STATE OF NEVADA

DEPARTMENT OF TRANSPORTATION

MATERIALS AND TESTING DIVISION

GEOTECHNICAL SECTION

FOUNDATION REPORT

SOUTH VIRGINIA OVERPASS

STRUCTURE NO. I-1831

March, 1993

E. A. No. 71565-1

WASHOE COUNTY, NEVADA

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I. INTRODUCTION

A. General

The soil investigation has been performed for the proposed overpass structure located at the intersection of South Virginia Street and 395 extension in Reno, Nevada. Plate 1 presents a site plan of the proposed project.

B. Purpose and Scope

The purpose of this soil investigation is to determine the subsurface soil conditions and to provide geotechnical design criteria for the proposed project based upon our findings. The Scope of this investigation included surface reconnaissance, subsurface exploration, soil sampling, laboratory testing, analysis of field and laboratory data, research of available geologic literature pertaining to the site, and report preparation. This report provides conclusions and recommendations concerning:

- General subsurface conditions and geology
- Engineering properties of the various strata which will influence the development, including:
  - Bearing Capacity
  - Settlement potential
- Foundation and Footing type and Design Criteria
- Seismic Design Criteria

C. Project Description

The proposed structure is a two span continuous composite plate girder structure with seat abutments.

One - half of the bridge and the abutments of the proposed structure were constructed in 1988. The proposed steel structure will span 170 feet in the southbound lanes and 190 feet in the northbound lanes. Abutment no.1 (East abutment) is located at station "P4" 672+11.76 P.O.T and Abutment no.2 (West Abutment) is located at station "P4" 675+81.62 P.O.T.

It was requested by N.D.O.T. Bridge Division to investigate the feasibility of using a single large diameter drilled shaft to support the pier columns with axial loading of 440 tons and 2 foot diameter cast in drilled hole to support the abutments.

Also later on, Bridge Division requested the feasibility of using 24 inch driven pipe piles with axial loading of 115 tons be evaluated.

D. Site Description

The subject site is located in the northeast corner of section 7, T.18 N., R.20 E., M.D.B. & M. The approximate elevation of the original ground is 4494.5 feet. The site is relatively flat.

E. Field Exploration and Laboratory Testing

On October 26, 1992, the Geotechnical Section of the Material and Testing Division of N.D.O.T. conducted a subsurface site investigation at the site.

The subsurface soil conditions were explored by drilling three borings to a maximum depth of 114.5 feet. The approximate location of the borings are shown on plate 1. Continuous logs of the subsurface conditions as encountered during the investigation were recorded at the time of drilling by a geotechnical engineer. The soils encountered were classified in accordance with ASTM D2487 based on the Unified Soil Classification System. Drilling was accomplished with a truck-mounted rotary drill rig equipped for soil sampling. Drilling fluid (bentonite slurry) was used to remove the cuttings from the borings. Soil samples were obtained utilizing Standard Penetration Testing procedures. Sampler driving resistance, expressed as blows per foot of penetration, is presented on the boring logs at the respective sampling depth. Selected soil samples were retained and transported to N.D.O.T. headquarters laboratory facilities for further testing. The laboratory testing program for these samples consisted of natural moisture content, gradation, Atterburg limit, chemical analysis, unit weight, and unconsolidated undrained (UU) triaxial tests to confirm field soil classifications and to provide insitu index values of soils. Results of these tests and the boring logs are presented in the Appendix.

II. DISCUSSION

A. Site Geology

The primary geologic reference for this area is a geologic map prepared by H.F. Bonham, Jr. and David K. Rogers, 1983 of the Bureau of Mines and Geology of Mackay School of Mines. According to this Geologic map, a Quaternary age formation underlies the site. It consists of Alluvial deposits (Qa) which includes fine to medium grained clayey sand and intercalated muddy, medium pebble gravel. These are the deposits of low gradient streams that reworked older gravelly outwash and alluvial fan deposits.

B. Seismicity and Geologic Hazards

The reference for this area is an earthquake hazards map prepared by Gail Cordy Szecsody, 1983 of the Bureau of Mines and Geology of Mackay School of Mines. According to this map, the potential for ground shaking during earthquakes is high in this area. Since the depth to groundwater is less than 10 feet and the soil is unconsolidated, there is a possible localized liquefaction potential (refer to liquefaction, page 6).
According to the Quaternary Fault Map of Nevada by John W. Bell, 1984, the subject site is within four miles of several Holocene faults (less than 12,000 years old) which are considered to be active. These faults are located to the west and during a seismic

event originating from these faults, the subject site may experience large ground motions.

C. Native Soils

The following section presents a generalized description of the native soil types encountered during our field investigation. The boring logs should be reviewed for a more through description. The site is underlain by medium dense silty sand and clay of low to medium plasticity. From twenty to fifty feet of depth, stiff silt and clay overlie dense silty sand. From fifty to ninety feet, very stiff to hard silt and clay of low plasticity and dense silty sand were encountered. Below this depth, ten to fifteen feet of very dense coarse sand and gravel was observed.

D. Groundwater

Groundwater was encountered at ten feet of depth at time of subsurface investigation and it has fluctuated to within five feet of depth since then. It may rise to the surface during the wet years. The groundwater will influence the method of excavation.

E. Liquefaction Analysis

In saturated loose to medium compact granular soils, seismic shocks may produce unacceptable shear strains. In such cases, the high shearing deformations and decreased shear strength is the consequence of the progressive buildup of high pore-pressure generated by seismic shaking. With no or limited drainage, cyclic shear stress can produce a progressive buildup of pore water pressure, significantly reducing the effective stress which controls the soil strength for practical purposes. The effective stress after several cycles of shear straining may ultimately be reduced to zero with total liquefaction.

Character of ground motion, soil type and in-situ stress conditions are the three primary factors controlling the development of cyclic mobility or liquefaction.

Case histories indicate that the liquefaction has occurred within a depth of 50 feet or less.

Liquefaction potential was evaluated for the site since it can cause the loss of side friction along the shaft or pile which resists the axial loading.

The analysis requires estimation of the bedrock acceleration generated by earthquake. Based on current practice at N.D.O.T., the NEHRP Map which was prepared by the U.S. Geological Survey, 1988 will be used. This is the map of Horizontal Acceleration (expressed as percent of gravity) in Rock with 90 percent probability of not being exceeded in 50 years.

The site is located in an area defined by the NEHRP map as having a horizontal acceleration coefficient in rock of 0.4g.

Two basic approaches were used for evaluation of liquefaction potential at the proposed site:

1. Empirical method is based on evaluation of liquefaction case histories, and in situ strength characteristics such as measured by the standard penetration resistance N.

2. Analytical Method is based on a comparison between field liquefaction strengths and earthquake-induced shearing stresses, using the Simplified Procedure by Seed and Idriss.

Based on the above techniques, there are some localized areas along the piles which are susceptible to liquefaction during earthquake.

It should be noted that there are considerable amounts of fi es in the subsurface soils and it is still not possible to evaluate the likelihood of liquefaction of a silty sand with the same confidence as for a clean sand.

Based on case histories (Ishihara, 1985) if the height of fill is greater than 9 feet, it will prevent the observable effects of an at-depth liquefaction from reaching the surface. This effect was shown in the field during the Niigata earthquake where soil under a 9 foot fill remained stable. Since the height of abutment fills is about 20 feet, it will reduce the liquefaction potential and increase the liquefaction safety factor.

In the design capacity, the side friction contribution from this susceptible zone has been neglected.

F. Soil Corrosion

Laboratory tests indicate that the subsurface soils have low concentrations of corrosive salts such as chlorides and sulfates with neutral pH and relatively high resistivity.

G. Settlement

The abutments were constructed in 1988 and the underlying native soils within the influence zone of the applied load have already been consolidated. No further settlement in the abutment area is expected

III. FOUNDATION RECOMMENDATIONS

This section presents two types of foundation supports for the proposed structure:

I. Drilled Shafts

The following are the foundation recommendations which are based on the Bridge Divisions request of using a single large diameter drilled shaft to support the center pier columns and total axial load of 440 tons, and two foot diameter cast in drilled hole piles to support the abutments. At this time no lateral loading was provided by Bridge Division.

All the design calculations are based on the Drilled Shafts Manual prepared for Federal Highway Administration, 1988.

<u>Center Pier</u>

Requested Design:

Calculations indicate an 8 foot diameter drilled shaft with a length of 95 feet below the existing ground surface should be capable of supporting a 440 ton axial load with a safety factor of 3.0 with 0.2 inches of immediate settlement. This unexpected length is due to the presence of silty clay between a depth of 50 and 90 feet which is subject to long term settlement upon loading. In order to overcome the adverse effect of settlement, the shaft should be extended below this compressive material. This design capacity is obtained by 48% skin friction and 52% end bearing. Since the center piers are spaced less than 3 shaft diameters center to center, a reduction factor of 0.67 was also applied. Since the length of the shaft is greater than 75 feet, this option may not be cost effective.

Recommended Design:

Two foot diameter drilled shafts with lengths of 50 feet below the ground surface are capable of supporting 60 ton axial load with a safety factor of 3.0. For center to center spacing of less than 3 shaft diameter, a reduction factor of 0.67 for axial loading should be applied. This option is more economical and preferable.

Abutments

Two foot diameter drilled shaft with the length of 50 feet below the natural ground surface (approximate ground surface elevation is 4494.6 feet) should be capable of supporting 70 tons of axial design load with a safety factor of 3.0 and settlement of less than 0.1 inch.

This design capacity is obtained by 70% skin friction and 30% end bearing. If the center to center spacing is less than 3 shaft diameter, a reduction factor of 0.67 should be applied to axial capacity.

II. Driven Piles

The most suitable foundation support for this site would be provided by driven piles. All the design calculations are based on the Manual on Design and Construction of Driven Pile Foundation prepared by Federal Highway Administration, 1986.

Twenty-four inch diameter hollow pipe piles driven with the end capped and then backfilled with concrete as it was suggested, were evaluated for design. These pipe piles with the length of 50 feet below the natural ground surface (approximate ground surface elevation is 4494.5 feet, and design tip elevation of approximately

4444.5) should be capable of supporting 115 tons of axial design load with a safety factor of more than 2.5 and the long term settlement of less than 1 inch. This capacity is for dead load plus live load. This design capacity is obtained by 44% skin friction and 56% end bearing.

Piles in groups should not be installed at spacing less than 3 times the pile diameter. In this case, the ultimate group load can be taken as the sum of the single pile capacity.

The fill can be pre-drilled and then piles be driven into the native soils. Once the pile is in place, the pre-drilled area can be backfilled with pea gravels.

During pile driving operation, the minimum design tip elevation should be no less than elevation 4449.5 (45 feet below the original ground surface). The targeted design tip elevation is specified at elevation 4444.5 feet (50 feet below the original ground surface). Pile driving inspectors should be careful to limit driving operations to WEAP driving criteria. This will insure that ultimate pile capacity is obtained without over-stressing the pipe piles.

To perform pile driving successfully, the pile must have sufficient stiffness to transmit driving forces large enough to overcome soil resistance. Also, the pile must have sufficient strength to withstand the driving forces without damage. To meet the above criteria and considering the site soil conditions and the pile length, the allowable driving stress should be limited to 0.9 of the steel yield strength (Fy). it is recommended to use steel pipe pile with 0.5 inch wall thickness if the steel yield strength is 35 ksi. Wall thickness of 0.25 inch is allowed if the steel yield strength is 45 ksi per ASTM A252.

Lateral Capacity

It was requested that the Geotechnical Section provides only P-Y Curves for seismic loading. To calculate the lateral design capacities and related lateral deflections, one should use the generated table of P versus Y which is presented in the Appendix of this report as input to a computer program COM622 for pile response computations. Computer program COM624 has been written in which the criteria for generating P-Y curves are subroutines of the program and the engineer only has to specify soil properties, pile geometry, and the kind of loading (static or cyclic).

As the pile penetration is increased, soil resistance at the bottom of the pile will increase and the groundline deflection will reach a limiting value where increased penetration will cause no groundline deflection. Thus, the designer should make computations for a series of pile penetrations and determine a penetration that yield an appropriate factor of safety.

Engineer should treat the results of this program as an aid in the overall process of engineering analysis and deign, not as the sole basis for design nor as the final word on how a laterally loaded deep foundation will perform.

Seismic Design Criteria

Seismic design criteria is based on AASHTO specifications. It is recommended to use soil profile type S2 with soil profile coefficient of 1.2 for the effect of site conditions on bridge response during an earthquake.

IV. Construction Specifications for Drilled Shafts

Excavation Stage

Outside drilled shaft locations should be excavated first and after waiting period of 5 to 7 days for concrete to cure, then the excavation for the inside shaft can proceed.

Concrete

The concrete must have a quality of workability suitable for uniform and proper placement throughout the duration of shaft construction and when cured must have the required strength and durability. The concrete must conform to the appropriate standards specifications. Items to be addressed by the standards include the following:

* proportioning of materials
* Cement
* Water Quality
* Aggregates
* Admixtures, including retarders and other additives
* Shrinkage or Expansion
* Curing
* Tests and their procedures
* Strength
* Slump

Slump

Since the concrete will be placed under water and or slurry, the slump should be in the range of 7 to 9 inches.

Steel

All the steel used in the construction of drilled shafts for reinforcing, permanent casing, or temporary casing should conform to the appropriate standards specified by the engineer. Reinforcing steel should have appropriate properties for strength, durability, and bond.

When calculating the stress in the steel, a minimum allowance of 1/16 in should be made for corrosion.

Drilling Slurry

Drilling slurry may be used for the maintenance of the stability of an uncased hole until a casing has been installed or concrete has been placed. The type of drilling slurry to be used should be approved by the engineer, with the subsurface conditions taken into account. The preferred method of forming the slurry is to use a mixing plant, or mixing machine, and prepare the slurry prior to its placement.

The properties of the drilling slurry must be controlled during the drilling of the excavation and during the placement of the concrete. The principal concern during drilling is that there may be a collapse of the borehole; a considerable variation in the slurry properties can be allowed if the borehole maintains its shape during the excavation. The principal concerns during the placing of the concrete are that the slurry does not weaken the bond between the concrete and the natural soil, that all of the slurry is discharged from the borehole by the rising column of fresh concrete, and that any sediment carried by the slurry is not allowed to be deposited in the borehole.

The following is one of the recommendations that may be satisfactory for construction in fine sands as in this site:

Density (pcf)	64.3 - 69.1
Viscosity (sec./quart)	28 - 45
pH	8 - 11

REFERENCES

FHWA, Drilled Shafts: Construction Procedures and Design Methods, U.S. Department of Transportation, Washington D.C., 1988.

NAVFAC DM-7.1, Design Manual, Soil Dynamics, U.S. Government Printing Office, Washington D.C., 1983.

NRC, Liquefaction of Soil During Earthquakes, National Academy Press, Washington, D.C., 1985.

Das, Braja M, Principles of Soil Dynamics, PWS-Kent, Boston, 1993.

FHWA, Soils and Foundations Workshop Manual, U.S. Department of Transportation, Washington D.C., 1982.

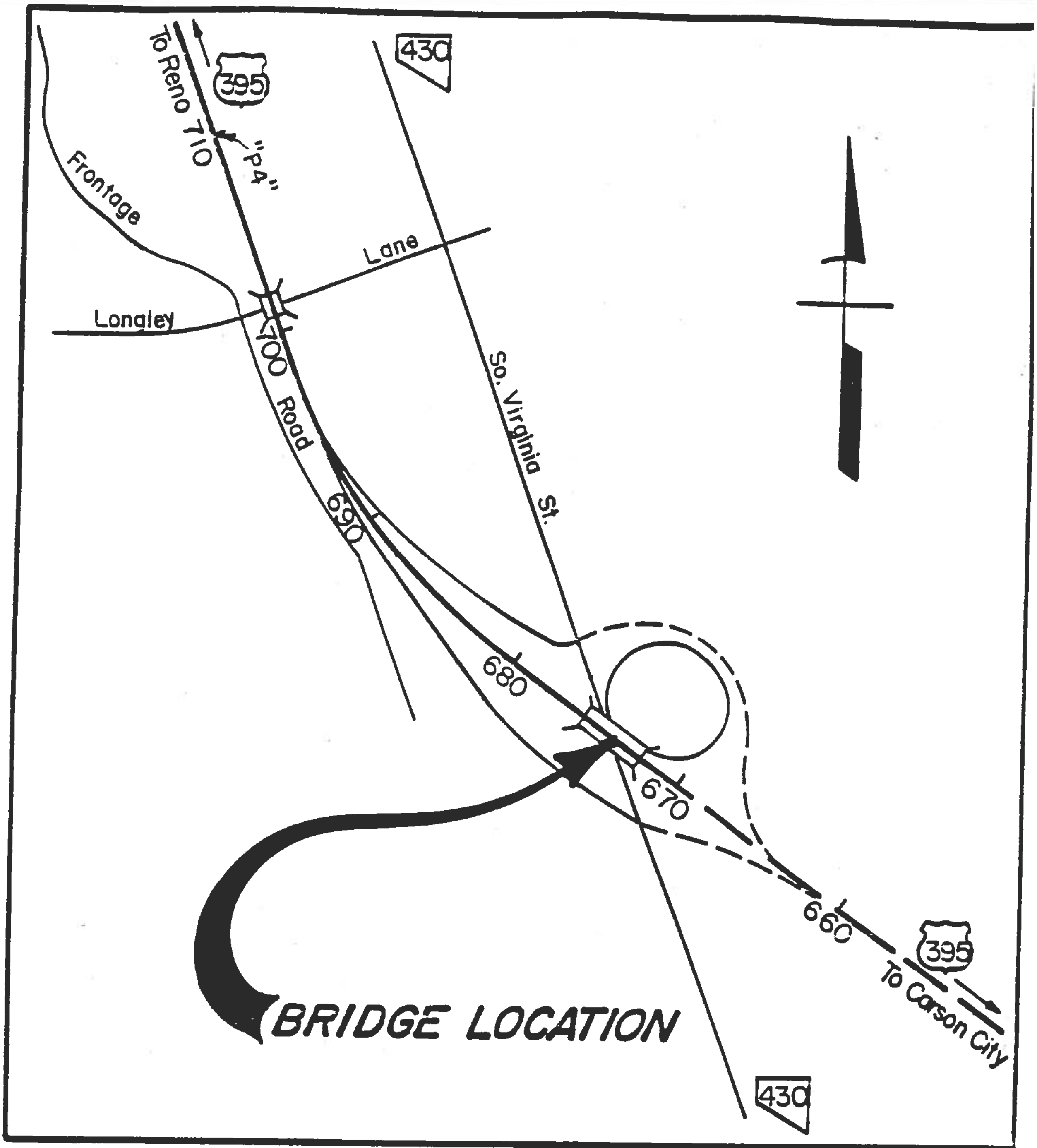
AASHTO, Guide Specifications for Seismic Design of Highway Bridges, AASHTO, Washington D.C., 1988.

FHWA-NHI, Seismic Design of Highway Bridges-Map of Horizontal Acceleration, Federal Highway Administration Publication No. FHWA-HI-91-019, Washington D.C., 1988.

FHWA, Manual on Design and Construction of Driven Pile Foundations, U.S. Department of Transportation, Washington D.C., 1985.

FHWA, Behavior of Piles and Pile Groups Under Lateral Load, Federal Highway Administration Publication No. FHWA/RD-85/106, Washington D.C., 1986.

APPENDIX



LOCATION SKETCH

STATE OF NEVADA
DEPARTMENT OF TRANSPORTATION

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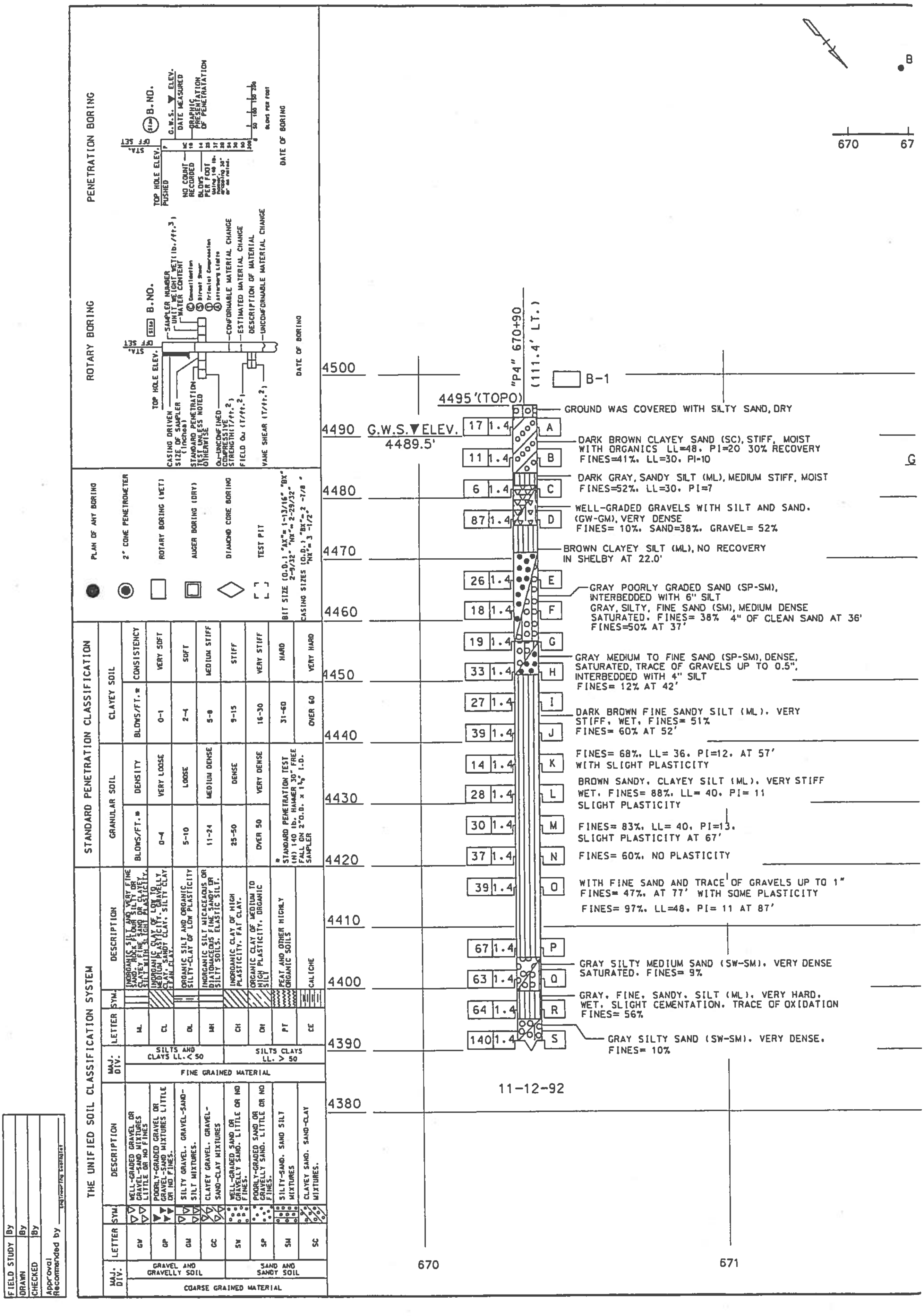
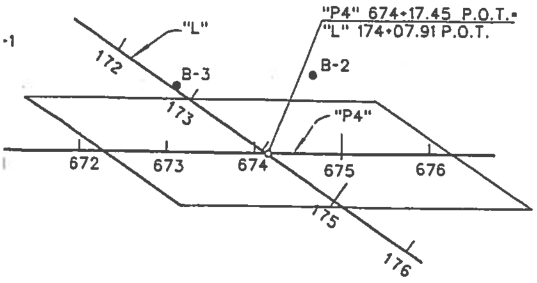
SO. VIRGINIA INTERCHANGE

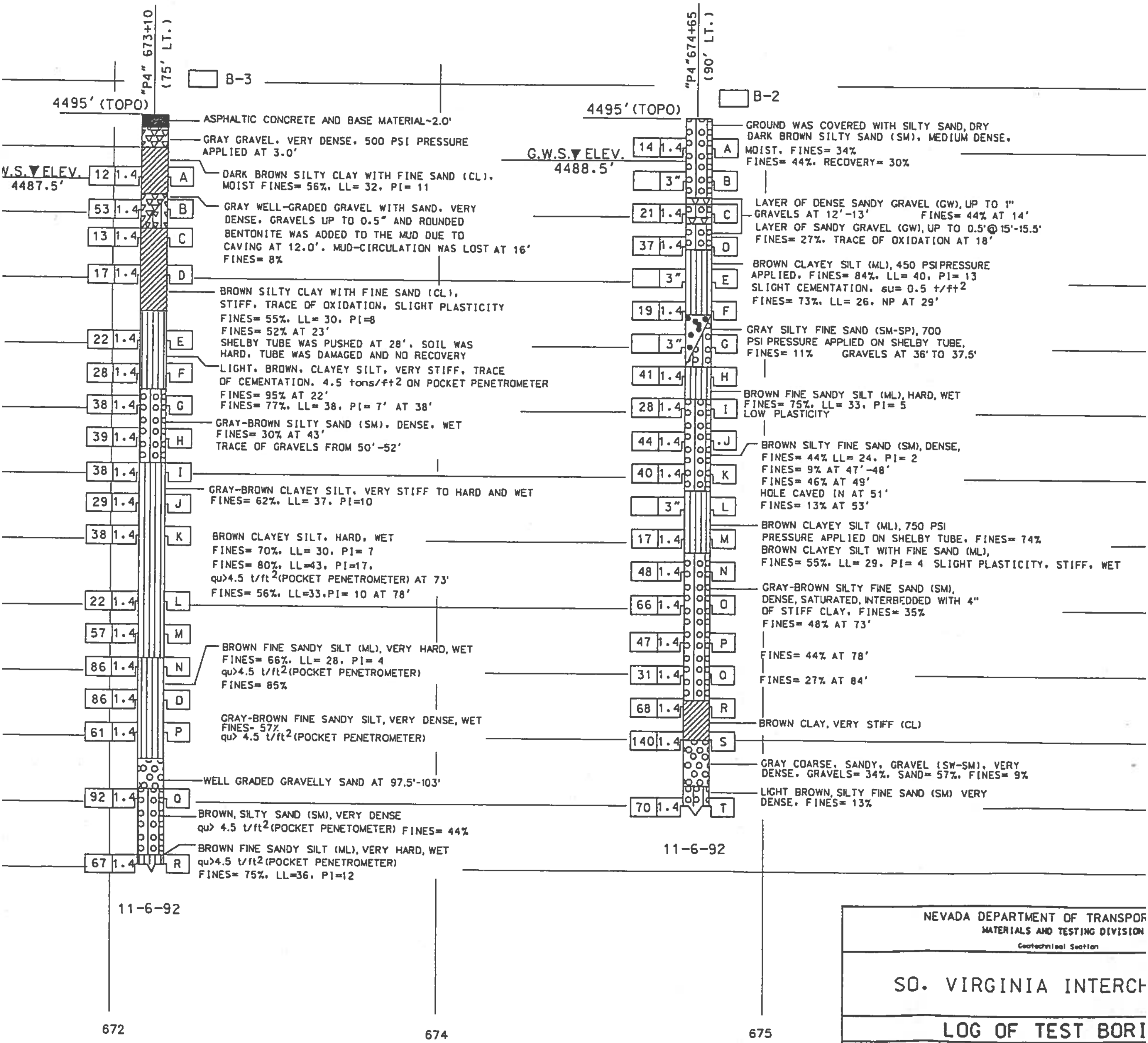

PENETRATION BORING
ROTARY BORING
STANDARD PENETRATION CLASSIFICATION
THE UNIFIED SOIL CLASSIFICATION SYSTEM
FIELD STUDY By
DRAWN By
CHECKED By
Approval Recommended by
B
670
67
4500
4495'(TOPO)
"P4" 670+90
(111.4' LT.)
B-1
GROUND WAS COVERED WITH SILTY SAND, DRY
4490
G.W.S.▼ELEV.
4489.5'
17 1.4 A
DARK BROWN CLAYEY SAND (SC), STIFF, MOIST WITH ORGANICS LL=48, PI=20 30% RECOVERY FINES=41%, LL=30, PI=10
11 1.4 B
DARK GRAY, SANDY SILT (ML), MEDIUM STIFF, MOIST FINES=52%, LL=30, PI=7
4480
6 1.4 C
WELL-GRADED GRAVELS WITH SILT AND SAND, (GW-GM), VERY DENSE FINES= 10%, SAND=38%, GRAVEL= 52%
87 1.4 D
4470
BROWN CLAYEY SILT (ML), NO RECOVERY IN SHELBY AT 22.0'
26 1.4 E
GRAY POORLY GRADED SAND (SP-SM), INTERBEDDED WITH 6" SILT
GRAY, SILTY, FINE SAND (SM), MEDIUM DENSE SATURATED, FINES= 38% 4" OF CLEAN SAND AT 36' FINES=50% AT 37'
4460
18 1.4 F
19 1.4 G
GRAY MEDIUM TO FINE SAND (SP-SM), DENSE, SATURATED, TRACE OF GRAVELS UP TO 0.5", INTERBEDDED WITH 4" SILT FINES= 12% AT 42'
4450
33 1.4 H
27 1.4 I
DARK BROWN FINE SANDY SILT (ML), VERY STIFF, WET, FINES= 51% FINES= 60% AT 52'
4440
39 1.4 J
FINES= 68%, LL= 36, PI=12, AT 57' WITH SLIGHT PLASTICITY
14 1.4 K
BROWN SANDY, CLAYEY SILT (ML), VERY STIFF WET, FINES= 88%, LL= 40, PI= 11 SLIGHT PLASTICITY
4430
28 1.4 L
30 1.4 M
FINES= 83%, LL= 40, PI=13, SLIGHT PLASTICITY AT 67'
4420
37 1.4 N
FINES= 60%, NO PLASTICITY
39 1.4 O
WITH FINE SAND AND TRACE OF GRAVELS UP TO 1" FINES= 47%, AT 77' WITH SOME PLASTICITY
FINES= 97%, LL=48, PI= 11 AT 87'
4410
67 1.4 P
GRAY SILTY MEDIUM SAND (SW-SM), VERY DENSE SATURATED, FINES= 9%
4400
63 1.4 Q
GRAY, FINE, SANDY, SILT (ML), VERY HARD, WET, SLIGHT CEMENTATION, TRACE OF OXIDATION FINES= 56%
64 1.4 R
4390
140 1.4 S
GRAY SILTY SAND (SW-SM), VERY DENSE, FINES= 10%
11-12-92
4380
670
671

FED. RD. REG. NO.	STATE	PROJECT NO.	COUNT
9	NEVADA		WASH

NOTE: FOUNDATION REPORT AVAILABLE FOR CONTRAC
IN DISTRICT OFFICE AND MATERIALS & TEST



BORING PLAN



NEVADA DEPARTMENT OF TRANSPOR
MATERIALS AND TESTING DIVISION
Geotechnical Section

SO. VIRGINIA INTERCH

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BRIDGE NO. I-1831 | MILE POST | E.A. NO. 71565

REVISION DATES

DISREGARD PRINTS BEARING EARLIER REVISION DATES

2-25-93 east.dgn

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL SECTION
SUMMARY OF TEST RESULTS

E.A. NO. 71565 - South Virginia Street

Boring No.: B - #1

Total Depth (ft) : 103.5

Station or Location :

Sample No.	Sample Depth (ft.)	Sampler Type	N Blows/ Foot	Soil Group	Unit Dry Wt. (pcf)	Unit Wet Wt. (pcf)	Water Content %	% Minus 200	PI	LL	Shear Strength Parameters				Other Tests Performed
											Test Type	ø	C	Qu	
B	7- 8.5	SS	11	SC			25	41	10	30					
C	12- 13.5	SS	6	ML			33	52	7	33					
D	17- 18.5	SS	87	GW			12								
E	27- 28.5	SS	20	ML			38	73	3	31					
F	32- 33.5	SS	18	SM			29	38							
G	37- 38.5	SS	19	SM			28	50							
H	42- 43.5	SS	33	SM			23	12							
I	47- 48.5	SS	27	ML			28	51							
J	52- 53.5	SS	29	ML			30	60							
K	57- 58.5	SS	14	ML			33	68	12	36					
L	62- 63.5	SS	28	ML			33	88	11	40					
M	67- 68.5	SS	30	ML			35	83	13	40					
N	72- 73.5	SS	37	ML			31	60							
O	77- 78.5	SS	39	ML			29								
P	87- 88.5	SS	67	ML			44	97	11	48					
Q	92- 93.5	SS	63	SP-SM			25	9							
R	97- 98.5	SS	64	ML			25	56							
S	102-103.5	SS	140	SW			15	10							

NOTATION:
UU = Unconsolidated Undrained
CD = Consolidated Drained
CU = Consolidated Undrained
UC = Unconfined Compression
S = Direct Shear

C = Cohesion - TSF
ø = Angle of Internal Friction - Degrees
Qu = Unconfined Compressive Strength - TSF
PI = Plasticity Index
LL = Liquid Limit

ES = Expansive Soils
GS = Specific Gravity
OC = Consolidation
SH = Shelby
SS = Split Spoon 1.4"

E.A. NO. 71565 - South Virginia Street

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL SECTION
SUMMARY OF TEST RESULTS

Boring No.: B - #2

Total Depth (ft) : 105

Station or Location :

Sample No.	Sample Depth (ft.)	Sampler Type	N Blows/ Foot	Soil Group	Unit Dry Wt. (pcf)	Unit Wet Wt. (pcf)	Water Content %	% Minus 200	PI	LL	Shear Strength Parameters				Other Tests Performed
											Test Type	ø	C	Qu	
A	3 - 4.5	SS	14	SM			21	34							
B	8 - 9.5	SH		SM			30	44	8	30					
C	13 - 14.5	SS	21	SM			32	44							
D	18 - 19.5	SS	37	SM			23	27							
E	23 - 25.5	SH		ML			40	84	5	35	UU		1000	PSF	Su=900 Lab Torvane
F	28 - 29.5	SS	19	ML			40	73	NP	26					
G	33 - 34.5	SH		SM			31	11							
H	38 - 39.5	SS	41	ML			34	75	5	33					
I	43 - 44.5	SS	28	SM			29	44	2	24					
J	48 - 49.5	SS	44	SM			23	46							
K	53 - 54.5	SH		SM			20	13							
L	58 - 60.5	SH		ML				74					900		Lab Torvane
M	63 - 64.5	SS	17	ML			30	55	4	29					
N	68 - 69.5	SS	36	SM			27	35							
O	73 - 74.5	SS	52	SM			35	48							
P	78.5- 80	SS	32	SM			28	44							
Q	83.5- 85	SS	31	SM			23	27							
R	88.5- 90	SS	68	SM			25								
S	93.5- 95	SS	140	SW			15	9							
T	103.5-105	SS	86	SM			24	13							

NOTATION:

UU = Unconsolidated Undrained
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UC = Unconfined Compression
S = Direct Shear

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SH = Shelby
SS = Split Spoon 1.4"

E.A. NO. 71565 - South Virginia Street

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL SECTION
SUMMARY OF TEST RESULTS

Boring No.: B - #3

Total Depth (ft) : 114.5

Station or Location :

Sample No.	Sample Depth (ft.)	Sampler Type	N Blows/ Foot	Soil Group	Unit Dry Wt. (pcf)	Unit Wet Wt. (pcf)	Water Content %	% Minus 200	PI	LL	Shear Strength Parameters				Other Tests Performed
											Test Type	ø	C	Qu	
A	8 - 9.5	SS	12	CL			25	56	11	32					
B	13 - 14.5	SS	53	GW			11.3	8							
C	17.5- 15.8	SS	13	CL			34	55							
D	23 - 24.5	SS	17	CL			29	52	8	30					
E	33 - 34.5	SS	22	CL			49.4	95							
F	38 - 39.5	SS	28	ML			37.3	77	7	38					
G	43 - 44.5	SS	38	SM			25	30							
H	48 - 49.5	SS	39	SM											
I	53 - 54.5	SS	38	ML			36	62	10	37					
J	58 - 59.5	SS	29	CL			25		11	29					
K	63 - 64.5	SS	38	CL			31.4	70	7	30					
L	73 - 74.5	SS	22	CL	95	126	32.6	80	17	43					
M	78 - 79.5	SS	57	CL			28.2	56	10	33					
N	83 - 84.5	SS	86	ML	98	124	26.7	66	4	28					
O	88 - 89.5	SS	86	ML	92	120.6	31.7	85							
P	93 - 94.5	SS	61	ML			31.4	57							
Q	103 -104.5	SS	92	ML			20.2								
R	113 -114.5	SS	67	ML	93	122	30.7	75	12	36					

NOTATION:
UU = Unconsolidated Undrained
CD = Consolidated Drained
CU = Consolidated Undrained
UC = Unconfined Compression
S = Direct Shear

C = Cohesion - TSF
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ES = Expansive Soils
GS = Specific Gravity
OC = Consolidation
SH = Shelby
SS = Split Spoon 1.4"

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL SECTION

CHEMICAL ANALYSES

LAB #. FL-7-93 E. A. NO. 71565

PROJECT 395 FREEWAY EXTENSION

SAMPLE NO.	B-1 R
CHEM NO.	C-157-93
DATE	11-6-92
CHLORIDES PPM.	60
SULFATES PPM.	Ø
PH	7.4
RESISITIVITY OHM / CM	12,987

SAMPLE NO.	B-2 G
CHEM NO.	C-162-93
DATE	11-10-92
CHLORIDES PPM.	30
SULFATES PPM.	Ø
PH	8.2
RESISITIVITY OHM / CM	6,849

SAMPLE NO.	B-2 E_1
CHEM NO.	C-164-93
DATE	11-10-92
CHLORIDES PPM.	30
SULFATES PPM.	Ø
PH	7.9
RESISITIVITY OHM / CM	13,158

SAMPLE NO.	B-3 E
CHEM NO.	C-168-93
DATE	11-16-92
CHLORIDES PPM.	60
SULFATES PPM.	Ø
PH	7.6
RESISITIVITY OHM / CM	6,289

SAMPLE NO.	B-3 F
CHEM NO.	C-169-93
DATE	11-16-92
CHLORIDES PPM.	30
SULFATES PPM.	Ø
PH	7.6
RESISITIVITY OHM / CM	6,061

PH2 PP
11-16-92

DEVELOPMENT OF P–Y CURVES FOR GRANULAR SOILS
For Route 395 Between S Virginia and Brown's School

LOCATION:		Bridge:	S. Virgina Street #I–1831
		Support:	Center Pier –– Driven Pipe Piles
		Remarks:	Cap ~5' below surface
DEPTH:	of P–y	x(ft.):@ bottom of pileca	4
	to water	Wtb(ft.):	0

INPUT PARAMETERS

Pile Diameter(in): 24

P–Y CURVE INTERCEPTS @ bottom of pilecap

Deflection–y(in.)	0.04	0.13	0.22	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	2.57E+02	3.41E+02	3.88E+02	4.50E+02	5.90E+02	5.90E+02

P–Y CURVE INTERCEPTS @ 2' below pilecap

Deflection–y(in.)	0.03	0.12	0.21	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	2.87E+02	5.00E+02	6.20E+02	7.85E+02	1.15E+03	1.15E+03

P–Y CURVE INTERCEPTS @ 6' below pilecap

Deflection–y(in.)	0.02	0.11	0.21	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	3.09E+02	7.35E+02	9.84E+02	1.34E+03	2.15E+03	2.15E+03

P–Y CURVE INTERCEPTS @ 10' below pilecap

Deflection–y(in.)	0.11	0.18	0.26	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	2.52E+03	3.20E+03	3.75E+03	4.64E+03	7.43E+03	7.43E+03

P–Y CURVE INTERCEPTS @ 15' below pilecap

Deflection–y(in.)	0.08	0.16	0.24	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	2.48E+03	3.43E+03	4.15E+03	5.29E+03	8.46E+03	8.46E+03

P–Y CURVE INTERCEPTS @ 22' below pilecap

Deflection–y(in.)	0.13	0.20	0.27	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	5.44E+03	6.60E+03	7.57E+03	9.19E+03	1.47E+04	1.47E+04

P–Y CURVE INTERCEPTS @ 32' below pilecap

Deflection–y(in.)	0.24	0.28	0.32	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	1.31E+04	1.42E+04	1.51E+04	1.69E+04	2.70E+04	2.70E+04

FOUNDATION DESIGN DATA

For Abutments located in Embankments

Soil Unit Weight (γ)	130 #/ft^3
Coef. of Active Earth Pressure (K_a)	.271 (level) .39 (2:1 slope)
Coef. of Passive Earth Pressure (K_p)	3.69 (level) 10.8 (2:1 slope) (not reliable for slope)
Coef. of Seismic Active Earth Pressure (K_{ae})	.581 (level w/o move) .396 (level w/ 4"mov)
Coef. of seismic Passive Earth Pressure (K_{pe})	2.82 (level w/o move)
Slopes at 2:1 will fail during earthquake	
Angle of Internal Friction (ϕ)	35°
Coefficient of friction for wall surfaces (δ)	0 (Rankine)
Coefficient of friction at footing bottom (δ) (footing concrete "neat" to embankment)	.35
Live Load Surcharge	250 #/ft^2