

GEOCRES No. 30M16-25

DIST. 7 REGION

W.P. No.

CONT. No.

W. O. No. 73-11221 M

STR. SITE No. -

HWY. No. LOC

LOCATION WESLEYVILLE GENERATING  
STATION

No of PAGES -

=====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

Geocres 30 M16-25  
copies made

73-11-22/M



**GENERATION  
PROJECTS  
DIVISION**

GEOLOGY No  
30M16-25

73-11-221 M

WESLEYVILLE GENERATING STATION

ACCESS ROAD BRIDGE

SOIL CONDITIONS AND FOUNDATIONS

REPORT NO. 177 - 75

M.T.C. S.L. No 21-399

PREPARED BY: F. DeBenedictis

APPROVED BY: J.G. E. Taylor

ISSUE DATE: SEPT. 28, 1973

STRUCTURE NO. 21-399



Wesleyville Generating Station  
Access Road Bridge  
Soil Conditions and Foundations

<u>CONTENTS</u>	<u>PAGE</u>
INTRODUCTION	1
FIELD WORK	1
SUBSURFACE CONDITIONS	2
DISCUSSION	6
General	6
North Abutment: Ch. 48+00	7
South Abutment: Ch. 51+20	9
Intermediate Support: Ch. 49+00	9
Intermediate Support: Ch. 50+19	10
Settlement	11
Alternative Foundations	11
Construction and Other Considerations	12
SUMMARY	14
REFERENCES	15
FIGURES	
1 to 5	
DRAWING	
No. NK11-DEH-10120-0030	

Wesleyville Generating Station  
Access Road Bridge  
Soil Conditions and Foundations

INTRODUCTION

The access road from Highway 401 to the Wesleyville Generating Station site crosses the CN and CP railroad tracks at CNR Mileage 275 and CPR Mileage 147 respectively. At this location the tracks pass through excavations about 20 feet deep below the original ground surface. A highway bridge is required over the tracks for an overpass to provide a minimum headroom of 23 feet 4 inches.

The bridge will be about 300 feet long and will consist of three spans supported on abutments at the top of the cuts and on intermediate piers about 100 feet apart.

Approach fills about 10 feet high above the original ground surface will be required at the abutments. The general arrangement of the proposed works is shown on Drawing No. NK11-DEH-13131-0002.

An investigation of the subsurface conditions for the foundations of the bridge was carried out in June - August 1973 and the findings are presented in this report.

FIELD WORK

At the location initially selected for the bridge eight boreholes were drilled, two at each support, for preliminary evaluation of the subsurface conditions. The bridge alignment finally chosen resulted in a minor shift from the original and four additional holes were drilled to extend the data to the new alignment. The location of the boreholes is shown on Drawing No. NK11-DEH-10120-0030. Standard penetration resistance tests were carried out in the boreholes which were advanced by augering. Samples of the soil strata encountered were sent to the Soils Laboratory for testing to determine the grain size distribution, Atterberg Limits, in-situ density and shear strength parameters.

Groundwater observations were made while advancing the boreholes and one observation well was installed for further measurements.

The results of the field work are shown on the sections on Drawing No. NK11-DEH-10120-0030 and laboratory test results are reported in Research Report No. S73-45-K.

#### SUBSURFACE CONDITIONS

The bridge is to be located approximately midway in 1000 feet long excavations through which the railways have been constructed. In the cuts the soils have been removed from around Elevation 400 to Elevation 375. The soil stratigraphy within the depth explored may be summarized as follows:

1. An upper silt till deposit between ground surface (Elevation 400) and Elevation 390.
2. Deposits of uniform fine sand, layered fine sand with some silt, or silt and fine sand with some coarse sand and fine gravel, between Elevation 360 and 390.
3. A lower silt till below Elevation 360.

In the excavations for the railways, the upper till and part of the sand deposits have been removed.

The details of results obtained from the boreholes are shown in section under each support and a longitudinal section is also shown on Drawing No. NK11-DEH-10120-0030.

Details of the subsurface conditions are as follows:

1. The topsoil cover is about 18 inches and consists mainly of sandy silt with organics and roots.
2. At the north abutment, there is an upper till deposit about 10 feet thick extending from below the topsoil (Elevation 402) to around Elevation 392. At the south abutment this stratum is also found but it is about 5 feet thick and extends from around Elevation 398 to Elevation 393.

The soil is a brown silt with about 30 percent fine to coarse sand and 10 percent fine to coarse gravel, occasional cobbles and boulders and a trace of clay. It exhibits slight plasticity with liquid limits of 13.6 percent and 15.7 percent and corresponding plasticity indices of 1.4 and 3.9 respectively. Typical grain size distribution curves for this deposit are shown on Figure 1. The natural moisture content of the deposit ranges between 7 percent and 10 percent.

The standard penetration resistance in the deposit ranges between 18 and 68 blows per foot with an average value of 30 blows per foot. The deposit may be considered to be generally in a compact state.

3. At around Elevation 392-394, the soil type changes to deposits of grey-brown silt and fine sand with about 10 percent medium to coarse sand and gravel, layered fine sand and silt, and uniform fine sand. Grain size curves for these soils are shown on Figures 2 and 3. These deposits are non-plastic. The distribution of these deposits is shown on the section on Drawing No. NK11-DEH-10120-0030. Generally the uniform fine sand occurs below the other two soils the behaviour of which is controlled largely by the higher silt content. These strata form a cohesionless mass extending across the site down to around Elevation 375 under the south abutment and to below Elevation 360 at the north abutment. The standard penetration resistance in the deposits ranges between 34 and 290 blows per foot (extrapolated from short penetration of the split-spoon) and the average value is around 120 blows per foot. It is inferred from these values that these soils are in a dense to very dense state.



Moisture contents of about 10 percent were measured on saturated disturbed samples of the silt and fine sand with a trace of medium to coarse sand and gravel. The moisture contents of the saturated uniform sand deposits were around 15 to 20 percent. The void ratios of these deposits are estimated at around 0.27 and 0.37 to 0.54 respectively. These values are associated with soil materials of similar composition in a dense to very dense state.

The dry density of samples from inserts in the split-spoon during testing ranges between 110 pcf. for the uniform material and 132 pcf. for the other soils. The void ratio calculated from these values ranges between 0.54 and 0.27 respectively and are in agreement with the values obtained from the moisture content analysis. Triaxial tests were carried out on laboratory compacted samples obtained from the split-spoon to determine the angle of shearing resistance of the deposits. The values range from 33 degrees for specimens in a relatively compact state to 40 degrees for specimens compacted to around the maximum density of 128 pcf. attainable in the laboratory.

From the standard penetration resistance values averaging 120 blows per foot and the high relative density inferred, the angle of shearing resistance of the natural soil strata is expected to be around 40 degrees to 45 degrees (Meyerhof 1956).

The existing cut slopes in these deposits are approximately 1.3 H to 1 V or at an angle of about 38 degrees to the horizontal. These slopes are stable and show no signs of distress. An angle of shearing resistance of about 40 degrees is therefore considered acceptable for these deposits in their very dense state.

The initial modulus of deformation (E) measured on an insert sample of the sand deposit in the triaxial compression tests was 70 Tons per square foot and the rebound modulus was 365 to 500 Tons per square foot. The initial value is probably low because of the effects of sample disturbance and the rebound value is believed to be a more reliable measure of the modulus of the undisturbed soil.

The standard penetration resistance (N) of the soil at the sample location was 61 blows per foot. Schmertmann proposes an E value equal to  $7 \times N$  for these soil types. For this case the rebound modulus is of the same order of magnitude as that obtained from the Schmertmann equation. The average modulus of the deposit is thus estimated to be  $7 \times 120$  or 840 Tons per square foot.

4. Below Elevation 375 at the south abutment a grey very dense silt till is encountered. The surface of this soil appears to slope gently downwards to the north and east and the deposit is encountered below Elevation 365 at the north abutment. The deposit is mainly a silt with some fine to coarse sand and gravel and a trace of clay. It is of low plasticity with a liquid limit of about 16 and a plasticity index of about 5. The natural moisture content of the soil is about 7 to 10 percent and the dry density is about 134 pcf. Grain size curves are shown in Figure 4.

There are also associated with the till mass, deposits of silt with a trace of fine sand. These deposits are all in a very dense state inferred from the high standard penetration resistance of more than 100 blows for a penetration of only a fraction of a foot.

5. The groundwater surface was observed in the boreholes during augering and sampling. At the south abutment the water surface is at around Elevation 382 and at the north abutment at around Elevation 376. At the intermediate pier locations the water surface has been lowered by the railway excavations and is found at around Elevation 373-376.

## DISCUSSION

### General

The proposed bridge is a 36 foot wide 3 span continuous beam structure with a cast in place concrete deck. The bridge is designed for H20 - S16 - 44 AASHO loading. Under maximum load conditions the total vertical loads on the intermediate supports will be about 1240 kips. Longitudinal loads from traffic and expansion on the fixed support will be about 45 kips and on the sliding support 23 kips. Lateral wind forces of about 40 kips are to be considered. At the abutments the vertical load is about 600 kips and the horizontal forces are about 80 kips. About 10 feet of fill will be required for the approaches at the abutments.

Widening of the existing railway excavations to provide for additional tracks is to be taken into account in the design of the structure and its foundations. The provision for the future widening is shown on Drawing No. NK11-DEH-13131-0002.

The dense to very dense soil deposits encountered at the site provide suitable subsurface conditions for use of spread footings for the foundations for the bridge.

The design of the foundations will depend not only on the soil characteristics but also on the loading conditions, foundation shape, effects of slopes particularly at the abutments, the position of the water table and construction limitations imposed by the railway operations near the foundations.

Spread footings will extend over the full width of the bridge and will be about 36 feet long. Preliminary estimates indicate a foundation width of about 12 feet will be required.

For protection against frost action, the foundations should be provided with a minimum soil cover of 4 feet. Reference should be made to Figure 5 on which are noted several of the comments covered in detail below.

North Abutment: Chainage 48+00

The ground surface is at around Elevation 402. The existing slope will be excavated for additional tracks as shown on Drawing No. NK11-DEH-13131-0002. Above Elevation 394 the soil is a compact to dense silt till. Below Elevation 394 the deposits are very dense and consist mainly of the cohesionless sands within the depth of influence of the foundations. The water table is at around Elevation 382.

The location of the foundations is controlled to a large extent by the limits of the proposed widening of the railway cut. In addition, the soil cover required for prevention of frost action should not be less than 4 feet. These considerations lead to a foundation grade at or below Elevation 394 below which the soil deposits are very dense.

Based on the standard penetration resistance of an average of about 120 blows per foot and a foundation width of about 12 feet, the allowable soil pressure would be of the order of 5 Tons per square foot if the footing is located more than about 30 feet from the top of slope. However, because of future widening of the railway excavations the proposed footing arrangement will more closely approximate a foundation on the face of a slope. The ultimate bearing capacity may be estimated according to Meyerhof (1957).

The abutment is also subjected to horizontal loads and the resultant of the horizontal and vertical forces will be at a small angle of about 8 degrees with the vertical. Meyerhof (1956) gives a correction factor for the allowable load for inclined forces on a footing.

The water table is about 12 feet below the proposed footing grade. The bearing capacity of a foundation is affected by a water table which is within a depth of the foundation less than twice the foundation width. The correction factor recommended by Meyerhof (1955) may be used.

Based on a soil bulk density of 130 pcf. and an angle of friction of 40 degrees, the vertical component of the ultimate bearing capacity of the abutment is estimated to be 6.4 TSF.

For a factor of safety of 3 against shear failure of the supporting soil, the safe average vertical pressure should not exceed 2.0 Tons per square foot.

In addition, the maximum toe pressure under the footing caused by the vertical and horizontal loads and moments should not exceed 4 Tons per square foot.

South Abutment: Chainage 51+20

The ground surface at the south abutment is at around Elevation 398. The compact to dense silt till occurs above Elevation 393. Below Elevation 393 the soils are the dense or very dense cohesionless sand deposits on which the foundations for the abutments can be placed. The bearing capacity considerations for the south abutment are similar to those discussed in detail for the North Abutment.

With foundations at or below Elevation 393, a water table at around Elevation 377, slope excavations for railway tracks and loading conditions similar to those on the north abutment, the safe vertical pressure on the soil should not exceed 2 Tons per square foot and the maximum toe pressure should not be greater than 4 Tons per square foot.

Intermediate Support: Chainage 49+00

At Chainage 49+00 the intermediate fixed support will be founded on the existing slope of the railway cut. The location of the support and frost protection requirements for the foundations indicate a foundation grade at or below Elevation 381.

The side ditch for the drainage of the existing railroad has been graded to Elevation 378. This ditch is about 20 feet from the edge of the proposed footing. If future additions to the railway will alter the ground configuration near the footing, it will be necessary to consider placing the foundation at a lower grade probably around Elevation 373. The water table under the proposed location is at around Elevation 376.

The excavation of the existing slope to provide space for construction of the footing will eliminate the effects of the slope on the bearing capacity of the footing if the slope is not rebuilt. Thus, for the small inclination of the resultant force on the footing, the position of the water table, soil density of 130 pcf. and friction angle of 40 degrees, the ultimate bearing capacity of the 12 foot wide footing will be reached under a vertical load of 15 Tons per square foot. For a factor of safety of 3, the safe average bearing pressure will be 5 Tons per square foot.

The maximum edge pressure under the foundation from moments or inclined loads should not exceed 7 Tons per square foot.

Intermediate Support: Chainage 50+19

At Chainage 50+19 a sliding support is to be provided for the superstructure. The vertical loading will be similar to that at the support at Chainage 49+00 but the horizontal forces due to friction and expansion will be less.

The foundation is located at the toe of the existing slope of the railway cut. This slope is to be excavated for future track widening and the ground surface configuration will be changed to that shown on Drawing No. NK11-DEH-13131-0002.

To provide the minimum cover for frost protection the foundation grade should be at Elevation 372 or lower. Below this grade are the very dense sand deposits to around Elevation 363 where a very dense silt till and very dense silt strata are encountered. The groundwater is at Elevation 372.

The bearing capacity considerations are about the same as those for the intermediate support at Chainage 49+00 and the safe average bearing pressure is estimated to be 5 Tons per square foot for the proposed footing size. The maximum edge pressure should not exceed 7 Tons per square foot.

### Settlement

The settlements which will occur under the foundations will be mainly due to elastic compression in the soil and a small, time-dependent movement is also likely. The settlements under each footing has been calculated by the Schmertmann (1970) approach. The modulus of Compressibility (E) is taken as 7 times the standard penetration resistance. An average value of 800 Tons per square foot has been used.

The settlement of the intermediate supports under the recommended safe loading is estimated to be about 1/2 inch most of which will occur rapidly under load. At the abutments, the settlement will also be around 1/2 inch because of the effects of the approach fill and the sloping ground. The differential settlement is not likely to exceed 1/4 inch.

The soils below the foundation grades are very dense it is not likely that there will be any adverse effects from the dynamic loads on the soil-foundation system which may arise from the passage of trains near the foundations.

### Alternative Foundations

For the foundations for the intermediate supports an alternative to spread footings with large diameter bored piles taken down to the very dense silt till below Elevation 365 is possible. The construction of these foundations will require a casing or slurry support in the sand deposits below the water table. The bearing capacity of the piled system will depend on the size and depth of the shaft. For a 3 foot diameter pile taken down to Elevation 360, the estimated allowable working load per pile will be about 300 Tons.

The piles can be extended upwards as columns to a transverse beam and designed to resist the lateral forces on the structure.



### Construction and Other Considerations

Widening of the railway cuts as shown on Drawing No. NK11-DEH-13131-0002 will be required to accommodate the additional tracks. Also, the construction of the intermediate supports will necessitate excavation of the existing slopes.

In the very dense sand deposits, slopes of 1 to 1 will be temporarily stable. Permanent slopes should be excavated to not steeper than 1.5 H to 1 V.

It is advisable that the excavations for future track additions at the bridge location should be completed during the construction of the bridge to permanent slopes which should be protected against erosion or future disturbance. This will remove the risk of disturbance of the soil around the foundations at the abutments and eliminate the adverse effects of locating a foundation on or near the face of a slope in the case of the intermediate supports.

The soil slopes will require protection against surface runoff and other agents of erosion. A transition layer will be required between the fine sand and the surface protection. Drainage near to the abutments must be adequately provided to prevent any movement of the fine sand deposits.

The upper silt till deposit is suitable for use as controlled compacted fill. The uniform fine sands will be difficult to control during compaction and are recommended for use where random and nominally compacted fill may be tolerated.

Because of the doubtful effectiveness of the relatively thin soil cover and backfill on the slope at the abutments, the passive resistance of the soil at the toe of the foundation should be neglected. The friction coefficient between the concrete and supporting sand may be taken as 0.35.

Because movement at the abutment is not desirable, the earth pressure on the abutment from the approach fill should be assumed higher than the minimum active earth pressure and closer to the earth-pressure-at-rest condition for which a factor of 0.5 times the vertical effective pressure is applicable. The bulk density of the backfill may be taken as 140 pcf.

During construction of the foundation the subgrade must be protected against frost action and excessive disturbance from construction equipment or method.

The excavations for the foundations at the abutments will be above the groundwater surface and no groundwater problems are anticipated. Adequate provision for control of surface runoff into the excavation and particularly on to the fine sands will be required to prevent disturbance and degradation of these deposits.

The excavations for the intermediate supports will be close to or below the groundwater surface. Because of the relatively shallow depth which may be involved, and the low to moderate permeability of the sand deposits, the quantity of water to be handled will be relatively small. Pumping from perimeter drains and sumps may be adequate for controlling the effects of seepage on the subgrade by lowering the water about 2 feet below foundation grade. A working mat consisting of a 6 inch concrete slab over the foundation area immediately after excavation will serve to maintain the sand subgrade intact during construction of the foundations.

The protection of the railway track near the excavation for foundation at Chainage 50+19 will need special attention during construction. The excavation toe will be about 12 feet from the track and the excavation depth will be about 6 feet.

To prevent any problems with instability of the ground near the railway tracks, it may be necessary to provide sheeting and bracing for the slope near the track.

#### SUMMARY

The subsurface conditions at the site are suitable for spread footings for the bridge foundations.

At the abutments the vertical pressure should not exceed 2 Tons per square foot. At the intermediate supports the vertical pressure should not exceed 5 Tons per square foot. The maximum edge pressures should not be greater than 4 and 7 Tons per square foot respectively.

Settlements under the safe loads for the proposed 12 feet wide footings will be about 1/2 inch and differential settlements are not likely to be greater than 1/4 inch.

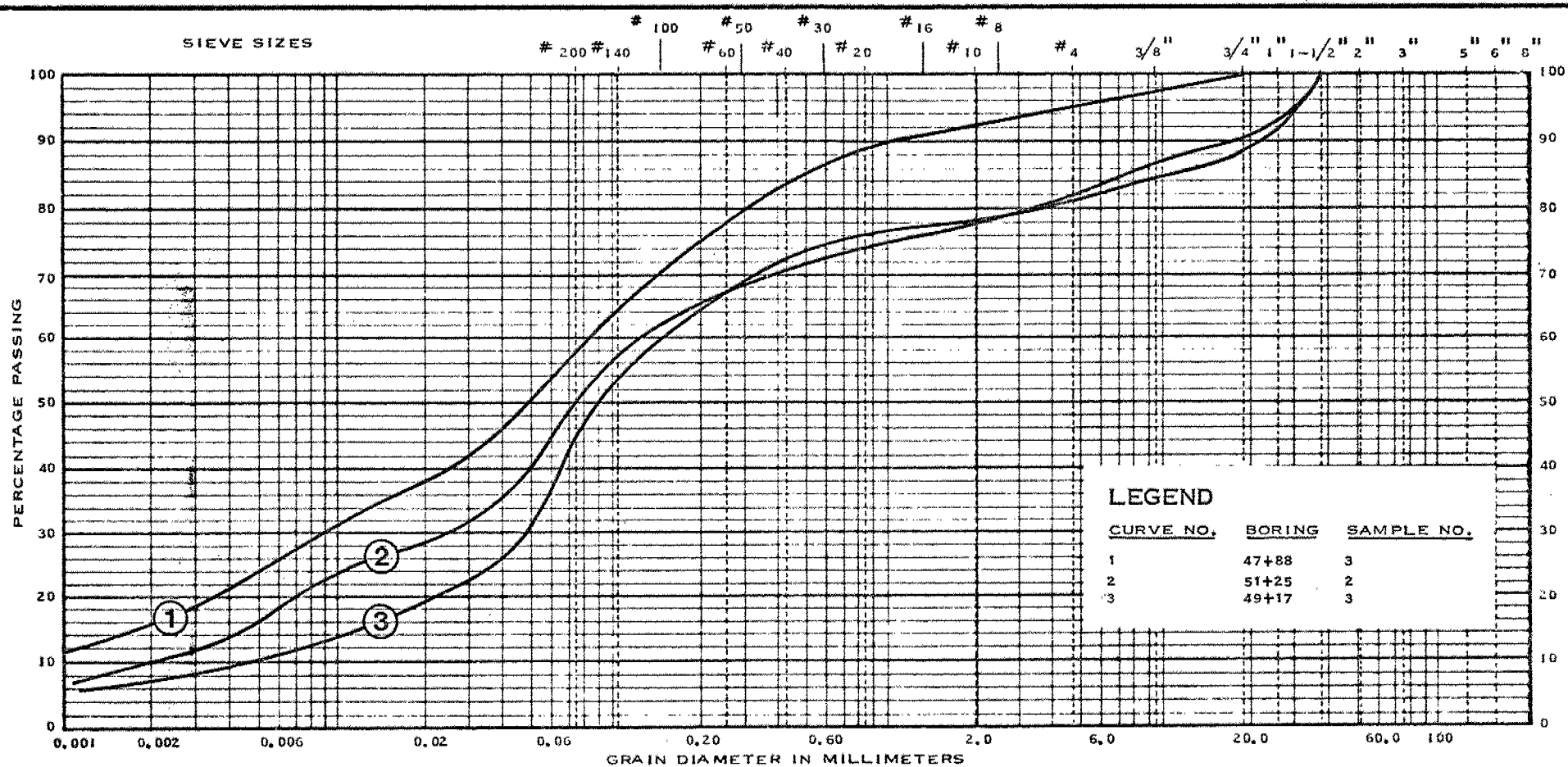
Permanent widening of the railway cuts at the bridge location is recommended to prevent future disturbance of the foundations when additional tracks are to be installed.

No problems from groundwater are expected at the abutments. Control of surface runoff and of the groundwater will be required at the intermediate supports. Normal pumping from sumps and perimeter ditches may be adequate. Close attention will also be required to the excavations near to the railway track and temporary bracing may be necessary.

REFERENCES

- MEYERHOF, G. G. (1955)      Influence of Roughness of Base  
and Groundwater conditions on the  
Ultimate Bearing Capacity of  
Foundations, Geotechnique.  
September 1955.
- MEYERHOF, G. G. (1956)      Discussion of "Rupture Surfaces  
in Sand Under Oblique Loads"  
J. ASCE, SM3, 1028-15.
- MEYERHOF, G. G. (1956)      Penetration Tests and Bearing  
Capacity of Cohesionless Soils,  
J. ASCE, SM1, 1956.
- MEYERHOF, G. G. (1957)      The Ultimate Bearing Capacity of  
Foundations on Slopes,  
Proc. 4th. ICSMFE, Vol. I.
- SCHMERTMANN, J. H.          Static Cone to Compute Static  
Settlement Over Sand  
J. ASCE, SM3, 1970.
- RESEARCH REPORT              Wesleyville Generating Station  
No. S73-45-K                  Access Road Bridge  
Foundation Studies  
September, 1973.

## MECHANICAL ANALYSIS



CLAY	TO SILT	SAND			GRAVEL		COBBLES
		FINE	MEDIUM	COARSE	FINE	COARSE	

UNIFIED SOIL CLASSIFICATION

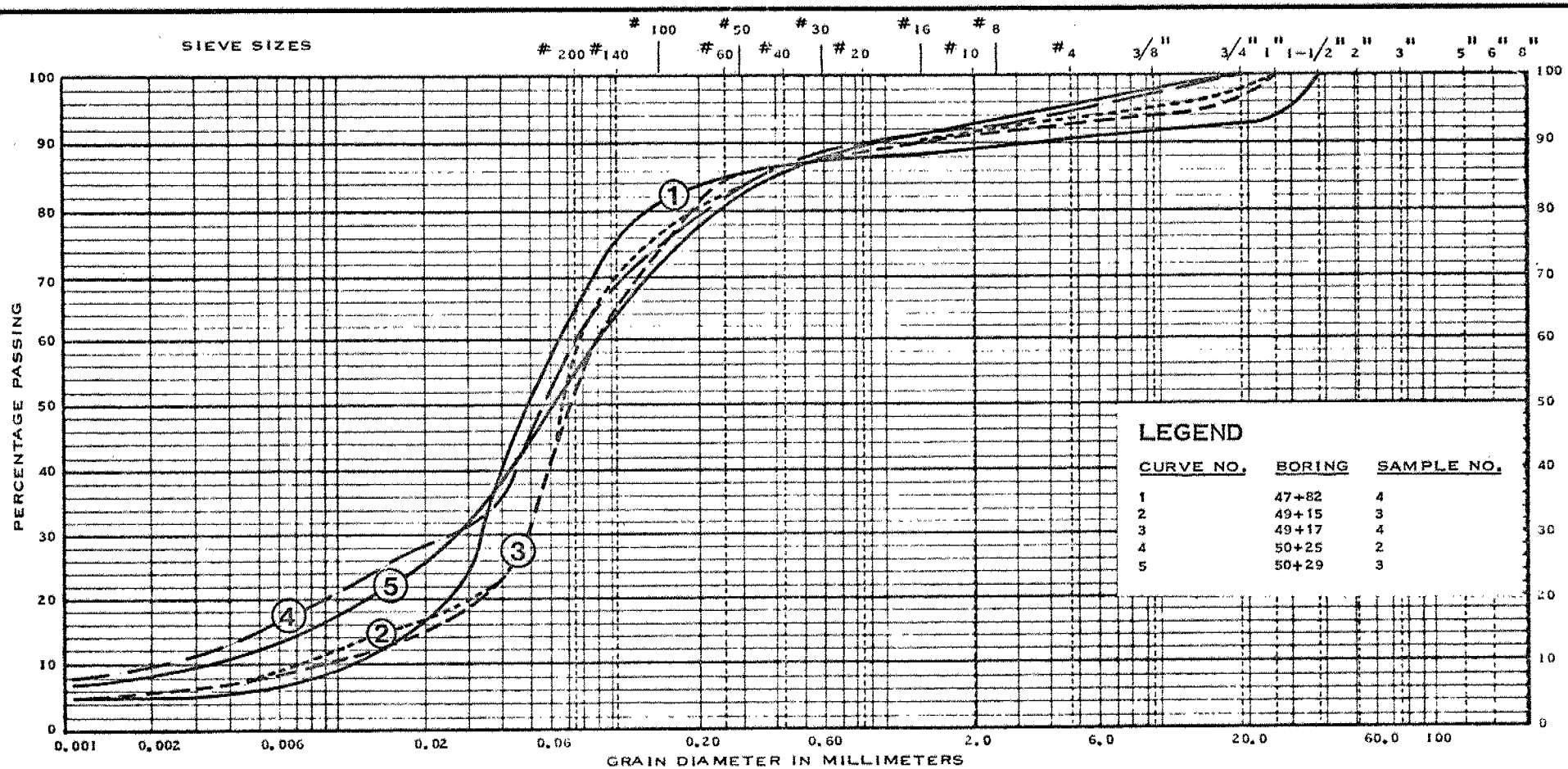
SILT WITH SAND, LITTLE GRAVEL, TRACE OF CLAY  
(UPPER TILL)

Fig. 1

REPORT NO. 177-75

E--

## MECHANICAL ANALYSIS



CLAY	TO SILT	SAND			GRAVEL		COBBLES
		FINE	MEDIUM	COARSE	FINE	COARSE	

UNIFIED SOIL CLASSIFICATION

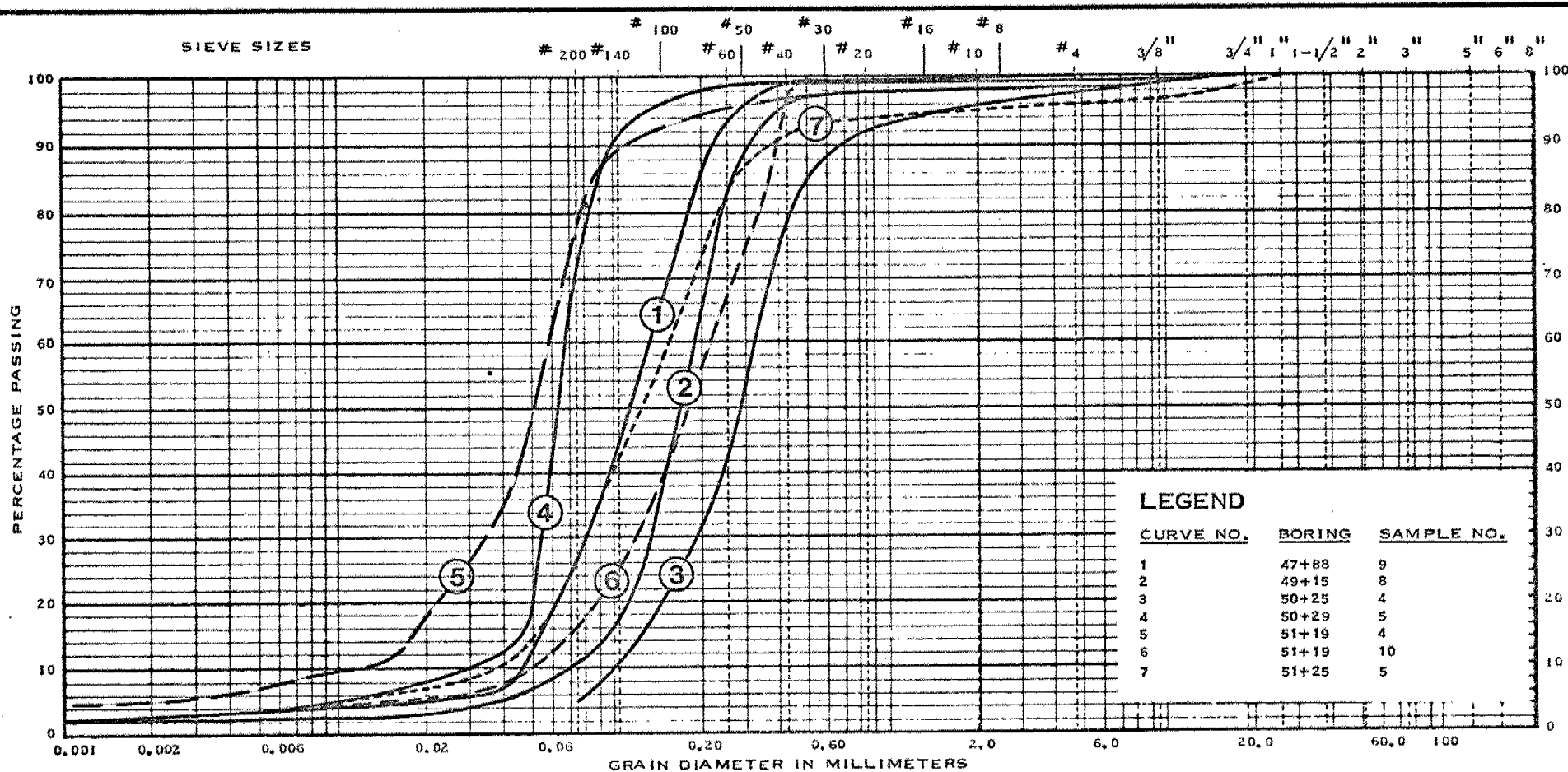
FINE SAND WITH SILT, TRACE OF COARSE SAND AND FINE GRAVEL

Fig. 2

REPORT NO. 177-73

E-

## MECHANICAL ANALYSIS



CLAY	TO SILT	SAND			GRAVEL		COBBLES
		FINE	MEDIUM	COARSE	FINE	COARSE	

UNIFIED SOIL CLASSIFICATION

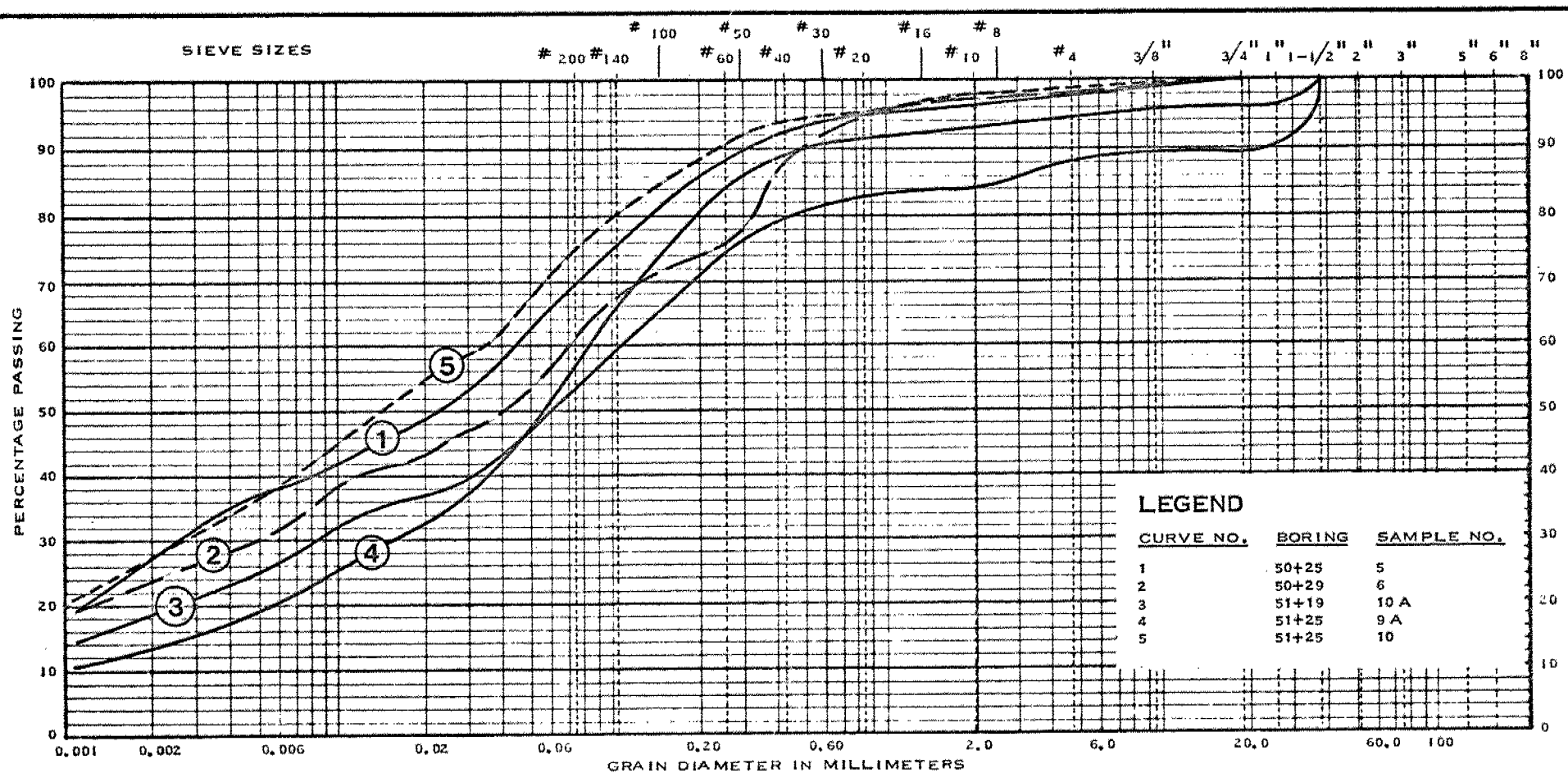
UNIFORM FINE SAND TO FINE SAND AND SILT

Fig. 3

REPORT NO. 177-75

E-

## MECHANICAL ANALYSIS



CLAY	TO SILT	SAND			GRAVEL		COBBLES
		FINE	MEDIUM	COARSE	FINE	COARSE	

UNIFIED SOIL CLASSIFICATION

SILT WITH SAND, LITTLE GRAVEL, TRACE OF CLAY  
(LOWER TILL)

Fig. 4

REPORT NO. 177-75

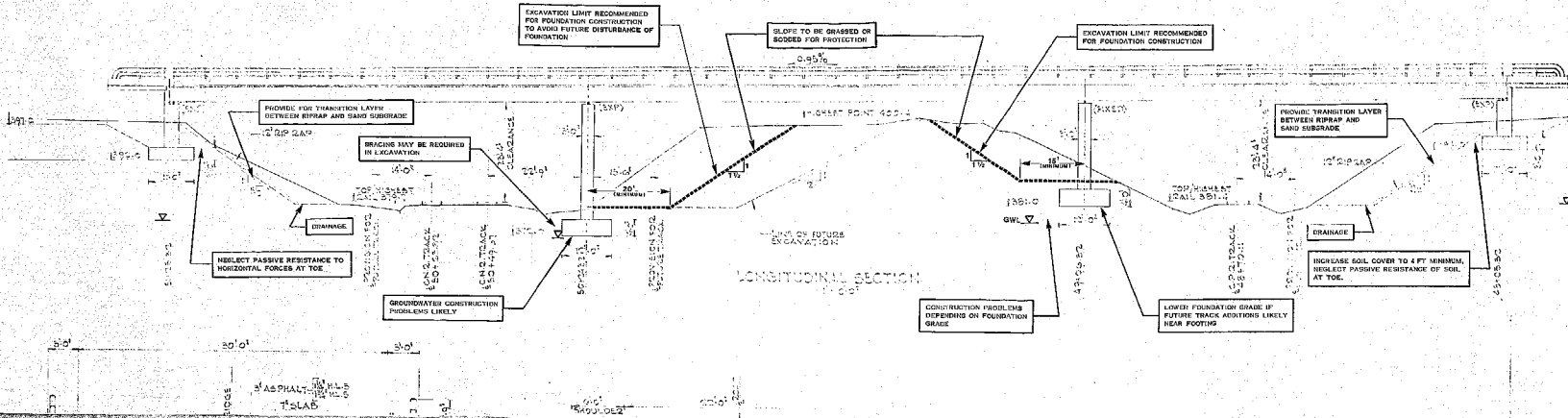
E-



# OVERSIZE DRAWING

**NOTES**

1. GROUNDWATER LEVELS (GWL) OBSERVED DURING DRILLING OPERATION JUNE TO AUGUST, 1973
2. LONGITUDINAL SECTION TAKEN FROM DWS, NO. NK11 DEH 13131 0002-R0.



**LONGITUDINAL SECTION**

WESLEYVILLE GS  
ACCESS ROAD BRIDGE  
LONGITUDINAL SECTION

Scale: 1" = 10'

**Fig. 5**

27 SEP 75

