

## MEMORANDUM

To: Mr. F. G. Allen,  
District Engineer,  
District #6 (Toronto).

From: Foundation Section,  
Materials & Testing Div.,  
Room 107, Lab. Bldg.

Attention: Mr. G. A. Metcalfe, DATE: July 26, 1968  
Construction Engineer

Our File Ref.

In Reply To

SUBJECT:

FOUNDATION INVESTIGATION REPORT  
For  
Slope Failures of Fill Section  
Between Stations 350+00 and 362+00  
Hwy. #401 Crossing of Hoggs Hollow  
Contract 62-252  
District No. 6 (Toronto)  
W.J. 68-F-46 -- W.P. 85-59-1

Attached, please find the Foundation Investigation Report for the above mentioned section of Hwy. 401.

The suggested remedial measures may, at this time of the year, seem somewhat elaborate. However, we feel that the stability problem will become acute again next Spring, or after prolonged wet periods.

Should you find it necessary or desirable to discuss with us, the report or any part of it, please feel free to call on this Office.

AGS/MdeF

Attach.

cc: Messrs. T. J. Kovich  
G. K. Hunter (2)  
D. M. Hopper

Foundations Files  
Gen. Files

*A. G. Stermac*  
A. G. Stermac  
PRINCIPAL FOUNDATION ENGINEER

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For  
Slope Failures of Fill Section  
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1. INTRODUCTION:

In 1963 and 1964, Hwy. #401, in the vicinity of Toronto, was reconstructed. In this reconstruction more traffic lanes were added and thus the capacity of the Expressway was increased. Between Stations 350+00 and 362+00, it was necessary to extend the roadway section to the north by placing fill over the natural slope. Between Spring 1964 and the present date, four distinct failure zones were noticed, three located within the areas in which fill was placed.

In a memo dated May 15, 1968, Mr. G. A. Metcalfe, District Construction Engineer, District #6 (Toronto), requested that this Section carry out an investigation. Subsequently an investigation was carried out to determine the subsoil and groundwater conditions as well as all other pertinent data.

This report will present all the factual information obtained from this investigation, together with our assessment of the causes of failure as well as recommended remedial measures.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The site in question is located on Hwy. #401, immediately west of the Hoggs Hollow bridge structure, in the Borough of North York. In this area the West branch of the Don River and its

2. DESCRIPTION OF THE SITE AND GEOLOGY: (cont'd.) ...

tributaries have cut deeply into the overburden deposits generally forming steep-sided, narrow valleys up to 120 feet in depth. The banks of these valleys are heavily wooded. Prior to the widening Hwy. #401, between Stations 350+00 and 362+00, was located on the top of such a bank, while the Don Valley Golf Club is located on the valley floor.

Physiographically, the site is situated in the section known as the "South Slope". In the vicinity of the site the South Slope is basically a smoothed ground moraine which has been faintly drumlinized and scored at intervals by valleys, as discussed previously. The material in the drift is mainly related to the underlying rock; at the site it is probably composed of clayey silt with some gravel (glacial till). This cohesive deposit is underlain by interglacial sands and gravels, which in turn, are underlain by grey and black shale bedrock of the Meaford - Dundas formation, Ordovician Period.

3. CONSTRUCTION DETAILS:

3.1) Conditions Existing Prior to Reconstruction of Hwy. #401 in 1963:

The north shoulder of Hwy. #401 was located near the crest of the slope leading to the valley floor, on which the Don Valley Golf Course is located. Between Stations 350+00 and 362+00, the bank varies from about 80 to 100 feet in height. Available information indicates that, in general, the natural slope was divided into an upper and lower portion by a broad flat terrace located at mid-height; this terrace is about 1,200

cont'd /3 ...

3. CONSTRUCTION DETAILS: (cont'd.) ...

feet in width. The upper portion of the slope was generally standing at between 2:1 and 3:1, while the lower portion was much flatter.

The existing north slope was heavily wooded. Numerous springs were noticed seeping out of the natural bank in the lower portion of the bank. Visual observations, made by construction personnel, prior to the reconstruction of Hwy. #401, indicated that the natural slope appeared to be stable.

3.2) Reconstruction of Hwy. #401 in 1963:

The widening required to the south, between Stations 350+00 and 362+00, was either in cut or fills of nominal height (less than 5 feet). The widening to the north involved reclamation of land by extending fill out over the natural valley slope. The valley bank, in this vicinity, varies from 80 to 100 feet in height; up to 60 feet of fill was placed above the face of the natural slope to facilitate the widening.

It was decided that the groundwater seeping out of the lower portion of the banks be drained prior to fill placement. According to available information, this groundwater control was provided by installing perforated drainage pipes placed at the base of open trenches about 4 feet deep; these trenches were cut into the natural slope prior to fill placement. The trenches were then backfilled with granular material. These sub-drainage systems were installed in the Spring of 1963 at the following locations:

cont'd /4 ...

3. CONSTRUCTION DETAILS: (cont'd.) ...

<u>Approx. Station</u>	<u>Extent of Pipe</u>	<u>Approx. Height of Fill Above Pipe</u>	<u>Outlet</u>	<u>Remarks</u>
358+70	To Mid-Height (with Tee)	At Toe - 0' At Mid-Height 25'	-	Water still flowing.
357+00	To Mid-Height	At Toe - 0' At Mid-Height 25'	To Catch Basin	No water in Catch Basin (Plugged?)
350+00	To Mid-Height (with Tee)	-	-	(Plugged?)

At the time of reconstruction, the drainage measures provided appeared to be effective in controlling the groundwater, particularly that portion seeping out of the lower areas of the bank. Based on the effectiveness of the drains and the dry favourable construction season, it was considered that fill placement could commence.

Fill placement began in the Spring and continued into the Fall of 1963. The fill provided was from numerous sources throughout the Toronto area, and as such, was of variable composition. In addition, the material was placed and compacted in lifts as supplied. Benches, generally, about 10 feet wide with a back face some 6 feet high, were cut into the slope in order to provide proper keying between the parent and fill material. The location of these benches are shown on Drawing 68-F-46A, attached to this report.

The crest of the finalized section now is between 800 and 2,000 feet north of the old section, within the area under consideration. The north face of the fill is presently standing at about 1-3/4:1. In addition, in the area where the fill is the highest (Section B-B, Drawing 68-F-46B), a bin type retaining wall,

3. CONSTRUCTION DETAILS: (cont'd.) ...

about 300 feet long and 16 feet high at the centre was constructed so as to prevent the toe of the fill from encroaching on the golf course fairway.

The outer face of the slope was protected by jute bagging pegged into the slope; subsequently the slopes were sodded. From available information, it is understood that this sodding caught on immediately and within a short period of time a thick mantle of growth existed over the slope.

4. DESCRIPTION OF FAILURES:

Four distinct failure areas have occurred along the north slope of Hwy. No. 401, between Stations 350+00 and 362+00. These failures, which are shown in plan on Drawing No. 68-F-46A, can be sub-divided into two main types which are discussed separately below.

4.1) Failure in the North-West Corner - Station 352+00:

In this area the west valley bank butts against the fill forming a 'V' notch; this notch channels the surficial drainage from this western corner down to the valley floor. Here, in the Spring of 1964, a failure occurred. It was about 50 feet wide by 64 feet long. The upper portion of the failure exhibits a steep scarp face, while at the toe, a wave of displaced material has formed. This cylindrical failure surface is characteristic of a deep-seated rotation type failure. It is probable that the failure might have reached a stabilized condition. However, the stability may be detrimentally affected at a later date by uncontrolled surface run-off, which would soften the upper portion

cont'd /6 ...

4. DESCRIPTION OF FAILURES: (cont'd.) ...

of the subsoil. At this stage we feel that specific remedial measures are not warranted.

4.2) Failures along the North Slope of Hwy. #401:

Three distinct failure areas are evident along the face of the north slope between Stations 352+00 and 362+00; their locations are listed below.

	<u>Station</u>	<u>Approx. Location on Slope</u>	<u>Max. Length of Area</u>	<u>Approx. Date Noted:</u>
i)	352+00 to 354+00 (See Section D-D)	Mid-third - 30 ft. wide	200 ft.	Spring, 1964.
ii)	355+00 to 358+00 (See Section B-B)	Lower Third - 60 ft. wide	250 ft.	Major movement in Spring, 1964.
iii)	358+00 to 359+45 (See Section A-A)	Lower Half - 40 ft. wide	85 ft.	Noticed in 1966 - Major movement in Spring, 1967.

These failures are surficial in nature - i.e., extend only 3 to 4 feet below the face of the slope; the causes of the failures will be discussed in detail - Section 6.1) to follow.

The sloughed material has, in the vicinity of Section B-B, spilled over the retaining wall onto the fairway of the golf course.

5. FIELD INVESTIGATION:

5.1) General:

The field work consisted of putting down 41 hand auger holes through the face of the north slope. In addition, 10 test pits were hand dug to supplement the information provided by the

cont'd /7 ...



5. FIELD INVESTIGATION: (cont'd.) ...

hand auger holes. The hand auger holes and test pits, which were put down by personnel from the Construction Section, District #6, extended for depths of between 3 and 13 feet below the face of the slope. Representative auger and bulk samples were obtained from the holes. Groundwater level observations were carried out, during the period of the investigation, in the open holes at all the locations.

The locations and elevations of the auger holes and test pits, which were surveyed by District #6 personnel, are shown on Drawing 68-F-46A. The detailed description of the soil types and groundwater conditions encountered across the site, which were estimated from the boring data, are shown on Drawing No. 68-F-46B.

All the samples were subjected to a visual examination in the field and subsequently in the laboratory. Following this examination, laboratory testing was carried out on selected representative samples. This testing is summarized on Figures 1 to 3, inclusive, contained in this report.

5.2) Subsoil Conditions:

The shallow hand auger holes and test pits put down indicate that, beneath the sod on the outer face of the north slope, the major portion of the fill is composed of a clayey silt with sand and a trace of gravel (glacial till). A trace of organic matter is often present within a zone 3 to 4 feet in thickness running parallel to the face of the slope. Grain-size

5. FIELD INVESTIGATION: (cont'd.) ...

distribution curves obtained on samples of the glacial till, are plotted on Figure 1.

Atterberg limit tests, carried out on samples of the glacial till, are summarized on the Plasticity Chart, Figure 3. The liquid and plastic limit varies from 17 to 26 and 13 to 18, respectively. These results indicate that the clayey silt is inorganic and of low plasticity. The natural water content varies from 14 to 23 percent - i.e., from a few percent above the plastic limit to a few percent above the liquid limit. The higher moisture content values were encountered in areas associated with a high groundwater level where unstable conditions prevailed, as discussed in detail later in this report.

The continuity of the clayey silt material is intermittently interrupted, particularly in the lower half of the slope, by pockets and seams composed of pervious silty sand and sandy silt material (see the sections shown on Drawing No. 68-F-46B). The thickness of the granular zones, varies from a few inches up to 1-1/2 feet. Two typical grain-size distribution curves for samples of the granular deposits are shown on Figure 2.

Visual observations made during the boring programme indicated that, in general, the fill was well compacted. In a few areas the surficial soil had been softened and loosened; these zones, however, were prevalent mainly in the failure areas, to be discussed in Section 4.1).

5.3) Groundwater Conditions:

Water level observations were carried out in the open

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5. FIELD INVESTIGATION: (cont'd.) ...

holes during the period of the investigation. These observations are summarized on Drawing No. 68-F-46B. These results indicate that in the upper portion of the slope (above the failure area) and near the toe of the slope, the holes were generally dry. In the intermediate zone, where the majority of the failures occurred, groundwater was often encountered relatively close to the face of the slope - i.e., within 2 to 3 feet. Further, occasional wet patches were noticed in this area. It is pertinent to note that in this section the depth of fill is relatively low - i.e., the natural slope face is covered by a minimum amount of fill. In view of this, it is not surprising that the groundwater level was high within this zone.

The test pits put down indicated that the numerous sandy silt and silty sand seams and pockets encountered are often water bearing, particularly those in the problem areas where the groundwater level is high. Once the excavations penetrated these zones, water seepage was observed. This flow tended to carry a high percentage of silt and clay-size particles with it - i.e., loss of ground. A few hours following excavation, major sloughing or loss of ground occurred in the pits; this sloughing was progressive in nature, eventually resulting in a complete collapse of the walls of the test pits. In some of the test pits it was observed that the water level rose to within 3 feet of ground surface overnight.

6. DISCUSSION AND RECOMMENDATIONS:

6.1) Reasons for the Failures:

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

The surficial failures that occurred along the north slope were primarily due to the following reasons:

i) The slope, between Stations 352+00 and 360+00, is between 80 and 100 feet in height. Further, the embankment is constructed with a slope of about 1-3/4:1. The surface run-off over such a high steep-sided slope would tend to erode and gully the less protected areas along the face. Such areas would be progressively softened over cyclic periods of high precipitation and thus tend to slough and become unstable with time.

ii) As discussed in Section 5.3), groundwater came into communication with granular seams and pockets within the fill material. These pervious water-bearing layers, which are confined top and bottom by impervious soil, act as small reservoirs saturating the material within this zone as well as softening the surrounding soil. This softened zone can then become unstable due to the loss of support. A surficial failure will then ensue; the failure will probably be progressive with time, with major movements occurring during or immediately following periods of heavy precipitation.

It is believed that the frost penetration in the area extends down some 4 to 5 feet below the face of the slope. Along the face of the slope, particularly in the problem areas, water would be readily available within this zone. This trapped water would freeze in the winter period. Upon thawing in the Spring, large hydraulic gradients would come into being; the stored water then would seep through preferential paths, such as granular seams, towards the face of the slope. This would create large seepage forces on the

cont'd /11 ...

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

outer face of the slope which would tend to cause surficial instability.

iii) In some areas the sod may not have firmly rooted. Further, the jute bagging placed on the slope at these points may have deteriorated due to weathering effects. This deterioration no doubt would create a softened saturated zone immediately below the inadequately rooted sodding. This zone may, therefore, have become unstable.

If suitable measures are not undertaken to protect the surficial zone of the slope, the failure areas now existing may become progressively more unstable - i.e., greater movements may take place with time. The failures that have occurred certainly substantiate this trend. It is therefore recommended that the surficial drainage characteristics of the slope be improved, as discussed in the next section.

6.2) Recommended Remedial Measures:

In order to control the groundwater seepage within the fill, it is recommended that a granular drainage blanket, at least 2 feet thick, be provided across the north face. The composition of the material within this blanket should be such that it acts as a filter to the fill material on the north slope. The optimum gradation for granular material to be used for such purposes is shown on Figure 4. A typical grading curve for the two most readily available granular sources in the Metro Toronto area, namely:

- i) Sand from the Superior Sand and Gravel Pit; and
- ii) Crushed stone from the Milton Quarry, are also

DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

plotted on Figure 4. Referring to this Figure, it can be seen that the Superior sand most closely approximates the optimum gradation. The drainage blanket should extend from the toe of the slope to a point about 10 feet above the crest of the failure areas. In the failure areas, prior to placing the granular blanket, all softened and loosened areas should be removed.

As well as providing drainage control of surface run-off, the granular blanket will prevent the groundwater within the slope from coming into communication with the face of the slope. This will prevent softening of the surficial zone and reduce the exit gradient, and by so doing, lessen the magnitude of the seepage pressures within this zone.

Topsoil and properly staked sod, as specified in D.H.O. Standard DD-403, should be placed over the blanket. This will protect the blanket from erosion and gullyng caused by surficial run-off.

In addition to the drainage blanket, it is recommended that a properly designed drainage ditch be provided at the toe of the slope. The surface run-off and groundwater seeping through the drainage blanket could then be channelled to this ditch. Provision should be made to lead the water from the ditch into the storm sewer system in the area.

7. SUMMARY:

Hwy. #401 was reconstructed in the Toronto area in 1963. Between Stations 350+00 and 362+00 fill was placed over the natural north slope, which is some 80 to 100 feet in height. Between the Spring of 1964 and the present time, 4 distinct failures have

7. SUMMARY: (cont'd.) ...

occurred in this area. Subsequently an investigation was carried out by this Section to determine the causes of the failures.

The fill, obtained from numerous random sources throughout Metro Toronto, is primarily composed of a competent clayey silt with sand and a trace of gravel (glacial till). Occasional seams and pockets of sandy silt to silty sand up to 1-1/2 feet thick were, however, encountered throughout the fill, particularly in the failure areas. The upper and lower portion of the slope is dry. However, in the intermediate zone, where the failures occurred, the groundwater was generally within 2 to 3 feet of the face of the slope. In this zone numerous wet patches were encountered on the face of the slope; these patches were most prevalent in areas where the height of fill over the original bank was a minimum.

The investigation indicates that the upper portion of the fill, within the failure area, has become saturated and thus softened; this led to a loss of support which eventually caused the surficial failures. These failures are noted to be progressive in nature. The factors most responsible for this condition occurring are discussed in detail in the report.

It is recommended that a granular drainage blanket, about 2 feet thick and extending at least 10 feet above the failure areas, be placed over the slope once it has been stripped of all unsuitable material. There are two readily available sources of granular borrow material in the area, namely, sand from the Superior Sand and Gravel Pit and crushed stone from the Milton Quarry. It is considered

cont'd /14 ...

7. SUMMARY: (cont'd.) ...

that the former would be the most acceptable for use as blanket fill, as discussed in the report.

8. MISCELLANEOUS:

The field work, performed during the period of June 4 to 13, 1968, was carried out under the supervision of Mr. P. B. Schnabel and Mr. T. Card, Project Foundation Engineers.

The report was written by Mr. B.T. Darch, Senior Foundation Engineer.

The project was carried out under the general supervision of Mr. M. Devata, Supervising Foundation Engineer, who also reviewed this report.

July, 1968.



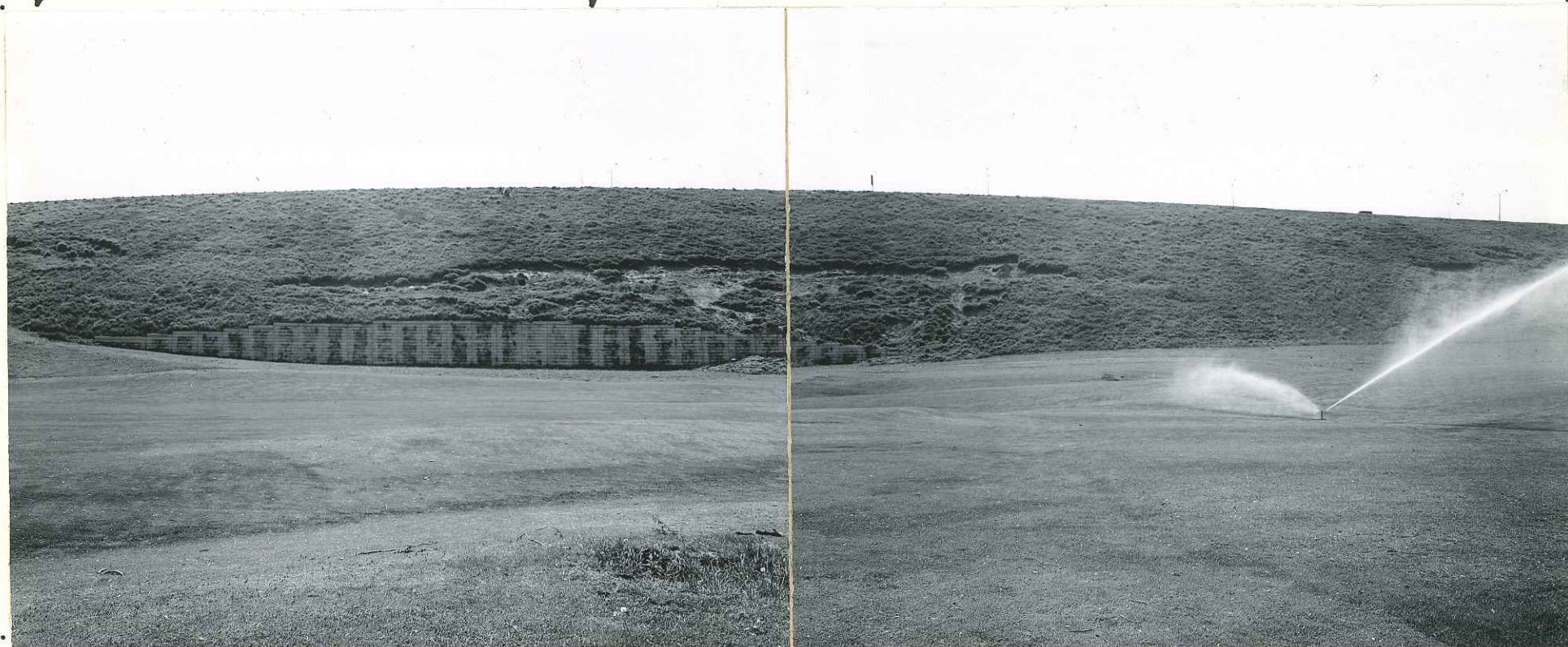
APPENDIX I

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FAILURE  
AREA 'A'

FAILURE AREA 'B'

FAILURE  
AREA 'C'

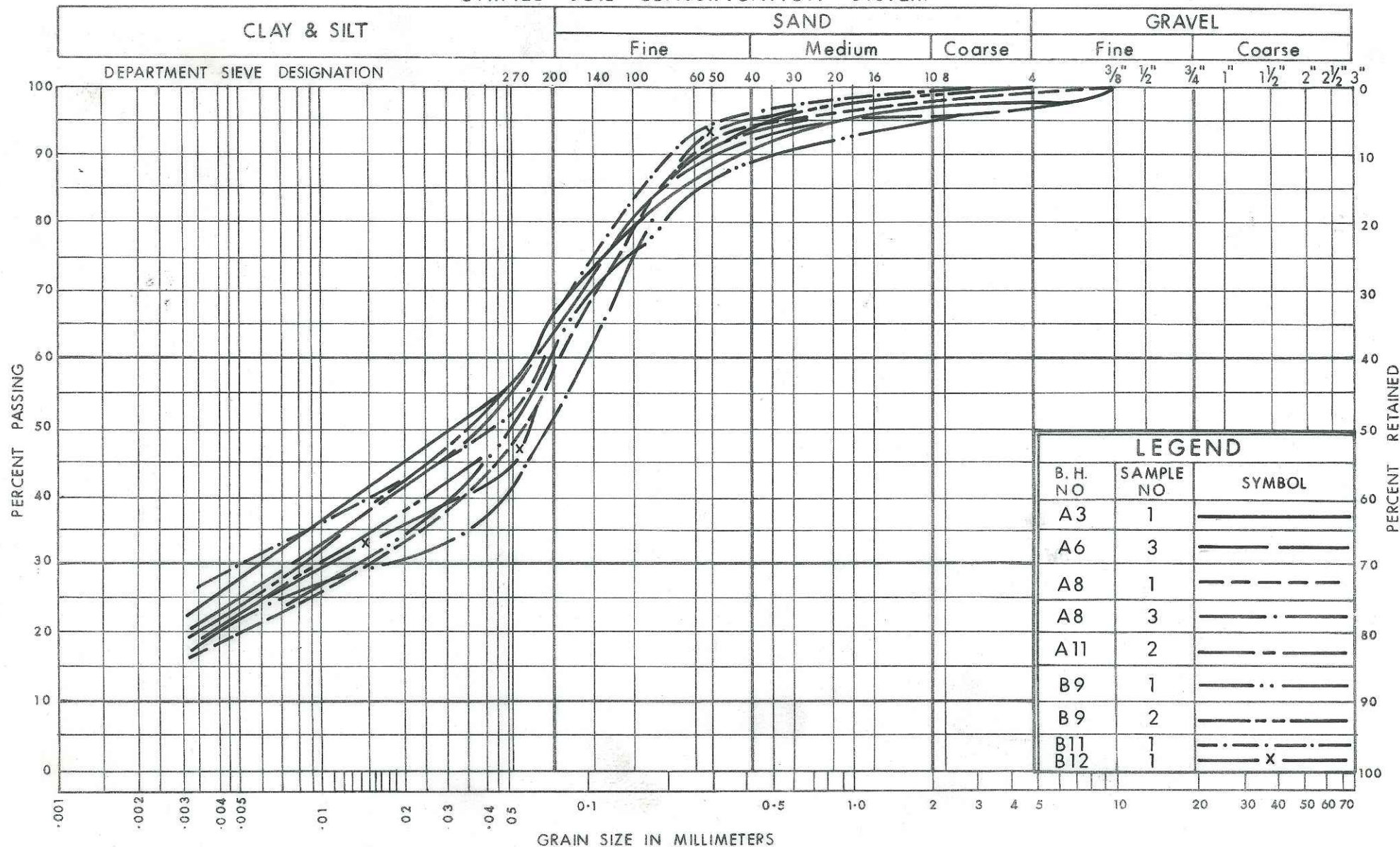


VIEW LOOKING SOUTH AT NORTH SLOPE AND RETAINING WALL

Sta. 350 + 00 TO Sta. 362 + 00



# UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

GRAIN SIZE DISTRIBUTION  
CLAYEY SILT  
WITH SAND & TRACE OF GRAVEL

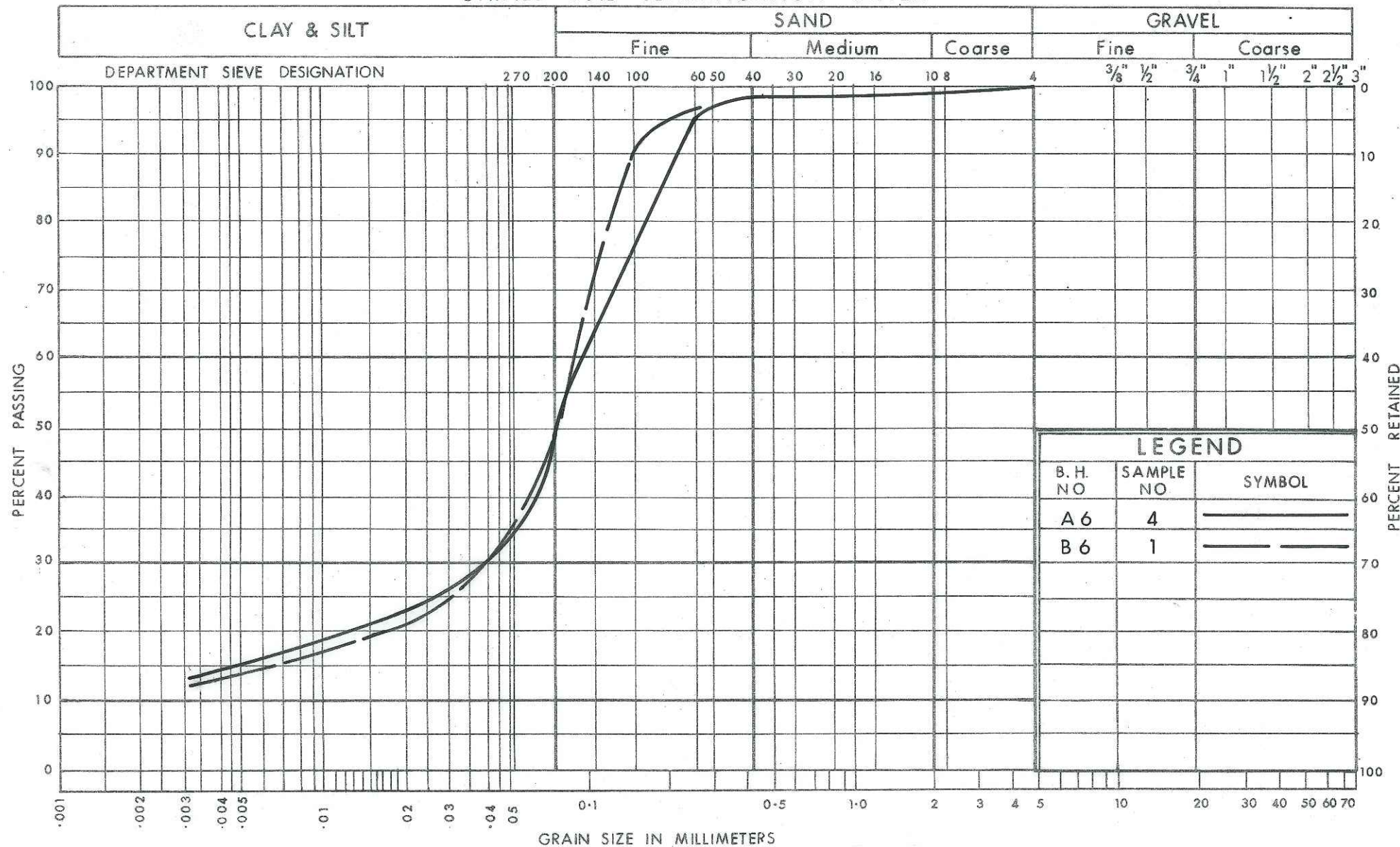
W.P. No.

JOB No. 68-F-46

FIG. 1



# UNIFIED SOIL CLASSIFICATION SYSTEM



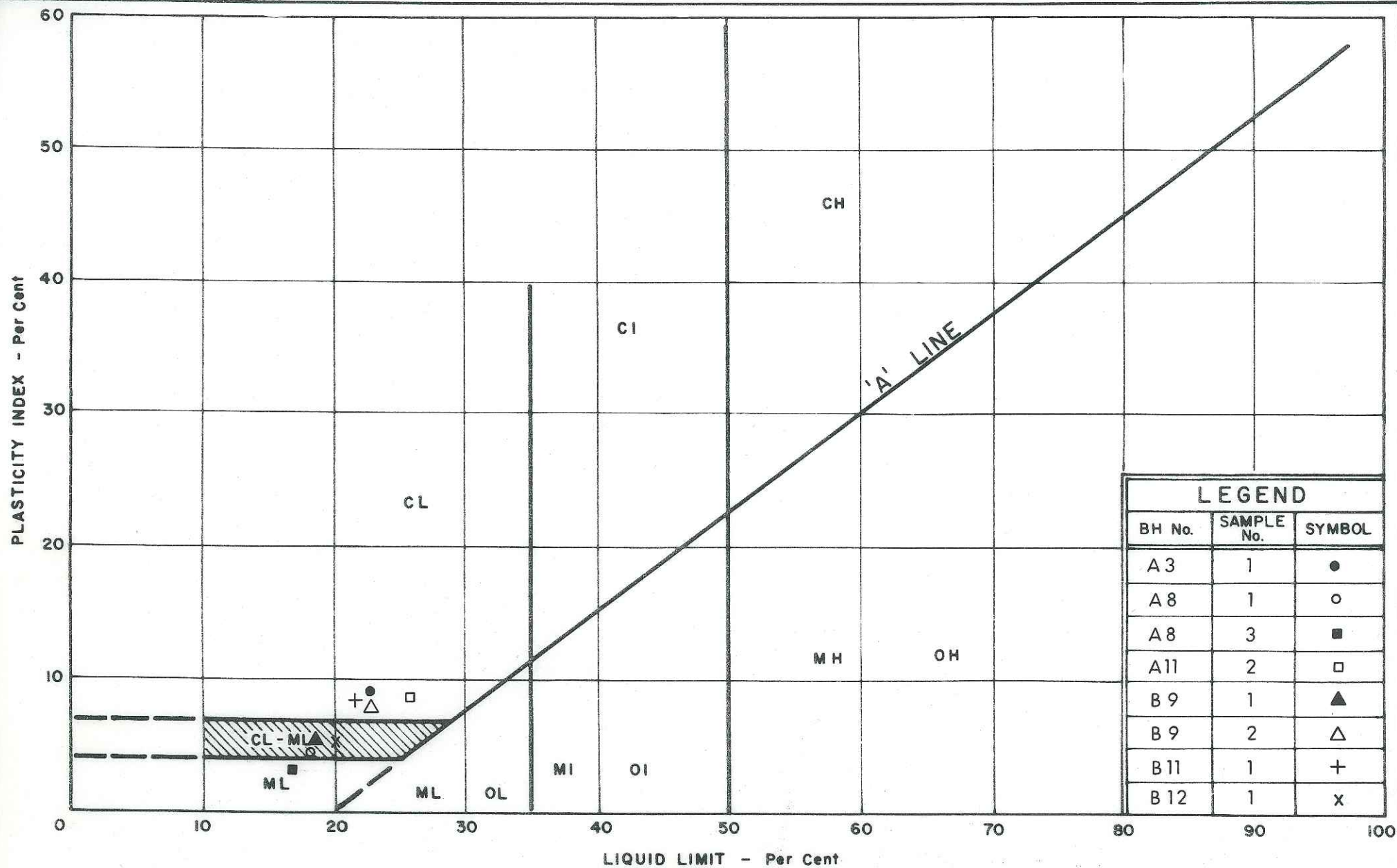
DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

GRAIN SIZE DISTRIBUTION  
SILTY SAND  
TRACE OF CLAY

W.P. No.

JOB No. 68-F-46

FIG. 2



DEPARTMENT OF HIGHWAYS  
 MATERIALS and  
 TESTING  
 DIVISION

# PLASTICITY CHART CLAYEY SILT WITH SAND & TRACE OF GRAVEL

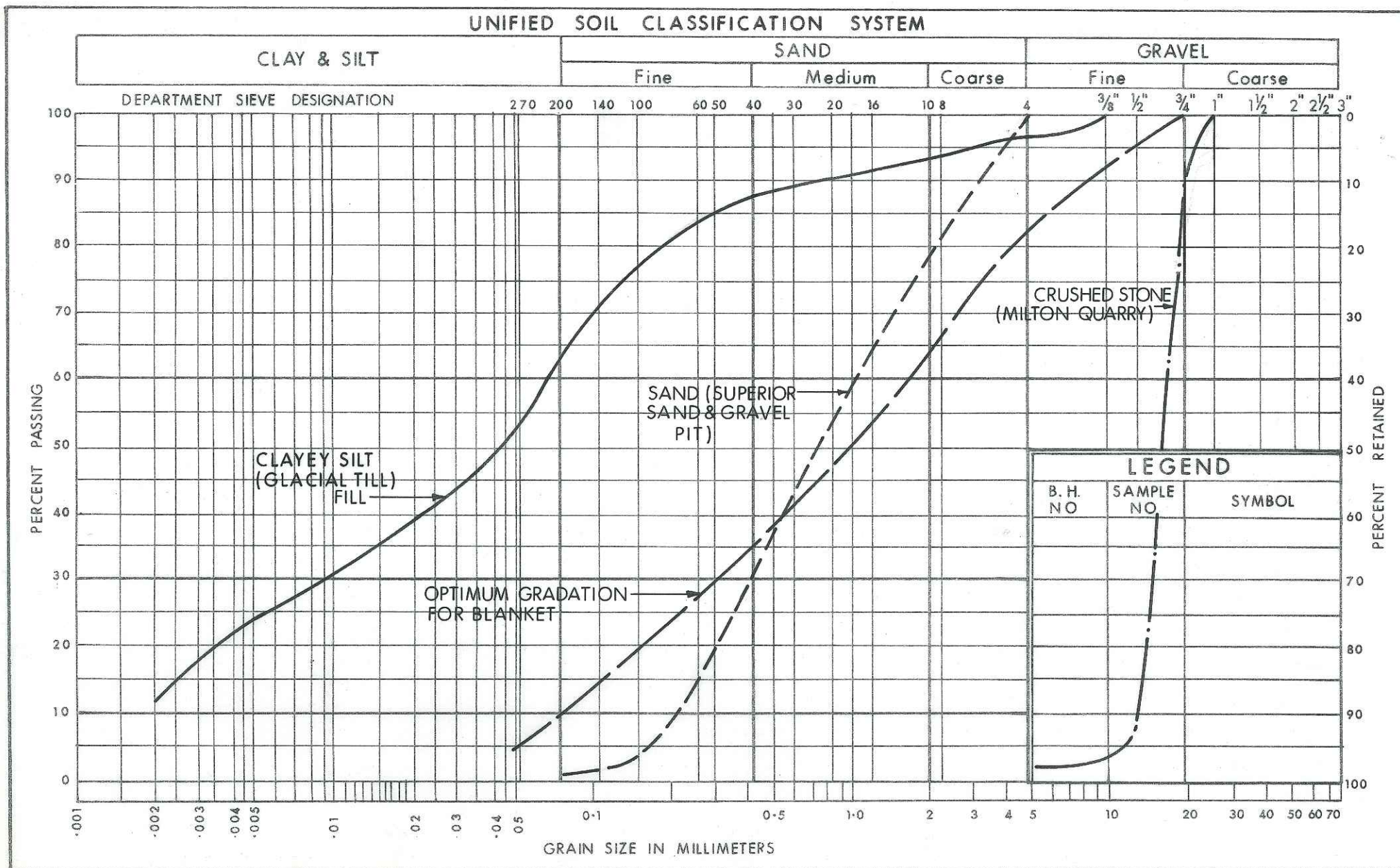
W.P. No.

JOB No.

68 - F - 46

FIG. 3





DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

# GRAIN SIZE DISTRIBUTION

RECOMMENDATION GRADATION FOR GRANULAR BLANKET —  
TYPICAL GRADATION OF AVAILABLE GRANULAR BORROW

W.P. No.

**JOB No.**

68 - F - 46

FIG. 4

## ABBREVIATIONS USED IN THIS REPORT

### PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

### DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS:-

<u>CONSISTENCY</u>	<u>'N' BLOWS / FT.</u>	<u>c LB. / SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

### TYPE OF SAMPLE

S.S	SPLIT SPOON	T.W	THINWALL OPEN
W.S	WASHED SAMPLE	T.P	THINWALL PISTON
S.B	SCRAPER BUCKET SAMPLE	O.S	OESTERBERG SAMPLE
A.S	AUGER SAMPLE	F.S	FOIL SAMPLE
C.S	CHUNK SAMPLE	R.C	ROCK CORE
S.T	SLOTTED TUBE SAMPLE		
	P.H	SAMPLE ADVANCED HYDRAULICALLY	
	P.M	SAMPLE ADVANCED MANUALLY	

### SOIL TESTS

Q <sub>u</sub>	UNCONFINED COMPRESSION	L.V	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V	FIELD VANE
Q <sub>cu</sub>	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Q <sub>d</sub>	DRAINED TRIAXIAL	S	SENSITIVITY

## ABBREVIATIONS USED IN THIS REPORT

### SOIL PROPERTIES

$\gamma$	UNIT WEIGHT OF SOIL (BULK DENSITY)
$\gamma_s$	UNIT WEIGHT OF SOLID PARTICLES
$\gamma_w$	UNIT WEIGHT OF WATER
$\gamma_d$	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
$\gamma'$	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
$S_r$	DEGREE OF SATURATION
$w_L$	LIQUID LIMIT
$w_p$	PLASTIC LIMIT
$I_p$	PLASTICITY INDEX
s	SHRINKAGE LIMIT
$I_L$	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
$I_c$	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
$e_{max}$	VOID RATIO IN LOOSEST STATE
$e_{min}$	VOID RATIO IN DENSEST STATE
$I_D$	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY $D_r$ IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
Q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
$m_v$	COEFFICIENT OF VOLUME CHANGE = $\frac{-\Delta e}{(1+e)\Delta\sigma}$
$C_v$	COEFFICIENT OF CONSOLIDATION
$C_c$	COMPRESSION INDEX = $\frac{\Delta e}{\Delta \log_{10} \sigma}$
$T_v$	TIME FACTOR = $\frac{C_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
$\tau_f$	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT
$\phi'$	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
$c_u$	APPARENT COHESION
$\phi_u$	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
$\mu$	COEFFICIENT OF FRICTION
$S_t$	SENSITIVITY

### GENERAL

$\pi$	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

### STRESS AND STRAIN

u	PORE PRESSURE
$\sigma$	NORMAL STRESS
$\bar{\sigma}$	NORMAL EFFECTIVE STRESS ( $\bar{\sigma}$ IS ALSO USED)
$\tau$	SHEAR STRESS
$\epsilon$	LINEAR STRAIN
$\gamma$	SHEAR STRAIN
$\nu$	POISSON'S RATIO ( $\mu$ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
$\eta$	COEFFICIENT OF VISCOSITY

### EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
$\delta$	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
$K_0$	COEFFICIENT OF EARTH PRESSURE AT REST

### FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
$k_s$	MODULUS OF SUBGRADE REACTION

### SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
$\beta$	ANGLE OF SLOPE TO HORIZONTAL



# 68 - F-46

W.P. # 85-59-1

HWY. # 401

HOGGS HOLLOW CROSSING

