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Abstract

Identification of landslide prone sites in Maine is imperative in order to maintain the safety of affected developed areas and future developments. Large landslides in Gorham, Maine in 1983 and Rockland, Maine in 1973, a number of smaller landslides, and evidence of historical and prehistorical landslides indicate that some areas of Maine, including developed areas will be susceptible to future landslides. Past landslides have occurred in deposits of the sensitive glaciomarine clay known as the Presumpscot Formation which covers the populated and rapidly developing eastern and southern Maine.

This paper examines two southern Maine sites - one at Brunswick and another at Westbrook- both of which have been subject to mass movements in the past and are suspected to be vulnerable to future mass movements. The landslides in the Brunswick area are single slides, whereas past slides in the Westbrook area included major flowslides such as the one which occurred in November, 1868, affecting an area of thirty-four acres. The flow or spreading slides, such as have occurred in Westbrook, are potentially far more destructive. This paper examines the geological characteristics and soil properties at the two sites with the purpose of identifying the causes of landslides, particularly flowslides or landslides that spread by retrogression. Historical investigations, subsurface investigations, laboratory testing and engineering slope stability analyses were performed in examining the two sites.

Although the mineral constituents and pore fluid of the deposits at each site were similar, the different structure of the deposit at each site led to the different stability behavior. The material at Westbrook had a higher water content, lower undrained peak shear strength, and a higher strength loss upon remolding than the deposit at Bunganuc bluff. Higher water content, lower peak strength, and higher strength loss have been related elsewhere to flow slides or retrogressive slides.

It is recommended that sites in Maine with deposits exhibiting similar geotechnical, geological and geomorphological characteristics as Westbrook and Bunganuc bluff be identified for possible site stability problems. It is also recommended for sites with possible stability problems that considerations of disturbance, loading and erosion by construction or natural causes be integrated into maintenance, planning and development decisions for these areas.

Introduction

Much of southern Maine's infrastructure and many of its residential communities are founded on the glaciomarine silty clay known as the Presumpscot Formation. This material has long been prone to geologic mass wasting processes, more popularly referred to as landslides (Amos and Sandford, 1987; Morse, 1869; Novak, Swanson, and Pollock, 1984; Novak, 1987). The present rapid growth of the area is a concern as development pushes into potentially landslide prone sites. This paper attempts to shed light on identification of potential sliding areas and on some natural and development-related mechanisms involved in causing landslides.

In this study funded by the U.S. Geological Survey - Office of Earthquakes, Volcanoes and Engineering, two developed sites, both of which have been the sites of landslides in the past, were investigated by means of field and laboratory testing and by engineering analyses. The first site is a receding coastal bluff located near Bunganuc Point in Brunswick, Maine. Houses have recently been constructed here for the view of the bay afforded from the top of the bluff. The characteristics of the topography, soil profile, and bluff failure pattern have previously been reviewed by Amos and Sandford (1987). In the present investigation, the stability of the bluff under various conditions was analyzed using the results of laboratory and field testing. The second site is located along the Presumpscot River in Westbrook, Maine in the vicinity of major historical landslides and is the site of a major housing development (more than 100 houses and a variety of underground utilities). The safety implications of new construction or natural disturbance on slope stability are investigated by means of a topographic survey, field and laboratory testing, and engineering stability analyses.

The Presumpscot Formation

Geologic History

The Presumpscot Formation is a glaciomarine silty clay found in the coastal lowlands of Maine and extending inland along the Penobscot and Kennebec River valleys (Thompson, 1987). It is a late glacial marine sediment consisting of clay, silt and fine sand-size particles deposited in the periglacial environment as the Laurentide ice sheet receded between about 14000 and 12000 years ago. As the ice sheet margin receded, isostatic crustal rebound resulted in the relative uplift of the Presumpscot sediments above the marine limit. The depositional environment and subsequent uplift have had a considerable effect on the geotechnical properties of the Presumpscot Formation.

Stratigraphy

Belknap et al. (in press) suggested the occurrence of three distinct forms of the Presumpscot sediments. A basal unit (GM-M) consisting of homogeneous to poorly stratified sediments draping over till and bedrock is interpreted as a sub-ice-shelf deposit in deep local basins. The basal unit is always overlain by a second unit (GM-D), which also drapes the topography and consists of a well stratified alternation of sandy silt and clay. It is interpreted as a proximal glaciomarine sediment, and the draping nature is indicative of rapid sedimentation from suspension. Immediately above the second unit a third unit (GM-P) is recognized to consist of a relatively homogeneous mass of silt and clay. This unit is interpreted as being deposited on the distal side of the calving ice shelf in ponded, sometimes discontinuous, lenses. Uplift of the deposits has resulted in lowered ground water levels yielding a surficial desiccated, weathered stiff brown crust from 0 to 20 feet thick. Underlying this crust and below the groundwater table the unweathered sediments are generally softer and often gray or blue-gray in color.

Since the deposits at Bunganuc bluff have been identified by Belknap et al. (in press) as belonging to the second unit (GM-D) and it appears that the deposit at Westbrook belongs to the third unit (GM-P), engineering properties and behavior typified by these two deposits may possibly be located elsewhere in Maine by identification of the geologic units.

Grain Size and Classification Properties

Grain size analyses from numerous sites have shown the variability of particle size distribution within these sediments. Reported grain size distributions are summarized by Amos

and Sandford (1987) as follows: fine sand (0.4 - 0.7 mm), 0 to 35%; silt (0.07 - 0.002 mm), 20 to 55%; and clay size (< 0.002mm), 35 to 75%. The mineral composition of the silt and clay size fraction has been found to consist of varying proportions of illite, chlorite, quartz and feldspar (Lambe and Martin, 1954, 1956; Mitchell, 1956; Kelley, 1988).

For engineering purposes knowledge of the exact grain size or mineral composition is not a useful measure of behavior, since the nature of the adsorbed ions, organic content, and pore fluid also affect behavior. Fine grained soil classification is most easily facilitated by readily obtained indices, such as the Atterberg limits, which are a function of grain size, mineral composition, and the depositional environment of the sediment. One limit, the liquid limit (LL), indicates the water content at which the soil consistency changes from a plastic state to a slurry state. Typical values of the liquid limit for the Presumpscot are 25 to 41 (Andrews, 1987). When the natural water content of the formation exceeds the liquid limit as it sometimes does in the Presumpscot, then the soil, upon disturbance, can flow as a viscous liquid. To quantify the relative position of the natural water content to the Atterberg limits, the liquidity index (LI), defined as $(w - PL) / (LL-PL)$ where w is the natural water content and PL is the plastic limit, is used. The Presumpscot clay typically has a plastic limit (PL), defined as the water content bounding the semisolid and plastic states, in the range of 15 to 25 (Andrews, 1987). A liquidity index greater than 1.0 describes a soil whose natural water content is above the liquid limit. The difference between the liquid limit and the plastic limit is refered to as the plasticity index (PI) and is the range of water contents over which a material behaves plastically. Typical PI values for the Presumpscot Formation (Andrews,1987) indicate a silty clay near the upper bound (PI = 20) grading into a clayey silt near the lower bound (PI = 6).

Engineering Properties

General

The most important engineering property affecting the stability of slopes is the shear strength. The undrained peak shear strength, remolded shear strength, and drained shear strength affect the behavior of slopes. For this project it is desired to relate the measured strengths to specific field behavior and attempt to use this as a basis for predicting behavior elsewhere. Index properties and geology will be used as indicators of the available strengths. The index properties and geology will then be used to identify problem areas existing elsewhere in deposits of the Presumpscot Formation.

Undrained Shear Strength (Peak)

For the soft Presumpscot clay, most all shear strength values for design are obtained from

the field vane shear test rather than by laboratory testing as is common practice for many clay deposits elsewhere. Typical values of the peak undrained shear strength measured by the vane test for the Presumpscot Formation range from 170 to 2400 psf. These values have been taken on a site by site basis with no attempt to generalize on the areal distribution of particular strength properties or the correlation to classification properties of the Presumpscot. Typical examples of Presumpscot strengths have been published by Poulos et al. (1982), MSHC (1969) and Sandford and Amos (1987). Undrained shear strength implies that shearing occurs rapidly enough to preclude drainage and the dissipation of excess pore water pressures generated during shear. The undrained shear strength is lower and thus more critical than the strength available after drainage has occurred.

Consolidation under an overburden increases the peak undrained strength of a clay deposit. A normally consolidated deposit, in which the effective overburden pressure existing at present is no less than it has ever been in the past, shows an increase in strength with depth related to the overburden pressure. This relation, termed the c/p ratio, where c is the undrained strength and p is the effective overburden pressure, has been found to correlate well with the plasticity index of the soil (Skempton, 1957). The c/p ratio for a given deposit will show variability dependent upon the method used to determine the undrained shear strength. Typical values of the c/p ratio in the Presumpscot clay range from as low as 0.13 to greater than 0.40 (Andrews, 1987).

In the geologic past a deposit may have been subjected to pressures greater than the existing overburden pressure resulting in a greatly increased peak shear strength. The maximum past pressure, or its equivalent, can be measured in the consolidation test and if it is greater than the present overburden pressure the soil is termed overconsolidated. The degree of overconsolidation is quantified by the overconsolidation ratio (OCR) which is defined as the ratio of the maximum past pressure to the existing overburden pressure. In addition to erosion of the overburden and secondary compression, weathering and capillary stresses from a fluctuating groundwater level result in the overconsolidation of a soil. The upper crust in the Presumpscot clay is always overconsolidated, apparently from weathering and groundwater fluctuations, thus yielding higher strengths than the softer strata beneath it. Secondary compression is a poorly understood phenomenon in which consolidation occurs at a constant effective stress, resulting in the overconsolidation of deposits thought to be normally consolidated.

Remolded Shear Strength

After straining, the undrained shear strength drops sharply until it reaches the remolded

shear strength where it becomes constant and independent of strain. The remolded strength affects the behavior of the shear failure. Massive landslides have occurred in Norway and the St. Lawrence valley of eastern Canada, and these landslides can be attributed to the large loss of strength upon disturbance that occurs in the sensitive clays found in these regions. The occurrence of these massive landslides is relevant to the Presumpscot Formation since Leda clay (of eastern Canada) and Norwegian marine clays share a common depositional and post- depositional uplift history with Maine clays.

The sensitivity of a clay, which is defined as the ratio of the undisturbed undrained shear strength to the remolded undrained shear strength, is usually determined by the field vane shear test. Rosenqvist (1953) suggests the system shown in Table 1 to describe the sensitivity of a clay. Presumpscot Formation deposits show sensitivities ranging from slightly sensitive to medium quick when this criterion is applied. Sensitivities considered extra quick have been measured on Leda clay, Swedish clays, and Norwegian marine clays, which all share a similar geologic history with the Presumpscot Formation. Bjerrum (1954) has found the senstivity to correlate well with the liquidity index of Norwegian marine clays such that as the LI increases so does the sensitvity.

Table 1
Sensitivity classification

Classification	Sensitivity
Insensitive	1
Slightly Sensitive	1 - 2
Medium Sensitive	2 - 4
Very Sensitive	4 - 8
Slightly Quick	8 - 16
Medium Quick	16 - 32
Very Quick	32 - 64
Extra Quick	> 64

The cause of the high sensitivity often exhibited by Norwegian clays has been related to the uplift of these glaciomarine sediments above the marine limit. Fresh water leaching has reduced pore water salinity from an assumed 35 g/l immediately after deposition to as low as 0.2 g/l today (Bjerrum, 1954). This change in pore water chemistry is indicated by the Atterberg limits since the resulting cation exchange produced by leaching has lowered the liquid limit of the Norwegian

clays, while little change has been observed in either the plastic limit or the natural water content (Bjerrum, 1954). These changes have resulted in an increased liquidity index since the time of deposition and, when Bjerrum's correlation is considered, an increased sensitivity.

Effective Shear Strength (Drained)

The effective shear strength parameters are rarely measured for the Presumpscot Formation, but Amos and Sandford (1987) found an effective angle of internal friction, ϕ', of 35.6° and a cohesion intercept, c', of 0, for a sample obtained from the site of the 1983 Gorham landslide. This relatively high value of the friction angle is indicative of low plastic clays, reflecting the large percentage of quartz and feldspar in the fine fraction. The value measured for the cohesion intercept on this sample reflects the normally consolidated stress history of the sample. The effective shear strength parameters govern the long term stability of slopes in which excess pore water pressures are allowed to dissipate (i.e., drainage occurs).

Bunganuc Bluff

Background

Bunganuc bluff is located on Maquoit Bay in Brunswick, Maine (Figure 1) and is one of numerous bluffs composed of the Presumpscot Formation which exist along the Maine coast. The bluff rises about 40 feet above the tidal flats and extends a distance of approximately 2200 feet along the bay between two bedrock outcrops. Numerous slumps have occurred along the bluff. One such slump (Photo 1) was investigated by Amos and Sandford (1987), and it is this same site which is treated in this report (Photo 2).

The rate of recession of bluffs formed by landslides, or slumps, along the coast is controlled by the coastal bluff recession process considered by Kelley and Belknap (1987). The numerous slumps and slump debris at Bunganuc bluff demonstrate the consequences of these processes. Bluff recession is the result of marine erosion by wave action at the toe of the bluff (Photo 3) and possible subaerial erosion which oversteepens the bluff until it fails. The slumped section achieves a more stable configuration after failure and will remain as such until it is eroded. Natural processes, such as the development of salt-water marsh vegetation in the slump debris, affect the rate of erosion of the toe.

Topographic Survey

A topographic survey was conducted in July, 1987 at a section along the bluff close to one previously surveyed in October, 1985 (Amos and Sandford, 1987) to determine if further steepening had occurred since the first survey. The 1987 section is shown in Figure 2, and for comparison the earlier survey is shown in Figure 3. The new section has an angle of inclination with the horizontal of about 51°, which amounts to a steepening of 3 to 4° from the previous survey. This steepening implies that up to four feet of erosion has occurred at the toe in the two year time span between surveys. However, since the same section was only approximately reestablished, this amount of erosion can only be considered an estimate.

Stratigraphy

Since only surficial samples were collected in the investigation at this bluff in 1986, it was desired to obtain strengths on material not directly affected by weathering at the face of the bluff. Accordingly, a boring was located behind the bluff to perform field testing and sampling.The exploratory boring was drilled vertically into the subsurface and located a distance of 32 feet from

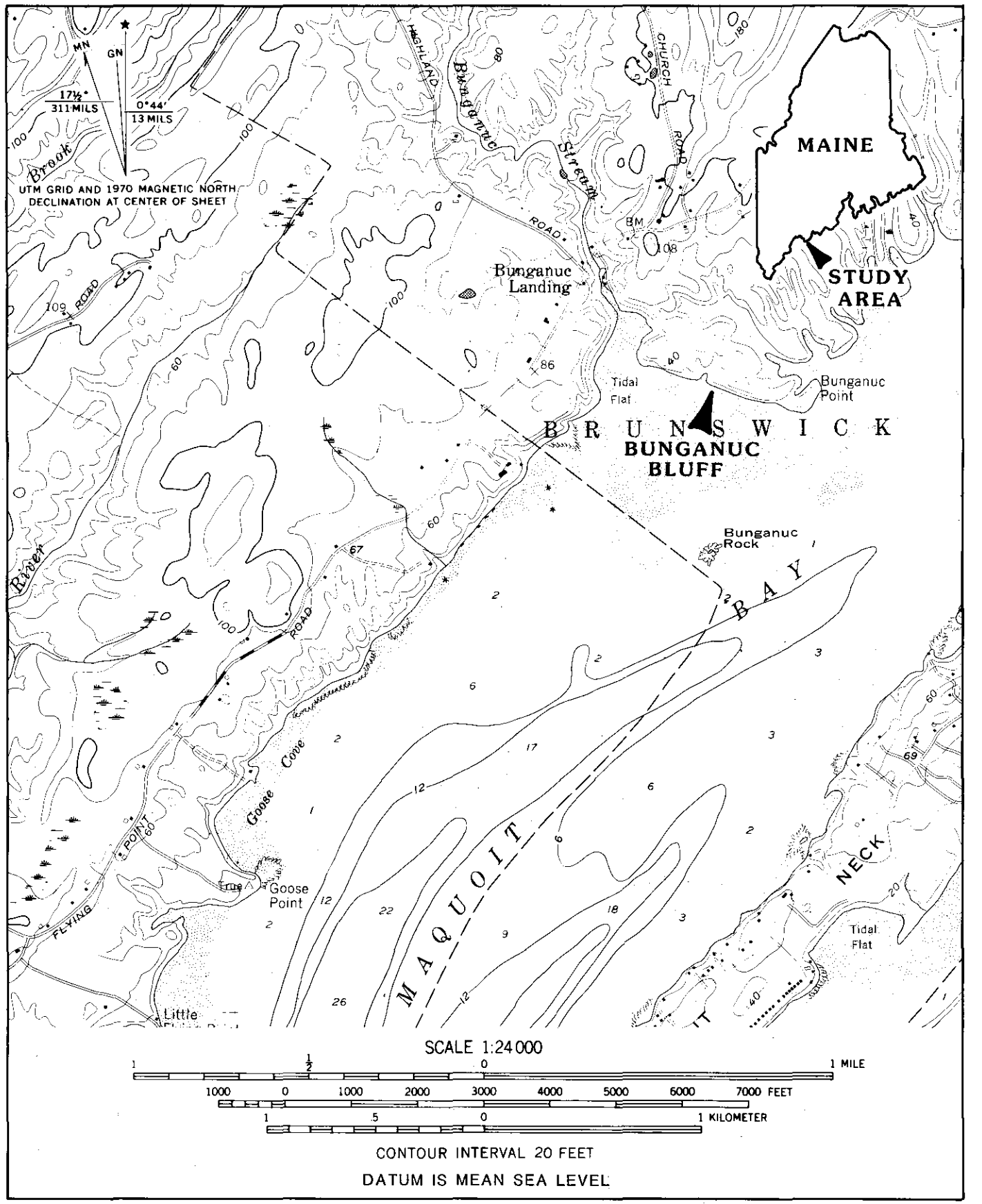


Figure 1. Location of Bunganuc bluff, Brunswick, Maine

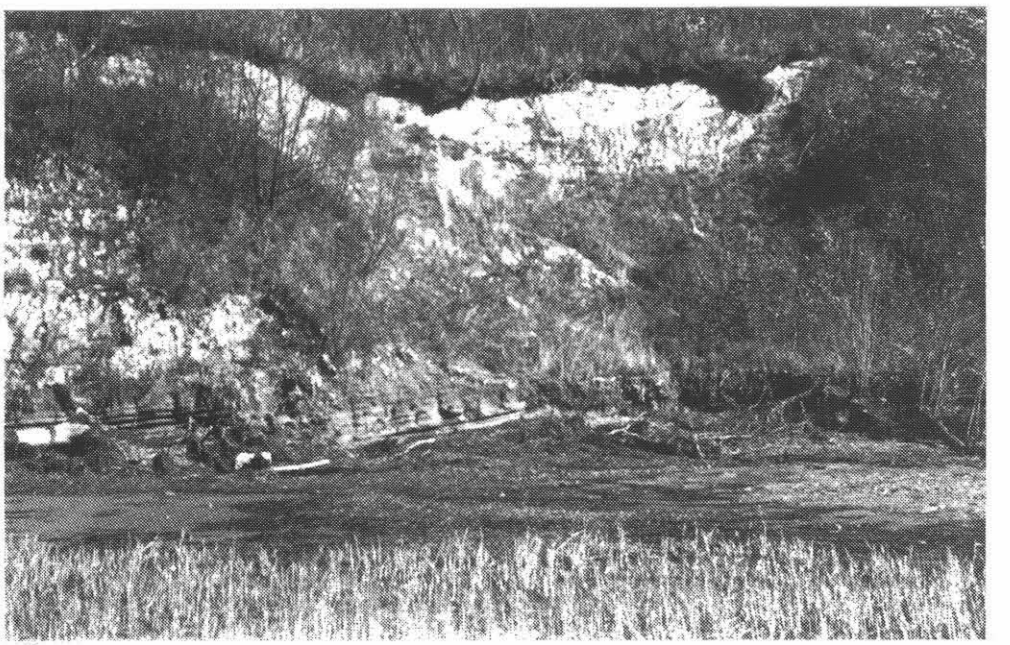


Photo 1. Slump investigated (1985)

Photo 2. Intact bluff investigated (1985)

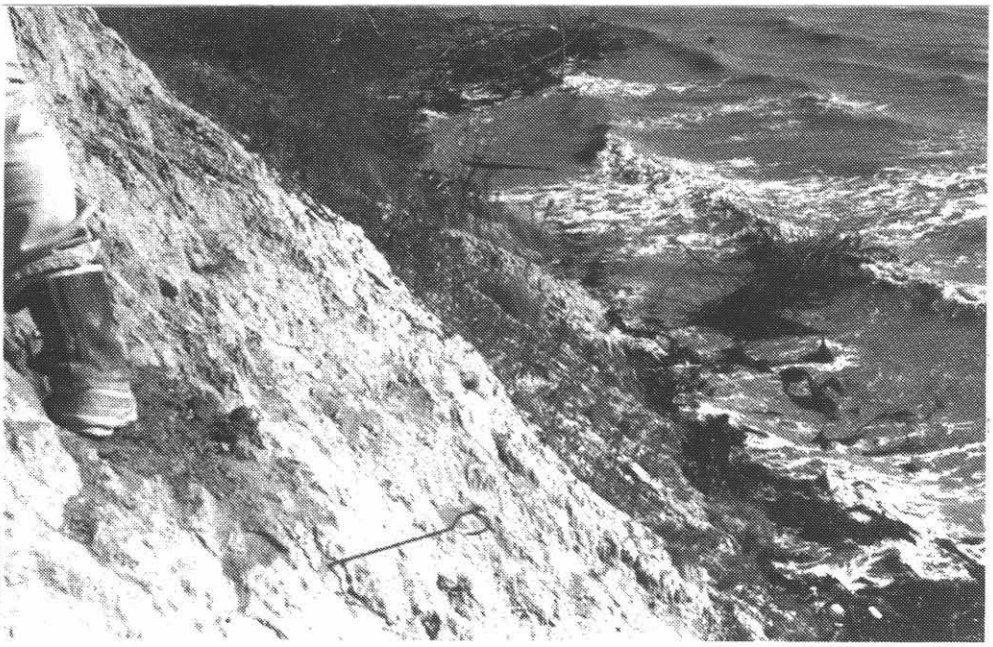


Photo 3. Wave erosion at toe (1985)

Photo 4. Erosion of silt laminations (1985)

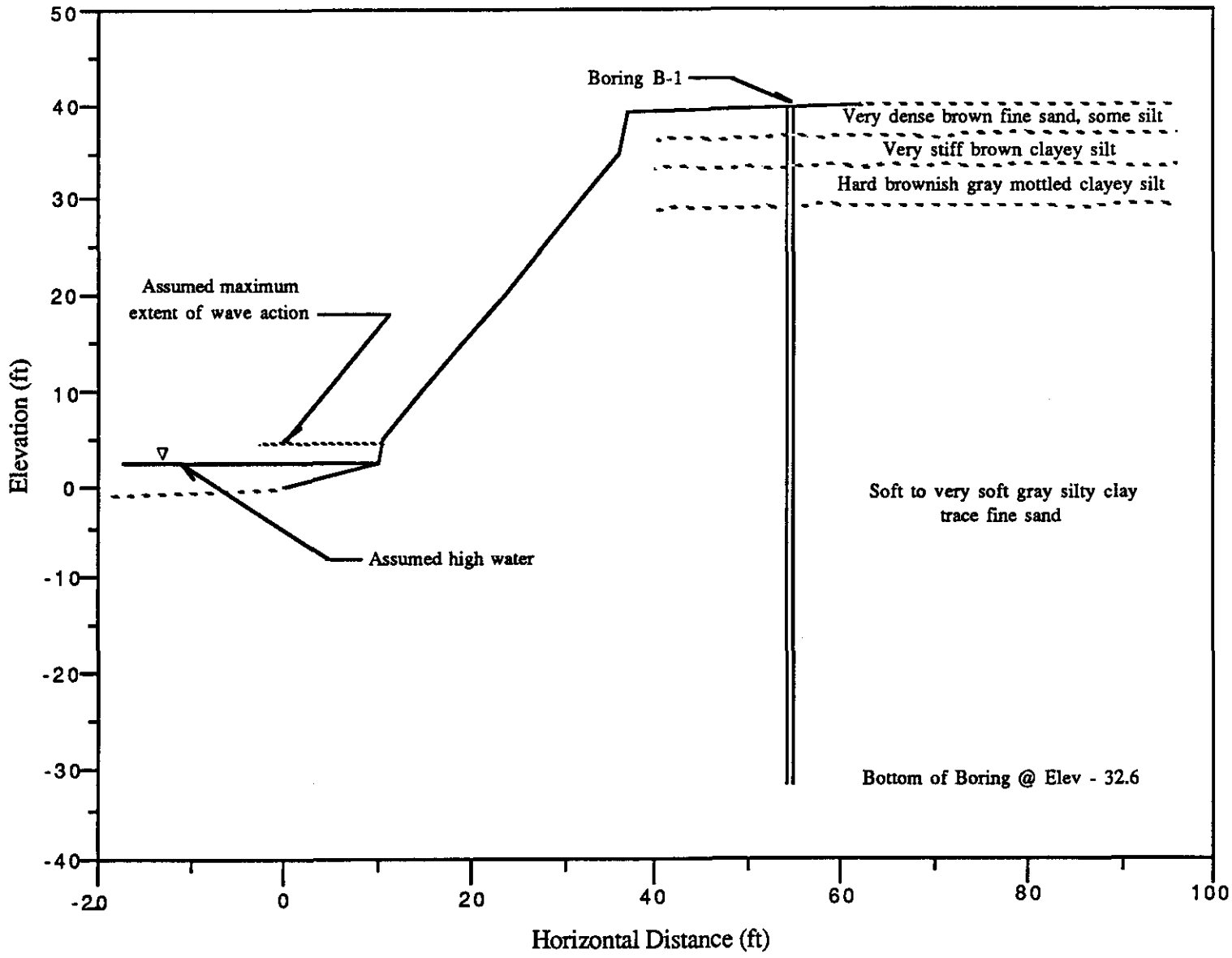
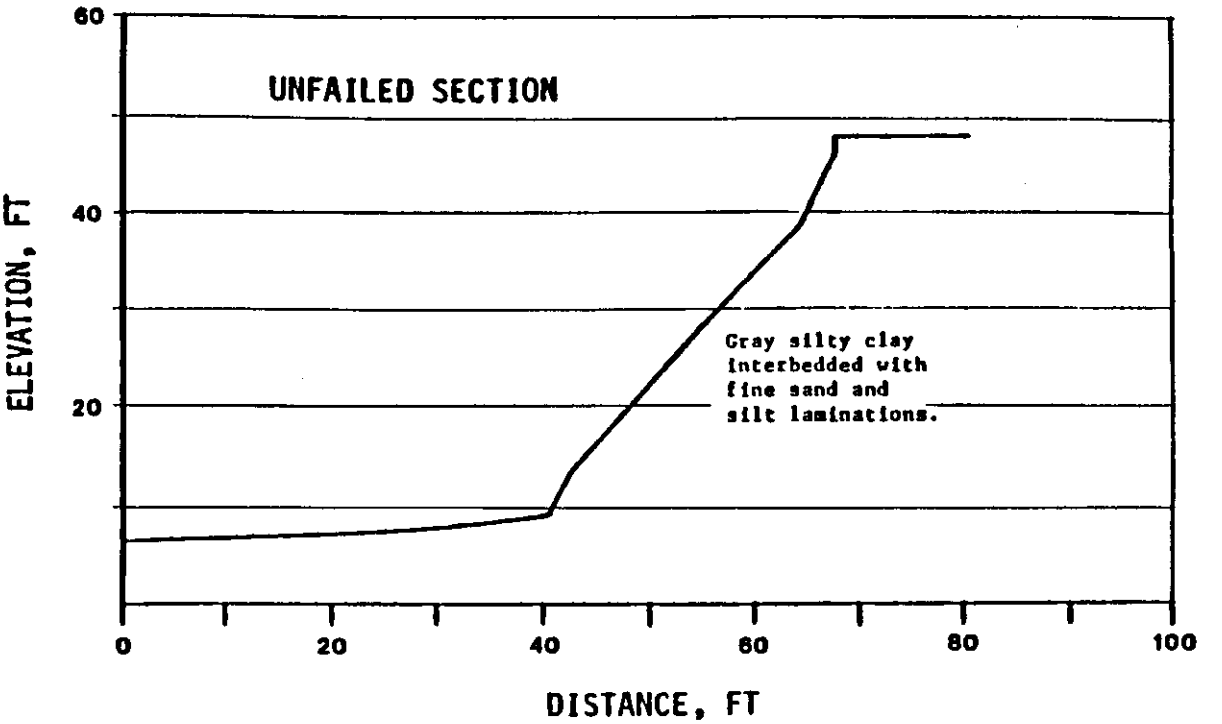
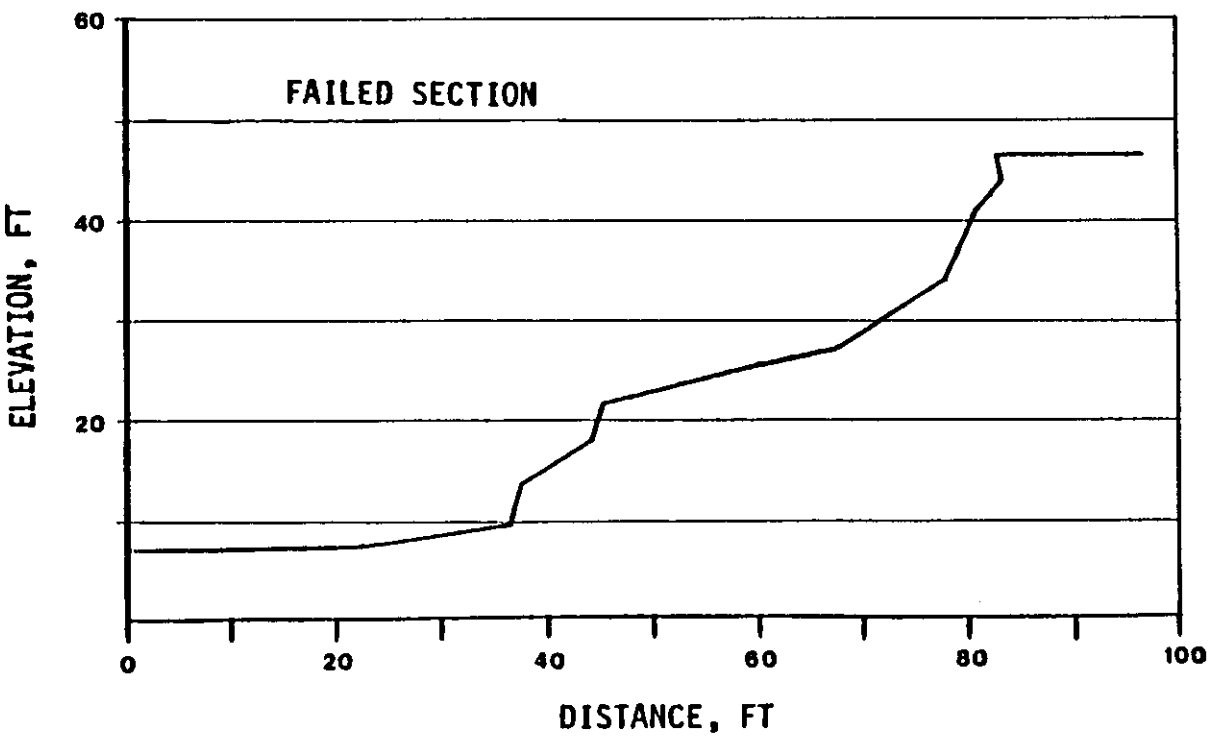


Figure 2. Section of Bunganuc bluff at Brunswick, Maine



PROFILE A-A



PROFILE B-B

Figure 3. Profiles of failed and unfailed sections at Bunganuc bluff

the top of the bluff and offset 16 feet from the survey section. Split spoon (disturbed) samples with standard penetration tests were taken at five-foot intervals, and 2.8 inch diameter undisturbed Shelby tube samples were taken at fifteen-foot intervals. Field vane shear tests were performed immediately below the undisturbed sampling locations. The boring was advanced to a depth of 72 feet (34 feet below the level of the tidal flat) without encountering bedrock. The log for this boring, designated B-1, is included in Appendix I.

The stratigraphy consists of about four feet of very dense brown fine sand overlying the Presumpscot Formation which extends to the bottom of the boring. The Presumpscot Formation here grades from a very stiff to hard brown clayey silt at the top to a soft to very soft gray silty clay at depth. Laminae of silt and sand were not apparent in the split spoon samples, although they were apparent in the face of the bluff (Photo 4). The crust is about 10 feet thick while the total thickness of the softer material is unknown.

Properties

Classification

To classify soil samples and determine their liquidity index, the Atterberg limits and natural water content were determined for each sample, while stress history was found by specific gravity and consolidation tests performed on selected samples. The classifications, according to the Unified Soil Classification System (USCS), on each sample are included in the boring log for boring B-1 (Appendix I) as are the Atterberg limits and natural water content determined for each sample. The specific gravity was determined for samples SS-2, SS-3, and SS-8 taken at depths of 5, 10, and 50 feet respectively. A complete summary of testing for boring B-1 is found in figure 5 and is discussed below. Pore water salinity (Table 2) determined by Mayer (in press) is consistent with deposition in a salt or brackish water followed by subsequent leaching.

Table 2

Salinity of samples in boring B-1

Sample Depth, ft	Salinity, ppt (g/l)
17	0.02
32	0.96
47	1.01
62	0.77
72	2.06

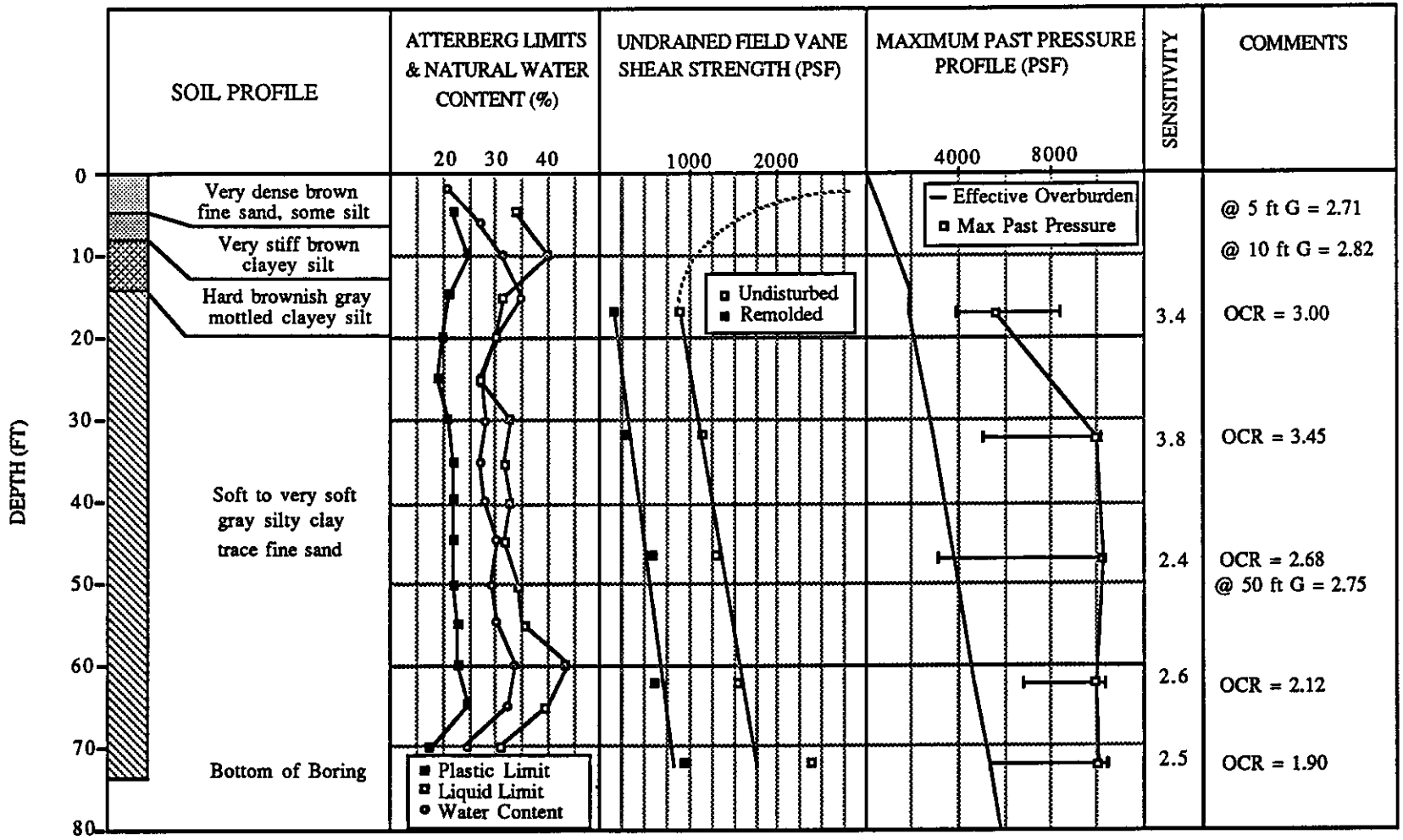


Figure 4. Summary of testing for boring B-1 at Bunganuc bluff, Brunswick, Maine

Since samples were taken from the face of the bluff in the previous investigation (Amos and Sandford, 1987), the Atterberg limits and natural water contents determined were possibly not representative of the material participating in the previously investigated failure. Since it was hoped to correlate the slope behavior to the classification properties, it was important to obtain representative samples near the failure surface (i.e. behind the face of the slope). For this reason, sampling was conducted in a boring that was located near the location of a potential failure surface. Comparison of the values from boring B-1 to values obtained from samples taken along the face of the bluff (Amos and Sandford, 1987) show little difference in both the Atterberg limits and natural water content (Table 3). Thus it seems that the face of the bluff had been exposed recently enough in 1985 to preclude changes in the Atterberg limits due to leaching or weathering and changes in the natural water content due to drying. The latter may be an indication of a continual replenishment of evaporating pore water at the face by groundwater seepage and capillarity.

Table 3
Classification property comparison

Test	Face of Bluff (1985)	Boring B-1 (1987)
Natural Water Content	12 to 33	21 to 35
Liquid Limit	32 to 40	27 to 44
Plastic Limit	16 to 22	18 to 25
Plasticity Index	11 to 18	8 to 21

Stress History

To obtain a profile of stress history, one dimensional consolidation tests were performed on each undisturbed sample and the results of these tests are summarized in Figure 5 (also shown are the effects of secondary compression). Consolidation tests indicate that the deposit at Bunganuc has been subjected in the past to stresses greater than those from the present overburden. The overconsolidation ratio (OCR) found from consolidation tests (Figure 5) varied from 1.90 to 3.45 (Figure 4). Secondary consolidation under sustained load (Bjerrum, 1973) can account for some, but not all, of the overconsolidation measured in the tests. To determine the effect of secondary consolidation on the overconsolidation ratio, the field virgin compression curve and the

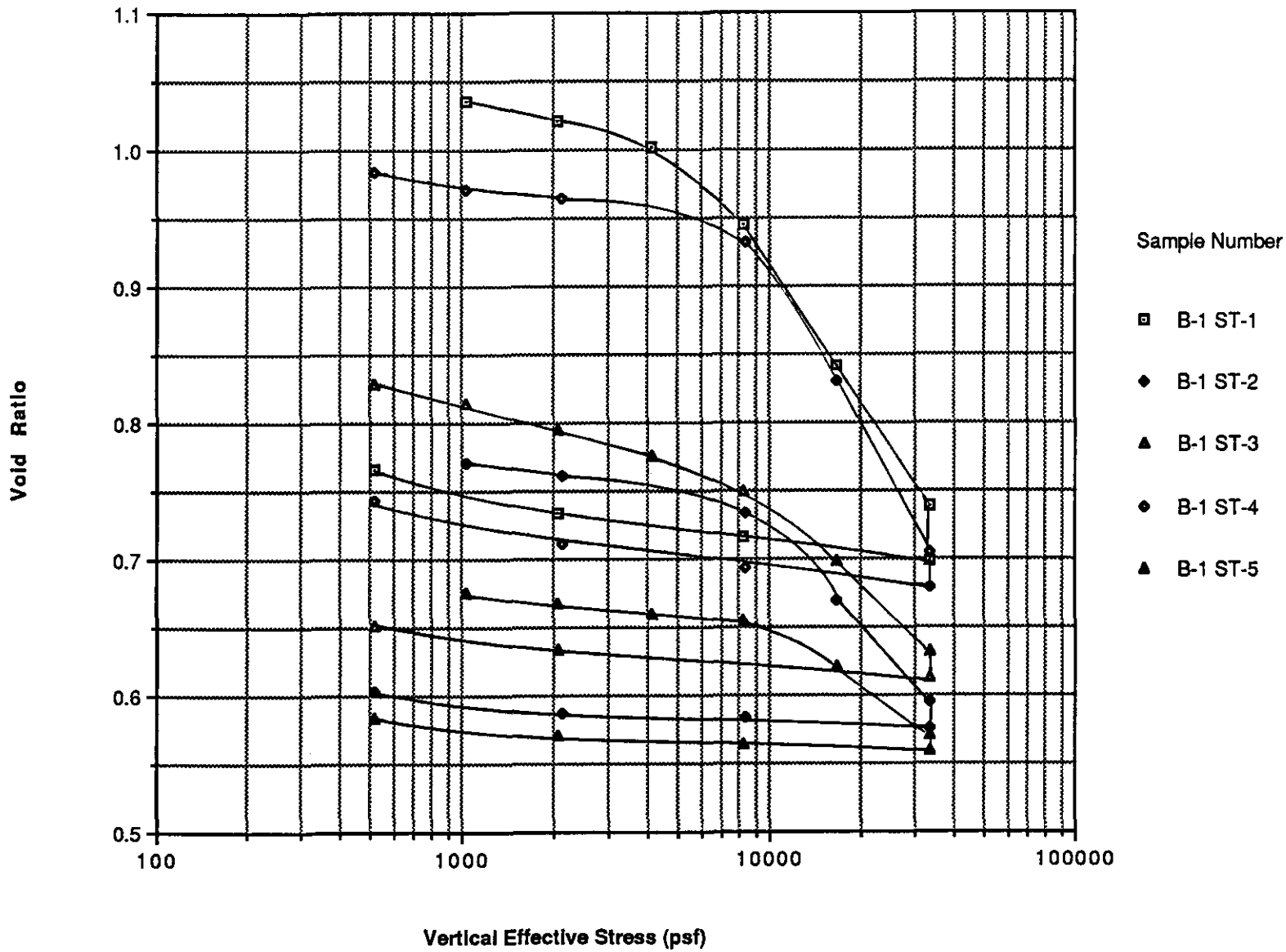


Figure 5. Bunganuc consolidation summary

compression curve for a sustained time of compression is estimated. For sample B-1 ST-4, obtained at a depth of 62 feet, an estimate was made of the field virgin compression curve (Figure 6) using Schmertmann's method (Holtz and Kovacs, 1981). The age of the deposit for sustained compression was assumed to be 12500 years, and the secondary compression index, C_α, was estimated from tests to be 0.0114. This resulted in an estimated decrease in the void ratio of 0.10 due to secondary compression since the time of deposition. If the overconsolidation at this depth were strictly the result of secondary compression a corresponding OCR of 1.91 would result. The OCR determined from the laboratory consolidation sample at this depth was 2.12 suggesting the past occurrence of stresses greater than the present overburden existing at this level. The laboratory consolidation curves were not adjusted to the field virgin compression condition, which would have given an even larger OCR than 2.12.

The presence of consolidation in addition to secondary consolidation suggests that a greater overburden and/or capillary stresses have existed previously at this depth. If the overconsolidation was due strictly to an additional overburden pressure, then the maximum past pressure curve should parallel the effective overburden pressure curve (Figure 5). Analysis of Figure 5 shows this not to be the case, suggesting the possibility of the existence of capillary stresses due to lowered groundwater levels contemporaneous with the sea-level lowstand. This conclusion, however, does not rule out the likelihood of erosion of the overburden subsequent to the time of deposition.

Mapping of the face of the bluff (Amos and Sandford, 1987) and examination of undisturbed Shelby tube samples indicate the existence of fine sand layers throughout the deposit. This reinforces the conclusion of Belknap et al. (in press) that this deposit is composed of the second unit (GM-D) of the Presumpscot Formation. Surficial erosion of the overburden would also suggest that the uppermost unit (GM-P) of homogeneous silt and clay has been eroded since deposition, exposing the second unit (GM-D) near the surface.

Peak Undrained Strength

The peak undrained shear strength, which is used for stability under rapid loading, is most commonly measured in Maine by the field vane test. The shear strength measured by the field vane increases with depth in a relatively linear manner from 680 psf at a depth of 17 ft to 1560 psf at 62 ft (Figure 4). The higher shear strength value at a depth of 72 ft appears to reflect a possible transition to another material. The shear strength as a ratio of the effective overburden pressure (c/p ratio) ranges from 0.34 to 0.39 (excluding the test performed at a depth of 72 ft).

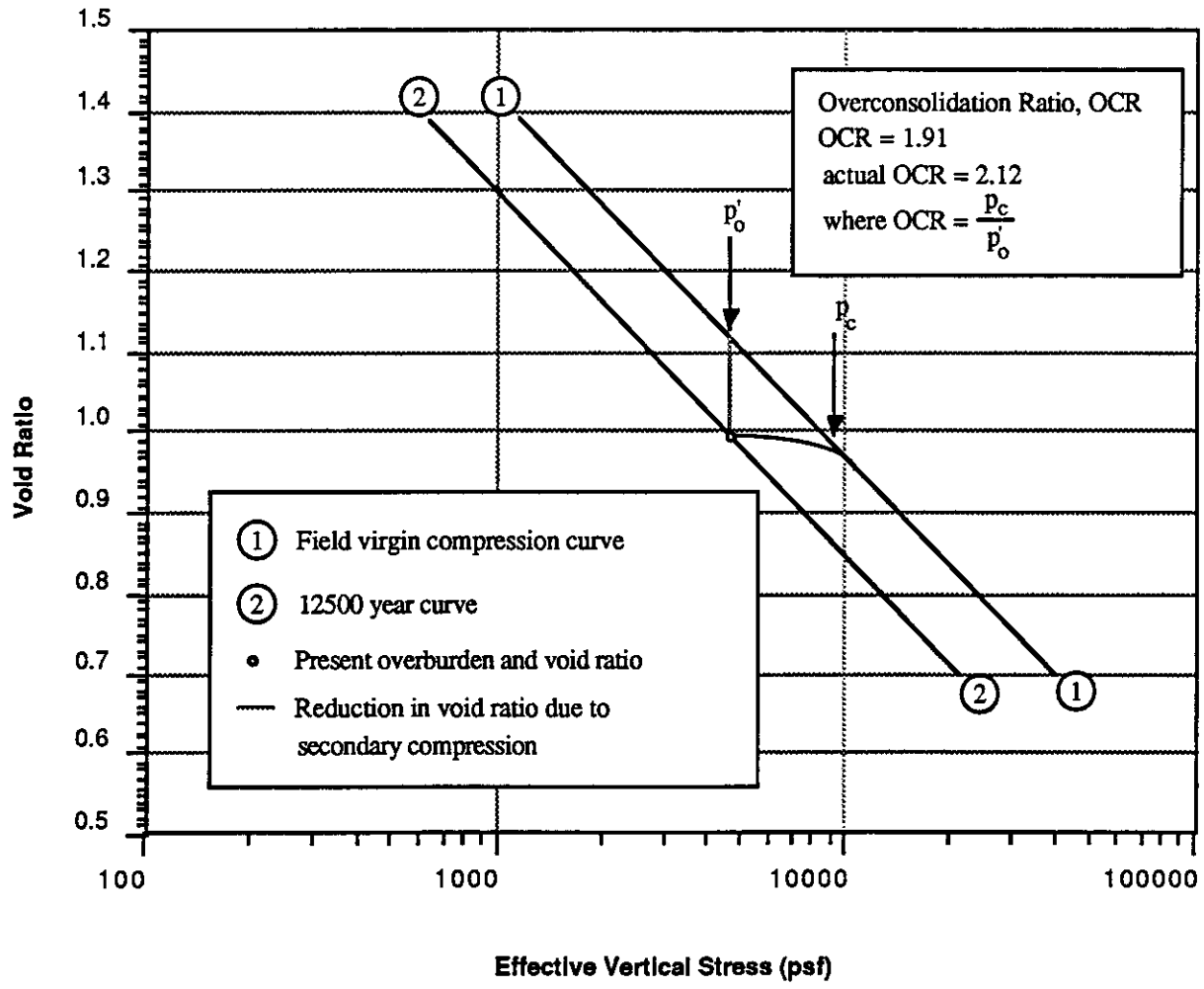


Figure 6. Estimate of secondary compression required to produce overconsolidation at a depth of 62 feet

Remolded Strength

The remolded strength, which controls the stability after movement has occurred, varies from 200 to 970 psf (Figure 5). The remolded strength increased with depth with a c/p ratio of 0.10 to 0.15 (excluding the deepest value). The ratio of the peak undrained strength and the remolded strength, i. e. the sensitivity, ranged from 2.4 to 3.8. These sensitivity values correspond to a material described as medium sensitive by Rosenqvist's (1953) criteria.

Drained Strength

The drained strength friction angle was not obtained for this site. It is likely that this strength is not much different from that measured at Gorham (Amos and Sandford, 1987) or Westbrook (following section).

Stability

Failure Pattern

Observations at Bunganuc indicate that erosion at the toe causes oversteepening of the bluff followed by stability failure. At Bunganuc, the failure surface is generally circular and does not retrogress into the slope with a series of failures as has occurred at Gorham in 1983 (Sandford and Amos, 1987) and at Rockland in 1973. The failure causes a general flattening of the slope which then becomes stable.

The remolded strength governs the stability of the slope exposed by the failed portion. For more sensitive material, i. e. material with a lower remolded shear strength, the failed material would move away from the slope and create another oversteepened slope. This slope would then immediately fail, and there would be a retrogression of failures as occurred at Gorham. Eventually a stable condition would be reached when the overall slope would be stable with respect to the remolded strength, and the retrogression would terminate. Thus the field evidence suggests that for the value of remolded strength at Bunganuc (i.e. as measured by the field vane at Bunganuc), there is no retrogression. The values of remolded strength for 31 ft and below correspond to a natural water content which remains below the liquid limit , i.e. the liquidity index averages 0.57. In the section from a 16 ft depth to a 26 ft depth, the liquidity index averages 1.13.

Analyses

A slope stability analysis was performed on the bluff section shown in Figure 2 assuming that failure would occur on a circular sliding surface, the soil strength at failure would be the same as that measured by the field vane test, and that the soil was saturated. Use of the field vane strengths implies that the pore pressures generated during shear by the field vane are the same as the pore pressures generated during failure of the slope. Stress conditions and possible drainage prior to failure are not explicitly considered in this approach. The overconsolidated condition of the soil suggests that the pore pressures generated during shear are small, and thus drainage conditions are less important.

Several analyses were performed utilizing both computer methods and stability charts. Computer analyses were performed with the aid of STABL6, a slope stability analysis program developed at Purdue University. The modified Bishop's method of slices was utilized in determining the factor of safety against rotational failure. The results of these analyses are summarized in Table 4.

As can be seen from Table 4, the analyses using the average strengths (uniform with depth) or the individual vane strengths (varying with depth) both give a factor of safety of 1.27 for the

Table 4

Summary of stability analyses performed at Bunganuc bluff

Method of Analysis	Assumed Conditions	Factor of Safety	Toe Erosion for Failure, ft
STABL6 Modified Bishop Method of Slices	Variable undrained shear strength measured by field vane corrected for rate of strain effect	1.27	15.5
Janbu Chart (Navfac, 1982)	Uniform undrained shear strength with depth; s_u=1080 psf	1.27	22.0
STABL6 Modified Bishop Method of Slices	Tension cracks to a 14 ft depth	1.00	----
Janbu Chart (Navfac, 1982)	Tension cracks to a 14 ft depth	1.12	9.0
Janbu Chart (Navfac, 1982)	Tension cracks to a 14 ft depth filled with water	1.08	7.0

profile analyzed. Since the bluff is standing, the factor of safety must be greater than unity. If these slopes were being engineered as permanent, the calculated safety factors would be inadequate since permanent slopes are designed with a minimum factor of safety of 1.50. Temporary slopes (construction) slopes are, however, designed with a smaller factor of safety of 1.25. Observations on the face of the bluff indicate that there are joint surfaces in this deposit, especially in the upper crust. When consideration is given to these by assuming that open tension cracks exist, then the factor of safety drops to 1.00 to 1.12. This is no surprise when one considers the numerous slumps evident along the bluff. The most critical failure surface shows that the failure circle emerges near the toe of the slope on the tidal flat. Field observations of past slumps confirm this predicted failure surface location.

Springtime slumping has been observed by nearby residents. In the spring, there is a slight increase in the total unit weight of the soil in the upper part of the profile and significantly increased pore pressure from water in the tension cracks. To determine the effect of the spring wet season on the factor of safety, the additional pore pressure in the slope was estimated to equal the water pressure in the tension cracks at the top while the increase in the unit weight of the upper 4 feet was considered small enough (due to the compactness, and hence low void ratio, of the material) that it could be neglected. This analysis indicates that the factor of safety for the analyzed slope is only slightly above failure in the spring (Table 4, tension cracks with water). According to this analysis, it would take very little (0 to 9 ft) toe erosion to cause a failure during the wet spring season. During the other drier seasons, the factor of safety is higher and thus it would likely require the amount of erosion corresponding to the upper limit of this range to cause failure. Although pore water pressures may be greatest during the spring season, and consequently effective strengths reduced, elimination of excess pore water pressures by drainage does not stop the slumping problem at Bunganuc; it only retards it until toe erosion proceeds to oversteepen the slope. The root of the problem at Bunganuc is the erosion, and consequent undercutting, of the toe of the slope. Continuing erosion of the toe means that it is only a matter of time before additional slumping occurs.

The use of the undrained peak shear strengths measured by the vane and corrected for strain rate, tension cracks, and possible water pressure in a stability analysis appears to closely approximate the field behavior.

Results

1. Unfailed slopes up to 51 degrees from the horizontal are standing at the 38 foot high Bunganuc bluff. Erosion of the toe by ocean waves has oversteepened some sections,

initiating numerous past failures. Each slide at Bunganuc bluff is a rotational slide which does not retrogress into the slope.

2. The stratigraphy of the bluff consists of four feet of very dense brown fine sand overlying the Presumpscot Formation. In the Presumpscot Formation a ten-foot thick very stiff to hard brown clayey silt crust grades to an underlying soft to very soft gray silty clay with fine sand and silt layers. The alternating fine sand and silty clay indicate that the deposit at Bunganuc is the second unit (GM-D) of the Presumpscot Formation as described by Belknap et al. (1988).
3. The peak undrained shear strength of the deposit ranged from 680 to 1560 psf (excluding the deepest sample) as measured by the field vane. The remolded strength ranged from 200 to 610 psf with a sensitivity range of 2.4 to 3.8. The ratio of the peak vane strength to the overburden pressure (c/p ratio) was 0.34 to 0.39 at this site.
4. When the peak undrained vane shear strength is used in an undrained stability analysis, corrected for tension cracks and associated increased hydrostatic water pressure, the results appear to closely approximate the field behavior.
5. The liquidity index averaged 0.67 for a depth of 70 feet. The relatively easily measured liquidity index is a likely indicator of the slope failure pattern, since it is related to the value of peak shear strength and remolded strength.

Recommendations

1. At Bunganuc bluff, the slope failures will be stopped if toe erosion is prevented.
2. Although failures occur preferentially in the spring when the ground is saturated, drainage of the slope will only delay the failure. Drainage will allow the slope to steepen more before failure occurs but will not prevent future failures.

Westbrook

Background

The site investigated in Westbrook is located on the north bank of the Presumpscot River one mile downstream from the S.D. Warren paper mill (Figure 7). This area was once farmland which has since been developed and is the site of a large housing subdivision situated on a plateau forty feet above the Presumpscot River (Figure 8). Numerous underground utilities cross the site, including sewer and water lines as well as a natural gas pipeline.

The susceptibility of the Westbrook area to landslides is well documented in the historical records. Several slides are known to have occurred both along the Stroudwater and Presumpscot Rivers since the area was first settled in the early 1700's. The evidence for the occurrence of landslides prior to settlement is also numerous as suggested by an excerpt taken from an article by C.W. Munson (1951).

> "About 1670, according to a legend or story told to the original settlers by the Indians, occurred a great slide of earth at the southwest and western side of Conant street. One wonders what could have occasioned this for there is no stream of water there today to undermine the land; yet great rains have been known to bring about such movement of the terrain. Many years ago there must have been a "slide" toward the river in the Dunn St. section for when excavating for a sewer years ago great trees were found on their sides, or so the story goes."

Evidence also exists which suggests the occurrence of a massive prehistoric landslide in the Saccarappa area of present day Westbrook (Morse, 1869; Anonymous, 1952). This slide is thought to have encompassed two hundred acres and redirected the flow of the Presumpscot River northward away from an assumed course through the Fore River estuary (Morse,1869). The evidence suggesting this scenario is implied by C. W. Munson in the later portion of the previous excerpt. However, it was Morse (1869) who first set forth the evidence supporting this occurrence.

> "As to the evidences of the Saccarappa slide, they are of the most positive character. In the first place, the village rests on a level plain of clay, and bordering this on all sides is an embankment from ten to twenty feet in height. The upper portion of this depression has always been called by the inhabitants Warren's Cellar, and indeed many have regarded this area as sunken land. In digging wells and sewers, trunks and branches of trees are met with at a depth of thirty feet from the surface."

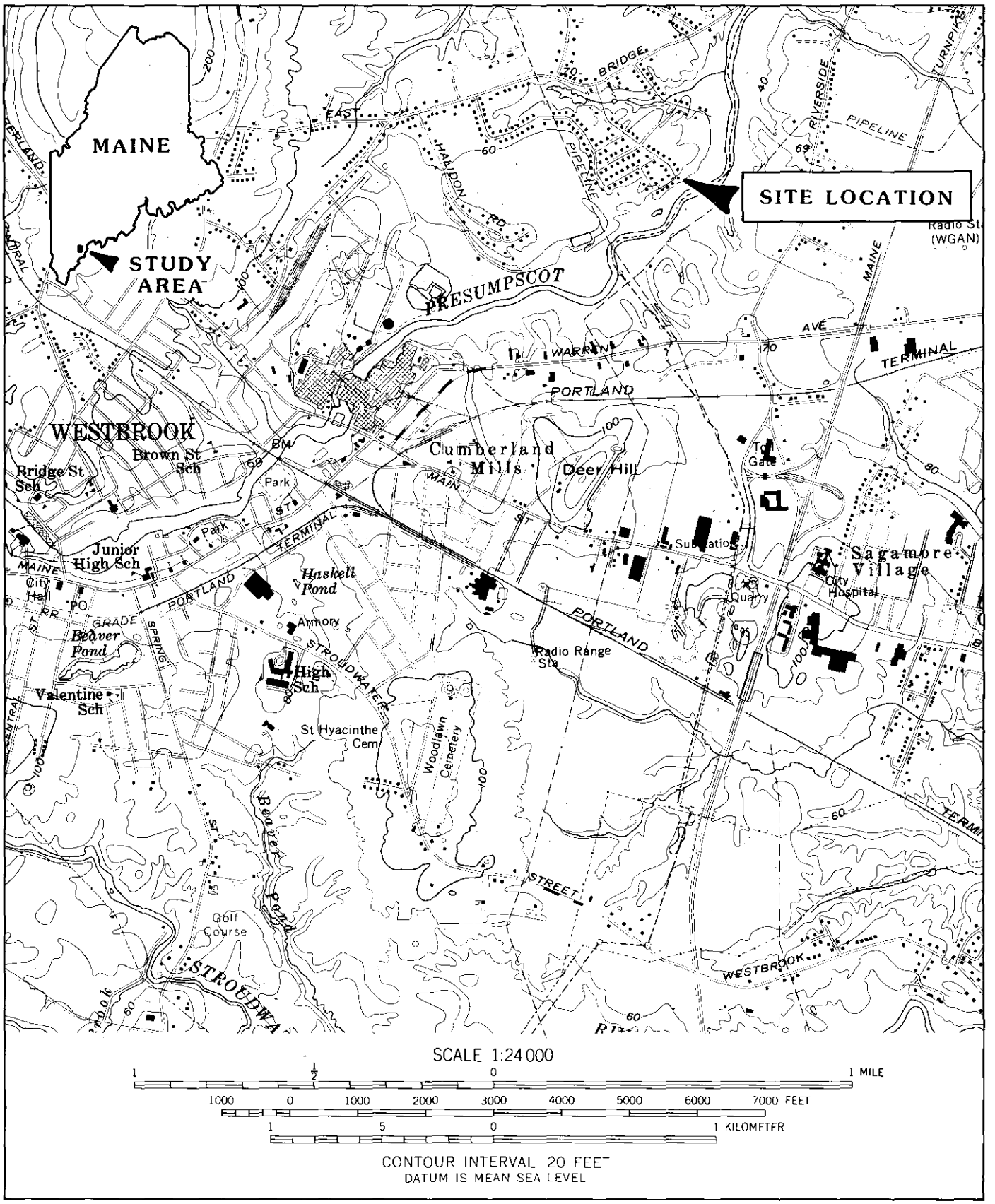
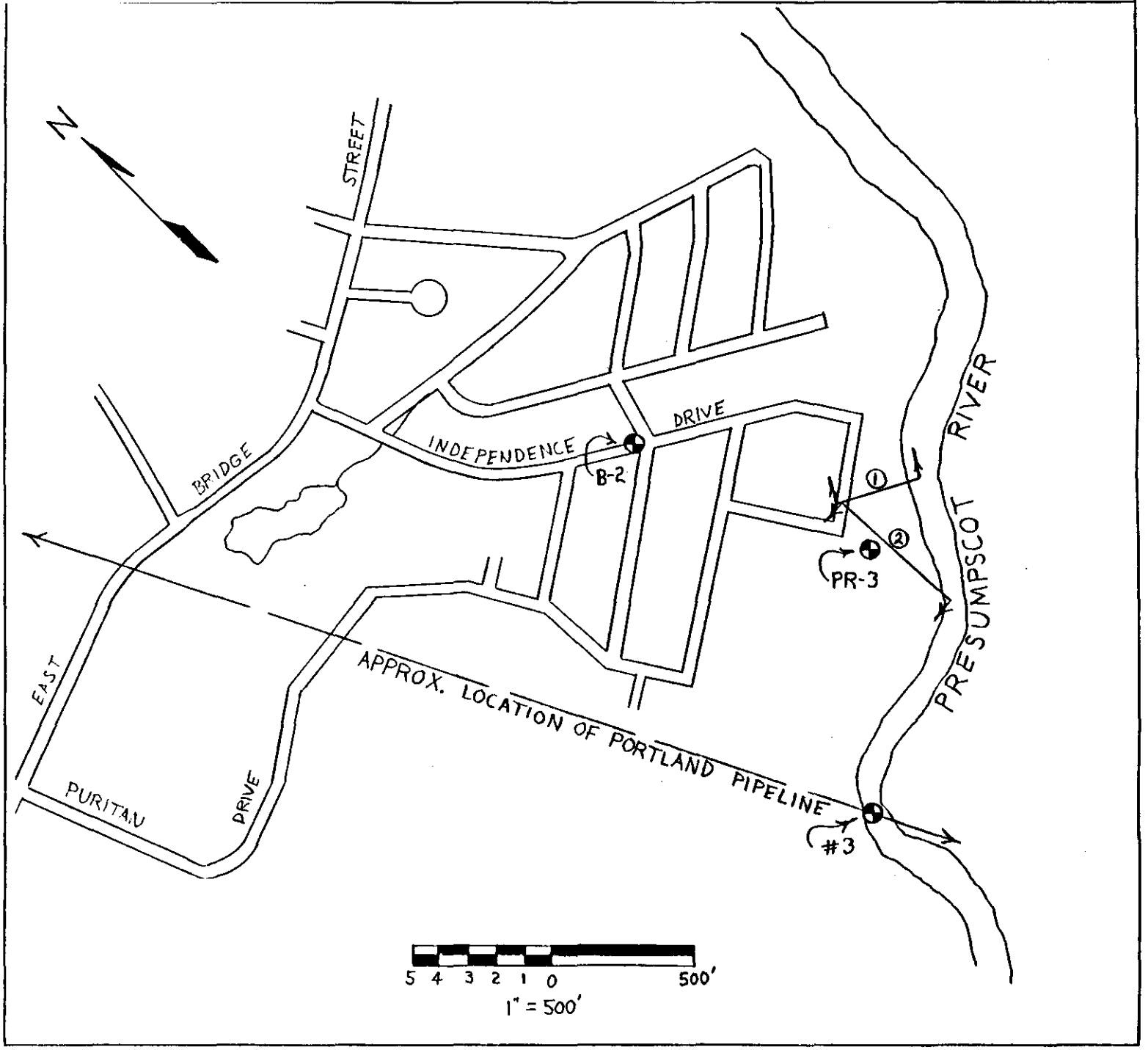


Figure 7. Location of Westbrook study area



Location of Boring

Location of Survey Line

Some details omitted for clarity

Figure 8. Plan view of study area; Westbrook, Maine

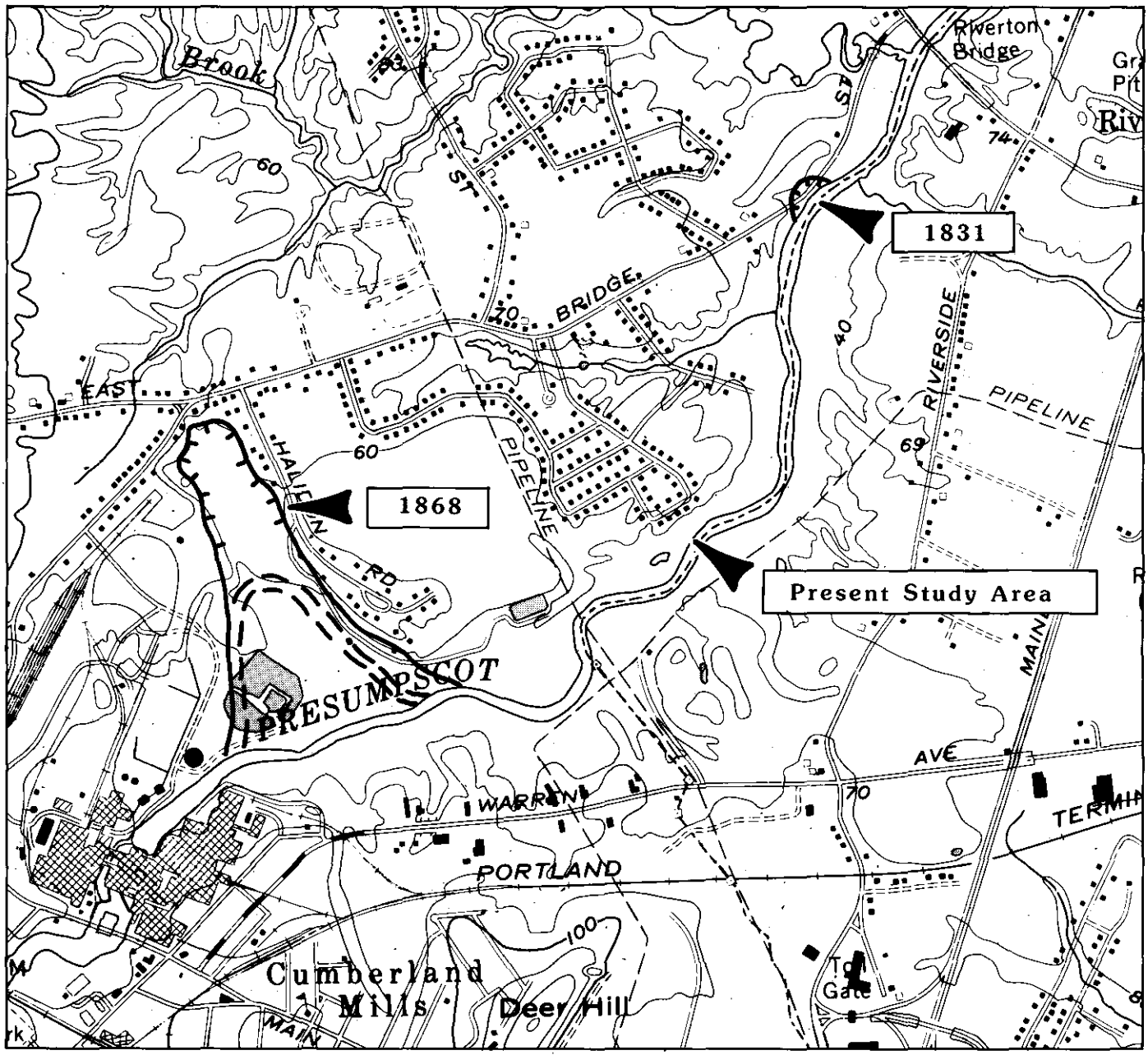


Figure 9. Historical landslides along the Presumpscot River in Westbrook, Maine

Two landslides which occurred on the north bank of Presumpscot River in the mid 19th century are located in Figure 9. The first slide ocurred on the night of May 5, 1831 and encomposed an area "one hundred and twenty yards in diameter" while exposing clay banks thirty feet above the river (Jackson, 1837). The description of this slide given by C.T. Jackson suggests that as many as 12 solid masses of clay (or sliding blocks) slid forward into the river, altering its course. This type of retrogressive sliding block failure is also evident in more recent slides such as at the nearby 1983 Gorham landslide (Sandford and Amos, 1987).

The second historical slide depicted in Figure 9 occurred on the night of November 22, 1868 immediately downstream of the S.D. Warren paper mill located at Cumberland Mills (or Ammoncongin). The magnitude and impact of this slide was much more severe than the earlier slide of 1831, blocking the 200 ft wide river channel for a distance of half a mile, raising the upstream water level 15 to 17 feet, and thus flooding the lower floors of the S. D. Warren mill (Morse, 1869). John E. Warren (Anonymous, 1954) summarizes the occurrence as follows.

> "Where formerly the River had meandered through a valley of intervale bordered by steep banks rising about forty-five feet to a level sedimentary plane, a portion of this high land, fifteen hundred feet long and six hundred wide had slid into the valley leaving a chasm averaging about twenty-five feet deep.
>
> This immense volume of soil totaling something like eight hundred thousand yards had within an hour or two moved a quarter of a mile. The dam which it formed raised the water to a height which prevented operating the mill. Eventually the water cut a new channel, this time following the south bank thus transferring thirty acres of Jonas Raymond's farm to the north side of the River.
>
> This is the land that we now use for the lower end of the woodyard and for piling bark."

Morse (1869) estimated the "sunken" area produced by the slide to be about twenty acres in extent (see Figure 10). The material ejected from this area then flowed into the river and beyond to encompass a total area of about 34.5 acres according to Morse's (1869) estimates. The description of this occurrence indicates behavior which can be more destructive than the retrogressive sliding block failures for the 1831 slide and the 1983 Gorham slide. The failure type for the 1868 landslide, in which massive amounts of material flow as a viscous fluid over an expanse, is referred to as a flowslide by Mitchell and Markell (1974). These authors note that the occurrence of flowslides has resulted in massive slides in Norway and eastern Canada . This type of slide is potentially very destructive as is suggested by Morse (1869) in reference to the 19th century slides in the Westbrook area.

Figure 10. Photographs of the November 1869 landslide
(Courtesy of P. Spiller, Westbrook, ME)

"These slides would have been disasterous to life had they occurred in inhabited regions. In the case of the Ammoncongin land-slide, the damage was estimated at over one hundred thousand dollars, and the checking of the Cumberland paper mills, by which three hundred operatives were thrown out of employment, and losses estimated at one thousand dollars per day incurred. A gang of one hundred and fifty men were required to aid in the opening of a new channel on the intervale, and this has been accomplished."

The frequency of major landslides in the Westbrook area clearly demonstrates the need to identify the sites susceptible to landsliding. This report examines one such site near to the 1868 flowslide.

Topographic Survey

A topographic survey was conducted to provide data for use in a slope stability analysis. The locations of the survey lines are indicated in Figure 8 while the corresponding profiles may be seen in Figure 11. Each survey was conducted utilizing the river level as a datum. Also, a profile of the entire site including line 1 and a survey line along Independence Drive are included in Figure 12 along with the general subsurface stratigraphy.

Stratigraphy

An exploratory boring was made at the site to determine stratigraphy and to perform sampling and field testing. Subsurface data from several sources were obtained in order to obtain a thorough picture of the subsurface conditions over the entire site. The boring, designated B-2, was located a distance of approximately 600 feet from the top of the slope and 930 feet from the river bank (Figures 8 and 11). After augering six feet, split spoon (disturbed) samples with standard penetration tests were taken at five foot intervals and 2.8 inch diameter undisturbed Shelby tube samples were taken at fifteen foot intervals. Difficulty in obtaining Shelby tube samples with a hydraulic push technique necessitated the use of an Osterberg piston sampler and modifications to the fifteen foot interval sampling criteria. The piston sampler provided enough suction to recover the samples. Field vane shear tests were performed immediately below the undisturbed sampling locations. The boring was advanced to a depth of 132 feet and was finished by coring 4.5 feet into bedrock. The summary of the field results is given in Figure 13 and more detail is given in the log for boring B-2 included in Appendix I.

The boring log for boring #3 supervised by Tippetts, Abbett, McCarthy, and Stratton, Consulting Engineers (1965) and the boring log for boring PR-3 supervised by E. C. Jordan, whose locations are given in Figure 8, can be found in Appendix I.

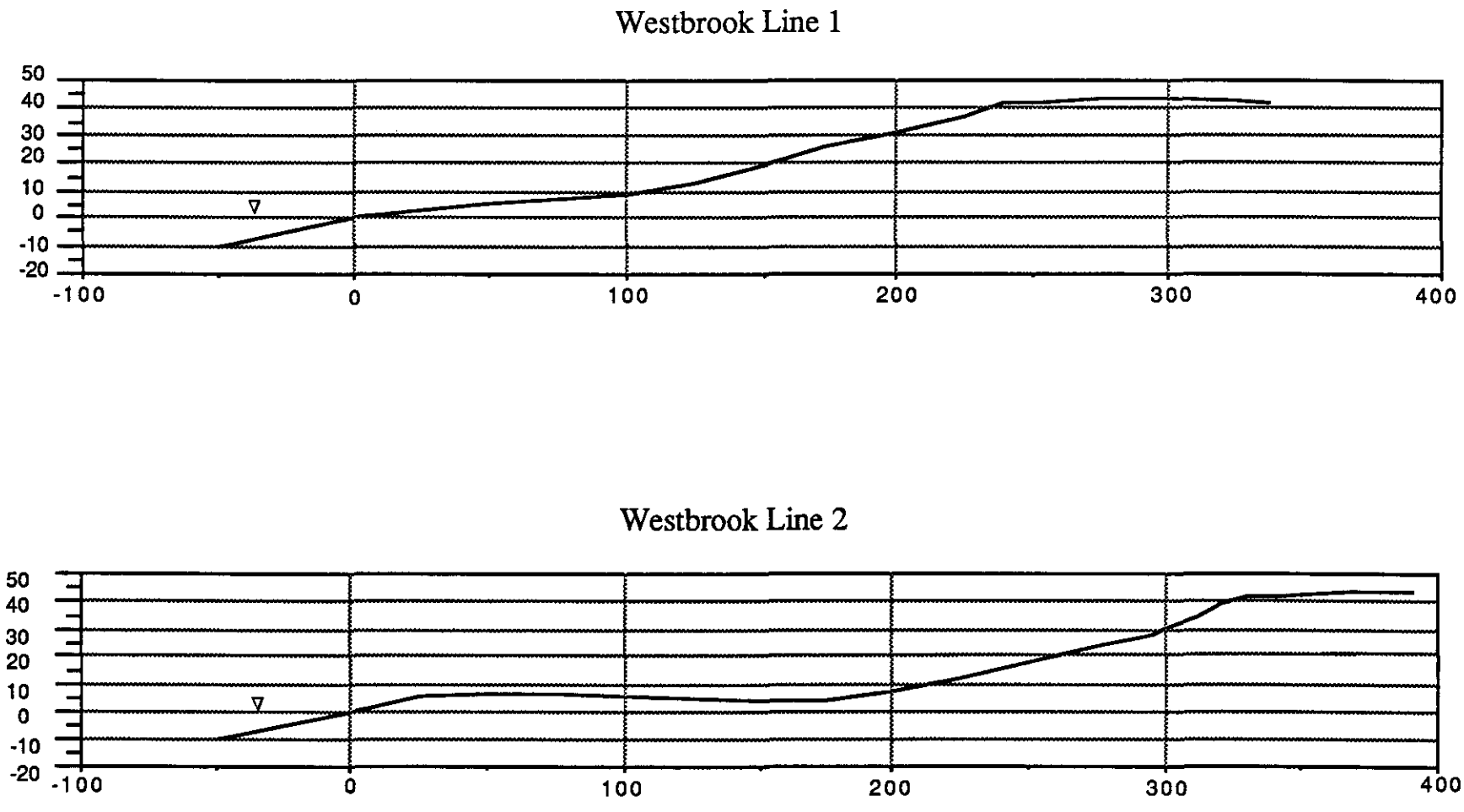


Figure 11. Sections from the Presumpscot River to Independence Dr. at Westbrook, Maine

Note: all distances and elevations are in feet

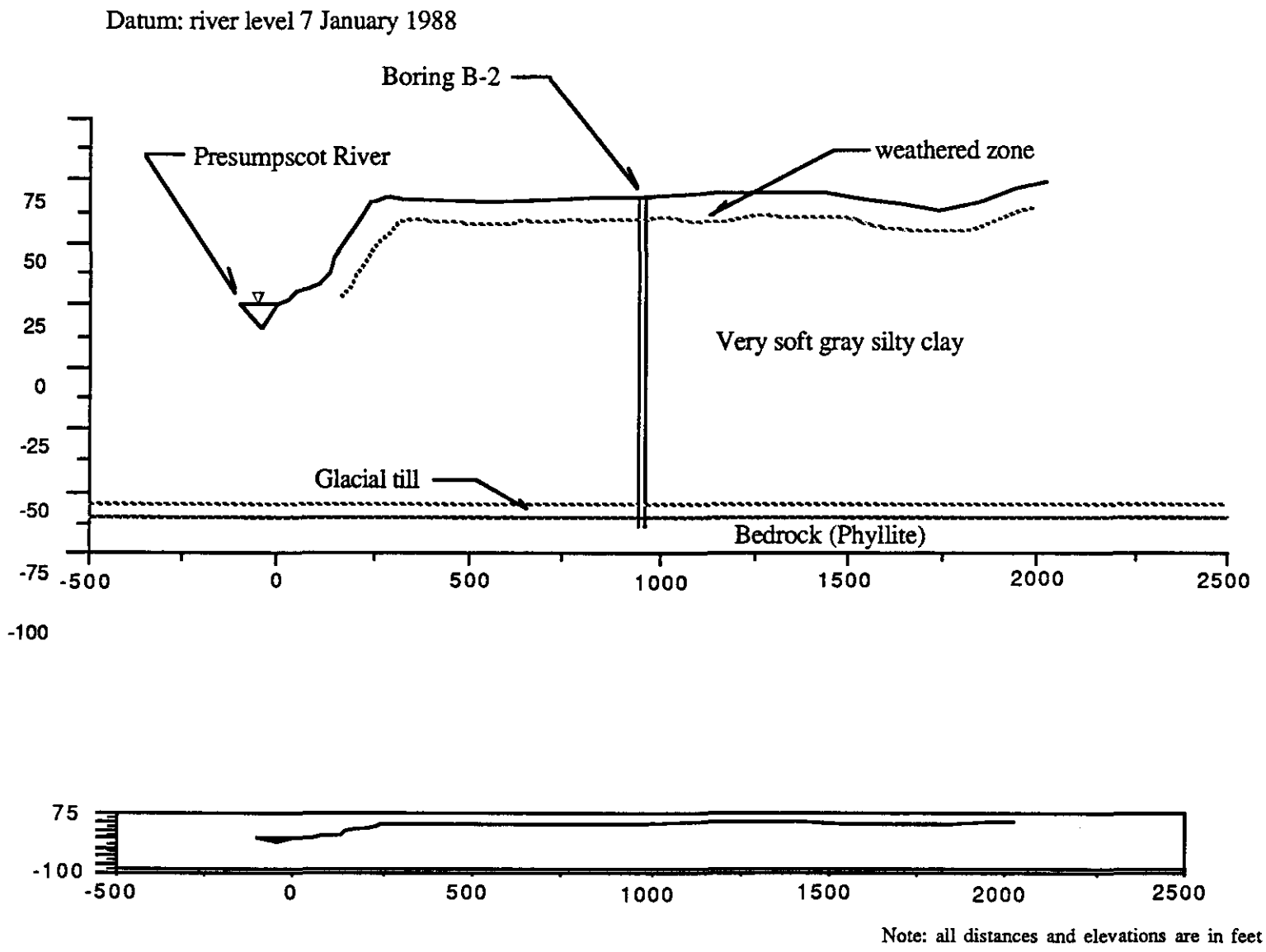


Figure 12. Section from the Presumpscot River to East Bridge Street at Westbrook, Maine

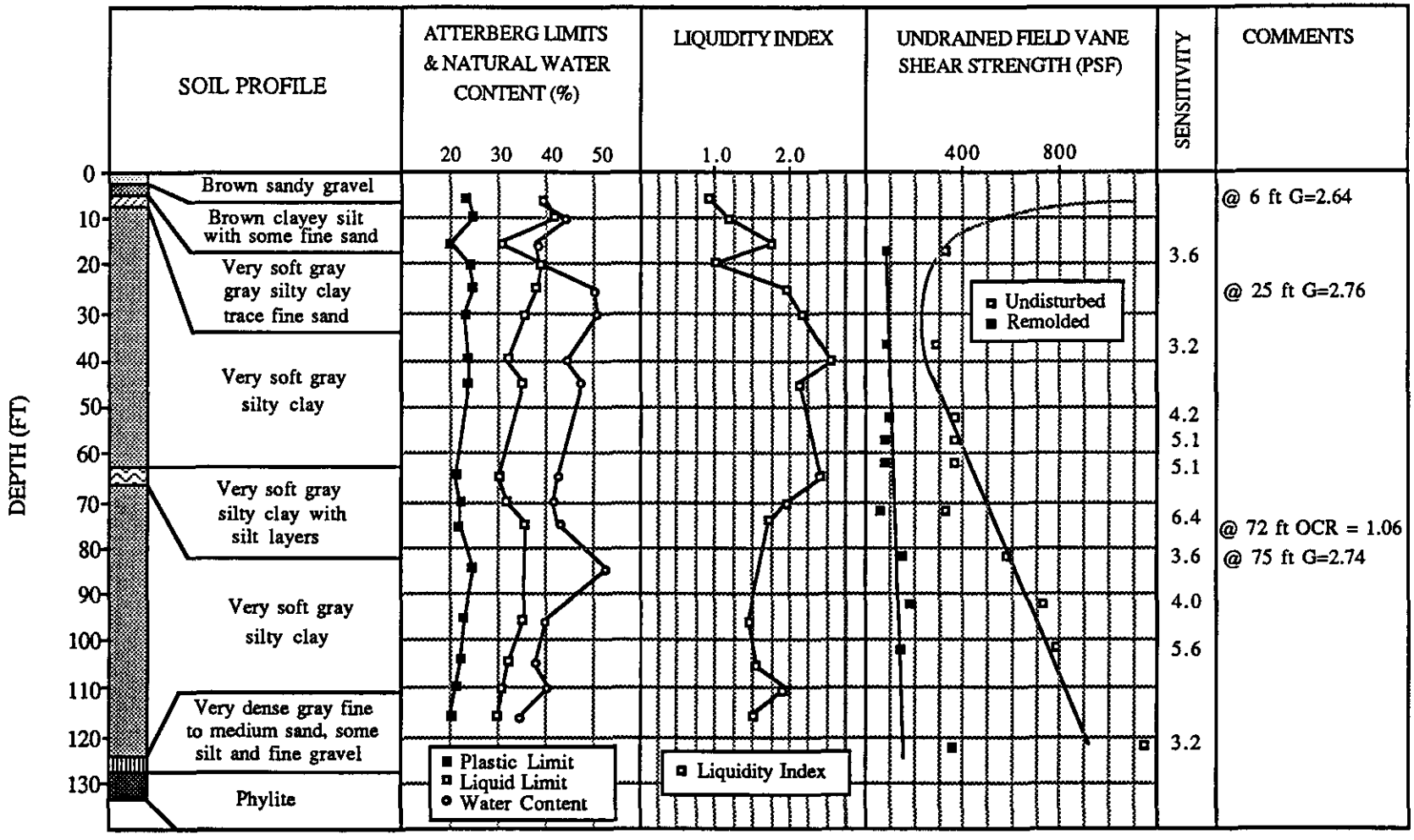


Figure 13. Summary of testing for boring B-2 at Westbrook, Maine

The stratigraphy indicated in Boring B-2 consists of 2 to 3 feet of brown sandy gravel overlying 3 to 4 feet of brown clayey silt crust. The Presumpscot Formation beneath this depth is generally a very soft gray silty clay extending to a depth of 123 feet. Below this silty clay, a 4.5 foot thick glacial till layer (very dense gray fine to medium sand, some silt and fine gravel) overlies bedrock (phyllite).

Properties

Classification

To classify soil samples and determine their liquidity index, the Atterberg limits and natural water content were determined for each sample, while stress history was found by specific gravity and consolidation tests performed on selected samples. The classifications, according to the Unified Soil Classification System (USCS), on each sample are included in the boring log for boring B-2 (Appendix I) as are the Atterberg limits and natural water content determined for each sample. The specific gravity was determined for samples SS-1, SS-4, and SS-9 taken at depths of 6, 25, and 75 feet respectively. A complete summary of testing for boring B-2 is found in Figure 13 and is discussed below. Pore water salinity (Table 5) determined by Mayer (in press) is consistent with deposition in a salt or brackish water followed by subsequent leaching.

Table 5

Salinity of samples in boring B-2

Sample Depth, ft	Salinity, ppt (g/l)
17	0.28
52	0.72
72	1.35
82	1.10
102	1.84
122	1.87

The liquid limit of the cohesive material ranged from 30 to 42 and the plasticity index ranged from 8 to 17 indicating a silt with clay or silty clay. In general, the upper 10 to 15 ft was more plastic than the remaining deposit. Results in Figure 13 show that natural water content values lie above the liquid limit, and thus the liquidity index (LI) is near or above unity (0.96 to 2.56) for the entire depth of the boring.

Peak Undrained Strength

The peak undrained shear strength, which is the available strength for stability under rapid loading, is most commonly measured in Maine by the field vane test. The shear strength measured by the field vane increases with depth in a relatively linear manner from 290 psf at a depth of 37 ft to 790 psf at 102 ft (Figure 13). The strength at a depth of 17 ft is 320 psf, which is higher than would be expected from the linear relation. This higher value reflects weathering or desiccation in the upper zone of the deposit. The higher shear strength value at a depth of 122 ft appears to reflect the transition to the underlying glacial till. The shear strength as a ratio of the effective overburden pressure (c/p ratio) ranges from 0.08 to 0.14 (excluding tests at a depth of 17 and 122 ft). The accepted Maine practice for the field vane shear test differs from ASTM D2573 (1982) standards (Appendix IV). The effect such differences have on the measured value of the undrained shear strength is difficult to ascertain, since compensating effects may result. However, the methods utilized seem to provide reliable or conservative results and therefore have not been widely questioned. Bjerrum's (1954) correlation of c/p ratio to the plasticity index for Norwegian clays indicates that a plasticity index of 8 to 12 as generally found in Westbrook would have a c/p ratio of 0.12 to 0.16. This c/p ratio is nearly the same as that found at Westbrook, which indicates that the practice in Maine may give similar results to the vane shear tests used by Bjerrum in his correlation.

The undrained shear strengths obtained from boring #3, using the field vane and laboratory CIU test results adjusted to field conditions, are plotted relative to the field vane shear strengths measured for boring B-2 utilizing the river level as a datum (Figure 14). Comparison of stratigraphy, classification properties, and undrained shear strength leads to the conclusion that variability in subsurface conditions across the investigated site is small.

The measured vane shear strength has been correlated to the average strength mobilized in stability failures by Bjerrum (1972). Bjerrum found that the correction factor needed to adjust the measured vane strength to the developed strength correlated well with the plasticity index. For this deposit, Bjerrum's correction is 1.05, i.e. the average developed shear strength is 5 per cent higher than the measured strength. This correction covers differences in the rate of failure between the vane test and the actual failure and includes the effects of progressive failure experienced in a large scale failure.

Undrained shear strength anisotropy is a direct consequence of in situ stress anisotropy (Hansen and Gibson, 1949). The degree of anisotropy becomes more pronounced in low plastic clays (Bjerrum, 1973) suggesting that the deposit at Westbrook, with an average PI near 10, is

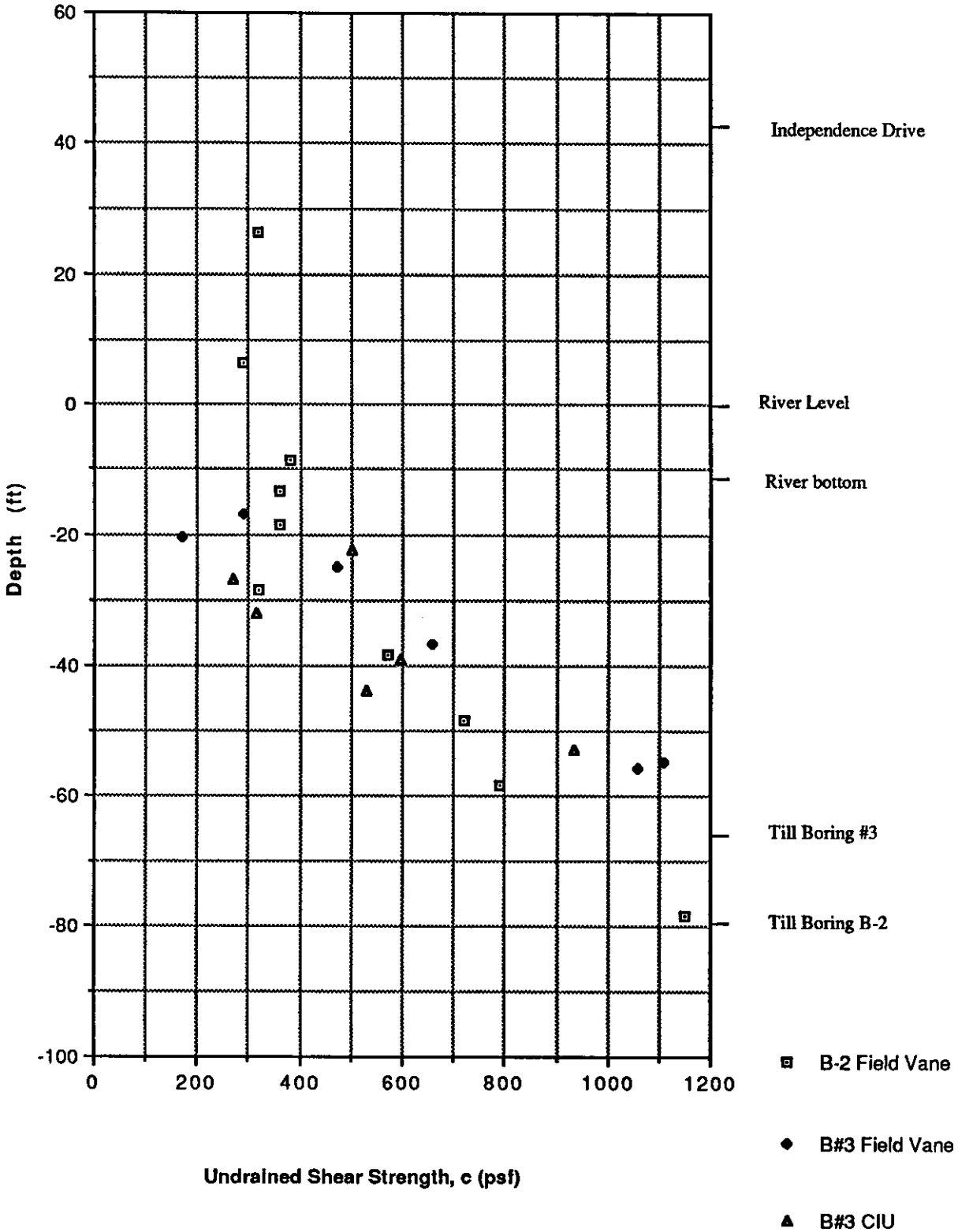


Figure 14. Comparison of undrained shear strengths between boring B-2 and boring #3

likely to exhibit considerable strength anisotropy, and thus the anisotropy should be considered in a stability analysis.

Several assumptions are necessary if the measured field vane shear strength values are to be utilized in such a modified analysis. First, and foremost, the values of the input parameters used in Bjerrum's (1973) theoretical expression for the variation of the undrained shear strength with the inclination of the failure surface are assumed valid for this deposit. These input parameters (such as; PI = 10 and OCR = 1.2) are used in his Figure 16 to develop a relation between the failure plane inclination and the ratio of the undrained shear strength on an inclined plane to the shear strength determined by the field vane. The second major assumption is that the shear strength determined by the 2" x 7" field vane is the shear strength on a vertical plane which can then be modified to estimate the shear strength on planes of various inclinations using the aforementioned relationship. Thus the Bjerrum anisotropic correction effectively indicates that the vane shear strength (vertical plane) is the lowest shear strength measured, and the highest strength measured on a plane inclined approximately 35° with the horizontal is 250 per cent higher.

Remolded Strength

The remolded strength, which controls the stability after movement has occurred, varies from 50 to 180 psf, excluding the value at a depth of 122 ft (Figure 13). The remolded strength increased with depth with a c/p ratio of 0.01 to 0.04 (.02 average). The ratio of the peak undrained strength and the remolded strength, i. e. the sensitivity, ranged from 3.2 to 6.4. These sensitivity values correspond to a material described as medium sensitive to very sensitive by Rosenqvist's (1953) criteria.

The likelihood of viscous flow has been confirmed for sample B-2 ST-2 taken at a depth of 32 feet and having a liquidity index of 2.15 which, when remolded in a beaker, was poured onto a glass plate.

Drained Strength

The drained strength friction angle obtained from the CIU tests is 35°. This compares to a value of 35.6 degrees measured at the Gorham landslide (Amos and Sandford, 1986).

Stress History

To obtain information on stress history, one-dimensional consolidation tests were performed on selected undisturbed samples. However, due to difficulty in performing such tests on sensitive material, the results of only one test was felt to be reliable. The results of this test

sample PO-4 taken at a depth of 72 feet are shown in Figure 15 (also shown is the effect of secondary compression).

Based upon limited consolidation test data, strength data, and natural water contents, the deposit, except the upper 10 to 15 feet, near Boring B-2 at Westbrook appears to be normally consolidated, i.e. it never had a stress larger than the existing overburden. Consolidation test samples are easily disturbed due to the high sensitivity of this material, but one test on sample B-2 PO-4 taken at a depth of 72 feet, yielded reliable results (Figure 15) which didn't show a high degree of sample disturbance. The result here was an overconsolidation ratio (OCR) of 1.06, which is close enough to unity that the deposit can be considered normally consolidated. Considering the effects of sample disturbance, it would seem that the actual OCR would likely be in the range of 1.10 to 1.20 for this sample.

Thus it appears that little surficial erosion has occurred at Westbrook since deposition, and the slight overconsolidation present at a depth of 72 feet can be attributed to secondary compression. If the slight overconsolidation is strictly the result of secondary compression, then the OCR should be relatively constant with depth throughout the deposit (Bjerrum, 1972). The lower water contents of the upper 10 to 15 feet of the deposit indicate that this upper zone has a higher degree of overconsolidation due to chemical weathering and drying. The normal consolidation and the lack of distinct varving indicates that this deposit is the uppermost Presumpscot Formation unit (GM-P) as defined by Belknap et al. (in press). The stress history of this deposit is reflected in low values of the undisturbed undrained shear strength when comparison is made to strengths measured for overconsolidated sites such as at Bunganuc.

Slope Stability

Failure Pattern

A large loss of strength will occur upon disturbance in a sensitive soil. During shear failure, the low remolded shear strength will allow the failed section to move away from the slope making the remaining slope less stable, such as happened near Anchorage, Alaska (Seed and Wilson, 1967). This results in numerous retrogressive failures subsequent to an initial failure. The value of the remolded strength is the most critical factor controlling the extent of a slide once a failure is initiated. The sensitivity and the liquidity index help provide an indication of a material's tendency to retrogress and flow after an initial failure.

Citing the work of Lawrence (1971), Mitchell and Markell (1974) suggest that flowslides (or earthflows using their terminology) occur when in situ stresses within a stable slope reach the

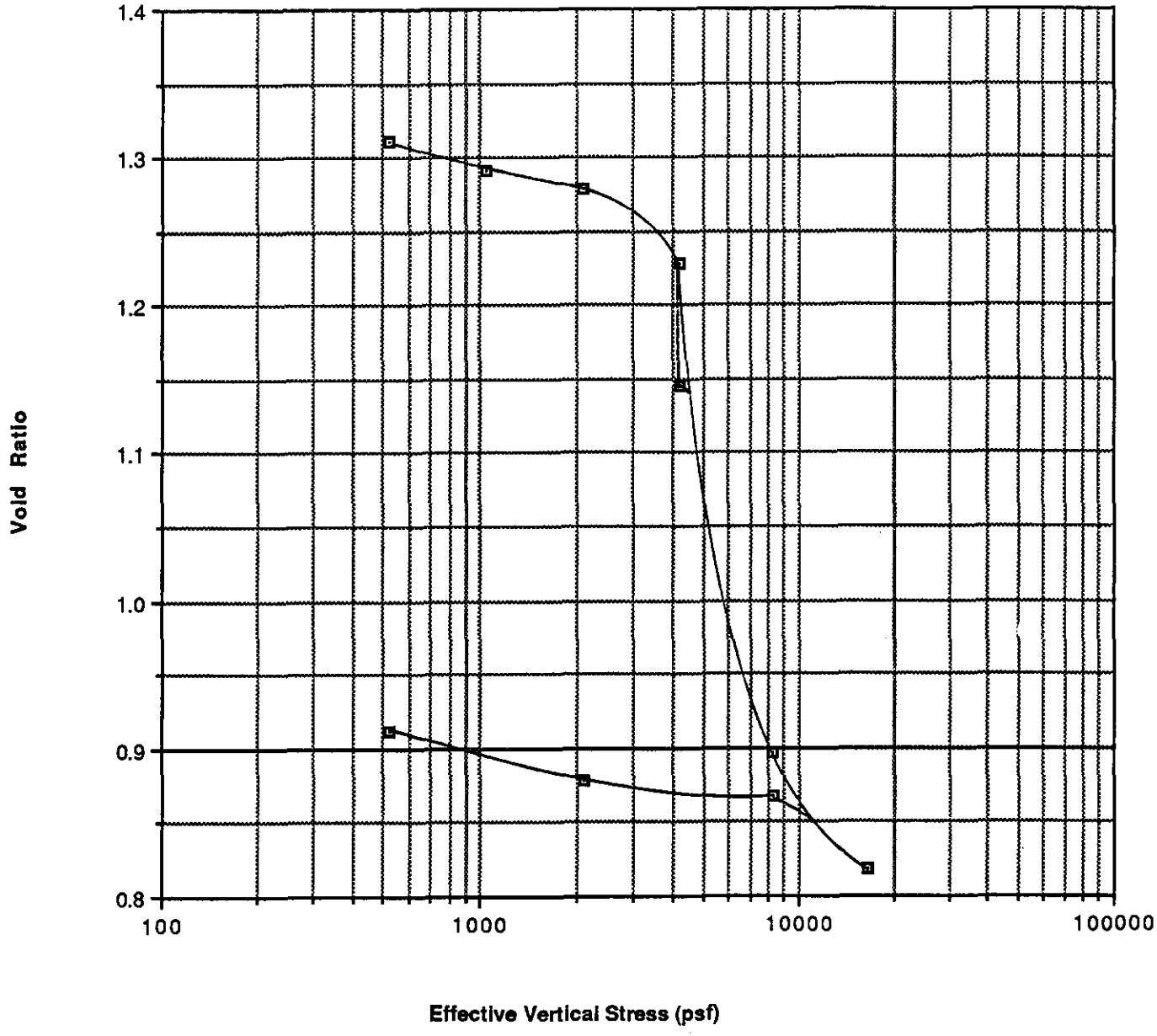


Figure 15. Consolidation results from sample B-2 PO-4

undrained shear strength resulting in a yielded zone. This happens when the factor of safety of the slope for undrained conditions approaches unity. In a sensitive soil, the material within the yielded zone will be extruded and flow upon rupture by an initial failure. Mitchell and Markell (1974) suggest that a flowslide should only occur when the deposit stability number, $\gamma H/c_u$, exceeds six, where H is the depth to points exhibiting an undrained shear strength measured with the field vane, c_u, and γ is the total unit weight of the soil comprising the deposit. For the Westbrook site the total overburden pressure, γH, is plotted vs depth along with $6c_u$ in Figure 16. From this figure it can be seen that γH exceeds $6c_u$ for virtually the entire depth of the boring. This criteria indicates that failure at this site will likely result in a flowslide. The previous descriptions of the 1868 landslide only serve to further reinforce this conclusion.

Analyses

The peak undrained shear strength controls the stability when the slope is rapidly loaded, because shear stresses generate pore pressures which cause shear failure. Rapid loading by steepening a slope or by surcharge loading can cause this type of failure. Therefore, slope stability analyses were performed using STABL6 and the modified Bishop method of slices on the slope corresponding to survey line 1 utilizing the field vane shear strengths measured in the field corrected for strain rate effect (Bjerrum, 1972). Such direct application of measured shear strength values is common practice in Maine. However, when such an application is made, and this slope is analyzed by the modified Bishop method of slices, a factor of safety of 0.75 results! The physical implication of any factor of safety less than unity is that the slope should have failed; it should not be standing. Since the slope is standing, the shear strength which is governing the stability of the slope at Westbrook is higher than that measured in the field vane test.

The existing slope at Westbrook likely has been in place for so long or formed so slowly that all excess pore pressures generated by the imposed shear stresses of the slope have dissipated before shear failure was reached. Thus, the excess pore water pressures generated by any subsequent stress changes will be small. This is not the case in the field vane shear test, and thus there has been a strength gain relative to the undrained condition. However, this condition of imposed shear stress does not apply behind the slope, and thus it is likely that the field vane shear value applies behind the existing slope when sudden stress changes are imposed and excess pore water pressures generated. This implies that if the existing slope should somehow fail, then the exposed slopes would be unstable and the failure would retrogress.

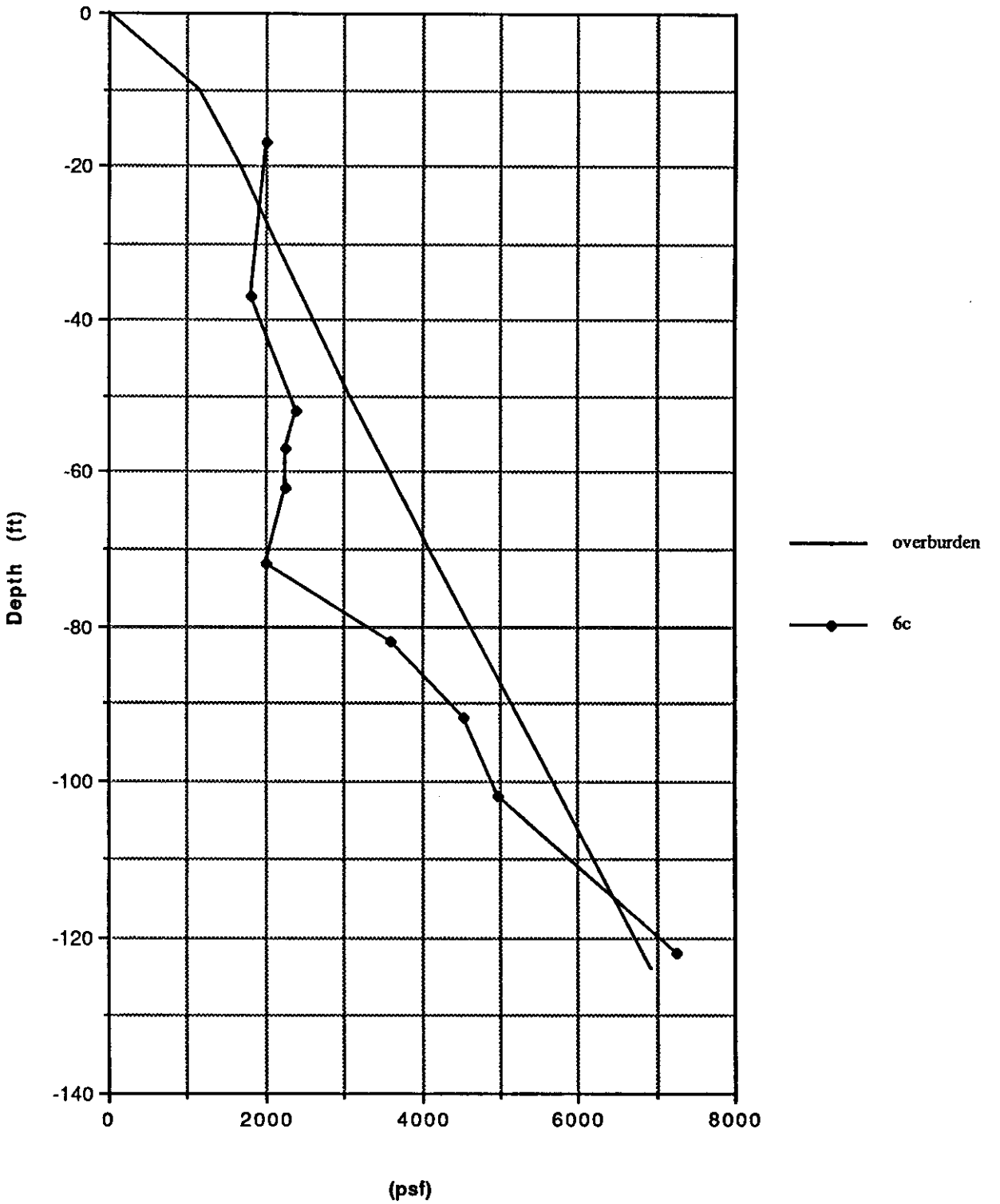


Figure 16. Flowslide criteria at Westbrook

Another explanation for the stable slopes at Westbrook is the presence of undrained shear strength anisotropy. STABL6 can use anisotropic modifications to the measured shear strengths reflecting the variation in strength with failure surface inclination. Such an analysis on survey line 1 results in a factor of safety of 1.19. The approximate location of the critical failure circle is shown in Figure 17.

A second analysis was performed utilizing an expression which effectively transforms vane shear strengths into 'average' isotropic strengths for soils of a given OCR. This relationship is written as $c_u/\sigma'_{vc} = (0.23 \pm 0.04)\ (OCR)^{0.8}$ (Jamiolkowski et al., 1985). Utilization of this relationship with a coefficient of 0.19 and an estimated OCR of 1.20 (based upon a measured value of 1.06 corrected for the effects of sample disturbance) yielded values of the undrained shear strength which were then input into a STABL6. Such an analysis of survey line 1 with STABL6 results in a factor of saftey of 1.13.

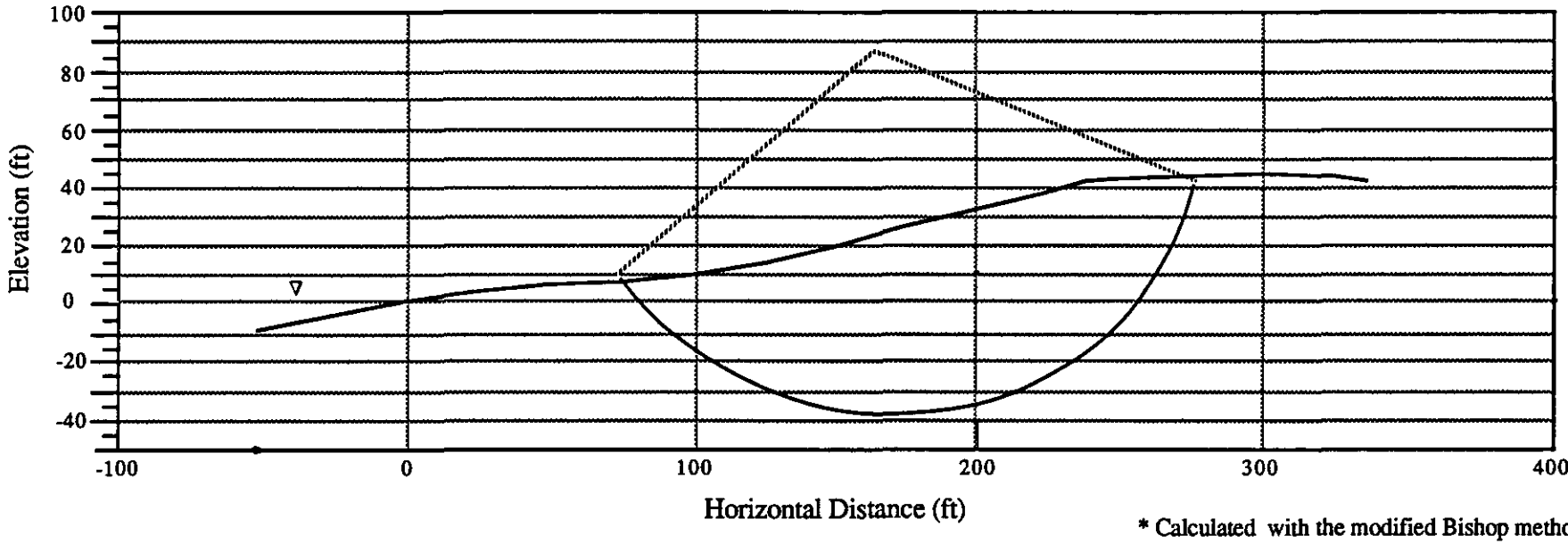
Assessment

To reduce the possibility of a future flowslide at this site, the emphasis must be placed upon the avoidance of an initial slide. Evidence indicates that once an initial slump occurs the subsequent events could be catastrophic and perhaps involve a flowslide of such enormous magnitude as to engulf much of the subdivision as far back as East Bridge Street.

Initial failure may result from a number of different occurrences. These mechanisms may be broadly grouped to include the following: 1) oversteepening of the slope; 2) an excessive surcharge loading near the top of the slope; and/or 3) the remolding of the soil as the result of a major disturbance.

Oversteepening of the slope may result from a number of different events with erosion being most obvious. If erosion at the river bank progresses enough, a slump immediately adjacent to the river may occur and may be followed by retrogression. Erosion may also result from man-made disturbances, such as the failure of the water mains which parallel the river near the toe of the slope. Also, an effective oversteepening may result from excavation near the toe. Thus excavations near the toe must be avoided or, when necessary, closely monitored.

An excessive surcharge may result from the placement of a large quantity of fill near the top of the slope. Therefore the placement of any sort of large surcharge near the top of the slope should be viewed with caution. Buildup of surcharge due to the building of a house near the slope was the likely cause of the massive Gorham landslide, although the failure appeared to have been triggered by vibration (Sandford and Amos, 1987).



* Calculated with the modified Bishop method
Factor of Safety = 1.19
Center of circle @ x = 165 ft
y = 87 ft
radius = 122.5 ft

Figure 17. Westbrook Line 1: Location of the Critical Failure Surface*

The extremely low value of the remolded shear strength arouses concern about disturbance. Sufficient disturbance can produce remolding and thus loss of strength. It is difficult to ascertain the amount of disturbance which would be caused by an earth tremor.

Thus, if precautions are observed then the likelihood of failure at this site may be diminished although not completely eliminated. It is hoped that identification of these mechanisms will provide working criteria that will prevent catastrophe.

Results

1. Prehistoric and historic landslides along the Presumpscot River in Westbrook indicate that this area is susceptible to landslides. One major historic landslide developed into a flowslide. If a flowslide occurred today in developed areas, it would likely be devastating in terms of life and property.
2. The stratigraphy at the site consists of 2 to 3 feet of brown sandy gravel overlying 3 to 4 feet of brown clayey silt crust. The Presumpscot Formation beneath this depth is generally a very soft gray clayey silt to silty clay extending to a depth of 123 feet. Below this depth 4.5 feet of glacial till (very dense gray fine to medium sand, some silt and fine gravel) overlies bedrock (phyllite). The massive, homogeneous character of the Presumpscot Formation at Westbrook suggests that it may be classified as the third unit (GM-P) described by Belknap et al. (in press).
3. Except for the upper 10 to 15 ft, the silty clay deposit is normally consolidated with a natural water content exceeding the liquid limit (liquidity index varying from 0.96 to 2.56). The peak undrained shear strength as measured by the field vane varies with depth from 290 psf at a depth of 37 ft to 790 psf at 102 ft. The c/p ratio ranges from 0.08 to 0.14. The remolded strength varies from 50 to 180 psf. The corresponding sensitivity ranges from 3.2 to 6.4. The drained friction angle is 35 degrees.
4. Conventional practice stability analysis using the peak undrained shear strengths from the field vane test results in a factor of safety of 0.75 indicating that the existing slope should be failing. Since the slope has been intact for some time, drainage and possible shortcomings in the measurement of undrained shear strength by the field vane likely account for the differences in the stability of the existing slope. However, if the existing slope should fail due to loading or disturbance, then the stability of the resulting slope would be controlled by the undrained shear strength and the remolded strength. The existing strength values indicate that a flowslide which would spread over a large area would result after the initial slide.

5. Prevention of the initial slide is a key to continued slope stability in this area. Oversteepening of existing slopes, excessive surcharge loading, or remolding of the soil by a major disturbance should be avoided.

Conclusions

1. Landslides will develop when a slope becomes high and steep enough or is loaded sufficiently to overcome the shear resistance of the soil. With the topographic relief, depth of deposit and shear strength that exists in the Presumpscot Formation of southern and eastern Maine, landslides have occurred in the past at numerous locations. Many landslides fail as a singular slide along a circular failure surface, and damage is limited to the single slide. However, other landslides begin with a single slide and then spread or retrogress to a large area with resulting damage to this area. It is important to be able to identify all potential landslides, but it is especially important to identify landslides that have the potential to spread.

   One site investigated in this study had a failure that didn't spread, and the other has had a past failure that spread to a large area. Bunganuc bluff is a receding coastal bluff due to landslides caused by oversteepening from erosion. The failure, with a generally circular failure surface, does not retrogress into the slope as has happened with prehistoric and historic landslides along the Presumpscot River in Westbrook. One major Westbrook landslide in 1868 developed into a flowslide, which spread to cover more than 20 acres. If a flowslide developed today in this developed area, it would likely be devastating in terms of life and property.

2. At each of the two sites investigated, the Presumpscot Formation is the predominant material, but each belongs to a different geologic unit of the Presumpscot Formation as described by Belknap et al. (in press). The stratigraphy of Bunganuc bluff consists of four feet of very dense brown fine sand overlying the Presumpscot Formation. At this locality the Presumpscot Formation consists of a ten-foot thick very stiff to hard brown clayey silt crust grading to an underlying soft to very soft gray silty clay with fine sand and silt layers. The alternating fine sand and silty clay indicate that the deposit at Bunganuc is the second unit (GM-D) of the Presumpscot Formation.

   The stratigraphy at the Westbrook site consists of 2 to 3 feet of brown sandy gravel overlying 3 to 4 feet of brown clayey silt crust. The Presumpscot Formation beneath this depth is generally a very soft gray silty clay to clayey silt extending to a depth of 123 feet. Below this depth 4.5 feet of glacial till (very dense gray fine to medium sand, some silt and

fine gravel) overlies bedrock (phyllite). The massive, homogeneous character of the Presumpscot Formation at Westbrook suggests that it may be classified as the third unit (GM-P).

3. The Presumpscot material at Bunganuc is 3 to 4 times stronger in undrained shear than the Presumpscot at Westbrook. At Bunganuc the peak undrained shear strength of the deposit ranged from 680 to 1560 psf (excluding the deepest sample) as measured by the field vane. The remolded strength ranged from 200 to 610 psf with a sensitivity range of 2.4 to 3.8. The ratio of the peak vane strength to the overburden pressure (c/p ratio) was 0.34 to 0.39 at this site.

   At Westbrook the peak undrained shear strength as measured by the field vane varies with depth from 290 psf at a depth of 37 ft to 790 psf at 102 ft. The c/p ratio ranges from 0.08 to 0.14. The remolded strength varies from 50 to 180 psf. The corresponding sensitivity ranges from 3.2 to 6.4. The drained friction angle is 35 degrees.

4. The height and steepness of slopes on the verge of failure at Bunganuc are consistent with estimates using vane shear strengths of the deposit, but at Westbrook the existing slopes are standing higher and steeper than indicated possible by vane shear strengths on the deposit. At Bunganuc, when the peak undrained vane shear strengths of the deposit are used in an undrained stability analysis, corrected for tension cracks and associated increased hydrostatic water pressure, the results appear to closely approximate the field behavior.

   Conventional practice stability analysis with the peak undrained strengths from the field vane test on the deposit at Westbrook indicates that the existing slope should be failing (factor of safety of 0.75). Since the slope has been intact for some time, drainage in the slope likely accounts for the differences in the stability of the existing slope. However, if the existing slope should fail due to loading or disturbance, then the newly created scarp would also fail because the stability is controlled by the undrained shear strength and the remolded strength of the deposit. The existing strength values indicate that a flowslide, which would spread over a large area, would result after the initial slide.

5. The material in which the slope failure spreads, such as happened in the past at Westbrook, has the potential for causing the greatest damage and should be identified first. Evidence of

past landslides which have spread is an indicator that conditions may still be present for causing such a landslide. Determination that soil conditions at these sites are still susceptible to retrogressive or flowslides is most directly done with shear strength testing. Shear strength can also determine susceptible areas even if there was no past history of sliding. The liquidity index may serve as a more convenient and less costly indicator of problem areas since the relatively easily measured liquidity index is related to the value of peak shear strength and remolded strength. At Bunganuc the liquidity index averaged 0.67 for a depth of 70 feet. Except for the upper 10 to 15 ft, the silty clay and clayey silt deposit at Westbrook had a liquidity index varying from 0.96 to 2.56. Since the Westbrook site was stratigraphically classified as the third unit GM-P, this unit, with more investigation, may turn out to be a broad indicator of Westbrook type material.

Recommendations

1. Controlling the mechanisms which change a stable slope to an unstable slope is the key to preventing a slide. These mechanisms may be broadly grouped to include the following: 1) oversteepening of the slope; 2) an excessive surcharge loading near the top of the slope; 3) increase in ground water level and/or 4) the remolding of the soil as the result of a major disturbance.

   Oversteepening may result from natural erosion by streams, rivers or ocean waves, or by erosion from a broken utility. It may also result from regrading or excavation near the toe. Surcharges can result from placement of fill or building near the top of the slope. Changes in drainage patterns from regrading or cutoff of natural drainage can cause an increase in ground water level. Pile driving, well drilling or possibly an earthquake can cause remolding of the soil. At each site, the extent of mechanism change which can be tolerated before the slope fails will depend upon specific conditions at the site.

2. Sites in Maine with deposits exhibiting similar geotechnical, geological and geomorphological characteristics as Westbrook and Bunganuc bluff should be identified and mapped for possible site stability problems. Special attention should be given to sites that exhibit soil characteristics similar to Westbrook, since the potential damage area for a single slide may extend for a large distance laterally and into the slope. For sites with possible stability problems, considerations of disturbance, loading, and erosion by construction or natural causes should be integrated into maintenance, planning and development decisions for these areas.

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Appendix I
Final Boring Logs

Boring Log Symbols and Abbreviations

SS Split spoon

ST Shelby tube

PO Osterberg Piston

CB Core Barrel

FV Field Vane

Abbreviations

GWT Groundwater table

HP Hydraulic push

WM Weight of man

WR Weight of rods

WH Weight of hammer

RQD Rock quality designation

Definition of terms

trace	0 - 10% by weight
some	10 - 20% by weight
adjective (-y)	20 - 35% by weight
and	35 - 50% by weight
name	> 50% by weight

FINAL LOG - SUBSURFACE EXPLORATION

Boring No. B-1
Sheet 1 of 3

Job Title: Bunganuc B-1
Location: Bunganuc Point, Brunswick, ME
Date Start: 11 June 1987 Date Finish: 12 June 1987
Drilling Contractor: Maine Test Borings
Driller: T Schaefer Inspector: S Devin

	Sampler	Casing	Core Barrel
Type	SS	HW	NQ2
Size (i.d.)	1 3/8"	4"	2 1/4"

Sample Hammer: wt 140 lb drop 30"
Casing Hammer: wt 300 lb drop 16"

Depth (Elevation) in feet	Blows per 6 inches on sampler	Unified Soil Classification Group Symbol	Sample Number	Description of Material	water content %	liquid limit %	plasticity index %	grain size	triaxial test	Remarks
0										
	4, 16, 35		SS-1	Very dense brown fine sand some silt	21.2	NP	NP			Augered to 5 ft
5	10, 11, 15	CL	SS-2	Very stiff brown clayey silt	25.7	34	12			GWT observ. @ 12.7' 6/11 5:30 PM @ 13.1' 6/12 8:00 AM
10	10, 22, 24		SS-3	Hard brownish gray mottled clayey silt	31.5	40	15			
15	HP		ST-1		35.3	32	11			undisturbed c = 680 psf
			FV-1							remolded c = 200 psf
20	WM		SS-4	Soft to very soft gray silty clay trace fine sand	30.0	30	10			
25	WM, 2, 2		SS-5		27.6	27	8			
30	HP		ST-2		28.2	33	12			undisturbed c = 1110 psf
			FV-2							remolded c = 290 psf

FINAL LOG - SUBSURFACE EXPLORATION

Boring No. B-1
Sheet 2 of 3

Job Title	Bunganuc B-1				
Location	Bunganuc Point, Brunswick, ME				
Date Start	11 June 1987	Date Finish	12 June 1987		
Drilling Contractor	Maine Test Borings				
Driller	T Schaefer	Inspector	S Devin		

	Sampler	Casing	Core Barrel
Type	SS	HW	NQ2
Size (i.d.)	1 3/8"	4"	2 1/4"

Sample Hammer: wt 140 lb drop 30"
Casing Hammer: wt 300 lb drop 16"

Depth (Elevation) in feet	Blows per 6 inches on sampler	Unified Soil Classification Group Symbol	Sample Number and Description of Material	Summary of Test Results: water content %	liquid limit %	plasticity index %	grain size	triaxial test	Remarks
35	2,4,6	CL	SS-6	27.0	32	10			
40	1,2,6		SS-7	28.0	33	11			
45	HP		ST-3 FV-3	30.4	32	10			undisturbed c = 1310 psf remolded c = 540 psf
50	3,6,7		SS-8	29.1	34	12			
			Soft to very soft gray silty clay trace fine sand						
55	3,6,8		SS-9	30.6	35	12			
60	HP		ST-4 FV-4	33.8	44	21			undisturbed c = 1560 psf remolded c = 610 psf
65	6,10,16		SS-10	32.5	40	15			

FINAL LOG - SUBSURFACE EXPLORATION

Boring No. B-1
Sheet 3 of 3

Job Title: Bunganuc B-1
Location: Bunganuc Point, Brunswick, ME
Date Start: 11 June 1987 Date Finish: 12 June 1987
Drilling Contractor: Maine Test Borings
Driller: T Schaefer Inspector: S Devin

	Sampler	Casing	Core Barrel
Type	SS	HW	NQ2
Size (i.d.)	1 3/8"	4"	2 1/4"

Sample Hammer: wt 140 lb drop 30"
Casing Hammer: wt 300 lb drop 16"

Depth (Elevation) in feet	Blows per 6 inches on sampler	Unified Soil Classification Group Symbol	Sample Number and Description of Material		Summary of Test Results: water content %	liquid limit %	plasticity index %	grain size	triaxial test	Remarks
70	HP	CL	ST-5 FV-5	Soft to very soft gray silty clay trace fine sand	23.9	31	13			undisturbed c = 2390 psf remolded c = 970 psf
				Bottom of boring @ 72.4'						
75										
80										
85										
90										
95										
100										

FINAL LOG - SUBSURFACE EXPLORATION

Boring No. B-2
Sheet 1 of 4

Job Title: Westbrook B-2
Location: Independence Dr., Westbrook, ME
Date Start: 15 June 1987 Date Finish: 19 June 1987
Drilling Contractor: Maine Test Borings
Driller: T Schaefer Inspector: S Devin

	Sampler	Casing	Core Barrel
Type	SS	HW	NQ2
Size (i.d.)	1 3/8"	4"	2 1/4"

Sample Hammer: wt 140 lb drop 30"
Casing Hammer: wt 300 lb drop 16"

Depth (Elevation) in feet	Blows per 6 inches on sampler	Unified Soil Classification Group Symbol	Sample Number and Description of Material		water content %	liquid limit %	plasticity index %	grain size	triaxial test	Remarks
0		GP		Brown sandy gravel						
		ML		Brown clayey silt some sand						Augered to 6 ft
5										GWT observ. @ 5.2" 6/16 6:00 AM
	WH,WH,2	CL	SS-1	Very soft gray silty clay, trace fine sand	39.3	40	16			@ 13.1' 6/18 9:30 AM
10	WH,WH,1		SS-2		44.7	42	17			
15	WR		ST-1	Very soft gray silty clay	37.6	30	10			
			FV-1							undisturbed c = 320 psf remolded c = 90 psf
20	WM		SS-3		38.8	39	15			
25	WR,WR,WM	ML	SS-4		50.2	38	13			
30	WR	CL	SS-5		51.0	36	13			

FINAL LOG - SUBSURFACE EXPLORATION

Boring No. B-2
Sheet 2 of 4

Job Title: Westbrook B-2
Location: Independence Dr., Westbrook, ME
Date Start: 15 June 1987 Date Finish: 19 June 1987
Drilling Contractor: Maine Test Borings
Driller: T Schaefer Inspector: S Devin

	Sampler	Casing	Core Barrel
Type	SS	HW	NQ2
Size (i.d.)	1 3/8"	4"	2 1/4"

Sample Hammer: wt 140 lb drop 30"
Casing Hammer: wt 300 lb drop 16"

Depth (Elevation) in feet	Blows per 6 inches on sampler	Unified Soil Classification Group Symbol	Sample Number and Description of Material		Summary of Test Results: water content %	liquid limit %	plasticity index %	grain size	triaxial test	Remarks
35	WR	ML	ST-2							highly disturbed
			FV-2							undisturbed c = 290 psf remolded c = 90 psf
40	WR		SS-6		44.5	32	8			
45	WR		SS-7	Very soft gray silty clay	47.4	35	11			
50	HP	CL	PO-1			33	10			
			FV-3							undisturbed c = 380 psf remolded c = 90 psf
55	HP		PO-2							
			FV-4							undisturbed c = 360 psf remolded c = 70 psf
60	HP		PO-3							
			FV-5							undisturbed c = 360 psf remolded c = 70 psf
65	10,5,3		SS-8	Very soft gray silty clay with silt layers	42.7	30	9			

FINAL LOG - SUBSURFACE EXPLORATION

Boring No. B-2
Sheet 3 of 4

Job Title: Westbrook B-2
Location: Independence Dr., Westbrook, ME
Date Start: 15 June 1987 Date Finish: 19 June 1987
Drilling Contractor: Maine Test Borings
Driller: T Schaefer Inspector: S Devin

	Sampler	Casing	Core Barrel
Type	SS	HW	NQ2
Size (i.d.)	1 3/8"	4"	2 1/4"

Sample Hammer: wt 140 lb drop 30"
Casing Hammer: wt 300 lb drop 16"

Depth (Elevation) in feet	Blows per 6 inches on sampler	Unified Soil Classification Group Symbol	Sample Number and Description of Material	Summary of Test Results: water content %	liquid limit %	plasticity index %	grain size	triaxial test	Remarks
70	HP	CL	PO-4	41.5	32	10			undisturbed c = 320 psf
			FV-6						remolded c = 50 psf
75	WR		SS-9	42.5	36	12			
80	HP		PO-5		41	16			
			FV-7						undisturbed c = 570 psf
			Very soft gray silty clay						remolded c = 160 psf
85	WR		SS-10	52.9					
90	HP		PO-6						
			FV-8						undisturbed c = 720 psf
									remolded c = 180 psf
95	WR		SS-11	40.2	35	12			
100	HP		PO-7		35	11			undisturbed c = 790 psf
			FV-9						remolded c = 140 psf

FINAL LOG - SUBSURFACE EXPLORATION

Boring No. B-2
Sheet 4 of 4

Job Title: Westbrook B-2
Location: Independence Dr., Westbrook, ME
Date Start: 15 June 1987 Date Finish: 19 June 1987
Drilling Contractor: Maine Test Borings
Driller: T Schaefer Inspector: S Devin

	Sampler	Casing	Core Barrel
Type	SS	HW	NQ2
Size (i.d.)	1 3/8"	4"	2 1/4"

Sample Hammer: wt 140 lb drop 30"
Casing Hammer: wt 300 lb drop 16"

Depth (Elevation) in feet	Blows per 6 inches on sampler	Unified Soil Classification Group Symbol	Sample Number and Description of Material		Summary of Test Results: water content %	liquid limit %	plasticity index %	grain size	triaxial test	Remarks
105	WR	CL	SS-12	Very soft gray silty clay	37.6	32	10			
110	WR	CL	SS-13	Very soft gray silty clay	40.5	31	10			
115	WR	CL	SS-14	Very soft gray silty clay	35.2	30	10			
120	HP	CL	PO-8, FV-10	Very soft gray silty clay		26	9			undisturbed c = 1150 psf remolded c = 360 psf
125	100,100	CL	SS-15	Very dense gray fine to medium sand, some silt and fine gravel				✓		100,100 for 0.2 ft
130	Run 1 (93 %) RQD 0.70		CB-1	Phyllite (Berwick Formation)						Rock @ 127.4'
135										

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. 3
SHEET 1 OF 2

SUBSURFACE EXPLORATION LOG

JOB NO. 2296 JOB TITLE PORTLAND PIPE LINE CORPORATION
LOCATION PORTLAND, MAINE COORDINATES STA. 1 + 34 DATE 7-20-65
DRILL ACKER RGT ANGLE VERTICAL REFERENCE ELEV. +19.5 Feet DATUM MEAN LOW WATER
DRILLING CONTRACTOR N.E. SOIL SERVICE DRILLER P. BROWN INSPECTOR B. TORIKOGLU
SAMPLE HAMMER: WT. 140 lbs. DROP 30 inches CASING HAMMER: WT 280 lbs. DROP 18 inches

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT %	LIQUID LIMIT %	PLASTICITY INDEX %	DRY UNIT WEIGHT PCF	SPECIFIC GRAVITY
0											
5											
10 (+8.7)			Bottom of river								
						WOH					
	(90%)		1	TSP	4"	2					
15	(80%)		2			2					
						4					
			V-1 (288 psf)			4					
	(80%)		3			5					
20		CL	Dark gray soft CLAY			8					
			V-2 (169 psf)			7					
						6					
	(70%)		4			5	42	39	15	82	2.79
25						6					
			V-3 (470 pfs)			7					
						5					
	(100%)		5			5	41	36	17	84	2.83
(-8.8)				13/8" S.S.		4					
	WOH					6					
30	(100%)		6			5					
						5					
			Gray black very soft CLAY	TSP		4	46	46	24	75	2.81
	(90%)	CL	7			6					
	WOR/12" WOH/12"			13/8" S.S.		6					
35	(100%)		8			5					

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. 3
SHEET 2 OF 2

SUBSURFACE EXPLORATION LOG

JOB NO. 2296 JOB TITLE PORTLAND PIPE LINE CORPORATION
LOCATION PORTLAND, MAINE COORDINATES STA. 1 + 34 DATE 7-20-65
DRILL ACKER RGT ANGLE VERTICAL REFERENCE ELEV. + 19.5 Feet DATUM MEAN LOW WATER
DRILLING CONTRACTOR N.E. SOIL SERVICE DRILLER P. BROWN INSPECTOR B. TORIKOGLU
SAMPLE HAMMER: WT. 140 lbs. DROP 30 inches CASING HAMMER: WT 280 lbs. DROP 18 inches

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT %	LIQUID LIMIT - %	PLASTICITY INDEX %	DRY UNIT WEIGHT PCF	SPECIFIC GRAVITY
35			V-4 (658 psf)			6, 7, 6					
	(90%)		9	TSP		7, 6, 6, 6, 6	41	33	11	81	2.76
40											
	(80%)		10 Gray - black soft to medium stiff CLAY	TSP		5, 4, 6, 8, 9, 9, 9, 12, 14	40	39	17	82	2.86
45		CL									
50											
	(85%)		11	TSP		13, 12, 17, 15, 15, 20	43	43	19	80	2.81
55			V-5 (1110 psf)	13 8" SS							
			V-6 (1060 psf)								
	WOH,1,3 / 12" (0%)		12			18, 22					
60 (-40.5)	(70%)		13	TSP							
	WOH,1,3,4 / 9" (100%)		14	13/8" S.S.							
		CL	Gray-black soft CLAY, some layers of black organic silt								
65 (-45.5)		CL OH	Gray-black medium stiff fine sandy CLAY & organic SILT								
(-47.5)	1,1,7,8 (90%)	GP	15 Gray silty fine GRAVEL & SAND								
(-48.1)			Bottom of boring								
70											

BREWER, MAINE 04412

Jordan-Gorrill Associates

HOLE NO. PR-3

DRILLER: Hatch	PROJECT NAME: Presumpscot Interceptor	LINE & STATION: PRI ℄ 54+10
JOB NUMBER: 914-76	LOCATION: Westbrock, Maine	OFFSET: 45' LT.

GROUND WATER OBSERVATIONS

AT 6.5' FT AFTER 24 HOURS

AT ____ FT. AFTER ____ HOURS

	CASING	SAMPLER	CORE BARREL
TYPE	BW	SS	
SIZE I.D.	2 1/2	1 3/8	
HAMMER WT.	300	140	
HAMMER FALL	16"	30"	

DATE START 3/13/73 DATE FIN. 3/14/73

SURFACE ELEV. 22.0 ±

GROUND WATER ELEV. ____

CASING BLOWS PER FOOT	SAMPLE NO.	O.D.	PEN.	REC.	DEPTH @ BOT.	BLOWS PER 6" ON SAMPLER 0-6	6-12	12-18	VANE READING	DEPTH	STRATUM DESCRIPTION
2											
3											
2	1D	2"	18"		3.5	1	1	1			
3											
3											Brown Silty Fine Sand WITH ROOTS, BARK & WOOD
1	2D	2"	18"		6.5	1	1	1			
1											
1											~ VERY LOOSE ~
6										9.0	
7											
5	1C	2"	24"		12.0	W/of 2 men				W=43.0 %	
3	Vane	2	x 7		12'7"				15/5	C=.13 TSF	
4	"	"	" "		13'2"				20/5	Sv=.30 "	
3										=.40 "	Gray Silty Clay WITH SILT AND SAND LAYERS
9	2C	2"	24"		17.0	W/of 2 men				W=42.8 %	
7	Vane	2	x 7		17'7"				15/5	Sv=0.30 TSF	
6	"	"	" "		18'2"				15/5	=0.30 "	
5											SOFT TO MEDIUM AND SENSITIVE
5											
	3C	2"	24"		22.0	W/of 2 men				W=39.8 %	
	Vane	2	x 7		22'7"				20/5	C=0.11 TSF	
	"	"	" "		23'2"				20/5	Sv=0.40 "	
										0.46 "	
										23.2	
											Bottom of boring @ 23.2

PROPORTIONS

AND = 35-50%

SOME = 10-35%

TRACE = 0-10%

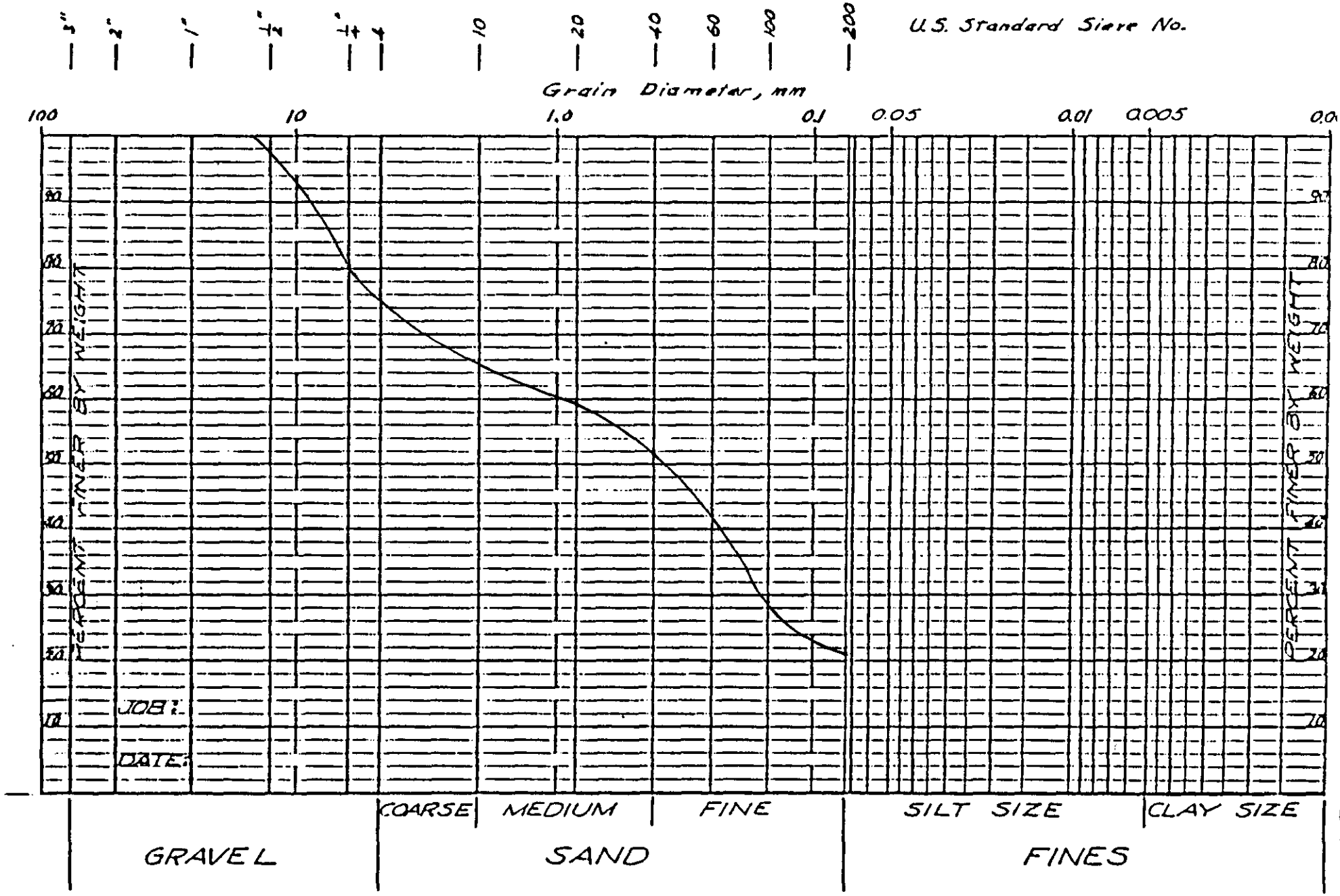
SOIL CLASSIFIED BY:

- [X] DRILLER-VISUALLY
- [✓] SOIL TECHNICAN-VISUALLY
- [✓] LABORATORY TESTS

REMARKS:

HOLE NO. PR-3

Appendix II
Grain Size Analysis
Sample B-2 SS-15



A.S.T.M. Classification
Designation D 422-61 T

~ GRAIN SIZE DISTRIBUTION ~

Appendix III
Driller's Logs

MAINE TEST BORINGS, INC.
BREWER, MAINE 04412

Maine Geological Survey

SHEET 1 OF 2
HOLE NO. B-1

DRILLER: Tom Schaefer
M.T.B. JOB NUMBER: 87-142
PROJECT NAME: Landslide Research
LOCATION: Brunswick, Maine
LINE & STATION:
OFFSET:

GROUND WATER OBSERVATIONS
AT ____ FT. AFTER ____ HOURS
AT ____ FT. AFTER ____ HOURS

	CASING	SAMPLER	CORE BARREL
TYPE	HW	SS	
SIZE I.D.	4"	1 3/8"	
HAMMER WT.	300	140	
HAMMER FALL	16"	30"	

DATE START 6/11/87 DATE FIN. 6/11/87
SURFACE ELEV.
GROUND WATER ELEV.

CASING BLOWS PER FOOT	SAMPLE NO.	O.D.	PEN.	REC.	DEPTH @ BOT	BLOWS PER 6" ON SAMPLER 0-6	6-12	12-18	VANE READING	DEPTH	STRATUM DESCRIPTION
Auger											Brown silty fine sand
"										2.5	
"	1D	2"	18"		3.5	4	16	35			Brown silty fine sand w/brown sandy silt layers
"											
"											
26	2D	2"	18"		6.5	10	11	15		6.2	Brown sandy clayey silt
38											
68										8.0	Mottled clayey silt
112											
122											
54	3D	2"	18"		11.5	10	22	24			
80											
107											
110										14.0	Gray silty clay
113											
77											
90	1S	3"	24"		17.0	Hyd.	Push				
85	2x7	vane			17'7"				20/6		
74	2x7	vane			18'2"				18/5		
67											
67	4D	2"	18"		21.5	Wt.	of 1	Man			
64											
64											
60										24.0	Gray silty clay w/silty fine sand layers
69											
80	5D	2"	18"		26.5	W01M	2	2			
63											
60											
60											
58											
82											
60	2S	3"	24"		32.0	Hyd.	push				
60	2x7	vane			32'7"				32/8		
60	2x7	vane			33'2"				30/8		
65											
40	6D	2"	18"		36.5	2	4	6			
46											
47											
60											
63											

SAMPLES
D - Split Spoon
C - 2" Shelby Tube
U - 3½" Shelby Tube

SOIL CLASSIFIED BY:
[X] Driller Visually
[ ] Soil Technican - Visually
[ ] Laboratory Tests

REMARKS

HOLE NO. B-1

MTB-14

MAINE TEST BORINGS, INC.
BREWER, MAINE 04412

CLIENT: Maine Geological Survey

SHEET 2 OF 2
HOLE NO. B-1

DRILLER: Tom Schaefer
M.T.B. JOB NUMBER: 87-142

PROJECT NAME: Landslide Research
LOCATION: Brunswick, Maine

LINE & STATION:
OFFSET:

GROUND WATER OBSERVATIONS
AT ____ FT AFTER ____ HOURS
AT ____ FT. AFTER ____ HOURS

	CASING	SAMPLER	CORE BARREL
TYPE	HW	SS	
SIZE I.D.	4"	1 3/8"	
HAMMER WT.	300	140	
HAMMER FALL	16"	30"	

DATE START 6/11/87 DATE FIN. 6/11/87
SURFACE ELEV. ____
GROUND WATER ELEV. ____

CASING BLOWS PER FOOT	SAMPLE NO	O.D.	PEN.	REC.	DEPTH @ BOT	BLOWS PER 6" ON SAMPLER 0-6	6-12	12-18	VANE READING	DEPTH	STRATUM DESCRIPTION
57	7D	2"	18"		41.5	1	2	6			
62											
69											
75											
82											
60											
51	3S	3"	24"		47.0	Hyd.	Push				
61	2x7	vane			47'7'				38/15		
64	2x7	vane			48'2'				35/15		
65											
66	8D	2"	18"		51.5	3	6	7			
67											
79											
90											Gray silty clay w/silty fine sand layers
109											
65	9D	2"	18"		56.5	3	6	8			
57											
52											
60											
65											
52											
59	4S	3"	24"		62.0	Hyd.	Push				
61	2x7	vane			62'7'				42/16		
60	2x7	vane			63'2'				45/18		
58											
54	10D	2"	18"		66.5	6	10	16			
61											
66											
71											
78											
	5S	3"	24"		72.0	Hyd.	Push				
	2x7	vane			72'7'				68/28	73.2	
	2x7	vane			73'2'				65/26		
											Bottom of Boring @ 73.2'

SAMPLES
D - Splite Spoon
C - 2" Shelby Tube
U - 3½" Shelby Tube

SOIL CLASSIFIED BY:
[X] Driller - Visually
[ ] Soil Technican - Visually
[ ] Laboratory Tests

REMARKS

HOLE NO. B-1

MTB-14

BREWER, MAINE 04412

Maine Geological Survey

SHEET 1 OF 4

HOLE NO. B-2

DRILLER	PROJECT NAME	LINE & STATION
Tom Schaefer	Landslide Research	
M.T.B. JOB NUMBER	**LOCATION**	**OFFSET**
87-142	Westbrook, Maine	

GROUND WATER OBSERVATIONS

AT ____ FT. AFTER ____ HOURS

AT ____ FT. AFTER ____ HOURS

	CASING	SAMPLER	CORE BARREL
TYPE	4"	SS	NQ2
SIZE I.D.	4"	1 3/8"	
HAMMER WT.	300	140	
HAMMER FALL	16"	30"	

DATE START 6/15/87 DATE FIN. 6/18/87

SURFACE ELEV. ____

GROUND WATER ELEV. ____

CASING BLOWS PER FOOT	SAMPLE NO.	O.D.	PEN.	REC.	DEPTH @ BOT	BLOWS PER 6" ON SAMPLER 0-6	6-12	12-18	VANE READING	DEPTH	STRATUM DESCRIPTION
										0.4	Pavement
Auger											
"											Brown sandy gravel
"										3.0	
"											Brown clayey silt w/some sand
"											
"										6.0	
"											
Hyd Push	1D	2"	18"		8.5	WOH	WOH	2			Gray silty clay w/fine sand
"											
"										10.0	
WOH	2D	2"	18"		11.5	WOH	WOH	1			
WOH											
WOH											
WOH											
2											
WOH											
WOH	1S	3"	24"		17.0	Hyd.	Push				
WOH	2x7	vane			17'7"				8/2		
WOH	2x7	vane			18'2"				10/3		
WOH											
WOH	3D	2"	18"		21.5	Wt.	of 1	man			
WOH											
WOH											
WOH											
WOH											
WOH	4D	2"	18"		26.5	WOR	WOR	WO1M			
WOH											
WOH											Gray silty clay
WOH											
WOH											
WOC											
WOC	S	3"	24"	0	32.0	Wt.	of rods				
WOC	5D	2"	24"		32.0	Wt.	of rods				
WOC											
WOC											
WOC											
WOC	2S	3"	24"		37.0	Wt.	of rods				
WOC	2x7	vane			37'7"				8/3		
WOC	2x7	vane			38'2"				8/2		
WOC											

SAMPLES

D = Split Spoon

C = 2" Shelby Tube

U = 3½" Shelby Tube

SOIL CLASSIFIED BY:

[x] Driller - Visually

[ ] Soil Technican - Visually

[ ] Laboratory Tests

REMARKS

HOLE NO. B-2

MTB-14

T BORINGS, INC.
ELWIN, MAINE 04412

Maine Geological Survey

SHEET 2 OF 4
HOLE NO. B-2

DRILLER	PROJECT NAME	LINE & STATION
Tom Schaefer	Landslide Research	
M.T.B. JOB NUMBER	**LOCATION**	**OFFSET**
87-142	Westbrook, Maine	

GROUND WATER OBSERVATIONS

AT ____ FT. AFTER ____ HOURS

AT ____ FT. AFTER ____ HOURS

	CASING	SAMPLER	CORE BARREL
TYPE	4"	SS	NQ2
SIZE I.D.	4"	1 3/8"	
HAMMER WT.	300	140	
HAMMER FALL	16"	30"	

DATE START 6/15/87 DATE FIN. 6/18/87
SURFACE ELEV. ____
GROUND WATER ELEV. ____

CASING BLOWS PER FOOT	SAMPLE NO.	O.D.	PEN.	REC.	DEPTH @ BOT	BLOWS PER 6" ON SAMPLER 0-6	6-12	12-18	VANE READING	DEPTH	STRATUM DESCRIPTION
WOC											
WOC	S	3"	24"	0	42.0	Wt. of rods					
WOC	6D	2"	24"		42.0	Wt. of rods					
WOC											
WOC											
WOC											
WOC	S	3"	24"	0	47.0	Wt. of rods					
WOC	7D	2"	24"		47.0	Wt. of rods					
WOC											
WOC											
WOC											
WOC	3S	3"	24"		52.0	Piston sampler					
WOC	2x7	vane			52'7"				10/3		Gray silty clay
WOC	2x7	vane			53'2"				11/2		
WOC											
WOC											
WOC	4S	3"	24"		57.0	Piston sampler					
WOC	2x7	vane			57'7"				10/2		
WOC	2x7	vane			58'2"				10/2		
WOC											
25											
WOH	5S	3"	24"		62.0	Piston sampler					
WOH	2x7	vane			62'7"				10/2		
WOH	2x7	vane			63'2"				10/2	64.0	
WOH											Gray silty clay w/silt layers
9											
WOH										67.0	
WOC											
WOC											
WOC											
2											
WOC	6S	3"	24"		72.0	Piston sampler					
WOC	2x7	vane			72'7"				10/2		
WOC	2x7	vane			73'2"				8/1		Gray silty clay
WOC											
WOH	8D	2"	18"		76.5	Wt. of rods					
WOC											
WOC											
WOC											
WOC										80.0	

SAMPLES
D = Split Spoon
C = 2" Shelby Tube
U = 3½" Shelby Tube

SOIL CLASSIFIED BY:
[X] Driller - Visually
[ ] Soil Technican - Visually
[ ] Laboratory Tests

REMARKS

HOLE NO. B-2

MTB-14

MAINE [illegible] S, INC.
BREWER, MAINE 04412

Maine Geological Survey, Inc.

SHEET 3 OF 4
HOLE NO. B-2

DRILLER: Tom Schaefer	PROJECT NAME: Landslide Research Inc	LINE & STATION
M.T.B. JOB NUMBER: 87-142	LOCATION: Westbrook, Maine	OFFSET

GROUND WATER OBSERVATIONS

AT _____ FT. AFTER _____ HOURS

AT _____ FT. AFTER _____ HOURS

	CASING	SAMPLER	CORE BARREL
TYPE	4"	SS	NQ2
SIZE I.D.	4"	1 3/8"	
HAMMER WT.	300	140	
HAMMER FALL	16"	30"	

DATE START 6/15/87 DATE FIN. 6/18/87

SURFACE ELEV. _____

GROUND WATER ELEV. _____

CASING BLOWS PER FOOT	SAMPLE NO.	O.D.	PEN.	REC.	DEPTH @ BOT.	BLOWS PER 6" ON SAMPLER 0-6	6-12	12-18	VANE READING	DEPTH	STRATUM DESCRIPTION
WOC											
WOC											
WOC	7S	3"	24"		83.0	Piston sampler					
WOC	2x7	vane			83'7"				14/4		
WOC	2x7	vane			84'2"				18/5		
WOC	9D	2"	18"		86.5	Wt. of rods					
WOC											
WOC											
WOC											
WOC											
WOC											
WOC	8S	3"	24"		92.0	Piston sampler					
WOC	2x7	vane			92'7"				20/5		
WOC	2x7	vane			93'2"				20/5		
WOC											
WOC	10D	2"	18"		96.5	Wt. of rods					
WOC											
WOC											
WOC											
WOC											Gray silty clay w/dark gray streaks
WOC											
WOC	9S	3"	24"		102.3	Piston sampler					
WOC	2x7	vane			102'11"				22/4		
WOC	2x7	vane			103'6"				22/4		
WOC											
WOC	11D	2"	18"		106.5	Wt. of rods					
WOC											
WOC											
WOC											
WOC											
WOC	12D	2"	18"		111.5	Wt. of rods					
WOC											
WOC											
WOC											
WOC											
WOC											
WOC											
WOC	13D	2"	46"		118.8	Wt. of rods					
WOC											
WOC											

SAMPLES
D = Splite Spoon
C = 2" Shelby Tube
U = 3½" Shelby Tube

SOIL CLASSIFIED BY:
[X] Driller - Visually
[ ] Soil Technican - Visually
[ ] Laboratory Tests

REMARKS

HOLE NO. B-2

MTB-14

MAINE TEST BORING CO., INC.
BREWER, MAINE 04412

Maine Geological Survey, Inc.

SHEET 4 OF 4
HOLE NO. B-2

DRILLER: Tom Schaefer
M.T.B. JOB NUMBER: 87-142

PROJECT NAME: Landslide Research Inc
LOCATION: Westbrook, Maine

LINE & STATION:
OFFSET:

GROUND WATER OBSERVATIONS
AT ____ FT. AFTER ____ HOURS
AT ____ FT. AFTER ____ HOURS

	CASING	SAMPLER	CORE BARREL
TYPE	4"	SS	NQ2
SIZE I.D.	4"	1 3/8"	
HAMMER WT.	300	140	
HAMMER FALL	16"	30"	

DATE START 6/15/87 DATE FIN. 6/18/87
SURFACE ELEV.
GROUND WATER ELEV.

CASING BLOWS PER FOOT	SAMPLE NO.	O.D.	PEN.	REC.	DEPTH @ BOT.	BLOWS PER 6" ON SAMPLER 0-6	6-12	12-18	VANE READING	DEPTH	STRATUM DESCRIPTION
68											
7	10S	3"	24"		122.0	Piston sampler					Gray silty clay w/dark gray streaks
5	2x7	vane			122'7"				38/10	123.0	
40	2x7	vane			123'2"				50+	124.0	Gray sandy clayey silt
85											
WA	14D	2"	8"		126.7	100	100			127.4	Gray medium to fine sand w/some gravel, coarse sand and silt
											Rock
	1R	3"	4.5	4.2	131.9	93%				131.9	
											Bottom of Boring @ 131.9'

SAMPLES
D = Split Spoon
C = 2" Shelby Tube
U = 3½" Shelby Tube

SOIL CLASSIFIED BY:
[X] Driller - Visually
[ ] Soil Technican - Visually
[ ] Laboratory Tests

REMARKS:

HOLE NO. B-2

MTB-14

Appendix IV
Comparison of Maine Practice versus ASTM
Vane Shear Test

1) The dimensions of the 2" x 7" field vane used at Westbrook and Bunganuc differ from the ASTM accepted standard height to diameter ratio of 2.0.
2) The accuracy of the torque wrench used to measure the applied torque does not meet the ASTM standards which state that the variation of the determined shear strength should not exceed ± 25 psf. Assuming that the torque wrench used to measure the applied torque in these tests was accurate to 1 lb-ft, the resulting variation in the shear strength would be approximately ± 36 psf.
3) The ASTM standards suggest that vane penetration below the bottom of the borehole should exceed five casing diameters while for the testing performed for this report the vane only penetrated 1.75 times the casing diameter.
4) The rate of torque application recommended by ASTM is not to exceed 0.1 degrees/sec, often resulting in a time to failure of 10 to 15 minutes in very soft clays. The rate of torque application for these tests was significantly faster resulting in a shorter time to failure.
5) The ASTM standards recommend that the effect of soil-rod friction be accounted for while no such account was made for the tests performed for this report.