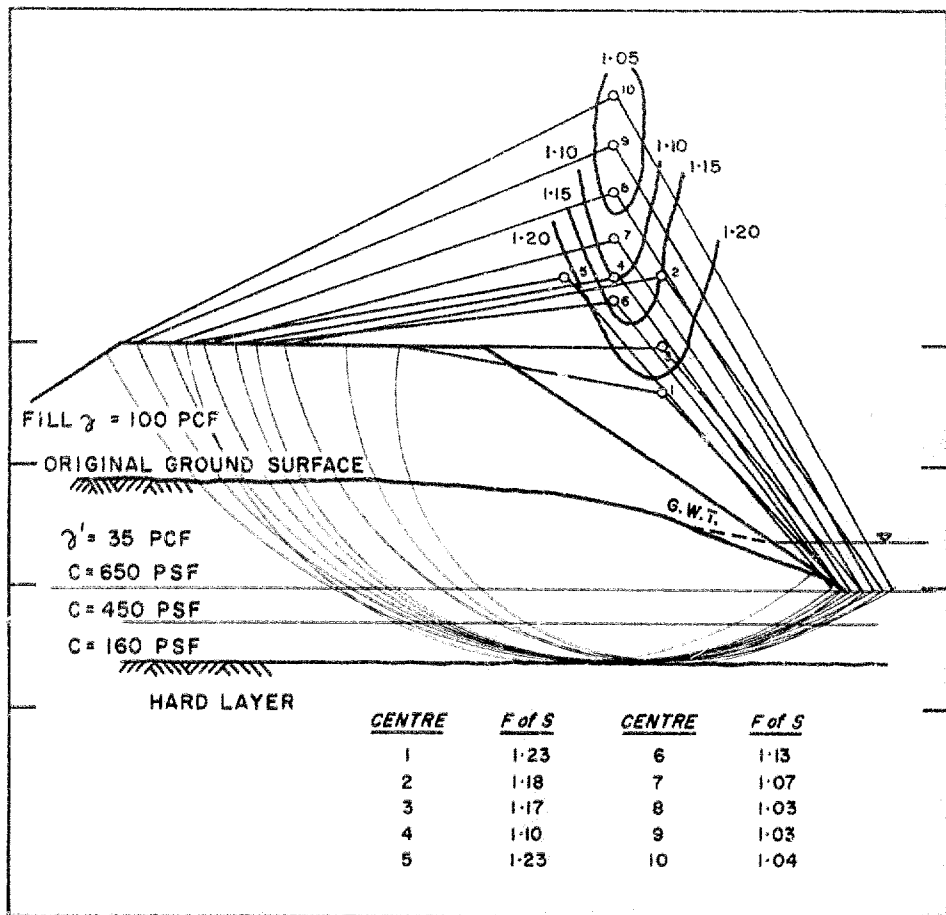


FIGURE 4
STABILITY ANALYSIS OF CIRCULAR SECTION

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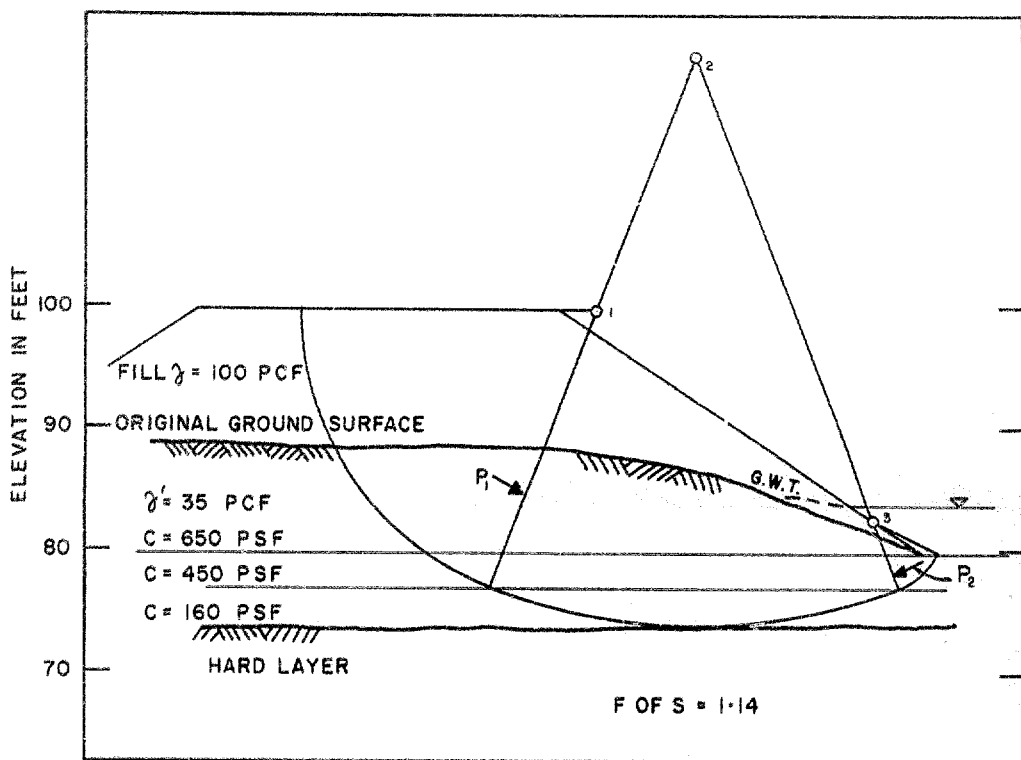


FIGURE 5
STABILITY ANALYSIS OF COMPOSITE SECTION