STATE OF NEVADA
DEPARTMENT OF TRANSPORTATION
MATERIALS AND TESTING DIVISION
GEOTECHNICAL SECTION

GEOTECHNICAL REPORT

US 395 FREEWAY EXTENSION FROM
S. VIRGINIA STREET TO NEAR BROWN SCHOOL.
STRUCTURES I-1950, I-1952, I-2007, H-2008, I-2009.

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WASHOE COUNTY, NEVADA

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INTRODUCTION

During the months of October, 1992 through February, 1993 the Geotechnical Section conducted subsurface site investigations at seven proposed structure sites for the future US 395 freeway extension project. Individual foundation reports (16) have already been distributed for two of the proposed grade separation structures (I-1831 and I-1951, to be located at S. Virginia Street (Sta. "H" 674) and Zolezzi Lane (Sta "H" 555, respectively). Additionally, separate reports (13,19) have been generated by outside consultants addressing the Mt. Rose Interchange structure (I-1949) and the highway embankment fills to be used to support most of the newly built freeway.

This report addresses the five remaining bridge sites investigated by NDOT (I-1950, 1952, 2007, 2009, & H-2008) between Sta. "H" 497 to "H" 628. These structures will serve as grade separations between the future extension of US 395 at the intersections of South Meadows Parkway (near former May's Lane), Old Virginia Road (aka....Old Virginia City Road), and South Virginia Street (near Brown School). All these bridge sites are located in the southwest portion of the Truckee Meadows south of the Huffaker Hills and just east of South Virginia Street (existing US 395).

Preliminary plans indicate that concrete, two span, cast-in-place, post-tensioned box girder type bridges, with 2:1 sloped earth embankments at the abutments, will be constructed at the South Meadows Parkway and South Virginia Street crossings. Concrete, simple span, cast-in-place, post-tensioned box girder bridges, with near vertical mechanically stabilized earth (MSE) abutment walls, will be used at the Old Virginia Road crossings.

Purpose of report

The intended purpose of this report is to provide specific recommendations concerning the geotechnical engineering design of the above mentioned structures. These recommendations are made considering:

1) The specific subsurface soil conditions as revealed during the site investigations at the individual bridge locations.

2) The quantitative laboratory test results from the selected soil samples transported to and tested at NDOT'S laboratory facilities.

3) Available geologic information from existing maps and reports.

4) The results of specific engineering analyses concerning pile design, soil liquefaction, slope stability, soil bearing capacity and settlement, and MSE wall external stability.

Exploration Program

A total of nine borings were completed at the various bridge sites. Borings were generally located at or near the proposed abutment locations for each bridge. Drilling was accomplished by a truck mounted Mobile B-80 rotary drill rig, using a 3.5 inch drill bit and bentonite drilling fluids. Continuous logs of the subsurface conditions were recorded at the time of drilling by a geotechnical engineer. Representative soil samples were taken from each boring using Standard Penetration Testing sampling procedures. Each soil sample was classified using the Unified Soil Classification System. Copies of the finished boring logs and boring location maps are included in Appendix 1.

Laboratory Testing Program

Selected soil samples were transported back to NDOT's Headquarters Laboratory facilities in Carson City for additional physical and chemical tests. A total of 58 Atterberg Limit tests and 118 sieve analyses (-200 sieve washes) were completed to aid in soil identification and classification. Seven chemical analyses were conducted to evaluate corrosive properties of the native soils. Additionally, soil tests conducted from near surface soil samples obtained along the entire project alignment were reviewed (17). The results of all the above mentioned tests are presented in Appendix 2.

GENERAL GEOLOGIC DISCUSSION

Soils

The bridge sites addressed in this report are all located within the southwestern portion of the Truckee Meadows. The Truckee Meadows form a topographic basin which separates the mountains of Carson Range to the west from the Virginia Range mountains located to the east. The basin generally drains to the North-Northeast toward the Truckee River. Existing references (4) denote the majority of mapped surface soils existing at the structure sites as unit Qa or "Alluvial bajada deposits". Qa soils are described as "Thin sheet-like aprons of fine to medium-grained clayey sand and intercalated muddy, medium pebble gravel". These soils were deposited during Holocene times and are the result of low gradient streams reworking the older and topographically higher gravelly outwash and alluvial fan deposits which flank the Carson Range. Some of the older (Pleistocene aged) soils are also mapped in fairly close proximity to the structure sites at the intersections at S. Meadows Parkway and S. Virginia Street. These older soils are denoted (4) as unit Qdm or the "Donner Lake Outwash--Mount Rose Fan Complex". Qdm soils are described as "Pediment and thin fan deposits" consisting of "brown to brownish-gray, sandy, muddy, poorly sorted large pebble gravel; with cobbles and small boulders common". These soils were deposited by major streams which drained the alpine glaciers on Mt. Rose during wetter climatic periods.

Regional Tectonics, Seismicity

The Truckee Meadows are located in a transitional zone between the Basin and Range Province and the Sierra Nevada Province. Existing references (2) show that four major structural tectonic features surround the project bridge sites. These features consist of:

1) The north-south trending Sierra Nevada frontal fault zone. Located less than 1 mile to the west of the project site.

2) The northwest trending Walker Lane Shear Zone located 30 to 35 miles to the northeast.

3) The east-west trending Olinghouse fault zone located approximately 10 to 15 miles to the north.

4) The northeast trending Carson Lineament located approximately 20 to 25 miles to the southeast.

According to Bell (2) at least 12 earthquakes with Richter Magnitude intensities of 7 or larger have occurred in the Reno Sheet Quadrangle within the last 12,000 years. Recent work (20) done by University of Nevada, Reno (UNR) suggests that the characteristic earthquake magnitude expected along the 2 largest and closest mapped Quaternary aged faults to the project site should range between 6.9 and 7.2. The UNR report also suggests that the maximum credible earthquake event which could occur along these two faults should fall between 7.0 and 7.5 in magnitude.

Maximum bedrock accelerations for the project area estimated at .4g with a 10% chance of exceeding this value in 100 years using a probabilistic method of analyses according to UNR's latest work

(20). This work also suggests that maximum bedrock accelerations calculated deterministically could be as high as .76g given that the maximum credible earthquake occurs on the nearest fault to the project site. Additionally, the Truckee Meadows is located in an area defined by the NHI's "Map of Horizontal Acceleration" (10) as having a maximum horizontal bedrock acceleration between .4g and .37g with a 10% chance of exceedence in 50 years.

All seismic design work completed for this report has assumed a maximum design earthquake of Richter magnitude 7.5 with a corresponding maximum horizontal bedrock acceleration of .4g. Vertical bedrock accelerations have been ignored.

Tectonic faults & Liquefaction Potential

Numerous mapped (21) north-northeast trending early to mid-Pleistocene aged faults are in very close proximity to the western edge of the project site. In fact, one of these faults (drawn as a queried concealed trace) may transect the South Meadows bridge site. However no evidence was uncovered during the site investigation to confirm this. Movement is estimated to have occurred along these fault traces greater than 100,000 years ago. The nearest mapped (21) Holocene aged fault trace to the project site is located approximately 4 miles southwest of the project along the eastern edge of the Carson Range near Whites Creek. Movement has been estimated to have occurred along this fault scarp within the last 3000 years.

According to Szecsody (21) the entire project alignment between South Meadows Parkway and South Virginia Street near Brown School is founded upon soils which may be moderately to severely susceptible to liquefaction during strong motion earthquakes.

SITE CONDITIONS

South Meadows Parkway, I-1952

Surface

The bridge site is placed upon a fairly level grassy meadow just south of the channelized Thomas Creek. The native ground slopes about 3% towards the north-northeast. The roadway grade for S. Meadows Parkway had already been constructed and paved. Ramp lines to the previously constructed portion of US 395 were also partially completed.

Subsurface

Both borings drilled at the site encountered approximately 21.5 feet of medium dense nonplastic micaceous silty sands and sandy silts from original ground elevations to approximately Elev. 4465′. Below this elevation both borings encountered denser silty sands and sandy silts which contained variable amounts of gravel and occasional thin interbeds and small pockets of clayey silt. These deeper soils were logged as dense in boring SMP-1 and dense to very dense in boring SMP-2 to approximately Elevation 4414′.

The groundwater surface was found to fluctuate between Elevs. 4484′ and 4488′ between November, 1992 and February, 1993.

Ground water levels are expected to fluctuate seasonally and may raise closer to the original ground surface during very wet years.

A very dense soil layer (N=115 blows) was logged from Elevs. 4490.6′ to 4485.1′ in boring SMP-2. Its unknown why this soil section is so dense as compared to the majority of the soils logged at the site. It's possible that this section may represent an old compacted roadway grade.

No evidence of faulting was found in the field during the site investigation. However as mentioned previously a questionable concealed fault trace has been mapped on existing references (4,21) which transects the bridge site. The exact location of this trace is not known. The age of the fault scarp is estimated to be greater than 100,000 years. Its potential for surface rupture is estimated to be relatively low (21).

Old Virginia Road, I-2009, I-2007, H-2008

Surface

At the time of investigation the proposed bridge abutment locations were occupied by grassy fenced pasture lands. These fields gently slope approximately 3% toward the northeast.

Subsurface

All five borings encountered 17 to 19 feet of loose to medium dense micaceous silty sands below original ground elevations.

The bottom contact of these upper soils was found to range from Elevs. 4520′ to 4517′. These soils generally were nonplastic to slightly plastic and contained variable amounts of gravel and organic material. Occasional thin interbeds and pockets of less silty cleaner sands and lean silty clays and clayey silts were also observed in drive samples taken from this soil unit. One boring (OVC-4) encountered a soft (N=2 blows) fine sandy clay from original ground elevation to approximately 8 feet below grade. However, this was the only boring to encounter a substantially thick cohesive surficial soil layer this soft in consistency. Typically, the surficial soils were more granular and exhibited uncorrected SPT blow counts greater than 5.

Beneath the loose to medium dense surficial soils each boring logged dense gravelly silty micaceous fine to medium grained sands. Soil samples from these dense soils usually exhibited substantial fines contents (>15% passing #200 sieve) and were generally at least slightly plastic (15 < PI > 3). Four of the five borings drilled at this site contained substantial (greater than 5 feet thick) very dense gravelly interbedded layers within this generally dense soil unit. The bottom contact elevation for this unit ranged from Elevs. 4493′ to 4484′.

The soils logged below the dense soil unit mentioned above were generally logged as very dense (N > 50 blows) silty sands. Generally, these soils were found to be less silty and gravelly than the overlying dense soils. Additionally, these very dense soils intermittently exhibited moderately to weakly cemented pockets and thin layers.

Groundwater measurements taken in borings drilled at this site remained fairly constant at Elev. 4521.1' during the time of field investigation. No evidence of faulting was discovered at any of these bridges sites during the field investigation or literature review for this project.

South Virginia Street, I-1950

Surface

This structure's eastern abutment is located on a fenced grassy field directly adjacent to existing US 395 just north of Brown School. The western abutment will be placed on a sparsely vegetated fenced field. The center pier support will be located on existing Virginia Street. Generally the native topography gently slopes north-northeast approximately 3%. The existing roadway fill at Virginia Street is at approximately Elev. 4550'.

Subsurface

Borings drilled for this structure encountered from 12 to 21 feet of loose to medium dense micaceous silty sands from the ground surface to approximately Elev. 4530'. Generally the fines content of these upper soils exceeded 15%. These soils were typically nonplastic to slightly plastic.

Similar soils were logged to completion depths in both borings. The majority of soils found from Elev. 4530′ to 4493′ were medium dense. Typically these medium dense soils were slightly more plastic than the overlying sands. However, both borings had substantial interbeds of non plastic dense to very dense gravelly sands.

Soils logged below Elev. 4493′ become dense to very dense in consistency. These deeper soils tend to have higher gravel contents than the overlying soils and occasionally contained rock fragments, small cobbles, and moderately cemented nodules and thin layers.

Ground water elevations measured in boring SV-1 remained stable at Elev. 4521.1′ during the field investigation. Measurements could not be taken in boring SV-2 due to caving problems. However, the elevation where caving occurred (4521.3′) in this boring was approximately at the suspected ground water elevation.

ENGINEERING ANALYSES & CONCLUSIONS

LIQUEFACTION ANALYSES

Soil liquefaction can be described as the sudden loss of soil strength due to dynamic cyclic loadings usually associated with major strong motion earthquakes. Liquefaction is a phenomena which primarily affects loose saturated cohesionless clean sands and silty sands. Because this project site is situated on soils which may be liquefiable (21) it was necessary to assess the liquefaction potential of the soils that will be used to support the bridge structures. Liquefaction analyses were performed using empirical methods based on historical liquefaction events, and in situ soil strength characteristics as measured by Standard Penetration Testing (11,12,14,15). A maximum horizontal bedrock acceleration value of .4g, and a 7.5 magnitude design earthquake were used in the analyses. These values are believed to be the most appropriate numbers to use for the project area considering the design life of the bridges. All calculations included corrections for fines content (% passing No. 200 sieve) and reflected the increased overburden pressures due to the earth fills to be placed at the site. Analyses results are presented for the individual bridge sites below:

South Meadows Parkway, I-1952

Analyses were conducted for this site using blow counts taken from boring SMP-1 (Elev. 4489.2') which contained the least dense soils. Calculations were done using a nonreduced .4g ground acceleration coefficient and a smaller value of .31g which has been reduced according to local site conditions (11,12,15). The following table summarizes the safety factors calculated for this bridge site:

	Safety Factor against Liquefaction			
	Abutment		**Center pier**	
Sample Elev.	**.4g**	**.31g**	**.4g**	**.31g**
4485.7′	.72	.93	.71	.92
4480.7′	1.02	1.31	.88	1.14
4475.1′	1.16	1.50	.96	1.25
4470.1′	1.09	1.40	.89	1.15
4465.1′	>1.9	>2.5	>1.6	2.06
4460.1′	1.19	1.54	.97	1.26
4455.1′	1.27	1.64	1.05	1.36
4450.1′	.73	.95	.62	.80
4445.1′	1.02	1.28	.86	1.12
4440.1′	.91	1.17	.79	1.02
4435.1′	.97	1.25	.85	1.09
4430.1′	>1.6	>2.1	>1.4	1.88
4425.1′	1.15	1.48	1.03	1.33
4415.1′	.87	1.13	.79	1.02

Results Summary

Calculations completed for the softest abutment soils indicate that for a 7.5 magnitude earthquake with a .4g maximum ground acceleration coefficient minor localized liquefaction may take place in medium dense to dense granular soils located below the water table (Elev. 4486′). For a .31g ground acceleration coefficient predicted soil liquefaction is even more limited.

Calculations completed for a .4g ground acceleration at the center pier location show that widespread liquefaction will occur in the saturated medium dense soils above Elev. 4565′. These calculations also indicate that saturated silty sands below Elev. 4455′ may also be subject to liquefaction. When the acceleration level is reduced to .31g only minor localized liquefaction occurs.

Old Virginia Road, I-2007, I-2009, H-2008

Blow counts from boring OVC-5 (Elev. 4533.8′) were used in this analyses. Calculations were done using a nonreduced .4g ground acceleration coefficient and a smaller value of .31g which has been reduced according to local site conditions. Two site design scenarios were considered. Case 1 assumed the water table to be at the original ground surface. Case 2 considered the ground water surface to be at Elev. 4521.1′ (12.7′ below the original ground elevation as measured during the site investigation). The following table summarizes the safety factors calculated for this bridge site:

Safety Factor against Liquefaction

	Case 1		Case 2	
Sample Elev.	**.4g**	**.31g**	**.4g**	**.31g**
4530.3′	.58	.75	---	---
4525.3′	.47	.61	---	---
4520.3′	.45	.59	.55	.72
4515.3′	.98	1.27	1.21	1.55
4510.3′	.55	.71	.68	.88
4505.3′	>1.7	>2.1	>2.0	>2.6

Results Summary

The analyses indicate that widespread liquefaction could occur beneath the abutment fills if the upper medium dense soils become saturated (Case 1) under either a .4g or .31g maximum horizontal ground acceleration coefficient. Under the ground water conditions revealed during the site investigation (Case 2) the analyses indicate that saturated medium dense soils above Elev.

4505′ may still be susceptible to liquefaction under either a .4g or .31g ground acceleration.

South Virginia Street, I-1950

Analyses were conducted using .4g and .31g ground acceleration coefficients as before. SPT blow counts taken in boring SV-2 (Elev 4549.7′) were used to simulate the softest soils logged at the site. Only the center pier location was analyzed with the ground water at Elev. 4521.1′ as revealed during the site investigation. No fill surcharges were considered. The table below summarizes the analytical results:

Safety Factor against Liquefaction

Sample Elev.	.4g	.31g
4521.1′	1.0	1.29
4516.2′	.87	1.12
4511.2′	1.21	1.47
4506.2′	.92	1.18
4501.7′	.80	1.04
4500.7′	.84	1.09
4496.2′	.77	1.00
4491.2′	>1.9	>2.4

Results Summary

The analyses indicate that localized liquefaction could occur in the center pier areas within medium dense silty sands below the ground water Elev. 4521.1′ to Elevation 4493′ during a magnitude 7.5 earthquake with a maximum ground acceleration coefficient equal to .4g. For a .reduced 31g seismic event the analyses indicate that center pier foundation soils should not liquify.

Final Conclusions And Discussion

As mentioned previously the results presented above are based upon an empirically derived analytical method which was originally developed for evaluating the liquefaction potential of clean sands under level ground conditions. Corrections have been applied in the method to account for native soil fines content and increases in total and effective earth pressures due to embankment surcharges. However, even with corrections applied in the method "it still is not possible to evaluate the likelihood of liquefaction of a silty sand with the same confidence as for a clean sand" (14).

Available empirical data concerning actual liquefaction of silty sands indicates no liquefaction has been documented for soils with fines contents of greater than 15% and corrected SPT blow counts of more than 25 blows (14). Case histories indicate that liquefaction phenomena is generally limited to 50 feet below the ground surface (15). Past experience (14) has suggested that the presence of an overlying nonliquefiable surface layer above liquefaction prone soils tends to prevent observable effects of at depth liquefaction from reaching the ground surface. Finally, in situ ground density improvements due to soil consolidation under the earth fills were ignored in the analyses. Consequently, the results presented above for this project site are clearly conservative. Given these facts the following conclusions have been made:

South Meadows Parkway, I-1952

Abutments

Liquefaction within the medium dense abutment foundation soils should be minimal at a .4g maximum ground acceleration. These soils were generally found to contain substantial fines (greater than 15%) and will be under 28 foot high nonliquefiable fills. In the worst case localized liquefaction should only occur between Elevs. 4486′ and 4467.2′ since soils lower than this will be over 50′ below the crest of the fills. Lateral load capacity of the abutment piles should not be substantially affected during the design seismic event as most of the loadings will be applied to the embankment fill soils.

Center pier

During a .31g ground acceleration localized liquefaction may take place between Elevs. 4486′ and 4483′. No liquefaction should take place below this elevation at this level of ground shaking.

During a .4g design event severe liquefaction is predicted by the analyses within the soils above Elev. 4465′. The analyses also indicates that widespread liquefaction may occur between Elevs. 4455′ and 4430′. But these soils are over 38 feet below the profile grade of South Meadows Parkway. Also, these soils typically contain over 35% fines and have corrected SPT blow counts greater than 25. Therefore liquefaction of these deep soil layers should not occur under design seismic events.

Generally, the fines content measured within the soil samples taken above Elev. 4465′ was over 15%. Corrected SPT blow counts exceeded 25 blows in all cases but one. Therefore severe liquefaction should not be a problem within this soil section. However, localized liquefaction will likely occur in the cleaner sand layers during a design seismic event within this soil section. Under the very worst conditions the lateral and vertical load capabilities of this upper soil section could be reduced. However, the bearing capacity of the soils below Elev. 4465′ should not be affected.

Old Virginia Road, I-2007, I-2009, H-2008

Localized liquefaction may occur in saturated medium dense to loose soils above Elev. 4505′. Soils below this will be located more than 50 feet below the crest of the roadway fills and should not be subject to liquefaction. Vertical and lateral load capability of these upper soils may be reduced during a design seismic event. However bearing capacity of the soils below this elevation should not be affected. Also, lateral loads should be adequately resisted by the reinforced earth fills and the nonsaturated native surficial soils.

South Virginia Street, I-1950

Abutments

Given the depth to groundwater (>28′) and the height of proposed earth fills (>28′) no liquefaction problems are anticipated at the abutments.

Center pier

Analyses indicate that saturated medium dense silty sands may liquefy between Elevs. 4521′ and 4500′. But these soils are very silty (% fines > 28%), slightly cohesive (PI>6), and will be overlain by at least 29′ of nonliquefiable soils. Therefore soil liquefaction should not be a problem at this site for the design seismic event.

SLOPE STABILITY

External slope stability was investigated using the computer program XSTABL™ and the subsurface soil conditions as revealed during the site investigations. Static and seismic design scenarios were analyzed. A maximum horizontal bedrock acceleration of .4g was assumed in the seismic analyses. A reduced value of .26g was also used. This level is approximately equal to the root mean squared average of the .4g maximum seismic event and is thought to be a more realistic representation of what the ground acceleration the embankments will see during the design seismic event. Vertical seismic accelerations were set at zero for all situations. Internal stability of the MSE walls was assumed to restrict any failure surfaces from occurring through the reinforced earth fills. The stabilizing effect of the pile foundations to be used for these structures was ignored. Slope deformations were calculated using Newmark's analytical method using yield accelerations determined by XSTABL™ and maximum horizontal embankment acceleration coefficients (Amaxe) of .4g and .6g.

Plots of the 10 most critical failure surfaces generated by XSTBL™ have been included in Appendix 3 for each structure except I-2007. Native soil conditions and embankment height proposed at this location are similar to those found at structures I-2009 and H-2008. Therefore, this site was not investigated individually. The table below summarizes the results of these analyses:

Safety Factor against Slope Failure				Estimated Downslope Movement @ A_{maxe}	
Structure	**STATIC**	**.26g**	**.4g**	**.4g**	**.6g**
I-1952	1.63	.92	.72	1"	6"
I-1950	1.55	.94	.76	1"	7"
H-2008	1.9	1.1	.91	0"	--
I-2009	1.96	1.19	.95	0"	--

Conclusions

Shallow surficial failures will occur within the embankment abutment fills under the design earthquake at the I-1950 and I-1952 structure sites. Downslope movement will be minor under design conditions at these locations. The MSE walls used to support the roadway at the Old Virginia Road crossings are expected to remain stable during the design earthquake event.

EMBANKMENT SETTLEMENT

Native soil settlement due to embankment loadings were calculated using Hough's Method (22) and specific soil properties correlated from in situ Standard Penetration Tests. Due to the granular nature of the native soils present at the bridge sites settlement should occur fairly quickly with the majority of consolidation taking place within one to three months. The table below summarizes the maximum predicted native soil settlement at the abutment centerline for each bridge site. Also shown is a calculated neutral settlement axis. Below this axis the remaining magnitude of total estimated ground settlement due to the earth fills should be less than 0.5 inches. Positive skin friction may be accumulated by piles which have been installed prior to abutment fill construction below this axis.

Structure	Maximum Settlement	Neutral Axis
I-1952	< 6 inches	n/a
I-1950	< 6 inches	n/a
I-2009	< 4 inches	Elev. 4502'
I-2007	< 4 inches	Elev. 4502'
H-2008	< 3 inches	Elev. 4502'

Conclusions:

The majority of settlement should take place beneath the embankment fills within 30 days. If embankments are allowed to settle prior to pile driving operations native soil settlement will not be a concern for axial pile designs. If piles are installed prior to fill construction negative skin friction will theoretically be applied to the pile above the calculated neutral axis. Also, piles founded above this axis may be subject to excessive settlements associated with the fill loads. Therefore if piles are installed prior to fill installation they should be designed to mitigate these possible problems.

FOUNDATION DESIGN

Foundation design for the bridges along this project is complicated by the high seismic potential of the Truckee Meadows area. Additionally, potentially liquefiable saturated soils underlay most of the various bridge sites. Finally, the surficial soils logged at the various bridge sites were loose to medium dense in consistency. Given these facts the Geotechnical Section, in conjunction with Bridge Division, decided to require pile foundations for these structures. Inherently piled foundations should provide several advantages over spread footings. First, they will be much less affected as compared to spread footings by surficial soil liquefaction and any associated fill slope failures or settlements that might be related to this phenomena. Second, piled foundations should improve the global slope stability of the abutment fills by providing an additional resisting force within the embankment and underlying native soils. In fact, the act of driving large displacement pipe piles may even improve the native soils resistance to liquefaction by dynamic soil densification. Finally, piled foundations can be designed to be nondependent on support from potentially liquefiable surficial soils.

Axial Pile Design

Axial pile design was completed using Nordland's method (22) in conjunction with soil properties correlated to in situ SPT results for closed ended 24" pipe piles. Specified pile tip elevations were established ignoring the capacity contribution of any potentially liquefiable soils. Pile tip elevations have also been placed below any soils judged to be susceptible to liquefaction during a design earthquake event. Native soil settlements due to embankment fill placements subsequent to pile driving operations were considered in the design analyses for the Old Virginia Road structures. The following table lists the results of this analyses for each structure. Design capacity and safety factors (S.F.) are given for seismic design conditions.

Structure	Max. Tip Elev.	Min. Tip Elev.	Design Capacity		F.S.
I-1952	4455′	4465′	Abutment	70 tons	>3.5
			Pier	70 tons	1.9
I-1950	4515′	4535′	Abutment	100 tons	>5.0
			Pier	100 tons	>3.0
I-2009					
I-2007	4483′	4493′	Abutment	100 tons	>3.4
H-2008					

Please note that pile designs for the Old Virginia Road structures have assumed that piles will be installed before the MSE abutment walls have been built. Also, analyses presume a friction reducing system will be applied to piles from the bottom of the MSE wall excavations to the bottom elevation of the pile caps. However, if the fills are constructed prior to pile driving, and a minimum 30 day waiting period is observed between fill construction and pile driving operations, the specified minimum pile tips may be raised to Elev. 4510′. In any case piles should be driven to the specified pile tip elevation unless required axial pile capacity is achieved as predicted by an appropriate Wave Equation Analyses below the specified minimum tip elevation.

Lateral Load Design

Lateral load capacity and deflection properties of the 24" pipe piles can be determined using the PY curves provided in Appendix 4 in conjunction with the software program COM624. These curves should be valid for all structures under assumed seismic design conditions. However, liquefaction analyses indicates that in the worst case substantial liquefaction may occur in the upper soils (above Elev. 4465′) at the center pier support for South Meadows

Parkway structure (I-1952). This could reduce the lateral load capacity of these soils substantially at the center pier supports. Therefore, in the worst case scenario the PY curves provided for the center pier support will not be valid. In this case, the bridge should be designed to resist seismic lateral loadings at the abutment foundation supports only.

Finally, localized liquefaction may occur beneath the abutment fills at the S. Meadows Parkway and the Old Virginia Road structures. This should not be a problem as the majority of lateral resistance against seismic loading will be applied against the nonliquefiable engineered fills and nonsaturated surficial soils at each site. The given PY curves for these structures should be valid under the project site seismic design conditions.

MSE WALL STABILITY

External stability of the MSE walls to be used at the Old Virginia Road structures was investigated using procedures established for nonextensible reinforced earth design (5). Safety factors concerning bearing capacity, sliding and overturning stability under static conditions and seismic design conditions (A_{max} = .4g) were investigated. Calculations indicate that to satisfy FHWA requirements the MSE walls should be designed with uniform, nonextensible reinforcing tendon lengths equal to at least 90% of the total abutment wall height for each structure (measured from the top of the leveling pad to the crest of the reinforced fills). Internal stability of the MSE walls is the responsibility of the NDOT approved patentee.

RECOMMENDATIONS

Seismic Design Conditions

Recommended seismic design criteria for this project is based upon AASHTO requirements. A maximum seismic horizontal ground acceleration coefficient of .4g with a Type II soil profile is recommended for design purposes for all the structures addressed in this report.

Bridge Foundations

Driven Pile Design, Axial Capacity

Twenty-four inch outside diameter, closed ended concrete filled pipe piles (PP24 x .5) have been specified for structural support of all the bridges addressed in this report. This pile type and size was agreed upon by the Geotechnical Section and Bridge Division during the latter portion of the site investigation phase of this project. At that time it was decided to require a uniform pile size for this project to expedite bridge designs and to minimize costs associated with several different foundation types. The table below outlines the recommended minimum and maximum specified pile tip elevations for the allowable pile design capacity for each structure support location.

Structure	Max. Tip Elev.	Min. Tip Elev.	Design Capacity	
I-1952	4455′	4465′	Abutment	70 tons
			Pier	70 tons
I-1950	4515′	4535′	Abutment	100 tons
			Pier	100 tons
I-2009 I-2007 H-2008	4483′	4493′	Abutment	100 tons

Little or no pile group settlement is expected upon application of the foundation loads. A design pullout capacity for piles driven to minimum tip elevation at I-1950 of 30 tons is recommended.

The above recommendations assume that a 30 day waiting period between final abutment fill construction and pile driving operations will be observed at structures I-1950 and I-1952. Recommendations for the Old Virginia Road structures have been made assuming that piles will be installed prior to MSE abutment wall construction. If fills are constructed prior to pile driving, and a 30 day waiting period between final fill construction and initial pile driving is observed, the specified minimum pile tip elevation for these structures may be raised to Elev. 4510′ given that sufficient "under the hammer" bearing capacity is reached according to the appropriate Wave Equation Analyses.

Pile load tests are recommended at the center pier foundations for structures I-1950 and I-1952. The tests should be conducted by NDOT personnel using a contractor supplied hydraulic load jack with a minimum capacity of 400 tons that has been load calibrated within one year of the test date by an AASHTO certified laboratory.

The next table shows the maximum predicted driving capacities pile driving systems should be able to overcome to seat piles to the specified minimum tip elevation recommended above for each structure. The driving capacities specified for the Old Virginia Road structures reflect two scenarios. The driving capacity given for pile tip Elev. 4493′ assume piles will be driven prior to MSE abutment construction. The value given for Elev. 4510′ assumes that the MSE fills have been constructed and a minimum 30 day waiting period has been observed between pile driving and fill construction.

Structure	**Min. Tip Elev.**	**Design Capacity**		**Max Driving Capacity**
I-1952	4465′	Abutment	70 tons	182 tons
		Pier	70 tons	403 tons
I-1950	4535′	Abutment	100 tons	539 tons
		Pier	100 tons	250 tons
I-2009				
I-2007	4493′		100 tons	458 tons
H-2008	4510′		100 tons	442 tons

These driving capacities have been calculated using static techniques. It′s anticipated that actual maximum driving capacities encountered during pile driving operations will be somewhat lower than predicted. However, field investigations indicate that dense to very dense gravelly zones may exist above the specified minimum pile tip elevation at the S. Meadows Parkway and Old Virginia Road structure sites. These zones could cause piles to meet refusal during driving prior to reaching the specified minimum pile tip elevation. Consequently, predrilling should be required at Abutment No. 1 for structure I-1952 to Elev. 4485′. This should allow the pipe piles to be driven through the very dense soil section logged between Elevs. 4491′

and 4485′. Predrilling to Elev. 4500′ should be required at the Old Virginia Road structures (I-2007, H-2008, I-2009) if piles are driven prior to MSE wall construction. If piles are driven 30 days subsequent to final MSE wall construction predrilling will not be necessary as the majority of native ground settlement due to fill surcharges will have already taken place.

Preliminary in-house Wave Equation Analyses indicates that maximum compressive stresses generated during pile driving may exceed 31.5 ksi at some of the bridge sites. These stress levels would exceed the recommended limit for ASTM A-252, Grade 2 steel. Therefore, in order to provide additional protection against pile damage during driving operations Bridge Division and the Geotechnical Section have agreed to recommend that the pipe piles used for these structures be constructed of ASTM A-252, Grade 3 steel. This will raise the maximum allowable pile compressive stress during driving to 41.5 ksi. Material cost increases due to this steel upgrade are expected to be minimal.

Driven Piles, Lateral Capacity

Any lateral load capacity design should use the PY curves provided in Appendix 4. These curves should be valid for design seismic conditions for all foundation support areas with the exception of the center pier location for I-1952. At this location, under the worst seismic conditions, significant reduction of the lateral load capacity of the upper soils may occur due to soil liquefaction during a strong motion earthquake.

Under these conditions it may be prudent to require all lateral load resistances to be achieved from the abutment foundations located within the nonliquefiable embankment fills.

Mechanically Stabilized Earth Fills

Construction of the MSE abutment fills will substantially affect the constructability of the pile foundations to be used to support the Old Virginia Road structures. Settlement calculations indicate that if piles are installed prior to MSE fill construction deeper minimum pile tip elevations will be necessary to balance anticipated negative skin friction forces and mitigate foundation settlement problems. If piles are installed 30 days subsequent to MSE fill construction settlement problems and negative skin friction forces should not be a concern. It appears that construction of the MSE fills prior to pile installation would be more attractive than the alternative. However this decision will be left to the Contractor. Should the Contractor decide to construct the MSE fills first the following recommendations are suggested:

1) Oversized 30 inch outside diameter cans should be placed at each pile location and brought up segmentally with the MSE fills as they are constructed.

2) These cans shall be constructed of smooth or corrugated galvanized steel pipe of sufficient thickness to prevent buckling during the placement and compaction of the MSE fills.

2) After a minimum 30 day waiting period the 24 inch pipe piles should be driven through the oversized cans to specified design capacity at or below Elev. 4510′.

3) Subsequent to pile driving the annular space between the

pile and oversized cans should be filled with sandy soils meeting NDOT's granular backfill requirements.

It should be noted that pile driving operations may be difficult to complete satisfactorily through the MSE fills. However, pile driving conducted through the oversized cans should still meet all required departmental specifications in Section 508 of NDOT's "Standard Specifications for Road and Bridge Construction".

Should the Contractor choose to drive piles prior to MSE fill construction the following recommendations are suggested.

1) A pile-soil friction reducing system should be required between the MSE fills and the 24" pipe piles. This system should be applied to the piles from the bottom of the MSE fill excavations to the bottom of the pile cap elevations.

2) A pile jacket slip layer such as the thermoplastic " Yellow Jacket Friction Protection System QC 2000" (manufactured by Queen Corporation, Lawrenceville, Georgia (Ph. 404-963-8970) is recommended for use as the pile-soil friction reducing system for these structures. These jackets are more safely and easily constructed than the typical bitumen slip coatings used on piles for friction reduction. Installation of these friction reducing systems should be done according to the manufacturer's recommendations.

Other recommendations concerning MSE fill construction are listed below:

1) MSE walls should be designed with uniform nonextensible tendon lengths equal to a minimum of 90% the total wall height (measured from the top of the leveling pad to the crest of the reinforced fills for external seismic stability.

2) Internal stability of the MSE walls is the responsibility of the NDOT approved patentee. However steps should be taken to insure that possible tendon corrosion problems associated with salt laden runoff coming into the MSE fills through the bridge deck expansion joints will not be a problem.

3) Soft cohesive sandy clays were encountered at the location of Abutment No.1 for structure H-2008 at the proposed base of the MSE fill. Upon excavation it may be necessary to stabilize the soft soils at this site prior to MSE wall construction. This may be achieved by working and compacting gravel into the native soils until a stable base is achieved for MSE wall construction. Alternatively the soft areas may be overexcavated and replaced with the same soils to be used for MSE fill construction compacted to 95% relative density as measured by AASHTO Test Method T-180.

2:1 Embankment Fills

Spread footings used to support wingwalls in these engineered fills may use an allowable design stress of 2 tsf if the following conditions are meet:

1) Footings are founded on compacted engineered fill soils (relative density at 95%) at least 2 footing widths above the native soils and the supporting fill soils.

2) Minimum earth cover over the footing top elevation is equal to at least 1.5 feet.

3) A minimum setback distance of 5 feet between the closest vertical footing edge and the face of the 2:1 slope is maintained.

Finally, a list of recommended foundation design parameters to be used for abutment design or retaining wall design within the 2:1 embankment fills has been included in Appendix 4.

Excavations

All excavations required for construction purposes (ie.. for center pier pile caps etc.) should be laid back or shored in accordance with current OSHA Excavation Standards as specified for Type C soils. These requirements may change based upon soil conditions exposed during construction. Any anticipated traffic or surcharge loadings placed in close proximity to these excavations should be accounted for in shoring designs.

REFERENCES

1) AASHTO, "Standard Specifications for Highway Bridges", Fifteenth Edition, Washington D.C., 1992.

2) Bell, J.W., "Quaternary Fault Map of Nevada--Reno Sheet", Map 79, Nevada Bureau of Mines and Geology, Reno, Nevada, 1984.

3) Bonham, H.F., "Geology and Mineral Deposits of Washoe and Storey Counties, Nevada", Nevada Bureau of Mines and Geology, Reno, Nevada, 1969.

4) Bonham, H.F. & Rogers D.K., "Mt. Rose NE Quadrangle Geologic Map", Map 4Bg, Nevada Bureau of Mines and Geology, Reno Nevada, 1983.

5) Christopher, B.R. et al, "Reinforced Soil Structures Volume I. Design and Construction Guidelines", Pub. No. FHWA-RD-89-043, U.S. Dept. of Transportation, FHWA, Mclean Virginia, 1990.

6) Cheny, R.S. & Chassie, R.G., "Soils and Foundation Workshop Manual", U.S. Dept. of Transportation, FHWA, Washington D.C., 1982.

7) Das, B.M., "Principles of Foundation Engineering", 2nd Edition, PWS-KENT Publishing Co., Boston, MA, 1990.

8) Das, B.M., "Principles of Geotechnical Engineering", 2nd Edition, PWS-KENT Publishing Co., Boston, MA, 1990.

9) Das, B.M., "Principles of Soil Dynamics", PWS-KENT Publishing Co., Boston, MA, 1993.

10) FHWA, "Seismic Design of Highway Bridges, Map of Horizontal Acceleration", Pub. No. FHWA-HI-91-019, Published 1988.

11) Imbsen & Associates, Inc., "Seismic Design of Highway Bridges Training Course Manual", U.S. DOT, FHWA, course given June, 1991.

12) Lam, I.P. & Martin G.R., "Seismic Design of Highway Bridge Foundations, Volume II Design Procedures and Guidelines", Report No. FHWA/RD-86/102, FHWA, June, 1986.

13) Kleinfelder--Reno office, "U.S. Highway 395 Embankment Study, South Meadows Parkway to Mt. Rose Highway, Washoe County, Nevada", Geotechnical Report submitted to NDOT on March 24, 1993.

REFERENCES (continued)

14) National Research Council, "Liquefaction of Soils During Earthquakes", National Academy Press, Washington D. C. 1985.

15) Dept of Navy, "NAVFAC Manuals DM-7.1, 7.2, 7.3", Naval Facilities Engineering Command, Alexandria, VA, published 1982 & 1983.

16) NDOT, "Foundation Reports- South Virginia Overpass & Zolezzi Lane Overpass, Reno Nevada" internal reports submitted in March 1993.

17) NDOT, "Soil Summary Report--US 395 Mays Lane to Mt. Rose Interchange, Reno Nevada", E.A. No. 71565-1, Internal report submitted in March, 1993.

18) OSHA Excavation Standards, Code of Federal Regulations--29, Chapter XVII (7-1-91 Edition), Office of the Federal Registrar, National Archives and Records Administration, Washington D.C., 1991.

19) Sergent, Hauskins & Beckwith, Inc., "Geotechnical Investigation-- Mt. Rose Interchange--NDOT Project I-1949", submitted to NDOT May, 1993.

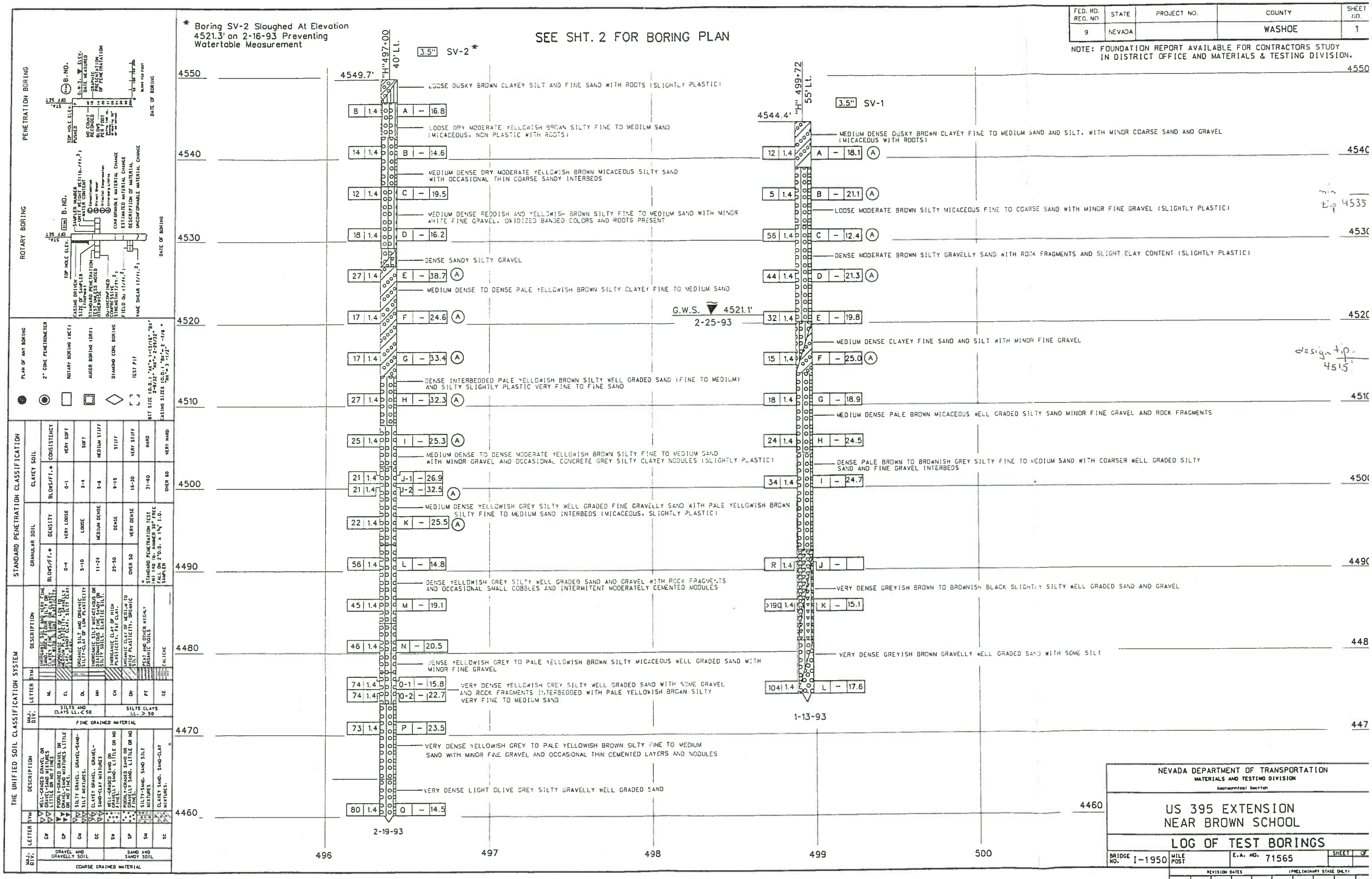
20) Siddharthan R. et al, "Peak Bedrock Acceleration for State of Nevada", Draft report submitted to NDOT, Univ. of Nevada, Reno, January, 1993.

21) Szecsody, G.C., "Mt. Rose NE Quadrangle Earthquake Hazards Map", Map 4Bi, Nevada Bureau of Mines and Geology, Reno, Nevada, 1983.

22) Vanikar,S.N., "Manual on Design and Construction of Driven Pile Foundations", Report No. FHWA-DP-66-1, U.S. Dept. of Transportation, FHWA, Washington D.C., 1986.

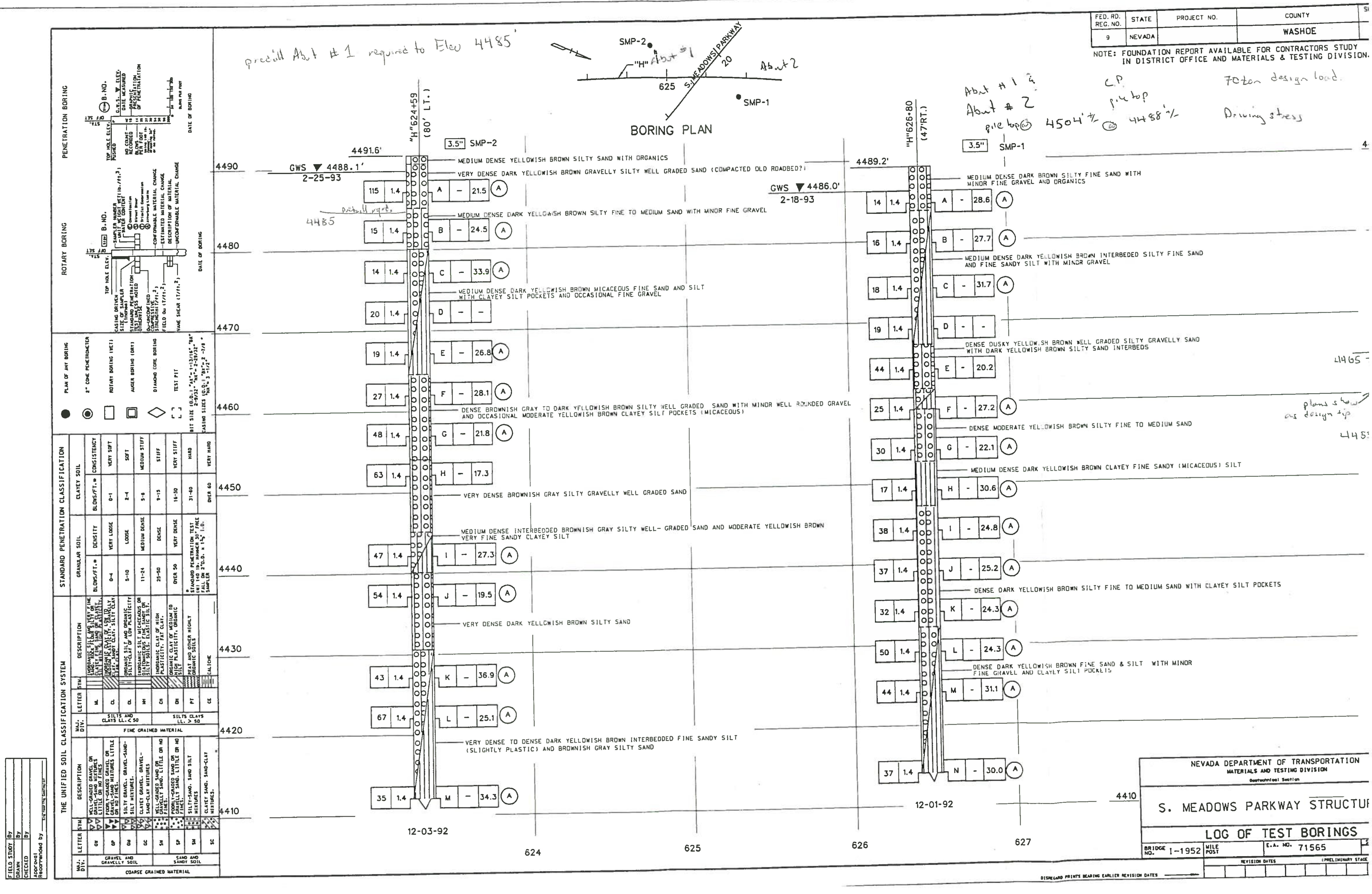
APPENDIX 1

Log of Exploration Borings and Location Map



FED. RD. REG. NO. 9
STATE NEVADA
PROJECT NO.
COUNTY WASHOE
SHEET NO. 1
NOTE: FOUNDATION REPORT AVAILABLE FOR CONTRACTORS STUDY IN DISTRICT OFFICE AND MATERIALS & TESTING DIVISION.
* Boring SV-2 Sloughed At Elevation 4521.3' on 2-16-93 Preventing Watertable Measurement
SEE SHT. 2 FOR BORING PLAN
3.5" SV-2 *
"H" 497+00 40' Lt.
4549.7'
LOOSE DUSKY BROWN CLAYEY SILT AND FINE SAND WITH ROOTS (SLIGHTLY PLASTIC)
LOOSE DRY MODERATE YELLOWISH BROWN SILTY FINE TO MEDIUM SAND (MICACEOUS, NON PLASTIC WITH ROOTS)
MEDIUM DENSE DRY MODERATE YELLOWISH BROWN MICACEOUS SILTY SAND WITH OCCASIONAL THIN COARSE SANDY INTERBEDS
MEDIUM DENSE REDDISH AND YELLOWISH BROWN SILTY FINE TO MEDIUM SAND WITH MINOR WHITE FINE GRAVEL, OXIDIZED BANDED COLORS AND ROOTS PRESENT
DENSE SANDY SILTY GRAVEL
MEDIUM DENSE TO DENSE PALE YELLOWISH BROWN SILTY CLAYEY FINE TO MEDIUM SAND
DENSE INTERBEDDED PALE YELLOWISH BROWN SILTY WELL GRADED SAND (FINE TO MEDIUM) AND SILTY SLIGHTLY PLASTIC VERY FINE TO FINE SAND
MEDIUM DENSE TO DENSE MODERATE YELLOWISH BROWN SILTY FINE TO MEDIUM SAND WITH MINOR GRAVEL AND OCCASIONAL CONCRETE GREY SILTY CLAYEY NODULES (SLIGHTLY PLASTIC)
MEDIUM DENSE YELLOWISH GREY SILTY WELL GRADED FINE GRAVELLY SAND WITH PALE YELLOWISH BROWN SILTY FINE TO MEDIUM SAND INTERBEDS (MICACEOUS, SLIGHTLY PLASTIC)
DENSE YELLOWISH GREY SILTY WELL GRADED SAND AND GRAVEL WITH ROCK FRAGMENTS AND OCCASIONAL SMALL COBBLES AND INTERMITENT MODERATELY CEMENTED NODULES
DENSE YELLOWISH GREY TO PALE YELLOWISH BROWN SILTY MICACEOUS WELL GRADED SAND WITH MINOR FINE GRAVEL
VERY DENSE YELLOWISH GREY SILTY WELL GRADED SAND WITH SOME GRAVEL AND ROCK FRAGMENTS INTERBEDDED WITH PALE YELLOWISH BROWN SILTY VERY FINE TO MEDIUM SAND
VERY DENSE YELLOWISH GREY TO PALE YELLOWISH BROWN SILTY FINE TO MEDIUM SAND WITH MINOR FINE GRAVEL AND OCCASIONAL THIN CEMENTED LAYERS AND NODULES
VERY DENSE LIGHT OLIVE GREY SILTY GRAVELLY WELL GRADED SAND
2-19-93
3.5" SV-1
"H" 499+72 55' Lt.
4544.4'
MEDIUM DENSE DUSKY BROWN CLAYEY FINE TO MEDIUM SAND AND SILT, WITH MINOR COARSE SAND AND GRAVEL (MICACEOUS WITH ROOTS)
LOOSE MODERATE BROWN SILTY MICACEOUS FINE TO COARSE SAND WITH MINOR FINE GRAVEL (SLIGHTLY PLASTIC)
DENSE MODERATE BROWN SILTY GRAVELLY SAND WITH ROCK FRAGMENTS AND SLIGHT CLAY CONTENT (SLIGHTLY PLASTIC)
G.W.S. 4521.1' 2-25-93
MEDIUM DENSE CLAYEY FINE SAND AND SILT WITH MINOR FINE GRAVEL
MEDIUM DENSE PALE BROWN MICACEOUS WELL GRADED SILTY SAND MINOR FINE GRAVEL AND ROCK FRAGMENTS
DENSE PALE BROWN TO BROWNISH GREY SILTY FINE TO MEDIUM SAND WITH COARSER WELL GRADED SILTY SAND AND FINE GRAVEL INTERBEDS
VERY DENSE GREYISH BROWN TO BROWNISH BLACK SLIGHTLY SILTY WELL GRADED SAND AND GRAVEL
VERY DENSE GREYISH BROWN GRAVELLY WELL GRADED SAND WITH SOME SILT
1-13-93
design tip 4515'
tip 4535
4550 4540 4530 4520 4510 4500 4490 4480 4470 4460
496 497 498 499 500
PENETRATION BORING
ROTARY BORING
STANDARD PENETRATION CLASSIFICATION
THE UNIFIED SOIL CLASSIFICATION SYSTEM
NEVADA DEPARTMENT OF TRANSPORTATION
MATERIALS AND TESTING DIVISION
Geotechnical Section
US 395 EXTENSION
NEAR BROWN SCHOOL
LOG OF TEST BORINGS
BRIDGE NO. I-1950
E.A. NO. 71565

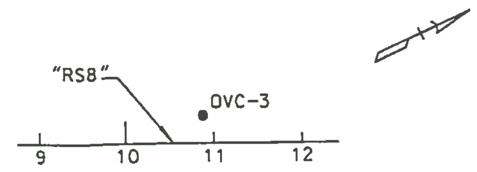
sv.dgn 4-15-93



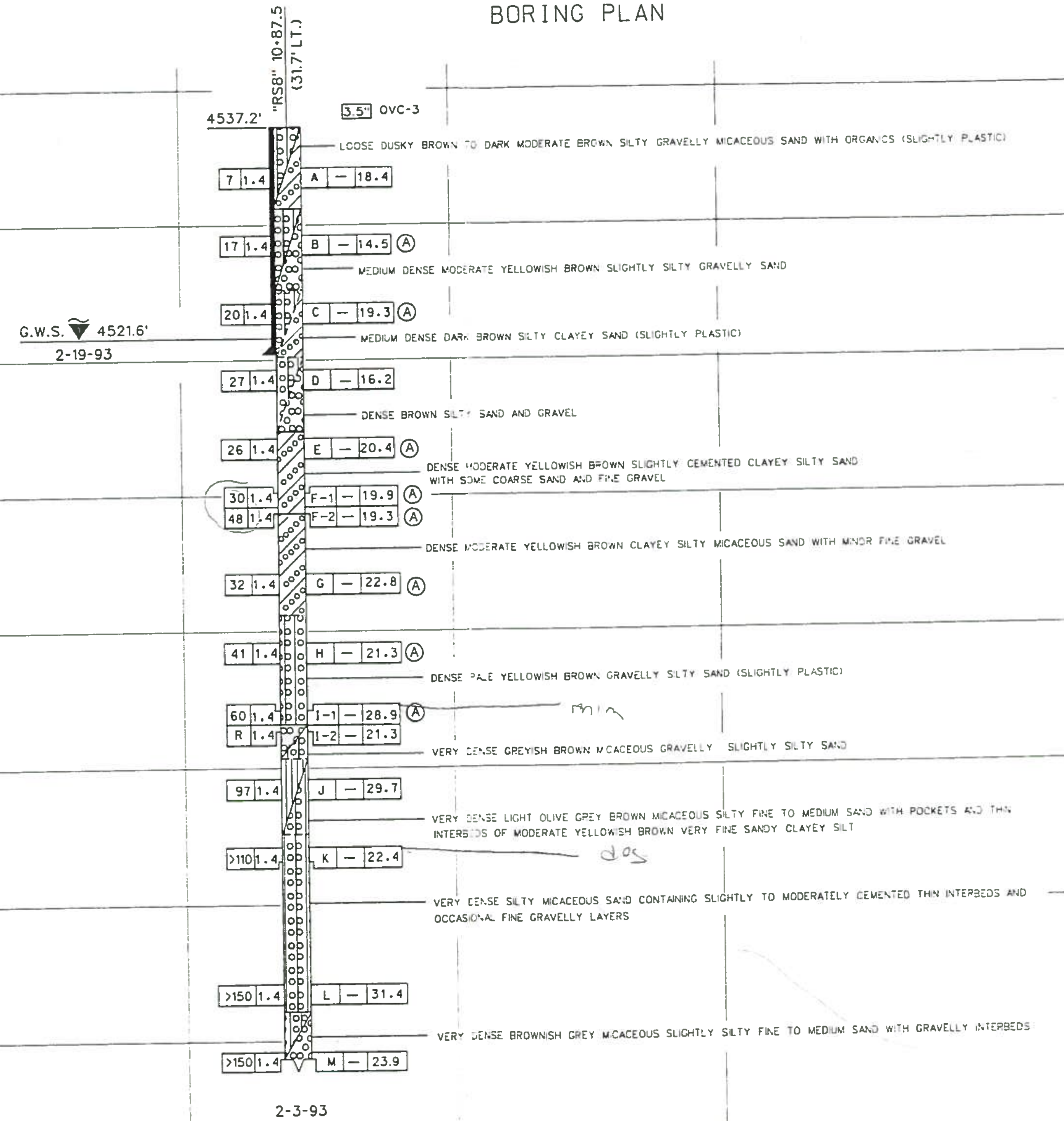
FED. RD. REG. NO. 9
STATE NEVADA
PROJECT NO.
COUNTY WASHOE
NOTE: FOUNDATION REPORT AVAILABLE FOR CONTRACTORS STUDY IN DISTRICT OFFICE AND MATERIALS & TESTING DIVISION.
predrill Abut #1 required to Elev 4485'
BORING PLAN
SMP-2
SMP-1
"H" Abut #1
S. MEADOWS PARKWAY
Abut 2
625
20
Abut #1 & Abut #2
C.P. pile top
pile top@ 4504'± @ 4488'±
70 ton design load.
Driving stress
"H" 624+59 (80' LT.)
3.5" SMP-2
4491.6'
GWS ▼ 4488.1' 2-25-93
MEDIUM DENSE YELLOWISH BROWN SILTY SAND WITH ORGANICS
VERY DENSE DARK YELLOWISH BROWN GRAVELLY SILTY WELL GRADED SAND (COMPACTED OLD ROADBED?)
predrill reqd.
4485
MEDIUM DENSE DARK YELLOWISH BROWN SILTY FINE TO MEDIUM SAND WITH MINOR FINE GRAVEL
MEDIUM DENSE DARK YELLOWISH BROWN MICACEOUS FINE SAND AND SILT WITH CLAYEY SILT POCKETS AND OCCASIONAL FINE GRAVEL
DENSE BROWNISH GRAY TO DARK YELLOWISH BROWN SILTY WELL GRADED SAND WITH MINOR WELL ROUNDED GRAVEL AND OCCASIONAL MODERATE YELLOWISH BROWN CLAYEY SILT POCKETS (MICACEOUS)
VERY DENSE BROWNISH GRAY SILTY GRAVELLY WELL GRADED SAND
MEDIUM DENSE INTERBEDDED BROWNISH GRAY SILTY WELL- GRADED SAND AND MODERATE YELLOWISH BROWN VERY FINE SANDY CLAYEY SILT
VERY DENSE DARK YELLOWISH BROWN SILTY SAND
VERY DENSE TO DENSE DARK YELLOWISH BROWN INTERBEDDED FINE SANDY SILT (SLIGHTLY PLASTIC) AND BROWNISH GRAY SILTY SAND
115 1.4 A – 21.5 (A)
15 1.4 B – 24.5 (A)
14 1.4 C – 33.9 (A)
20 1.4 D – –
19 1.4 E – 26.8 (A)
27 1.4 F – 28.1 (A)
48 1.4 G – 21.8 (A)
63 1.4 H – 17.3
47 1.4 I – 27.3 (A)
54 1.4 J – 19.5 (A)
43 1.4 K – 36.9 (A)
67 1.4 L – 25.1 (A)
35 1.4 M – 34.3 (A)
12-03-92
"H" 626+80 (47' RT.)
3.5" SMP-1
4489.2'
GWS ▼ 4486.0' 2-18-93
MEDIUM DENSE DARK BROWN SILTY FINE SAND WITH MINOR FINE GRAVEL AND ORGANICS
MEDIUM DENSE DARK YELLOWISH BROWN INTERBEDDED SILTY FINE SAND AND FINE SANDY SILT WITH MINOR GRAVEL
DENSE DUSKY YELLOWISH BROWN WELL GRADED SILTY GRAVELLY SAND WITH DARK YELLOWISH BROWN SILTY SAND INTERBEDS
DENSE MODERATE YELLOWISH BROWN SILTY FINE TO MEDIUM SAND
MEDIUM DENSE DARK YELLOWISH BROWN CLAYEY FINE SANDY (MICACEOUS) SILT
DENSE DARK YELLOWISH BROWN SILTY FINE TO MEDIUM SAND WITH CLAYEY SILT POCKETS
DENSE DARK YELLOWISH BROWN FINE SAND & SILT WITH MINOR FINE GRAVEL AND CLAYEY SILT POCKETS
14 1.4 A - 28.6 (A)
16 1.4 B - 27.7 (A)
18 1.4 C - 31.7 (A)
19 1.4 D - -
44 1.4 E - 20.2
25 1.4 F - 27.2 (A)
30 1.4 G - 22.1 (A)
17 1.4 H - 30.6 (A)
38 1.4 I - 24.8 (A)
37 1.4 J - 25.2 (A)
32 1.4 K - 24.3 (A)
50 1.4 L - 24.3 (A)
44 1.4 M - 31.1 (A)
37 1.4 N - 30.0 (A)
12-01-92
4465
plans show as design tip
4453
4490
4480
4470
4460
4450
4440
4430
4420
4410
624
625
626
627
PENETRATION BORING
ROTARY BORING
STANDARD PENETRATION CLASSIFICATION
THE UNIFIED SOIL CLASSIFICATION SYSTEM
GRANULAR SOIL
CLAYEY SOIL
FINE GRAINED MATERIAL
COARSE GRAINED MATERIAL
FIELD STUDY BY
DRAWN BY
CHECKED BY
Approved Recommended by
NEVADA DEPARTMENT OF TRANSPORTATION
MATERIALS AND TESTING DIVISION
Geotechnical Section
S. MEADOWS PARKWAY STRUCTUR
LOG OF TEST BORINGS
BRIDGE NO. I-1952
MILE POST
E.A. NO. 71565
REVISION DATES
PRELIMINARY STAGE
DISREGARD PRINTS BEARING EARLIER REVISION DATES

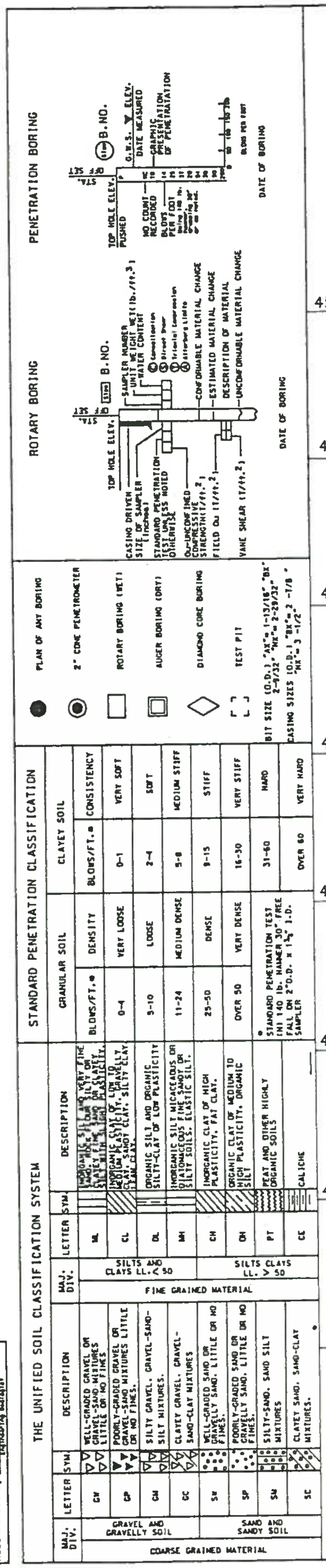
FED. RD. REG. NO.	STATE	PROJECT NO.	COUNTY
9	NEVADA		WASHOE

NOTE: FOUNDATION REPORT AVAILABLE FOR CONTRACTORS STUDY IN DISTRICT OFFICE AND MATERIALS & TESTING DIVISION



BORING PLAN





FIELD STUDY	By
DRAWN	By
CHECKED	By
Approval Recommended by	

NEVADA DEPARTMENT OF TRANSPORTATION
MATERIALS AND TESTING DIVISION
Geotechnical Section

US 395 EXTENSION
NEAR BROWN SCHOOL

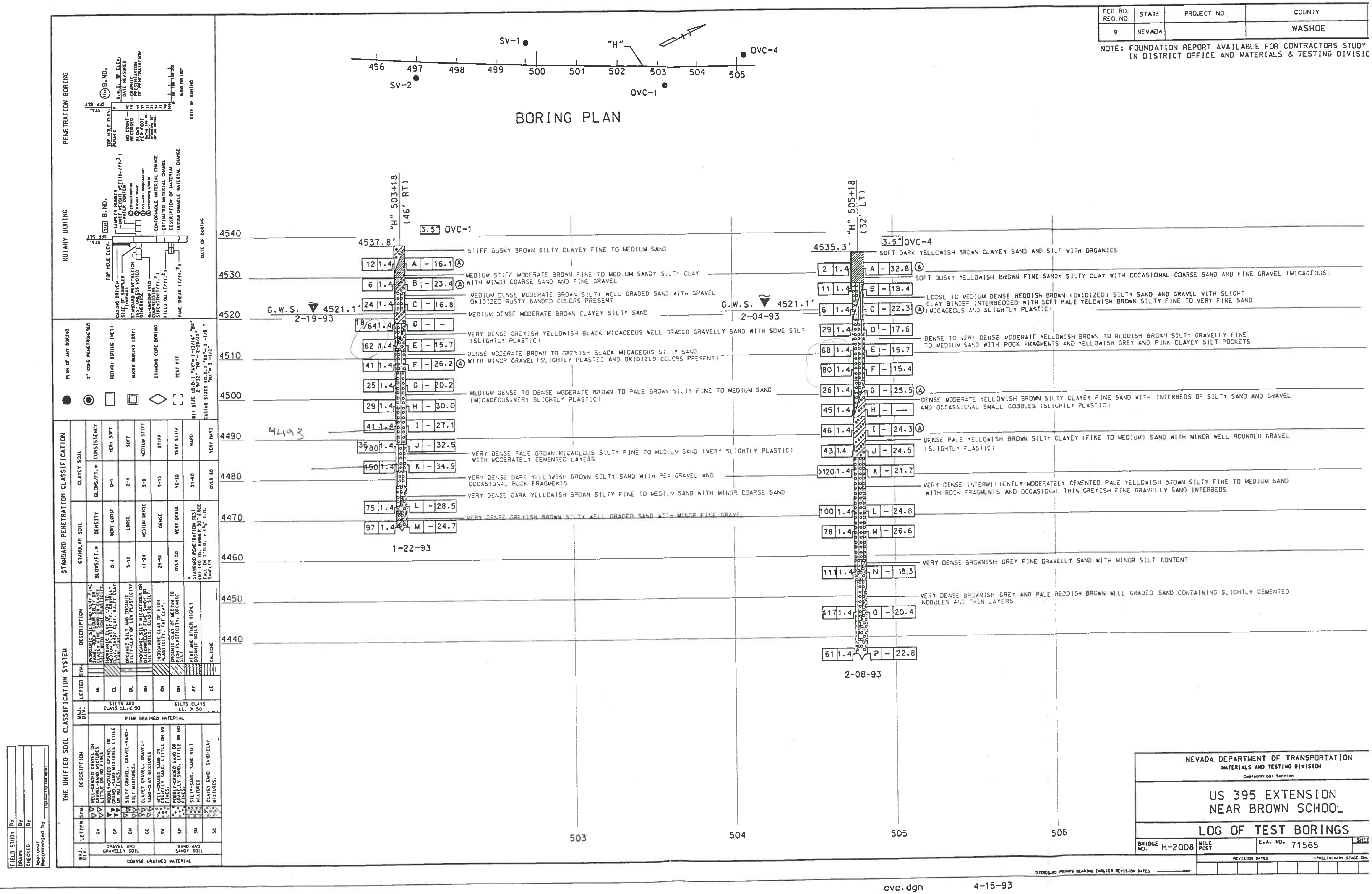
LOG OF TEST BORINGS

BRIDGE NO. I-2007 | MILE POST | E.A. NO. 71565 | SHEET

REVISION DATES | PRELIMINARY STAGE ONLY

DISREGARD PRINTS BEARING EARLIER REVISION DATES

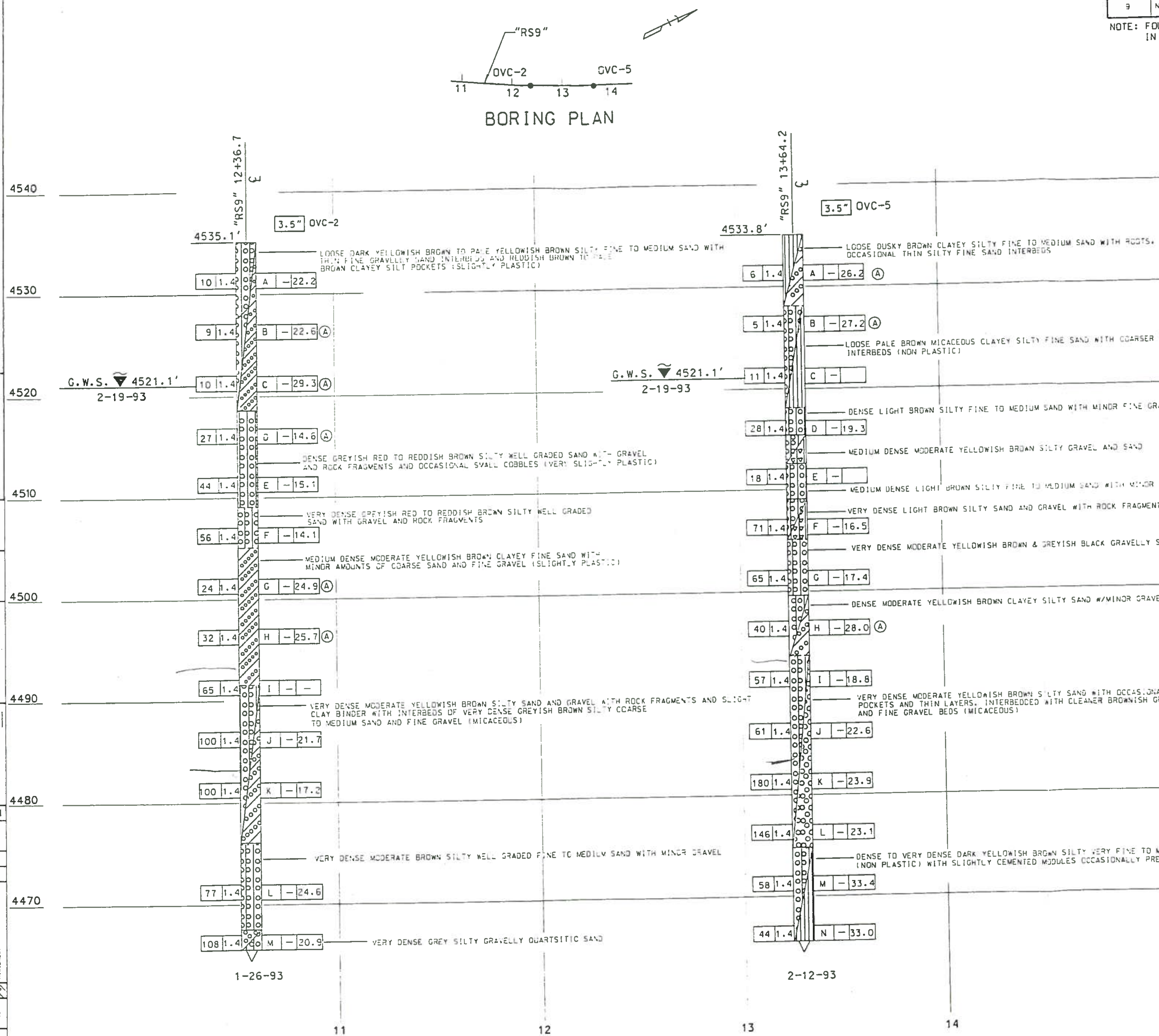
ovc3.dgn 4-09-93



FED. RD. REG. NO. 9
STATE NEVADA
PROJECT NO.
COUNTY WASHOE
NOTE: FOUNDATION REPORT AVAILABLE FOR CONTRACTORS STUDY IN DISTRICT OFFICE AND MATERIALS & TESTING DIVISION
BORING PLAN
SV-1
SV-2
"H"
OVC-1
OVC-4
496 497 498 499 500 501 502 503 504 505
"H" 503+18 (46' RT)
3.5" OVC-1
4537.8'
G.W.S. 4521.1' 2-19-93
STIFF DUSKY BROWN SILTY CLAYEY FINE TO MEDIUM SAND
MEDIUM STIFF MODERATE BROWN FINE TO MEDIUM SANDY SILTY CLAY WITH MINOR COARSE SAND AND FINE GRAVEL
MEDIUM DENSE MODERATE BROWN SILTY WELL GRADED SAND WITH GRAVEL OXIDIZED RUSTY BANDED COLORS PRESENT
MEDIUM DENSE MODERATE BROWN CLAYEY SILTY SAND
VERY DENSE GREYISH YELLOWISH BLACK MICACEOUS WELL GRADED GRAVELLY SAND WITH SOME SILT (SLIGHTLY PLASTIC)
DENSE MODERATE BROWN TO GREYISH BLACK MICACEOUS SILTY SAND WITH MINOR GRAVEL (SLIGHTLY PLASTIC AND OXIDIZED COLORS PRESENT)
MEDIUM DENSE TO DENSE MODERATE BROWN TO PALE BROWN SILTY FINE TO MEDIUM SAND (MICACEOUS, VERY SLIGHTLY PLASTIC)
VERY DENSE PALE BROWN MICACEOUS SILTY FINE TO MEDIUM SAND (VERY SLIGHTLY PLASTIC) WITH MODERATELY CEMENTED LAYERS
VERY DENSE DARK YELLOWISH BROWN SILTY SAND WITH PEA GRAVEL AND OCCASIONAL ROCK FRAGMENTS
VERY DENSE DARK YELLOWISH BROWN SILTY FINE TO MEDIUM SAND WITH MINOR COARSE SAND
VERY DENSE GREYISH BROWN SILTY WELL GRADED SAND WITH MINOR FINE GRAVEL
12 1.4 A - 16.1
6 1.4 B - 23.4
24 1.4 C - 16.8
18/64 1.4 D - -
62 1.4 E - 15.7
41 1.4 F - 26.2
25 1.4 G - 20.2
29 1.4 H - 30.0
41 1.4 I - 27.1
36/80 1.4 J - 32.5
150 1.4 K - 34.9
75 1.4 L - 28.5
97 1.4 M - 24.7
1-22-93
4493
"H" 505+18 (32' LT)
3.5" OVC-4
4535.3'
G.W.S. 4521.1' 2-04-93
SOFT DARK YELLOWISH BROWN CLAYEY SAND AND SILT WITH ORGANICS
SOFT DUSKY YELLOWISH BROWN FINE SANDY SILTY CLAY WITH OCCASIONAL COARSE SAND AND FINE GRAVEL (MICACEOUS)
LOOSE TO MEDIUM DENSE REDDISH BROWN (OXIDIZED) SILTY SAND AND GRAVEL WITH SLIGHT CLAY BINDER INTERBEDDED WITH SOFT PALE YELLOWISH BROWN SILTY FINE TO VERY FINE SAND (MICACEOUS AND SLIGHTLY PLASTIC)
DENSE TO VERY DENSE MODERATE YELLOWISH BROWN TO REDDISH BROWN SILTY GRAVELLY FINE TO MEDIUM SAND WITH ROCK FRAGMENTS AND YELLOWISH GREY AND PINK CLAYEY SILT POCKETS
DENSE MODERATE YELLOWISH BROWN SILTY CLAYEY FINE SAND WITH INTERBEDS OF SILTY SAND AND GRAVEL AND OCCASIONAL SMALL COBBLES (SLIGHTLY PLASTIC)
DENSE PALE YELLOWISH BROWN SILTY CLAYEY (FINE TO MEDIUM) SAND WITH MINOR WELL ROUNDED GRAVEL (SLIGHTLY PLASTIC)
VERY DENSE INTERMITTENTLY MODERATELY CEMENTED PALE YELLOWISH BROWN SILTY FINE TO MEDIUM SAND WITH ROCK FRAGMENTS AND OCCASIONAL THIN GREYISH FINE GRAVELLY SAND INTERBEDS
VERY DENSE BROWNISH GREY FINE GRAVELLY SAND WITH MINOR SILT CONTENT
VERY DENSE BROWNISH GREY AND PALE REDDISH BROWN WELL GRADED SAND CONTAINING SLIGHTLY CEMENTED NODULES AND THIN LAYERS
2 1.4 A - 32.8
11 1.4 B - 18.4
6 1.4 C - 22.3
29 1.4 D - 17.6
68 1.4 E - 15.7
80 1.4 F - 15.4
26 1.4 G - 25.5
45 1.4 H - —
46 1.4 I - 24.3
43 1.4 J - 24.5
>120 1.4 K - 21.7
100 1.4 L - 24.8
78 1.4 M - 26.6
111 1.4 N - 18.3
117 1.4 O - 20.4
61 1.4 P - 22.8
2-08-93
4540 4530 4520 4510 4500 4490 4480 4470 4460 4450 4440
503 504 505 506
PENETRATION BORING
ROTARY BORING
B. NO.
TOP HOLE ELEV.
DATE OF BORING
PLAN OF ANY BORING
2" CONE PENETROMETER
ROTARY BORING (WET)
AUGER BORING (DRY)
DIAMOND CORE BORING
TEST PIT
STANDARD PENETRATION CLASSIFICATION
GRANULAR SOIL
CLAYEY SOIL
THE UNIFIED SOIL CLASSIFICATION SYSTEM
FINE GRAINED MATERIAL
COARSE GRAINED MATERIAL
FIELD STUDY By
DRAWN By
CHECKED By
Approval Recommended by
NEVADA DEPARTMENT OF TRANSPORTATION
MATERIALS AND TESTING DIVISION
Geotechnical Section
US 395 EXTENSION
NEAR BROWN SCHOOL
LOG OF TEST BORINGS
BRIDGE NO. H-2008
MILE POST
E.A. NO. 71565
REVISION DATES
ovc.dgn
4-15-93

FED. RD. REG. NO.	STATE	PROJECT NO.	COUNTY
9	NEVADA		WASHOE

NOTE: FOUNDATION REPORT AVAILABLE FOR CONTRACTORS STUDY IN DISTRICT OFFICE AND MATERIALS & TESTING DIVISION.



Boring	Sample	Blows	Size	Moisture
OVC-2	A	10	1.4	22.2
OVC-2	B	9	1.4	22.6 (A)
OVC-2	C	10	1.4	29.3 (A)
OVC-2	D	27	1.4	14.6 (A)
OVC-2	E	44	1.4	15.1
OVC-2	F	56	1.4	14.1
OVC-2	G	24	1.4	24.9 (A)
OVC-2	H	32	1.4	25.7 (A)
OVC-2	I	65	1.4	–
OVC-2	J	100	1.4	21.7
OVC-2	K	100	1.4	17.2
OVC-2	L	77	1.4	24.6
OVC-2	M	108	1.4	20.9
OVC-5	A	6	1.4	26.2 (A)
OVC-5	B	5	1.4	27.2 (A)
OVC-5	C	11	1.4	
OVC-5	D	28	1.4	19.3
OVC-5	E	18	1.4	
OVC-5	F	71	1.4	16.5
OVC-5	G	65	1.4	17.4
OVC-5	H	40	1.4	28.0 (A)
OVC-5	I	57	1.4	18.8
OVC-5	J	61	1.4	22.6
OVC-5	K	180	1.4	23.9
OVC-5	L	146	1.4	23.1
OVC-5	M	58	1.4	33.4
OVC-5	N	44	1.4	33.0

PENETRATION BORING
ROTARY BORING
STANDARD PENETRATION CLASSIFICATION
THE UNIFIED SOIL CLASSIFICATION SYSTEM

FIELD STUDY By
DRAWN By
CHECKED By
Approval Recommended by

NEVADA DEPARTMENT OF TRANSPORTATION
MATERIALS AND TESTING DIVISION
Geotechnical Section

US 395 EXTENSION
NEAR BROWN SCHOOL

LOG OF TEST BORINGS

BRIDGE NO. I-2009 | MILE POST | E.A. NO. 71565

REVISION DATES

DISREGARD PRINTS BEARING EARLIER REVISION DATES

ovc2.dgn 4-15-93

APPENDIX 2

Summary of Test Results Sheets

E.A. No. 71565-1; South Meadows Parkway
Structure I-1952

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL SECTION
SUMMARY OF TEST RESULTS

Boring No.: SMP-1

Total Depth (ft) : 75.1

Station or Location : "H" 626+80 47' Right

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	∅	C	Qu	
a	3.0 - 4.5	1.4 SS	14	SM			28.6	17	NP	24					
b	8.0 - 9.5	1.4 SS	16	SM/ML			27.7	41	NP	23					
c	13.6 - 15.1	1.4 SS	18	ML			31.7	60	NP	24					
d	18.6 - 20.1	1.4 SS	19	SM/ML											Chemical Analyses
e	23.6 - 25.1	1.4 SS	44	SM			20.2	15							
f	28.6 - 30.1	1.4 SS	25	SM/ML			27.2	50	1	25					
g	33.6 - 35.1	1.4 SS	30	SM			22.1	40	NP	23					
h	38.6 - 40.1	1.4 SS	17	ML			30.6	69	3	28					
i	43.6 - 45.1	1.4 SS	38	SM			24.8	32	NP	18					
j	48.6 - 50.1	1.4 SS	37	SM			25.2	36	NP	19					
k	53.6 - 55.1	1.4 SS	32	SM			24.3	35	NP	21					
l	58.6 - 60.1	1.4 SS	50	ML/SM			24.3	55	NP	23					
m	63.6 - 65.1	1.4 SS	44	ML/SM			31.1	55	NP	25					
n	73.6 - 75.1	1.4 SS	37	SM/ML			30.0	64	NP	27					

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

C = Cohesion - TSF
∅ = 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"

F\SUMMARY

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL SECTION
SUMMARY OF TEST RESULTS

E.A. No. 71565-1
South Meadows Parkway, Structure I-1952

Boring No.: SMP-2

Total Depth (ft) : 79.5

Station or Location : "H" 624+59 80' LF

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	∅	C	Qu	
a	3.0 - 4.5	1.4" SS	115	SM			21.5	18	NP	24					
b	8.0 - 4.5	1.4" SS	15	SM			24.5	30	NP	21					
c	13.0 - 14.5	1.4" SS	14	ML			33.9	59	NP	24					
d	18.0 - 19.5	1.4" SS	20	ML/SM											Chem Analyses
e	23.0 - 24.5	1.4" SS	19	SM/ML			26.8	44	NP	24					
f	28.0 - 29.5	1.4" SS	27	SM			28.1	29	NP	21					
g	33.0 - 34.5	1.4" SS	48	SM			21.8	11	NP	19					
h	38.0 - 39.5	1.4" SS	63	SM			17.3	10	--	--					
i	48.0 - 49.5	1.4" SS	47	SM			27.3	37	NP	24					
j	53.0 - 54.5	1.4" SS	54	SM			19.5	24	NP	19					
k	63.0 - 64.5	1.4" SS	43	ML			36.9	72	1	30					
l	68.0 - 69.5	1.4" SS	67	SM/ML			25.1	47	NP	24					
m	78.0 - 79.5	1.4" SS	35	ML/SM			34.3	60	NP	29					

NOTATION:
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CU = Consolidated Undrained
UC = Unconfined Compression
S = Direct Shear

C = Cohesion - TSF
∅ = Angle of Internal Friction - Degrees
Qu = Unconfined Compressive Strength - TSF
PI = Plasticity Index
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ES = Expansive Soils
GS = Specific Gravity
OC = Consolidation
SH = Shelby
SS = Split Spoon 1.4"

F\SUMMARY

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL SECTION
SUMMARY OF TEST RESULTS

E.A. No. **71565**
U.S. 395 Extension Near Brown School
Structure # H-2008
Boring No.: **OVC-1**

Total Depth (ft) :**69.5**

Station or Location :**"H" 503+18**
46' RIGHT

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	∅	Cu	Qu	
A	3.0- 4.5	SS	12	SC/CL			16.1	47	9	29					
B	8.0- 9.5	SS	6	SC/CL			23.4	51	14	34					
C	13.0- 14.5	SS	24	SM			16.8	17							
D	18.0- 19.5	SS	18/64	SC											
E	23.0- 24.5	SS	62	SM/SW			15.7	10							
F	28.0- 29.5	SS	41	SM			20.2	28	3	26					
G	33.0- 34.5	SS	25	SM			26.2	30							
H	38.0- 39.5	SS	29	SM			30.0	30							
I	43.0- 44.5	SS	41	SM			27.1	37							
J	48.0- 49.5	SS	36/80	SM			32.5	31							CHEM
K	53.0- 54.0	SS	150	SM			34.9	37							
L	63.0- 64.5	SS	75	SM/SW			28.5	12							
M	68.0- 69.5	SS	97	SM/SW			24.7	10							

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

*Shelby Tube subsample depths shown in inches.

Cu = Undrained Cohesion - TSF
∅ = Angle of Internal Friction - Degrees
Qu = Unconfined Compressive Strength - TSF
PI = Plasticity Index
LL = Liquid Limit
SH = Shelby Tube
SS = Split Spoon 1.4"
DB = Diamond Core Barrel

ES = Expansive Soils
GS = Specific Gravity
OC = Consolidation
T = Triaxial Compression
Su = Undrained Shear Strength-Tsf
H = Hydrometer
P = Pushed under weight of hammer and drill stem.
R = Refusal

E.A. No. **71565**
U.S. 395 Extension Near Brown School
Structure # I-2009
Boring No.: **OVC-2**

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL SECTION
SUMMARY OF TEST RESULTS

Total Depth (ft) :**69.5**

Station or Location :**"RS9" 12+36.7**
CENTERLINE

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	Cu	Qu	
A	3.0- 4.5	SS	10	SM/SC			22.2	38							CHEM
B	8.0- 9.5	SS	9	SM/SC			22.6	27	8	31					
C	13.0- 14.5	SS	10	SM/SC			29.3	35	5	27					
D	18.0- 19.5	SS	27	SM			14.6	21	3	22					
E	23.0- 24.5	SS	44	SM			15.1	15							
F	28.0- 29.5	SS	56	SM			14.1	13							
G	33.0- 34.5	SS	24	SC			24.9	40	14	34					
H	38.0- 39.5	SS	32	SC			25.7	42	14	35					
I	43.0- 44.5	SS	65	SM/SC											
J	48.0- 49.5	SS	100	SM/SC			21.7	10							
K	53.0- 54.0	SS	100	SM/SC			17.2	9							
L	63.0- 64.5	SS	77	SM			24.6	15							
M	68.0- 69.5	SS	108	SM/SW			20.9	7							

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

*Shelby Tube subsample depths shown in inches.

Cu = Undrained Cohesion - TSF
ø = Angle of Internal Friction - Degrees
Qu = Unconfined Compressive Strength - TSF
PI = Plasticity Index
LL = Liquid Limit
SH = Shelby Tube
SS = Split Spoon 1.4"
DB = Diamond Core Barrel

ES = Expansive Soils
GS = Specific Gravity
OC = Consolidation
T = Triaxial Compression
Su = Undrained Shear Strength-Tsf
H = Hydrometer
P = Pushed under weight of hammer and drill stem.
R = Refusal

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL SECTION
SUMMARY OF TEST RESULTS

E.A. No. **71565**
U.S. 395 Extension Near Brown School
Structure # I-2007
Boring No.: **OVC-3**

Total Depth (ft) :**68.5**

Station or Location :**"RS8" 10+87.5**
31.7' LEFT

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	Cu	Qu	
A	3.0- 4.5	SS	7	SM/SC			18.4	9							
B	8.0- 9.5	SS	17	SM/SW			14.5	6	NP	20					
C	13.0- 14.5	SS	20	SM/SC			19.3	35	5	25					
D	18.0- 19.5	SS	27	SM/SW			16.2	11							
E	23.0- 24.5	SS	26	SC			20.4	33	11	29					
F-1	28.0- 28.5	SS	30	SC			19.9		10	27					
F-2	28.5- 29.5	SS	48	SC			19.3	27	14	30					
G	33.0- 34.5	SS	32	SC			22.8	35	14	33					
H	38.0- 39.5	SS	41	SM			21.3	27	NP	24					
I-1	43.0- 44.0	SS	60	SM			28.9	32	NP	26					
I-2	44.0- 44.5	SS	R	SW/SM			21.3	9							
J	48.0- 49.5	SS	97	ML/SM			29.7	21							
K	53.0- 54.0	SS	>110	SM			22.4	11							
L	58.0- 58.5	SS	>150	SM			31.4	16							
M	68.0- 68.5	SS	>150	SM/SW			23.9	7							

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

*Shelby Tube subsample depths shown in inches.

Cu = Undrained Cohesion - TSF
ø = Angle of Internal Friction - Degrees
Qu = Unconfined Compressive Strength - TSF
PI = Plasticity Index
LL = Liquid Limit
SH = Shelby Tube
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ES = Expansive Soils
GS = Specific Gravity
OC = Consolidation
T = Triaxial Compression
Su = Undrained Shear Strength-Tsf
H = Hydrometer
P = Pushed under weight of hammer and drill stem.
R = Refusal

E.A. No. **71565**
U.S. 395 Extension Near Brown School
Structure # H-2008
Boring No.: **OVC-4**

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL SECTION
SUMMARY OF TEST RESULTS

Total Depth (ft) :**99.5**

Station or Location :**"H" 505+18**
32' LEFT

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	Cu	Qu	
A	3.0 - 4.5	SS	2	CL			32.8	65	14	36					
B	8.0 - 9.5	SS	11	SM/ML			18.4	13							
C	13.0 - 14.5	SS	6	SM/ML			22.3	23	NP	23					
D	18.0 - 19.5	SS	29	SM			17.6	10							
E	23.0 - 24.5	SS	68	SM			15.7	18							
F	28.0 - 29.5	SS	80	SM			15.4	15							CHEM
G	33.0 - 34.5	SS	26	SM/SC			25.5	43	6	27					
H	38.0 - 39.5	SS	45	SM/SC											
I	43.0 - 44.5	SS	46	SC			24.3	30	10	31					
J	48.0 - 49.5	SS	43	SC			24.5	15							
K	53.0 - 53.5	SS	>120	SM			21.7	16							
L	58.0 - 59.5	SS	100	SM			24.8	16							
M	68.0 - 69.5	SS	78	SM			26.6	22							
N	78.0 - 79.5	SS	111	SM/SW			18.3	8							
O	88.0 - 89.5	SS	117	SM/SW			20.4	6							
P	98.0 - 99.5	SS	61	SM/SW			22.8	11							

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CU = Consolidated Undrained
UC = Unconfined Compression
S = Direct Shear

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Qu = Unconfined Compressive Strength - TSF
PI = Plasticity Index
LL = Liquid Limit
SH = Shelby Tube
SS = Split Spoon 1.4"
DB = Diamond Core Barrel

ES = Expansive Soils
GS = Specific Gravity
OC = Consolidation
T = Triaxial Compression
Su = Undrained Shear Strength-Tsf
H = Hydrometer
P = Pushed under weight of hammer and drill stem.
R = Refusal

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL SECTION
SUMMARY OF TEST RESULTS

E.A. No. **71565**
U.S. 395 Extension Near Brown School
Structure # I-1950
Boring No.: **SV-1**

Total Depth (ft) :**69.5**

Station or Location :**"H" 499+72**
55' LEFT

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: Test Type	ø	Cu	Qu	Other Tests Performed
A	3.0- 4.5	SS	12	SC			18.1	41	9	29					
B	8.0- 9.5	SS	5	SM			21.1	26	4	24					
C	13.0- 14.5	SS	56	SM			12.4	18	4	24					
D	18.0- 19.5	SS	44	SM			21.3	24	NP	30					
E	23.0- 24.5	SS	32	SM			19.8	11							
F	28.0- 29.5	SS	15	SM/SC			25.0		6	27					
G	33.0- 34.5	SS	18	SM			18.9	19							
H	38.0- 39.5	SS	24	SM			24.5	24							
I	43.0- 44.5	SS	34	SM			24.7	20							
J	53.0- 53.5	SS	R	GW/GM											
K	58.0- 58.5	SS	>190	GW/GM			15.1	5							
L	68.0- 69.5	SS	104	SM/SW			17.6	7							

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UC = Unconfined Compression
S = Direct Shear

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Su = Undrained Shear Strength-Tsf
H = Hydrometer
P = Pushed under weight of hammer and drill stem.
R = Refusal

→

E.A. No. **71565**
U.S. 395 Extension Near Brown School
Structure # I-1950
Boring No.: **SV-2**

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL SECTION
SUMMARY OF TEST RESULTS

Total Depth (ft) :**89.5**

Station or Location :**"H" 497+00**
40' LEFT

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 Test Type	ø	Cu	Qu	Other Tests Performed
A	3.0- 4.5	SS	8	SM			16.8	44							CHEM
B	8.0- 9.5	SS	14	SM			14.6	31							
C	13.0- 14.5	SS	12	SM			19.5	14							
D	18.0- 19.5	SS	18	SM			16.2	25							
E	23.0- 24.5	SS	27	SC			38.7		25	45					
F	28.0- 29.5	SS	17	SC			24.6	42	16	34					
G	33.0- 34.5	SS	17	SC/CL			33.4	48	14	37					
H	38.0- 39.5	SS	27	SM			32.3	35	7	35					
I	43.0- 44.5	SS	25	SM			25.3	31	14	41					
J-1	48.0- 49.0	SS	21	SM			26.9	28							
J-2	49.0- 49.5	SS	21	SM			32.5	45	19	46					
K	53.0- 54.5	SS	22	SM			25.5	32	20	46					
L	58.0- 59.5	SS	56	SM			14.8	16							
M	63.0- 64.5	SS	45	SM			19.1	12							
N	68.0- 69.5	SS	46	SM			20.5	14							
O-1	73.0- 74.0	SS	74	SM			15.8	13							
O-2	74.0- 74.5	SS	74	SM			22.7	31							
P	78.0- 79.5	SS	73	SM			23.5	18							
Q	88.0- 89.5	SS	80	SM			14.5	13							

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CU = Consolidated Undrained
UC = Unconfined Compression
S = Direct Shear

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ø = Angle of Internal Friction - Degrees
Qu = Unconfined Compressive Strength - TSF
PI = Plasticity Index
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SH = Shelby Tube
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ES = Expansive Soils
GS = Specific Gravity
OC = Consolidation
T = Triaxial Compression
Su = Undrained Shear Strength-Tsf
H = Hydrometer
P = Pushed under weight of hammer and drill stem.
R = Refusal

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL SECTION

CHEMICAL ANALYSES TEST RESULTS

PROJECT SOUTH MEADOWS PARKWAY, 395 EXT. E. A. NO. 71565

SAMPLE NO.	SMP-1 D
CHEM NO.	C-212-93
DATE	12-04-92
CHLORIDES PPM.	70
SULFATES PPM.	0
PH	7.7
RESISITIVITY OHM / CM	10,309

SAMPLE NO.	SMP-2 D
CHEM NO.	C-232-93
DATE	12-10-92
CHLORIDES PPM.	40
SULFATES PPM.	0
PH	7.1
RESISITIVITY OHM / CM	13,514

PH2 PP☺
11-16-92

NEVADA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL SECTION

CHEMICAL ANALYSES TEST RESULTS

PROJECT U.S. 395 FREEWAY EXT. E. A. NO. 71565

SAMPLE NO.	OVC-1 J
CHEM NO.	C-253-93
DATE	1-28-93
CHLORIDES PPM.	30
SULFATES PPM.	0
PH	7.2
RESISITIVITY OHM / CM	12,048

SAMPLE NO.	OVC-2 A
CHEM NO.	C-278-93
DATE	1-28-93
CHLORIDES PPM.	40
SULFATES PPM.	0
PH	8.3
RESISITIVITY OHM / CM	11,628

SAMPLE NO.	OVC-4 F
CHEM NO.	C-290-93
DATE	2-11-93
CHLORIDES PPM.	30
SULFATES PPM.	0
PH	7.5
RESISITIVITY OHM / CM	9,434

SAMPLE NO.	OVC-5 G
CHEM NO.	C-291-93
DATE	2-18-93
CHLORIDES PPM.	60
SULFATES PPM.	TRACE
PH	7.3
RESISITIVITY OHM / CM	11,494

SAMPLE NO.	SV-2 A
CHEM NO.	C-293-93
DATE	2-22-93
CHLORIDES PPM.	120
SULFATES PPM.	0
PH	6.8
RESISITIVITY OHM / CM	10,417

STATE OF NEVADA
DEPARTMENT OF TRANSPORTATION

MEMORANDUM

March 10, 1993, 19

To Steve Oxoby, Chief Road Design Engineer

From Dean Weitzel, Assistant Chief Materials Engineer DCW

Subject: Depth Checks - US 395 Freeway, from Mays Ln. to Mt. Rose Interchange; E.A. NO. 71565.

The attached Soil Summary is forwarded for your information.

DW:DM
Attach.
cc: Mark Salazar
Rod Johnson
Gary Anderson
John Bradshaw
Todd Montgomery
Jay Van Sickle
District 2 Eng.
Resident Eng.

MATERIALS DIVISION
SOIL SUMMARY

E.A. NO.: 71565
PROJECT: US 395, MAYS LN TO MT ROSE INT.
COUNTY: WASHOE DATE SAMPLED: MAY AND NOV 1992

	STATION	LOCATION FROM CL	DEPTH	R VALUE	LL	PI	%GRVL	%SAND	%SAND & SILT	SOLUBLE SALTS IN PPM CL	SO4	Ph	RESIST.
'H'	623 +29	24' LT	0'– 5'	71	28	5	7	68	25	150	0	6.9	2584
	623 +29	48' RT	0'– 5'	75	29	1	2	65	33	90	0	7.0	3663
	620 +00	48' RT	0'– 5'	69	22	NP	17	65	18	80	0	7.3	4237
	617 +00	48' LT	0'– 5'		31	10	3	57	40				
	615 +00	24' RT	0'– 5'		39	12	1	54	45				
	613 +00	24' LT	0'– 5'	38	40	17	3	54	43	50	0	6.4	2793
	610 +00	48' RT	0'– 5'		71	14	0	29	71				
	608 +00	48' LT	0'– 5'		52	5	0	39	61				
	605 +00	30' RT	0'– 5'		25	5	5	62	33				
	603 +00	24' LT	0'– 5'		26	8	20	52	28				
	600 +00	48' RT	0'– 5'		25	2	2	70	28				
	598 +00	48' LT	0'– 5'		29	9	2	62	36				
	595 +00	24' RT	0'– 5'	50	32	12	2	63	35	70	120	6.9	2740
	593 +00	24' LT	0'– 5'		28	9	0	61	39				
	590 +00	48' RT	0'– 5'		27	7	1	61	38				
	588 +00	48' LT	0'– 5'	19	27	6	1	61	38	40	0	6.1	5780
	585 +00	24' RT	0'– 5'		26	4	2	66	32				
	583 +00	24' LT	0'– 5'		30	9	3	60	37				
	580 +00	60' RT	0'– 5'		30	6	2	59	39				
	578 +00	48' LT	0'– 5'		27	8	0	62	38				
	575 +00	24' RT	0'– 5'		30	19	1	61	38				
	573 +00	24' LT	0'– 5'		26	2	0	62	38				
	570 +00	48' RT	0'– 5'	66	24	2	2	68	30	70	210	6.6	3096
	568 +00	48' LT	0'– 5'		25	3	1	64	35				
	565 +00	24' RT	0'– 5'		21	1	3	74	23				
	563 +00	24' LT	0'– 5'	14	23	4	4	58	38	50	0	7.0	4202
	560 +00	48' RT	0'– 5'		24	2	0	58	42				
	558 +00	48' LT	0'– 5'		24	3	3	69	28				
	555 +00	24' RT	0'– 5'		25	5	1	65	34				
	553 +00	24' LT	0'– 5'		24	4	1	56	43				
	550 +00	39' RT	0'– 5'		29	8	1	56	43				
	548 +00	48' LT	0'– 5'		32	8	1	48	51				

MATERIALS DIVISION
SOIL SUMMARY

E.A. NO.: 71565
PROJECT: US 395, MAYS LN TO MT ROSE INT.
COUNTY: WASHOE DATE SAMPLED: MAY AND NOV 1992

	STATION	LOCATION FROM CL	DEPTH	R VALUE	LL	PI	%GRVL	%SAND	%SAND & SILT	SOLUBLE SALTS IN PPM CL	SO4	Ph	RESIST.
'H'	545 +00	24' RT	0'– 5'	53	23	2	3	65	32	120	0	6.9	4049
	543 +00	24' LT	0'– 5'		29	5	0	51	49				
	540 +00	48' RT	0'– 5'		22	NP	3	64	33				
	538 +00	48' LT	0'– 5'	34	23	2	2	70	28	80	0	6.7	6061
	535 +00	24' RT	0'– 5'		28	6	3	59	38				
	533 +00	24' LT	0'– 5'		29	6	1	58	41				
	510 +00	48' RT	0'– 5'		31	6	1	61	38				
	508 +00	48' LT	0'– 5'		28	6	1	62	37				
	505 +00	24' RT	0'– 5'		27	4	2	53	45				
	503 +00	24' LT	0'– 5'		25	4	2	64	34				
	500 +00	48' RT	0'– 5'	60	26	6	4	68	28	70	0	5.5	12821
	498 +00	50' LT	0'– 5'		24	3	5	66	29				
	495 +00	24' RT	0'– 5'	75	25	3	4	70	26	60	120	5.7	6711
	493 +00	24' LT	0'– 5'		24	2	11	62	27				
	490 +00	48' RT	0'– 5'		24	2	1	77	22				
'P2A'	489 +00	48' LT	0'– 5'	77	25	NP	29	55	16	50	0	5.6	8621
	486 +00	24' RT	0'– 5'		23	4	23	57	20				
	484 +00	24' LT	0'– 5'		19	1	10	68	22				
	481 +00	48' RT	0'– 5'		20	2	10	68	22				
	479 +00	48 LT	0'– 5'		21	1	32	54	14				
	476 +00	24' RT	0'– 5'	70	22	3	14	66	20	50	0	5.8	10101
	474 +00	24' LT	0'– 5'		22	5	9	68	23				
	471 +00	48' RT	0'– 5'		27	11	33	48	19				
	469 +00	48' LT	0'– 5'		20	2	9	69	22				
	466 +00	24' RT	0'– 5'		25	10	8	69	23				
	464 +00	24' LT	0'– 5'	18	24	6	3	69	28	40	0	6.6	6289
	461 +00	48' RT	0'– 5'		24	4	4	66	30				
	459 +00	48' LT	0'– 5'		26	12	7	62	31				
	456 +00	24' RT	0'– 5'		21	5	3	68	29				
	454 +00	24' LT	0'– 5'		24	8	12	62	26				
	451 +00	48' RT	0'– 5'	30	25	4	14	57	29	970	0	6.8	772
	451 +00	48' LT	0'– 5'	70	23	3	3	70	27	6100	0	6.8	150

MATERIALS DIVISION
SOIL SUMMARY

E.A. NO.: 71565

PROJECT: US 395, MAYS LN TO MT ROSE INT.

COUNTY: WASHOE DATE SAMPLED: MAY AND NOV 1992

	STATION	LOCATION FROM CL	DEPTH	R VALUE	LL	PI	%GRVL	%SAND	%SAND & SILT	SOLUBLE SALTS IN PPM CL	SO4	Ph	RESIST.
'RD1'	12 +10	5' LT	0'– 5'	49	24	NP	9	64	27	80	0	8.3	3937
	14 +00	ON CL	0'– 5'	71	25	NP	14	65	21	80	0	7.9	1919
'RD3'	4 +00	ON CL	0'– 5'	29	40	8	1	53	46	60	0	8.1	4444
'RD4'	12 +00	ON CL	0'– 5'	24	26	5	10	61	29	80	0	7.9	3921
	12 +75	ON CL	0'– 5'	51	27	5	6	64	30	50	0	7.8	3731
'Z'	11 +00	ON CL	0'– 5'	50	26	5	1	69	30	50	0	7.5	10869
	15 +00	ON CL	0'– 5'	14	23	3	1	71	28	30	0	7.7	11111
	20 +00	ON CL	0'– 5'	24	26	3	2	67	31	70	0	7.3	2591
	25 +00	ON CL	0'– 5'	19	29	8	1	67	32	70	0	7.5	2513
	30 +00	ON CL	0'– 5'	7	22	NP	3	70	27	80	0	8.6	2066
	35 +00	ON CL	0'– 5'	43	26	4	1	61	38	70	0	8.4	2762
	40 +00	ON CL	0'– 5'	14	28	8	2	61	37	60	0	8.5	2506
'RS8'	0 +00	ON CL	0'– 5'	50	24	NP	11	68	21	220	0	7.7	1931
	5 +50	ON CL	0'– 5'	65	26	4	1	65	34	60	0	6.8	14286
	9 +50	ON CL	0'– 5'	39	24	NP	3	72	25	90	0	7.2	4651
'RS8A'	0 +30	ON CL	0'– 5'	74	22	4	34	52	14	100	0	8.0	3413
'RS9'	27 +00	ON CL	0'– 5'	25	21	NP	14	69	17	60	0	8.1	3876
	31 +00	ON CL	0'– 5'	32	22	NP	18	63	19	170	0	7.9	2141
'RS10'	0 +00	ON CL	0'– 5'	58	25	NP	26	56	18	50	0	7.6	3876
	5 +00	12' LT	0'– 5'	30	23	NP	19	57	24	30	0	7.8	3077
	10 +00	17' RT	0'– 5'	65	26	5	1	72	27	40	0	7.2	4717
	15 +00	ON CL	0'– 5'	63	22	NP	5	70	25	40	0	7.0	7407
'W'	77 +12	5' LT OE	0'– 5'	5	20	NP	10	70	20	60	0	7.7	8000
	82 +12	5' LT OE	0'– 5'	67	17	NP	7	79	14	70	0	8.2	9259
	87 +12	5' LT OE	0'– 5'	57	21	NP	7	69	24	100	0	7.8	4082
	92 +12	5' LT OE	0'– 5'	20	22	NP	7	72	21	100	0	8.8	2421
	97 +12	5' LT OE	0'– 5'	37	32	4	12	58	30	150	0	8.5	2427
	102 +12	5' LT OE	0'– 5'	68	33	NP	20	49	31	260	0	7.7	1548
	107 +12	5' LT OE	0'– 5'	74	21	NP	19	65	16	90	0	7.6	4608
	112 +12	5' LT OE	0'– 5'	14	51	NP	14	35	51	1100	0	7.5	571
	117 +12	5' LT OE	0'– 5'	30	22	NP	8	65	27	230	0	7.8	1623
	123 +12	5' LT OE	0'– 5'	75	21	NP	23	59	18	100	0	8.4	2933

MATERIALS DIVISION
SOIL SUMMARY

E.A. NO.: 71565
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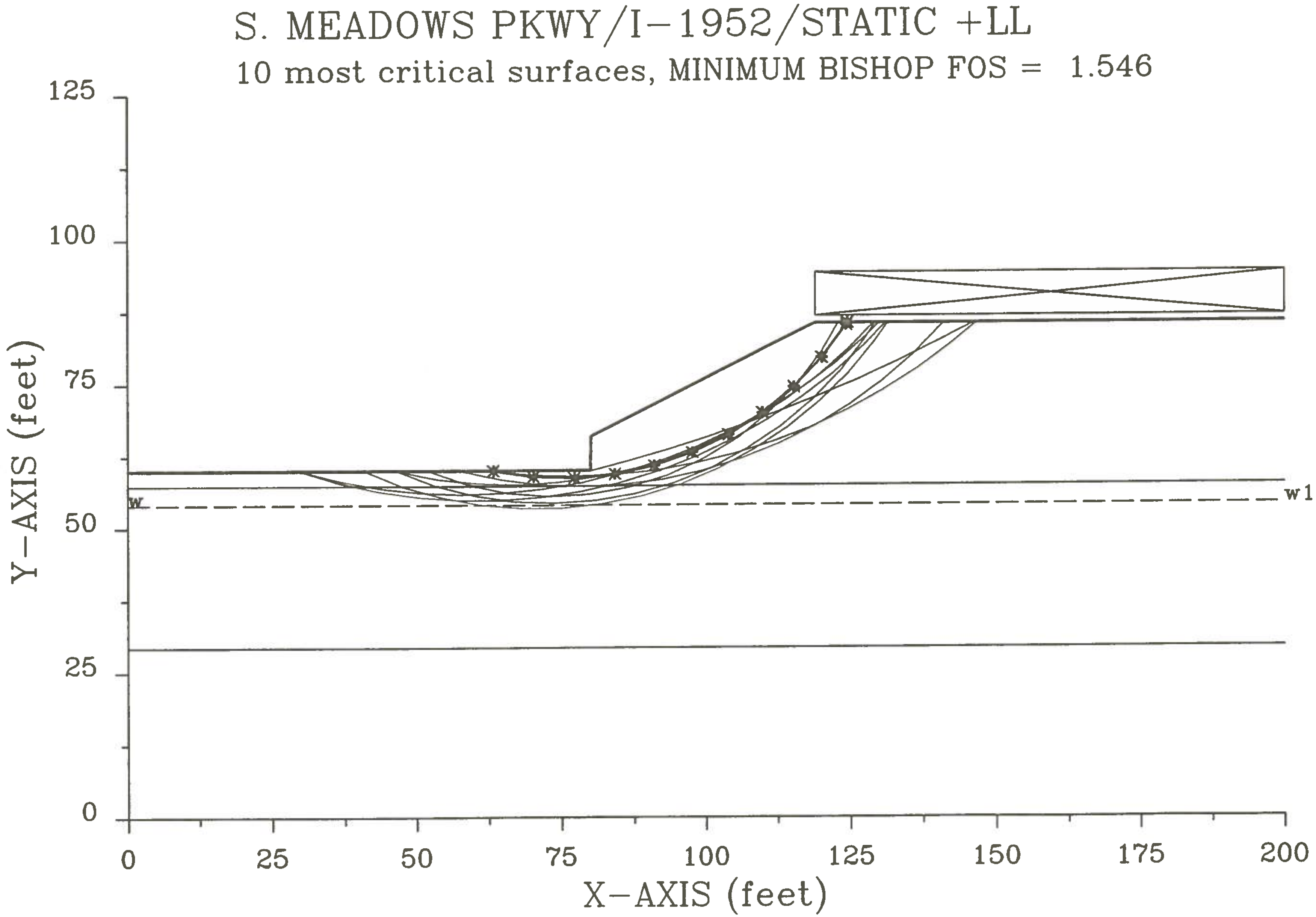
COUNTY: WASHOE DATE SAMPLED: MAY AND NOV 1992

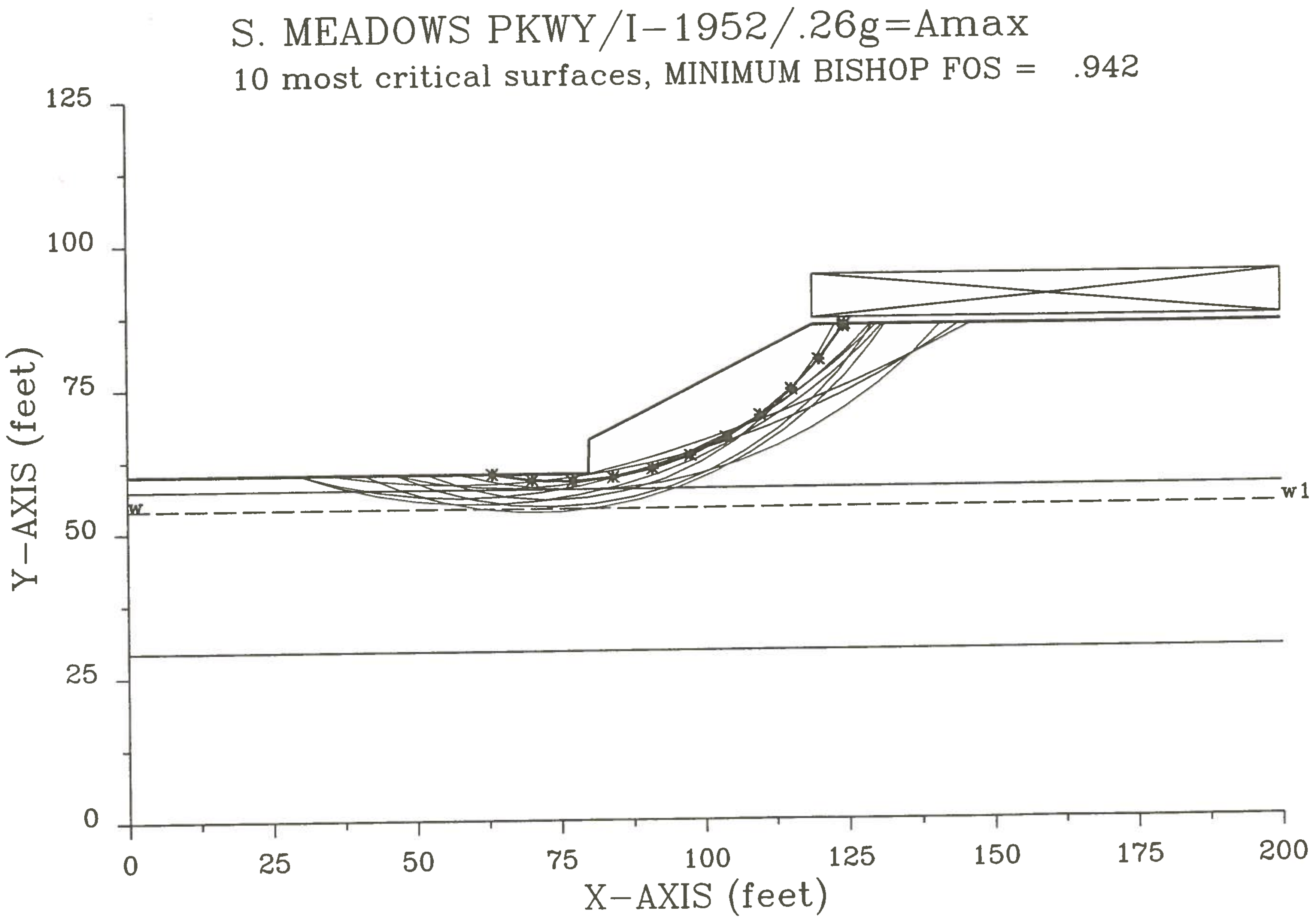
	STATION	LOCATION FROM CL	DEPTH	R VALUE	LL	PI	%GRVL	%SAND	%SAND & SILT	SOLUBLE SALTS IN PPM CL	SO4	Ph	RESIST.
	CHEM SAMPLES ONLY BELOW												
'H'	618 +87	ON CL								90	525	5.4	1577
	611 +79	ON CL								110	180	6.5	3300
	607 +82	ON CL								60	350	6.2	2268
	604 +08	ON CL								70	0	6.5	4739
	595 +60	ON CL								80	120	5.5	6494
	576 +70	ON CL								60	0	6.7	4926
	566 +94	ON CL								80	140	4.8	5848
	563 +55	100' LT								60	150	5.5	3861
	550 +42	ON CL								70	0	5.9	9804
	547 +35	12' RT								60	0	5.2	10753
	541 +74	ON CL								60	0	6.0	9524
	536 +00	ON CL								70	0	5.2	6711
	532 +15	ON CL								60	125	5.1	8929
	526 +71	ON CL								80	0	5.5	6289
	520 +00	ON CL								60	0	5.3	6915
	513 +12	ON CL								70	260	3.9	3185
	511 +70	ON CL								60	170	5.1	6579
	504 +40	ON CL								60	0	5.7	10204
	499 +20	ON CL								40	0	4.9	13158
	488 +65	ON CL								60	0	6.1	6803
'P2A'	474 +52	ON CL								60	0	4.4	14925
	470 +42	ON CL								40	0	5.9	9259

APPENDIX 3

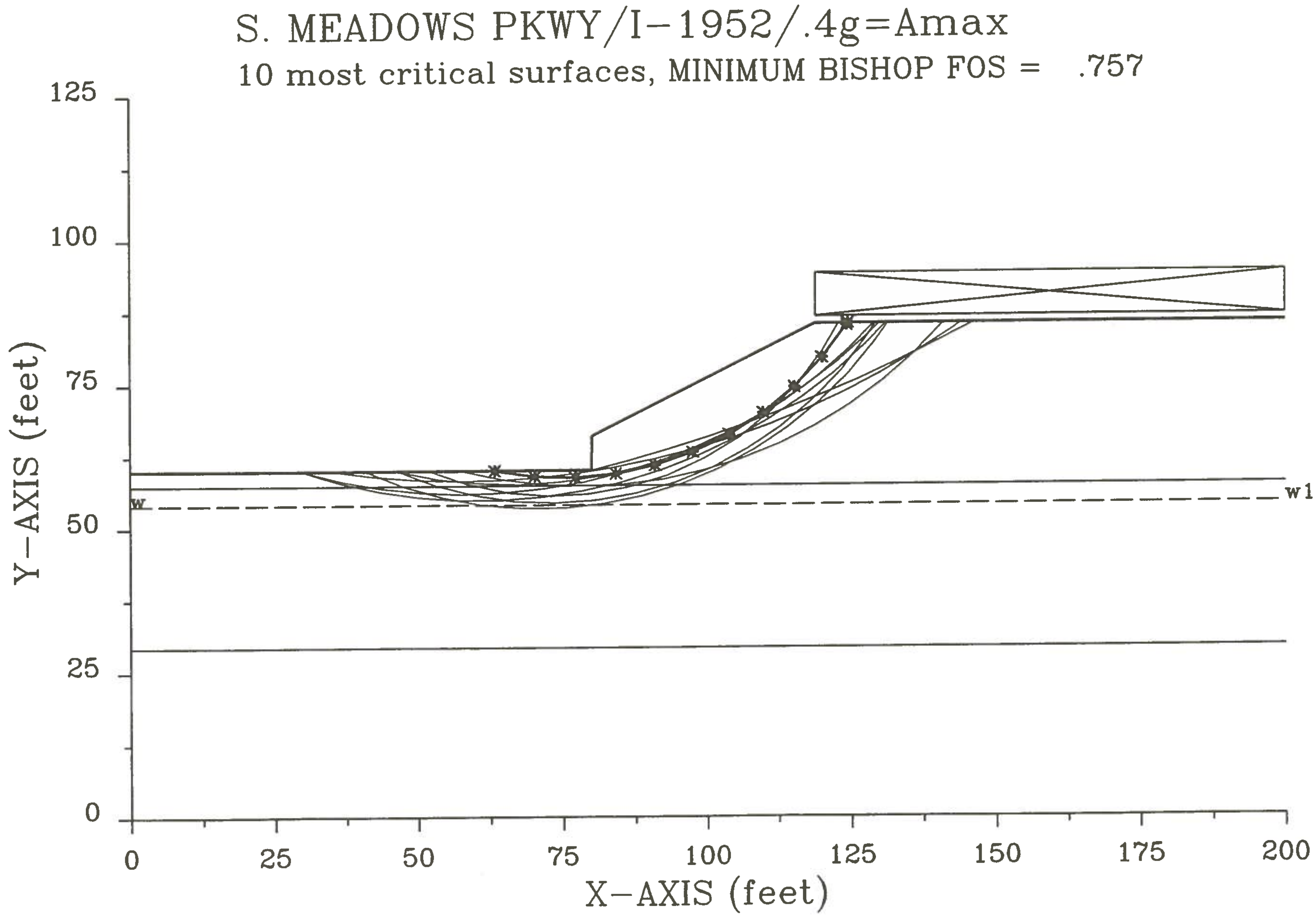
XSTABL™ External Slope Stability Plots


S. MEADOWS PKWY/I-1952/STATIC +LL
10 most critical surfaces, MINIMUM BISHOP FOS = 1.546
Y-AXIS (feet)
X-AXIS (feet)
125
100
75
50
25
0
0
25
50
75
100
125
150
175
200
w
w1

SMEDORMS 4-23-93 8:48



SMEDOEQ 4-23-93 9:01



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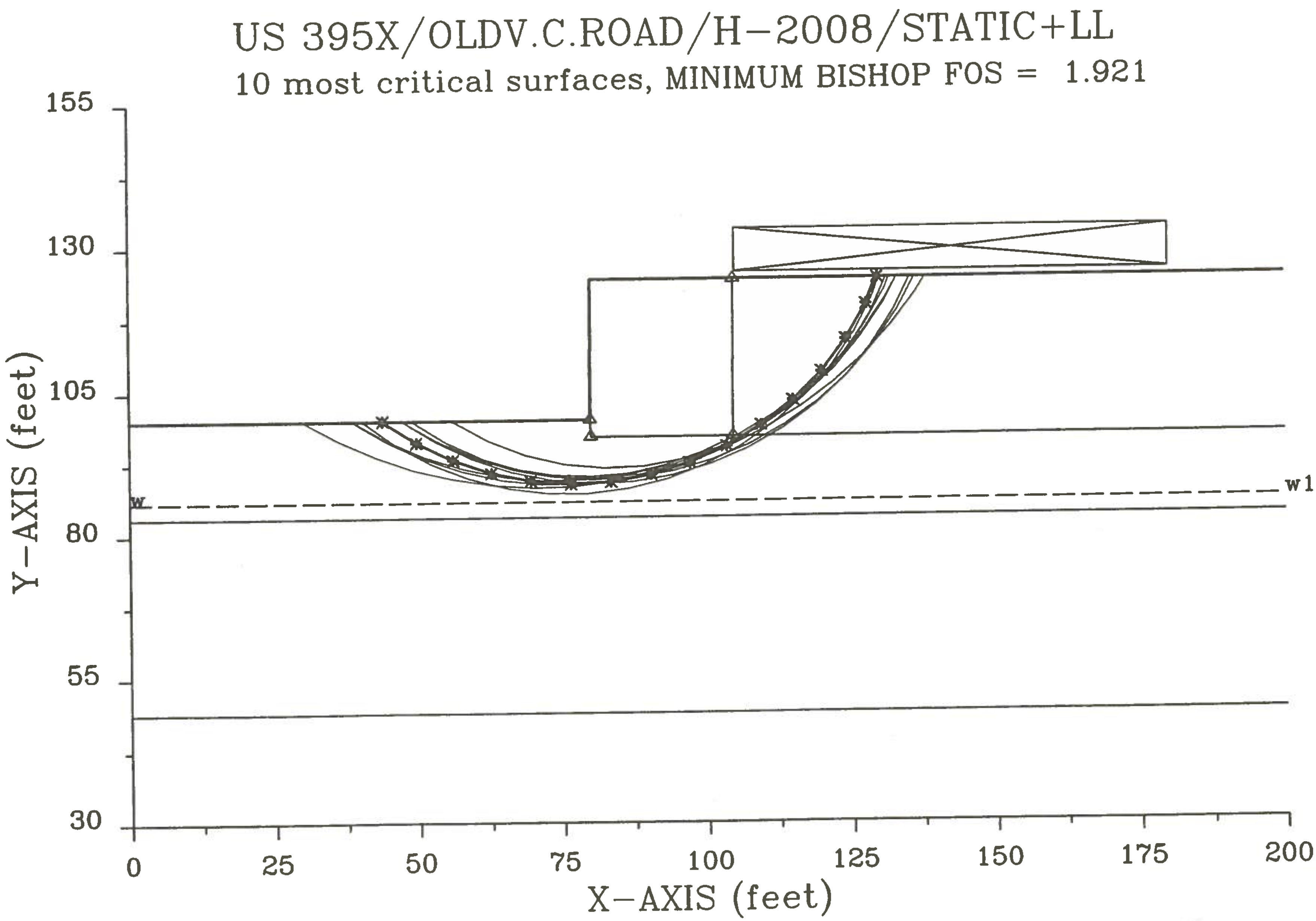

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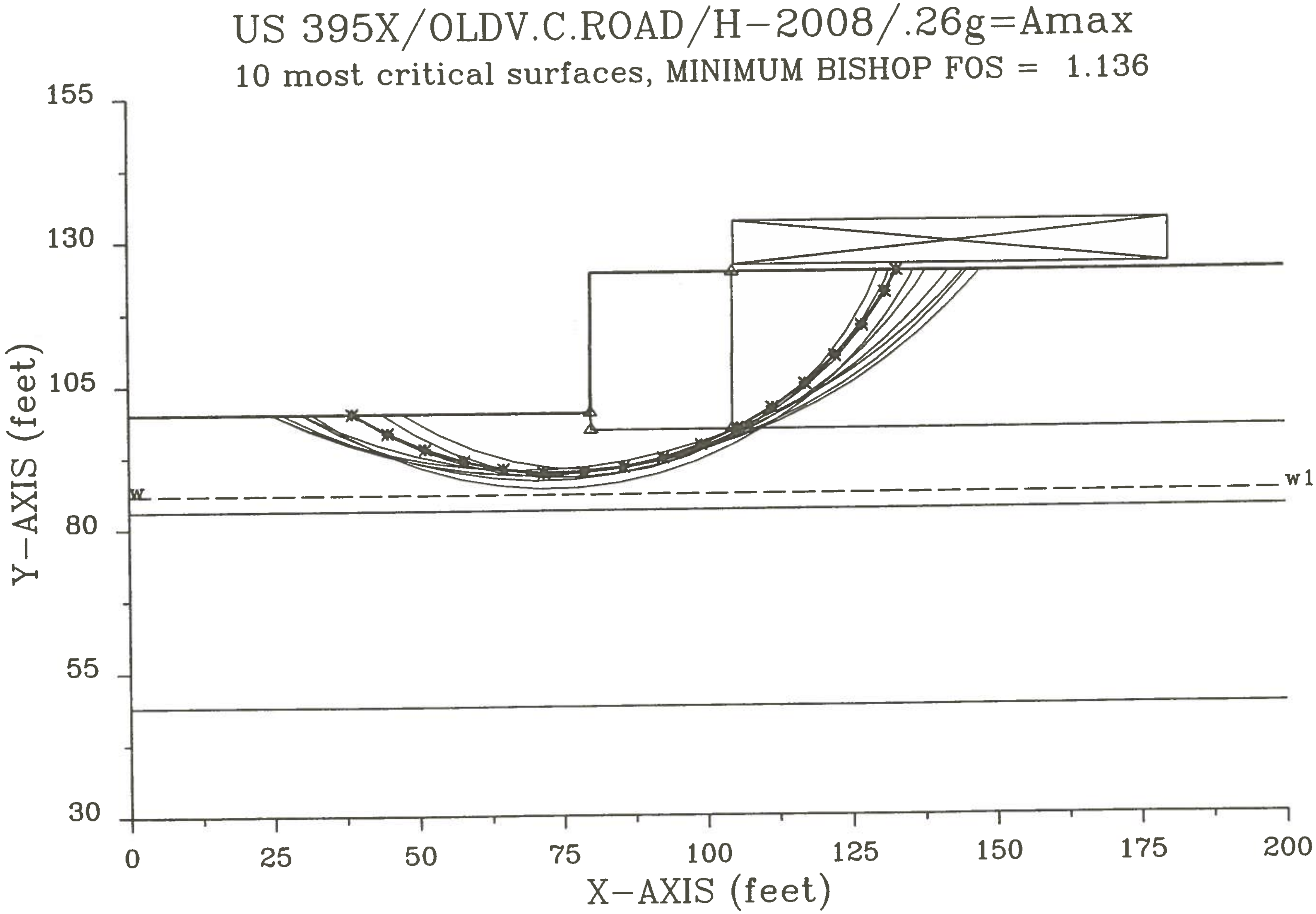

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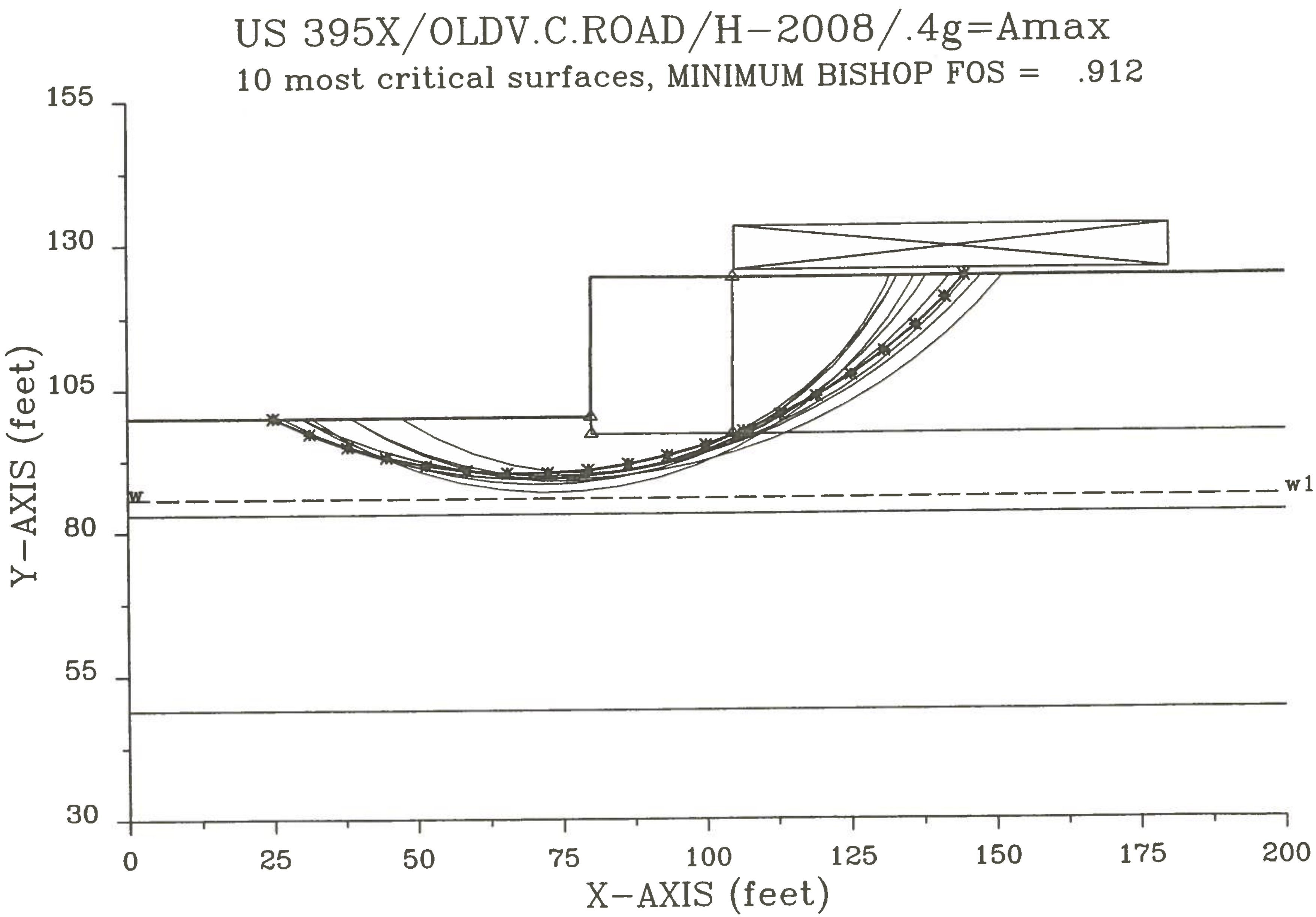
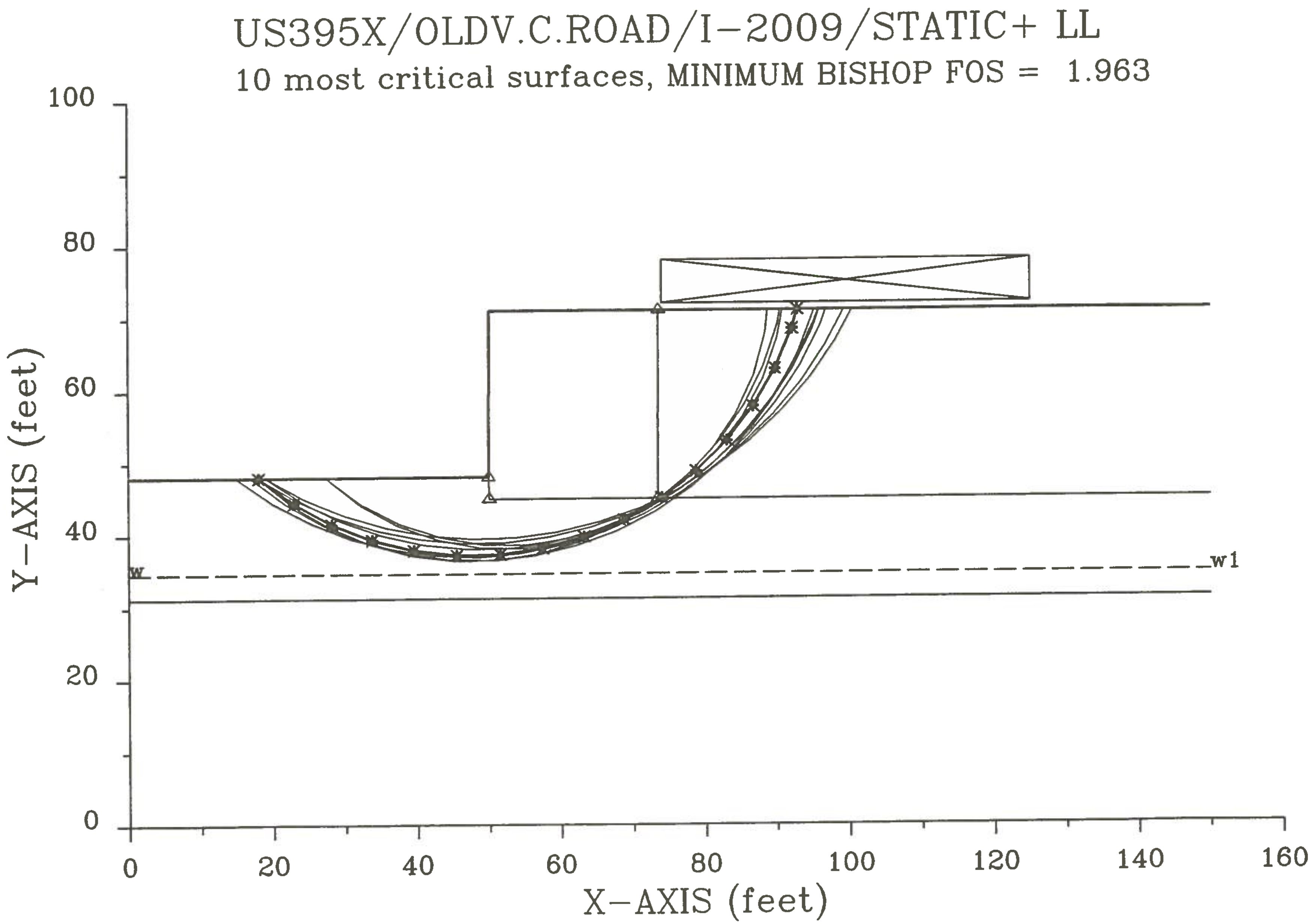
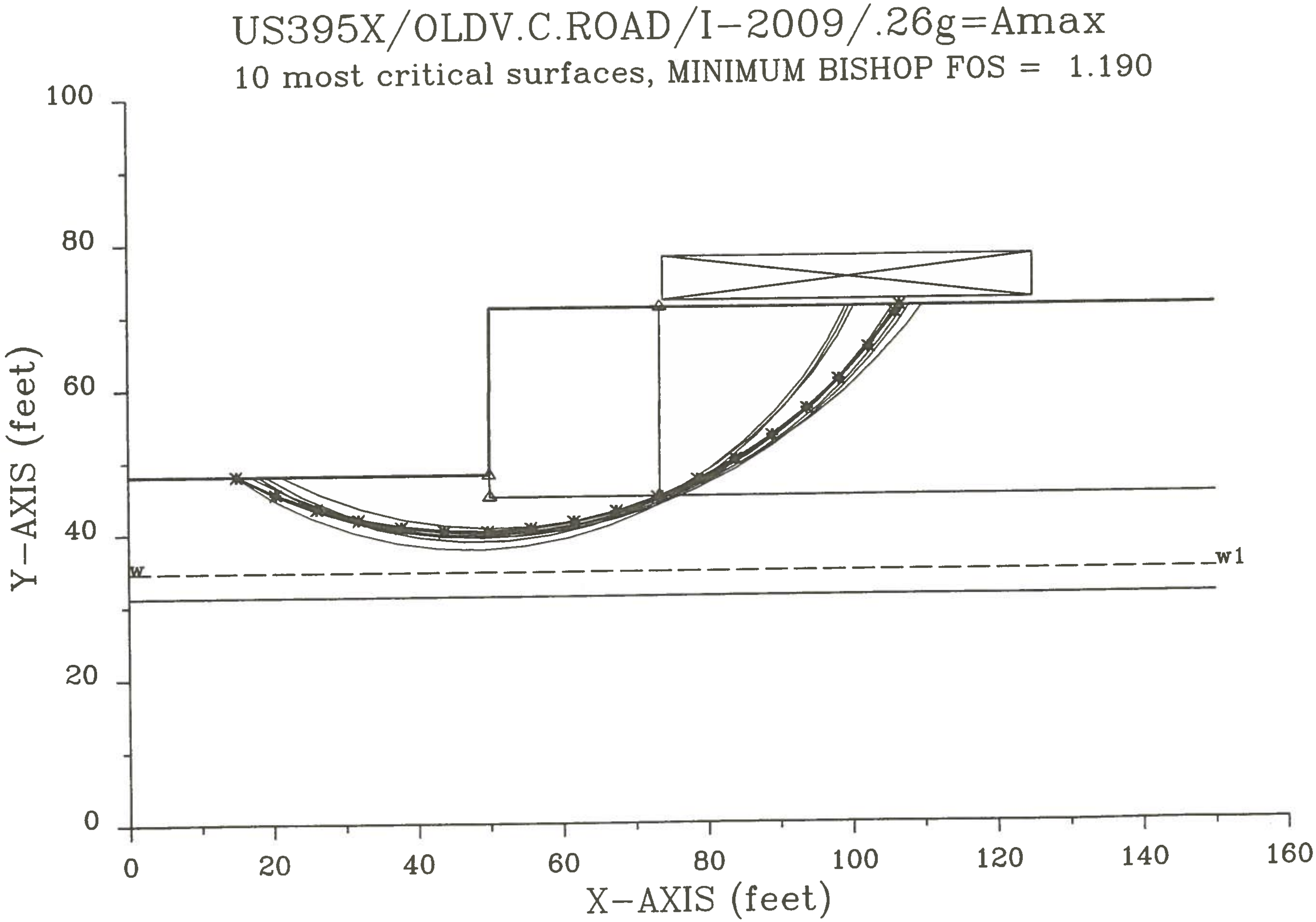
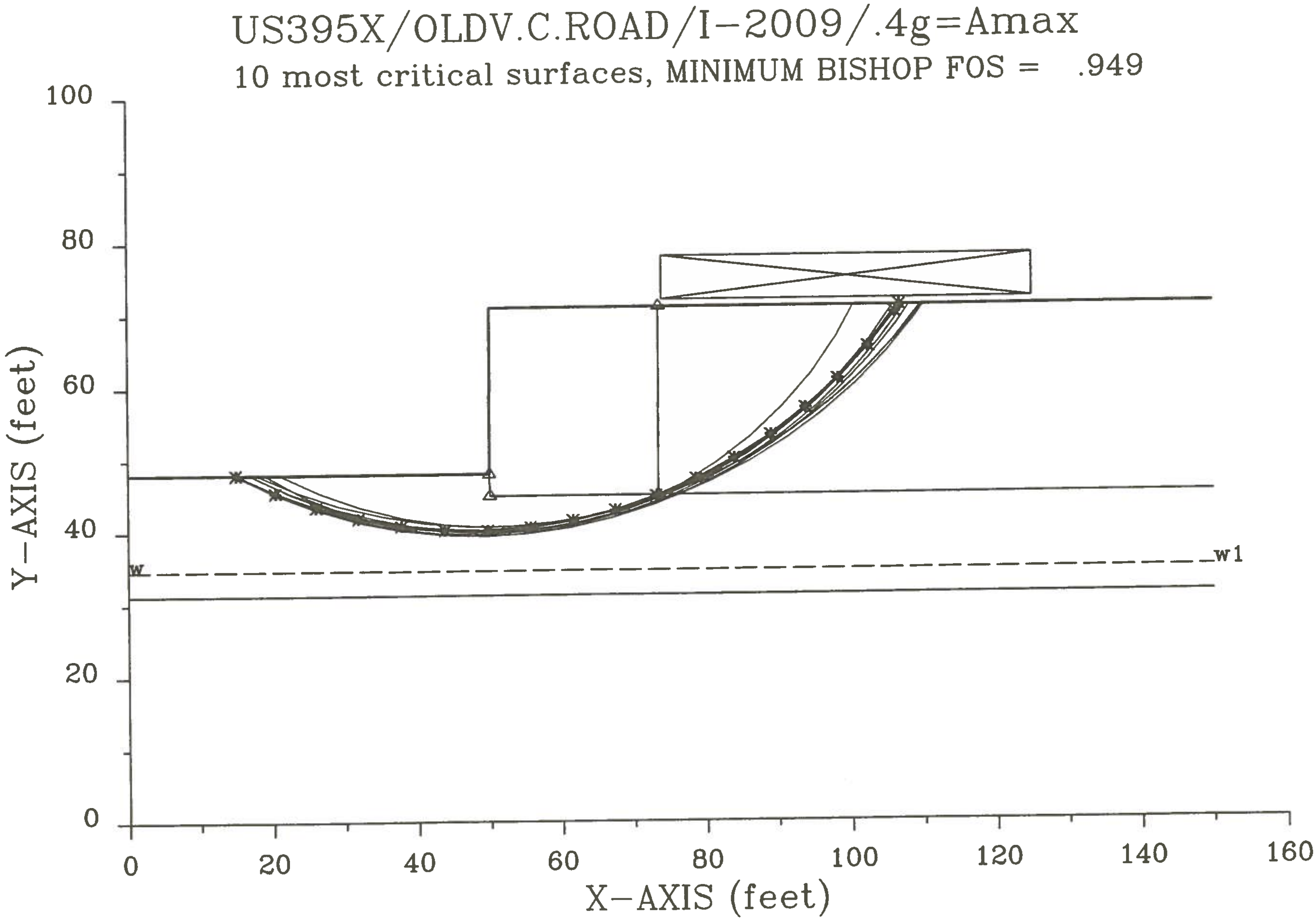



US395X/OLDV.C.ROAD/I-2009/STATIC+ LL
10 most critical surfaces, MINIMUM BISHOP FOS = 1.963
100
80
60
40
20
0
Y-AXIS (feet)
w
w1
0
20
40
60
80
100
120
140
160
X-AXIS (feet)

RS9RMSEQ 4-23-93 8:06



RS9MAXEQ 4-23-93 7:55



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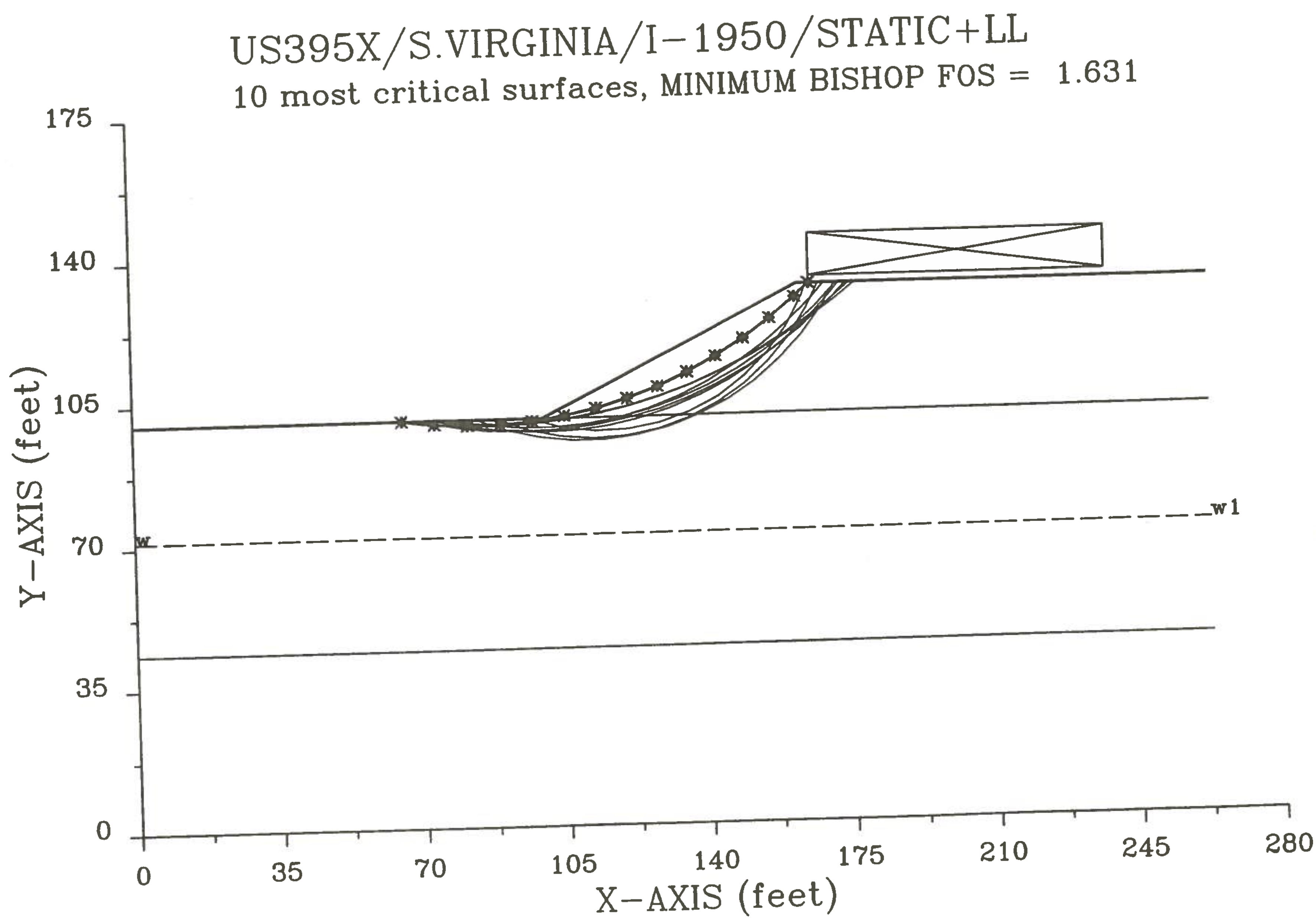

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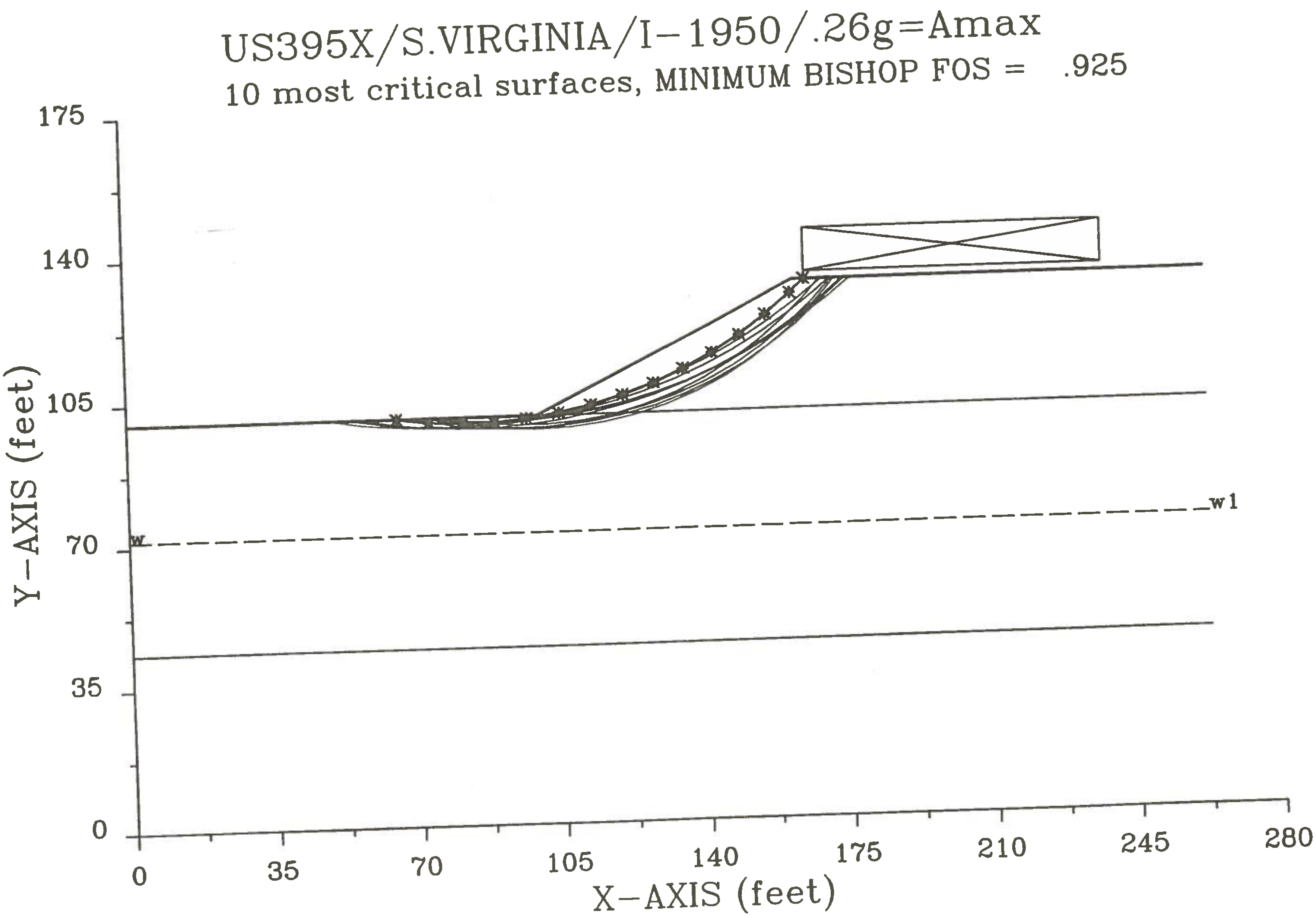

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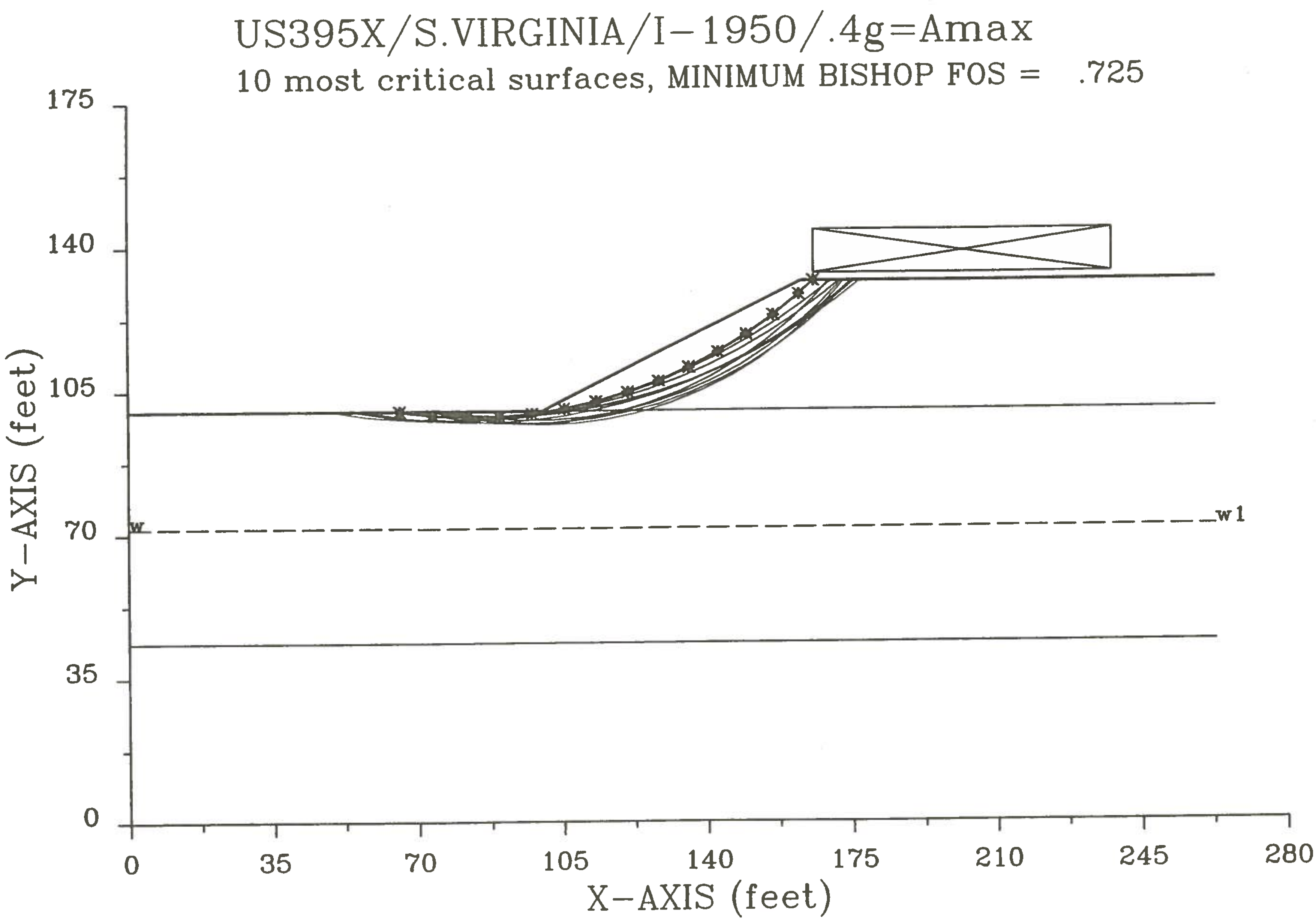

APPENDIX 4

PY Curves and Earth Embankment Foundation Design Parameters

FOUNDATION DESIGN DATA FOR SVERDRUP

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

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

LOCATION: Bridge: S. Meadows Parkway #I–1952
Support: Abutments # 1&3
Remarks: Cap ~6' below surface of sloping fill

P–Y CURVE INTERCEPTS @ pilecap

Deflection–y(in.)	0.12	0.19	0.26	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	7.57E+02	8.81E+02	9.76E+02	1.12E+03	1.57E+03	1.57E+03

P–Y CURVE INTERCEPTS @ 2' below pilecap

Deflection–y(in.)	0.09	0.17	0.25	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	8.11E+02	1.06E+03	1.24E+03	1.54E+03	2.39E+03	2.39E+03

P–Y CURVE INTERCEPTS @ 6' below pilecap

Deflection–y(in.)	0.14	0.21	0.27	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	1.85E+03	2.22E+03	2.52E+03	3.04E+03	4.86E+03	4.86E+03

P–Y CURVE INTERCEPTS @ 13' below pilecap

Deflection–y(in.)	0.31	0.33	0.35	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	6.32E+03	6.54E+03	6.75E+03	7.16E+03	1.15E+04	1.15E+04

P–Y CURVE INTERCEPTS @ 17' below pilecap

Deflection–y(in.)	0.31	0.33	0.35	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	7.60E+03	7.87E+03	8.14E+03	8.64E+03	1.38E+04	1.38E+04

P–Y CURVE INTERCEPTS @ 25' below pilecap

Deflection–y(in.)	0.40	0.90	1.20

Soil Rest.P(#/in.) (force/length of pile)	1.35E+04	2.16E+04	2.16E+04

DEVELOPMENT OF P–Y CURVES FOR GRANULAR SOILS

For Route 395 Between S Virginia and Brown's School

LOCATION:	Bridge:	S. Meadows Parkway #I–1952
	Support:	pier #2
	Remarks:	Cap ~5' below surface of ground

P–Y CURVE INTERCEPTS @ pilecap

Deflection–y(in.)	0.12	0.19	0.26	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	4.19E+02	4.76E+02	5.19E+02	5.83E+02	7.77E+02	7.77E+02

P–Y CURVE INTERCEPTS @ 2' below pilecap

Deflection–y(in.)	0.09	0.17	0.25	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	4.54E+02	5.73E+02	6.61E+02	7.94E+02	1.17E+03	1.17E+03

P–Y CURVE INTERCEPTS @ 4' below pilecap

Deflection–y(in.)	0.06	0.15	0.23	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	4.20E+02	6.15E+02	7.56E+02	9.70E+02	1.53E+03	1.53E+03

P–Y CURVE INTERCEPTS @ 7' 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)	6.84E+02	9.57E+02	1.16E+03	1.49E+03	2.38E+03	2.38E+03

P–Y CURVE INTERCEPTS @13' 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)	1.72E+03	2.10E+03	2.41E+03	2.92E+03	4.68E+03	4.68E+03

P–Y CURVE INTERCEPTS @ 20' below pilecap

Deflection–y(in.)	0.34	0.35	0.37	0.40	0.90	1.20

Soil Rest.P(#/in.) (force/length of pile)	6.12E+03	6.25E+03	6.37E+03	6.62E+03	1.06E+04	1.06E+04

P–Y CURVE INTERCEPTS @ 30' below pilecap

Deflection–y(in.)	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	1.27E+04	2.04E+04	2.04E+04

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

LOCATION:		Bridge:	Old Virgina Road– #'s I–2007, H–2008, I–2009
		Support:	Abutments
		Remarks:	MSE Walls ~25' high
DEPTH:	of P–y		25 assume full effect of wall
	to water	Wtb(ft.):	41

INPUT PARAMETERS

P–Y CURVE INTERCEPTS @ base of MSE Wall

Deflection–y(in.)	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	1.15E+04	1.84E+04	1.84E+04

P–Y CURVE INTERCEPTS @ 5' below MSE Wall

Deflection–y(in.)	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	1.60E+04	2.55E+04	2.55E+04

P–Y CURVE INTERCEPTS @ 15' below MSE Wall

Deflection–y(in.)	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	2.21E+04	3.54E+04	3.54E+04

P–Y CURVE INTERCEPTS @ 20' below MSE Wall

Deflection–y(in.)	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	3.49E+04	5.59E+04	5.59E+04

P–Y CURVE INTERCEPTS @ 30' below MSE Wall

Deflection–y(in.)	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	3.89E+04	6.23E+04	6.23E+04

P–Y CURVE INTERCEPTS @ 35' below MSE Wall

Deflection–y(in.)	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	1.26E+05	2.01E+05	2.01E+05

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

LOCATION:	Bridge:	S. Virgina @ Brown's School – #I–1950
	Support:	Abutments
	Remarks:	Pilecap ~7' below sloping surface
DEPTH: to water		~ 38

INPUT PARAMETERS Pile Diameter(in): 24

P–Y CURVE INTERCEPTS @ bottom of pilecap

Deflection–y(in.)	0.11	0.18	0.25	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	8.23E+02	9.97E+02	1.13E+03	1.34E+03	1.97E+03	1.97E+03

P–Y CURVE INTERCEPTS @ 2' below pilecap

Deflection–y(in.)	0.10	0.18	0.25	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	9.82E+02	1.27E+03	1.49E+03	1.85E+03	2.91E+03	2.91E+03

P–Y CURVE INTERCEPTS @ 6' below pilecap

Deflection–y(in.)	0.16	0.22	0.28	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	2.29E+03	2.66E+03	2.97E+03	3.52E+03	5.63E+03	5.63E+03

P–Y CURVE INTERCEPTS @ 10' below pilecap

Deflection–y(in.)	0.07	0.15	0.23	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	3.17E+03	4.63E+03	5.70E+03	7.37E+03	1.18E+04	1.18E+04

P–Y CURVE INTERCEPTS @ 17' below pilecap abut

Deflection–y(in.)	0.10	0.18	0.25	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	6.54E+03	8.53E+03	1.01E+04	1.27E+04	2.03E+04	2.03E+04

P–Y CURVE INTERCEPTS @ 27' below pilecap

Deflection–y(in.)	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	2.23E+04	3.56E+04	3.56E+04

P–Y CURVE INTERCEPTS @ 40' below pilecap

Deflection–y(in.)	0.40	0.90	1.20

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

LOCATION:	Bridge:	S. Virgina @ Brown's School – #I–1950
	Support:	Pier
	Remarks:	Assume pilecap ~5' below surface
DEPTH: to water		23
INPUT PARAMETERS	Pile Diameter(in):	24

P–Y CURVE INTERCEPTS @ bottom of pilecap

Deflection–y(in.)	0.03	0.12	0.22	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	3.57E+02	4.98E+02	5.71E+02	6.66E+02	8.74E+02	8.74E+02

P–Y CURVE INTERCEPTS @ 2' below pilecap

Deflection–y(in.)	0.10	0.18	0.25	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	7.67E+02	9.43E+02	1.08E+03	1.28E+03	1.88E+03	1.88E+03

P–Y CURVE INTERCEPTS @ 6' below pilecap

Deflection–y(in.)	0.10	0.17	0.25	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	1.17E+03	1.54E+03	1.83E+03	2.29E+03	3.67E+03	3.67E+03

P–Y CURVE INTERCEPTS @ 10' below pilecap

Deflection–y(in.)	0.06	0.14	0.23	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	2.34E+03	3.62E+03	4.53E+03	5.92E+03	9.47E+03	9.47E+03

P–Y CURVE INTERCEPTS @ 17' below pilecap pier

Deflection–y(in.)	0.11	0.18	0.26	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	6.66E+03	8.45E+03	9.90E+03	1.23E+04	1.96E+04	1.96E+04

P–Y CURVE INTERCEPTS @ 27' below pilecap

Deflection–y(in.)	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	1.53E+04	2.45E+04	2.45E+04

P–Y CURVE INTERCEPTS @ 40' below pilecap

Deflection–y(in.)	0.36	0.37	0.38	0.40	0.90	1.20
Soil Rest.P(#/in.) (force/length of pile)	2.46E+04	2.49E+04	2.52E+04	2.57E+04	4.12E+04	4.12E+04