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62-F. 287m

MILL STREET

BRIDGE

CHELTENHAM

BA-1396
FOUNDATION
R.M. 113

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS
LABORATORY TESTING
SOIL MECHANICS CONSULTATION

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Project: J820

February 27, 1962

County of Peel,
1 Wellington Street East,
Brampton, Ontario

62-F-287M

Attention: Mr. C. Krudsen, P.Eng.,
Engineering Department

Re: FOUNDATION CONDITIONS
MILL STREET BRIDGE
CHEL TENHAM, ONTARIO

Dear Sirs:

We have completed an investigation of foundation conditions at this crossing of the Credit River in the village of Cheltenham, Ontario.

Two borings and one cone penetration test were made at this site. They disclosed that the subsoil is essentially similar to the dense silty materials encountered in recently completed bridge projects located a few hundred yards upstream and downstream from this site. The only variation occurs in the upper few feet just below river bed level where a stratum of uniform very fine sand was encountered. This fine sand exists in a medium dense to dense condition and it extends to depths of 4 to 12 feet below the river bed under the east and west banks respectively. The soil below the fine sand consists of a very dense, medium to coarse stratified silt which contains some thin layers of clay. The borings were terminated in this deposit at a depth of about 30 feet below the surface of the water, or about 24 feet below maximum stream bed level.

Abutments and piers for this bridge can be supported on simple footings in this sand or on the silt at a depth of about 4 feet below the level of the river bed. The recommended safe net bearing pressure to apply in this material is 5000 psf. The factor of safety against failure, when utilizing this pressure is in the order of 3 for rectangular footings about 5 feet wide. The settlement to be anticipated should be about 1 inch or probably less and it should occur as soon as load is applied.

Since the upper levels of soil consist of very fine sand, which will have a low resistance to scouring forces, some type of erosion protection should be provided around the footings. The use of light steel piling

is suggested as a means of permanent protection. This piling will also serve to support the walls of the excavation for the footings during the initial stages of construction of the bridge.

No embankment stability problem exists at this location.

The factual information and soil mechanics reasoning which form the bases for these recommendations are presented under the headings that follow.

Site Description

The site of this investigation constitutes the Mill Street Crossing of the Credit River in the Village of Cheltenham. Access across this river is provided at the present time by a Bailey structure which was installed as a temporary measure following the failure of the original bridge. It is understood that the original bridge was built immediately prior to the first World War. The concrete abutments, which support the Bailey crossing, are badly deteriorated and the wing walls are cracked and pushed out to some extent. The span between the abutments is just over 77 feet.

The east bank of the Credit River rises about 3 to 4 feet above the present water level and the west banks are about 6 to 8 feet high. The back yards of homes extend right to the river's edge at all locations, except in the southeast corner which is overgrown with shrubs, weeds and some trees.

The river flows in a southerly direction at this crossing, but its course bends toward the west a short distance downstream from the bridge. The ice level at the time of this investigation was about 12 feet below the bridge deck. It is understood that the high flood level is about 8 feet above the present ice surface. A short length of approach fill has been used to raise the level of Mill Street up to bridge entrances.

Geology

The Credit River, in the vicinity of Cheltenham, passes along the westerly limits of the relatively flat Peel Till Plain. It flows in a deep valley bounded by this plain and by the Niagara escarpment to the west.

The upper levels of this valley represented the drainage path of glacial melt water which followed a confined course between the escarpment and an ice lobe to the east. The flow was to the south over the escarpment, where the stream finally emptied into glacial Lake Warren near Brantford. Gravel deposits of this glacial stream are strewn in the upper levels of the valley. The western limits of the irregular Oakridges moraine lie a short distance to the north of Cheltenham.

Subsoil

The results of the two borings at this site are presented on Dwg. 2 and 3 of this report and are summarized in the estimated stratigraphical profile of Dwg. 1.

Two soil strata are indicated. The predominating material is a very dense deposit of stratified medium to coarse silt which contains some thin clay seams. Both borings were terminated in this soil about 30 feet below the present level of the river. Its upper limit begins about 10 feet below the ice surface at the southeast side of the crossing and at a depth of 18 feet in the vicinity of the northwest corner of the bridge.

The silt is overlain by uniform very fine sand which exists in a somewhat less dense state. A typical grading for this sand is shown on Dwg. 4. The existing abutment footings presumably are founded in, or just below, this material.

In hole 2, at the southeast corner, the sand is overlain by about $7\frac{1}{2}$ feet of loose organic silt. The bottom limits of this silt coincide, more or less, with the deepest sections of the river bed.

Foundations

In view of the very dense nature of the soil at this site, the support of the bridge replacement on simple footings taken about 4 feet below the deepest levels of the river bed would seem to be the logical foundation proposal for this structure. At this foundation depth the footings, generally, will bear in the very fine sand, except possibly at the east side of the river, where dense silt rises close to the surface.

The penetration resistance, or N value, for this sand is equal approximately to 30. According to conventional relationships between penetration resistance and foundation capacity, the safe bearing value associated with this resistance is about 3500 psf. This value allows for the reduction in capacity resulting from submergence and from the possible erosion of sand above footing level.

Recent investigations by the U.S. Bureau of Reclamation,* however, indicate that this relationship is unduly conservative, particularly for application close to the ground surface or below the water table. Making allowance for this fact and for the probability that the bridge structure will not be particularly sensitive to differential settlements, the use of a net bearing pressure of 5000 psf is recommended.

The ultimate capacity, Q, of a strip footing bearing on this submerged sand can be estimated from the expression:

$$Q = A(\frac{1}{2} \gamma B N_{\gamma} + \gamma D N_q)$$

* "Research on Determining the Density of Sands by Spoon Penetration Testing" Gibbs and Holtz, Fourth Int. Conference on Soil Mechanics and Foundation Engineering, - 1957

where:

- A is the bearing area of the footing of width B feet
- γ is the submerged weight of the soil, estimated to be 70 pcf approximately
- D is the depth of the footing below the river bed
- N_q & N_γ are bearing capacity factors dependent upon the value of the angle of internal friction ϕ . For this application, N_q and N_γ are conservatively estimated to be equal at least to 40

Insertion in the above expression, shows that for a footing of width $B = 5$ feet, the maximum or failure stress is equal to $(7000 + 2800D)$ psf. Taking $D = 4$ feet the ultimate capacity is computed to be about 18,000 psf. The factor of safety against failure when using 5000 psf bearing pressure, therefore, is in excess of 3. If it is assumed that the soil is eroded down to bearing level around the footing, the ultimate capacity will be reduced to 7000 psf and the factor of safety will be very low.

Because of this latter possibility, it is recommended that vulnerable sections of footings be surrounded by light interlocking steel sheet piling which should be driven down into the underlying silt or to a depth of 10 feet below the footing level if silt is not encountered. This sheeting will serve two other purposes as well. It will assist in the support of the footing excavations and the control of seepage pressures when digging in the sand. It is for this control that the sheeting must be driven to the depths indicated above. Secondly, it will transfer the abutment load to greater depths and thereby increase the ultimate carrying capacity of the sand.

Excavations

Excavation work for the bridge footings will be below the water table in wet, very fine sand. By using the interlocking sheet piling, driven into the silt, as specified in the previous section, the seepage path from the adjacent river bed to the footing excavation will be lengthened considerably and a barrier of low permeability will lie in this path. The hydraulic gradient in the fine sand and the associated tendency for "boiling" of the sand particles will thereby be reduced to safe limits. It should be possible, therefore, to dig out the sand confined within the sheeting down to a depth of 4 feet below the stream bed. The ground water in the excavation can be pumped from gravel-lined sumps placed at convenient locations in the cut. The sides of the sheeting must, of course, be braced against the external earth pressures.

The alternative means for obtaining a stable footing excavation is to divert the river away from the work and to surround the excavation with a vacuum well point system. This system will gradually depress the water table and thereby permit the installation of footings in the dry.

Embankments

Since the approaches to the bridge take the form of a local increase in the height of fill, the earth pressures exerted by these approaches against the bridge abutments should be less than would be the case if the fill had a considerable lateral extent. The magnitude of earth pressure also will depend upon the type of material used in the embankment construction. The value of this earth pressure force at any depth, h , below the road surface can be conservatively estimated from the expression:

$$P = k\gamma h$$

where the earth pressure coefficient, k can be assumed equal to 0.3 and the unit weight of the fill, $\gamma = 125$ pcf.

Because the soil under the approaches is dense and granular, no stability problem exists.

We hope that the information in this report assists you in the preparation of foundation plans for this bridge.

Yours very truly,



WAT/gc
Encls.

W. Trow
William A. Trow, P.Eng.



View Looking Toward West Abutment



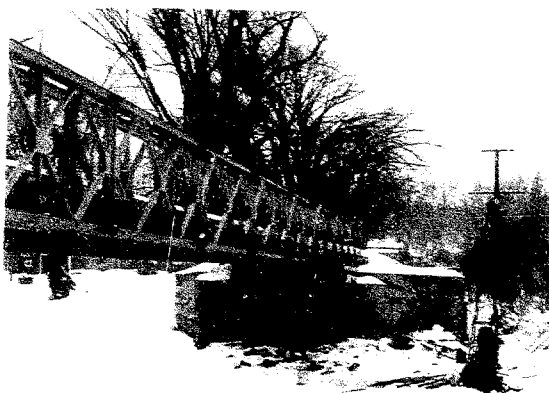
View of East Abutment
Drill on Hole 2



View Looking West
From East Approach



View Looking Down Driveway



View Looking Down Driveway
From East Approach



View Looking Down Driveway
From East Approach



View of Bridge
Looking Upstream



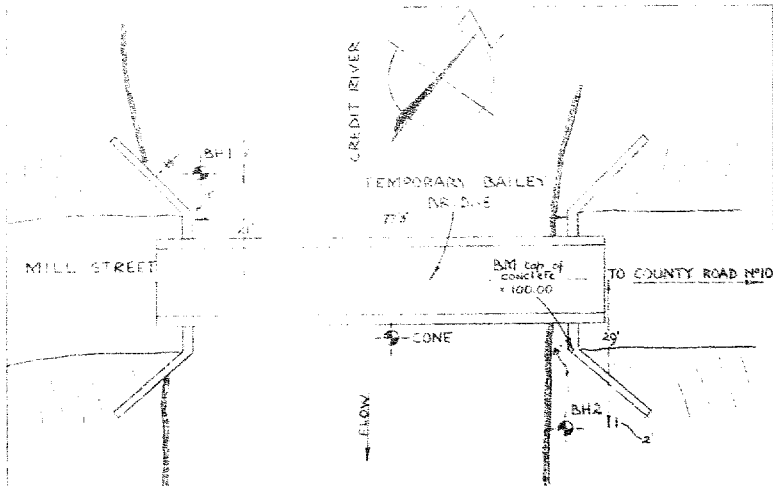
View of Bridge
Looking Downstream



View of Bridge
Looking N. - 100 ft.

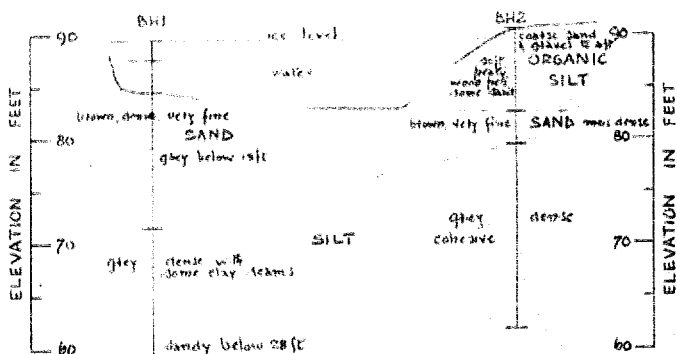


View of Bridge
Looking N. - 100 ft.

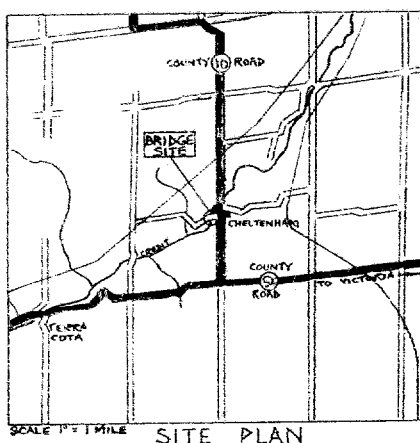


BOREHOLE LOCATION PLAN
SCALE 1" = 20'

2/1/66 Jan 95-9



ESTIMATED SUBSOIL STRATIGRAPHY
SCALES: HOR 1" = 20' VERT 1" = 10'



SCALE 1" = 1 MILE SITE PLAN

FOUNDATION INVESTIGATION
MILL STREET BRIDGE
CHELTENHAM, ONTARIO
W. A. TROW & ASSOCIATES LTD
PROJECT NO 820 DRAWING NO 1

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

LEGEND

DRAWING NO 3
PROJECT NO J820

BOREHOLE NO 1
PROJECT Mill Street Bridge
LOCATION Cheltenham, Ontario
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 89.6 ft.
DATUM See Dwg. 1.

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—○—
2" I.D. SHELBY TUBE *—*—*—*—
2" DIA. CONE ————

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ①
UNCONFINED COMPRESSION ②
VANE TEST AND SENSITIVITY (S) +^s

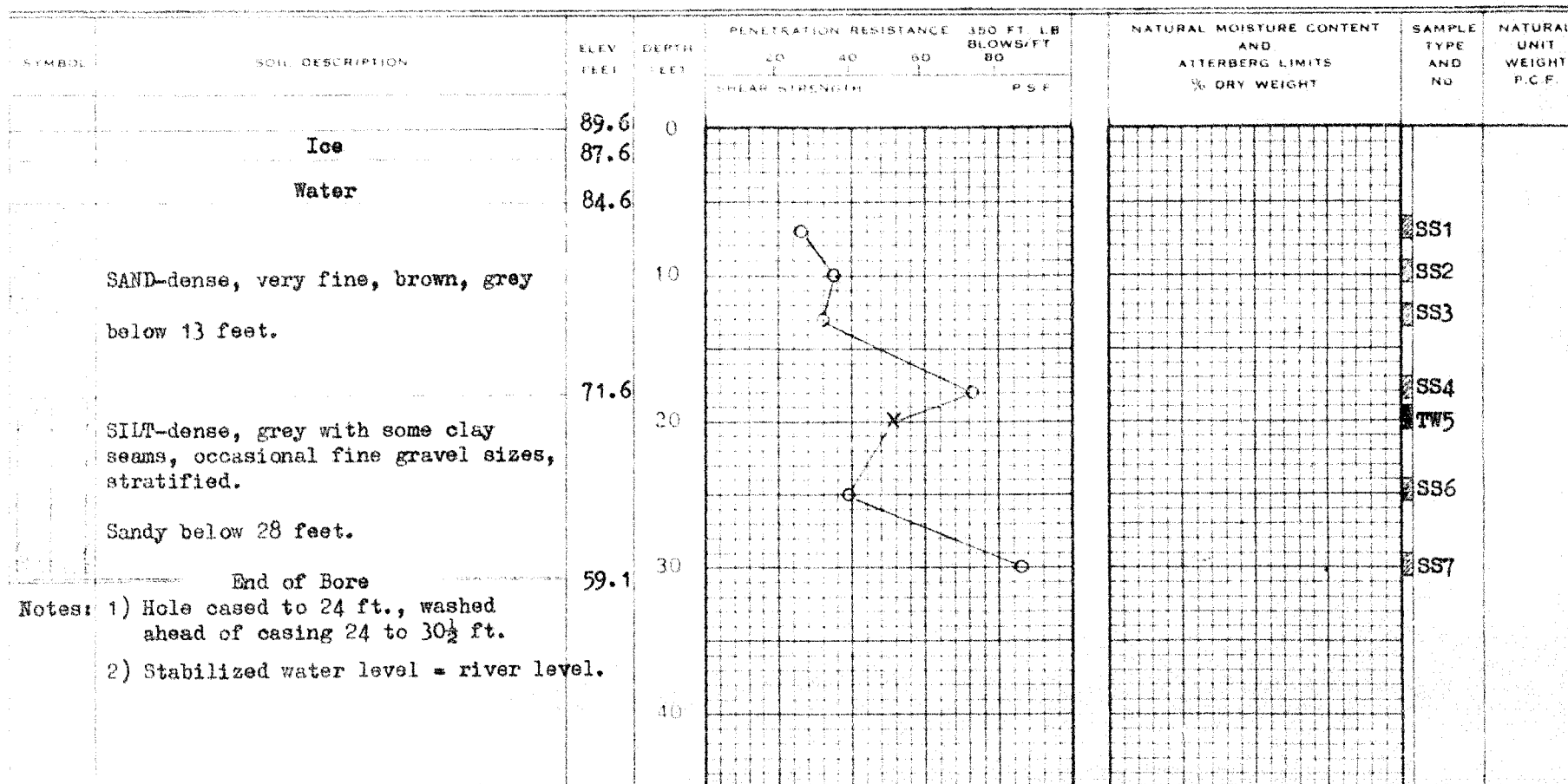
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS

LIQUID LIMIT —○—
PLASTIC LIMIT ———

SAMPLE TYPE

2" O.D. SPLIT TUBE [Symbol]
2" I.D. SHELBY TUBE [Symbol]
3" O.D. SHELBY TUBE [Symbol]



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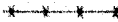

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING NO. 3
PROJECT NO. J820




LEGEND

BOREHOLE NO. 2
PROJECT Mill Street Bridge
LOCATION Cheltenham, Ontario
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 90.4 ft.
DATUM See Dwg. 1.

PENETRATION RESISTANCE


2" O.D. SPLIT TUBE 
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SHEAR STRENGTH

UNDRAINED TRIAXIAL
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NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

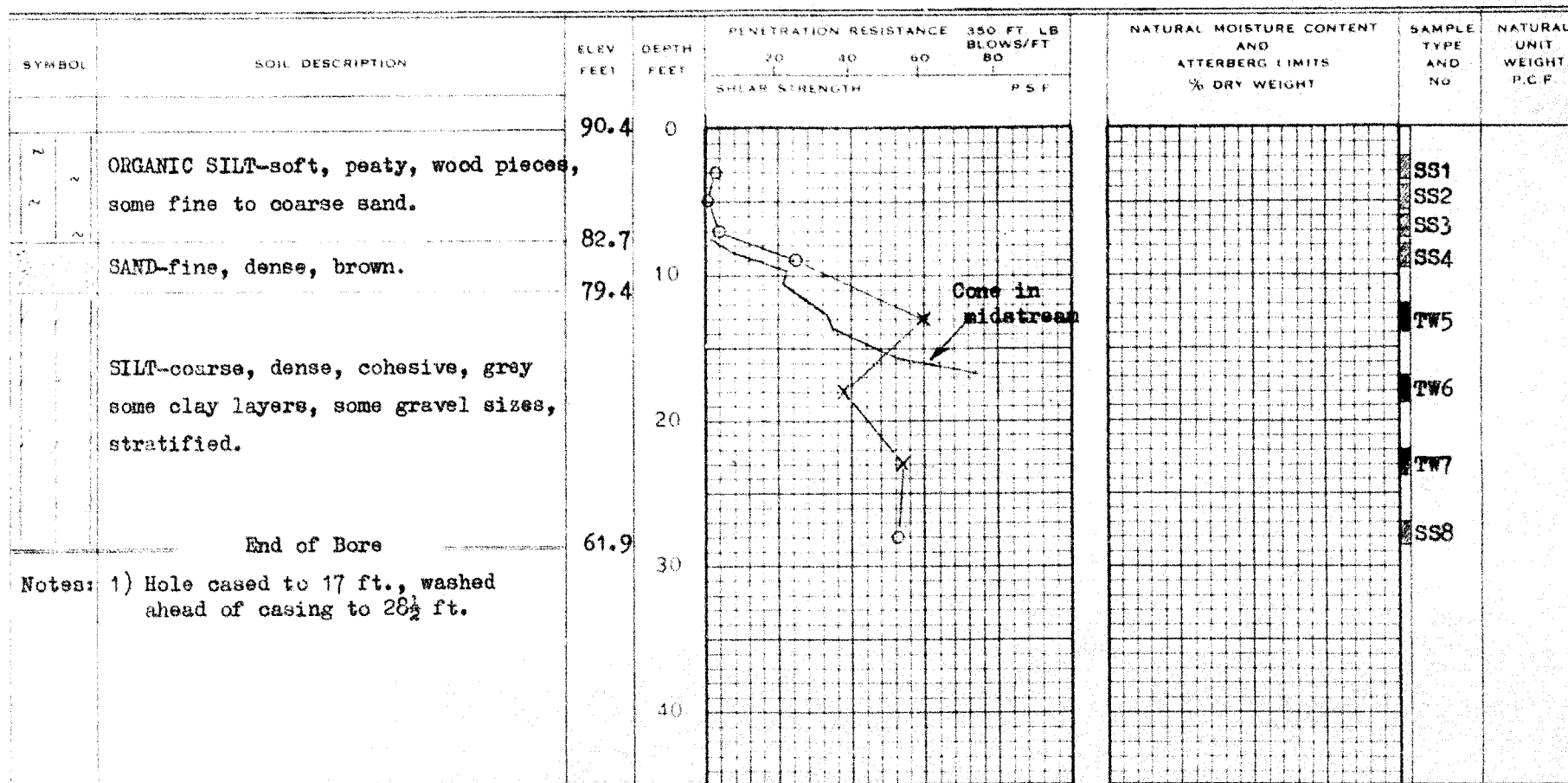
ATTERBERG LIMITS

LIQUID LIMIT 

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SAMPLE TYPE

2" O.D. SPLIT TUBE 
2" I.D. SHELBY TUBE 
3" O.D. SHELBY TUBE 



MECHANICAL ANALYSIS

