GEOTECHNICAL REPORT

BOULDER CITY BYPASS
PHASE 1

CLARK COUNTY, NEVADA

May 2011

VOLUME 1: GEOTECHNICAL DESIGN REPORT

NEVADA DOT
GEOTECHNICAL ENGINEERING

BOULDER CITY
BYPASS

BOULDER CITY BYPASS

DEPARTMENT OF TRANSPORTATION

MATERIALS DIVISION

GEOTECHNICAL SECTION

VOLUME 1

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PHASE 1

May 2011

E.A. No. 73307

CLARK COUNTY, NEVADA

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I. INTRODUCTION

1.1 Project Description

The Boulder City Bypass Project involves traffic improvements to US 93 in the cities of Henderson and Boulder City, Nevada, area. Project limits are between a western boundary at the south end of Interstate 515 on US 93/US 95 in Henderson, approximately 1 mile north of the Railroad Pass Hotel and Casino, and an eastern boundary on US 93, approximately 0.75 mile east of the Hacienda Hotel and Casino.

The project purpose is to:

- Reduce traffic problems in the vicinity of Boulder City
- Extend freeway status to the US 93/95 interchange
- Improve operations at the junction of US 93/95
- Create a safer transportation corridor
- Accommodate future transportation demand
- Improve system linkage on US 93 and maintaining route continuity

The project is separated into two phases (Phase 1 and Phase 2), shown in the exhibit below:

Phase 1 extends from the Foothills grade separation to the US 95 interchange south of the existing US 93/US 95 interchange for a project length of approximately 2.75 miles. A visual simulation of the Railroad Pass interchange is shown below, depicting what it might look like when complete:

Phase 1 consists of package 1, 2, 3, and 4:

Package 1: Right of Way Setting for US 93/US 95 Freeway Improvements.

Package 2: Construct the Frontage Road and Utility Relocations:

Package 3: Construct realigned US 93/ US 95 mainline to the intersection with the frontage road and the new interchange at Railroad Pass:

Package 4: Complete the US 93/US 95 Interchange at Railroad Pass, and construct the new US 95 Connection, bypassing the existing US 93/US 95 interchange:

This report addresses the geotechnical related issues of the key features of the proposed plans for **Phase 1** such as:

"P" line is the main line for I-515 corridor.

"F" line includes a two-lane flyover, spanning over the I-515 freeway that provides a connection to U.S. 93 toward Boulder City. It comprises of a four-span structure (**Structure I-2871**; Boulder City Bypass) and a single-span structure (**Structure I-2870S**; Boulder City Bypass-I-515/US93 Interchange). The location of Structure I-2871 is between station "F" 14+47 and station "F" 21+29, and Structure I-2870S is located between station "F" 27+79 and station "F" 29+14, spanning over "RR" line.

"DC" line connects U.S. 93 to I-515 North and it comprises of a single-span structure (**Structure I-2870N**; Boulder City Bypass-I-515/US93 Interchange). This structure is located between station "DC" 22+68 and station "DC" 23+97.

"RR" line connects the east and the west frontage roads to the adjacent subdivisions and to the Railroad Pass Casino. It comprises of a bridge structure (**Structure I-2869**; Railroad Pass Interchange) that would be used to cross over I- 515. This structure is located between station "RR" 107+88 and station "RR" 109+82.

"SL" line includes a three-span, two-lane flyover (**Structure I-2868**; US 95 Southbound over I-515 Ramps) spanning over "V" line (on-ramp from U.S. 95 to I-515 North) and "S" line (off-ramp from I-515 to U.S. 95 South). "SL" line conveys the traffic from I-515 and U.S.95 North to U.S. 95 Southbound. The structure is located between station "SL" 14+12 and station "SL" 19+14.

Railroad Bridge (**Structure G-2872**) is a grade separation which crosses over I-515 and is located approximately at Station "P" 98+54.

Direct Connect (**Structure H-2972N/S**) is a grade-separation between "P" line and "V" line and is located approximately at Station "P" 209+37.

There will be roadway cuts in the northern section of "P" line (I-515). Also, there will be cut rock slopes on the west of "WF" line (west frontage road), and on the east side of "R3" line (ramp 3 connects to Structure I-2869).

There will be embankment fill at various locations to connect the roadway alignment to the structures with the maximum height of approximately fifty-three feet at station "F" 24+50.

There will be various hydraulic structures and energy dissipaters along the alignment.

Notes:

A bridge structure was proposed to cross over the Silverline Drive, at the vicinity of station "P" 185+00, in the original scope of the project but was omitted later. However, the geotechnical investigation was conducted for this structure and the geotechnical exploration logs (BSL1, BSL2, BSL3, and BSL4) are presented in the Appendix of this report.

An interchange was proposed to cross over US 95, at the vicinity of station "P" 214+00, in the original scope of the project but was omitted later. However, the geotechnical investigation was conducted for this structure and the geotechnical exploration logs (BCB1 and BCB2) are presented in the Appendix of this report.

1.2 Purpose and Scope

The purpose of this geotechnical investigation was to determine the subsurface soil conditions of the site, to provide geotechnical design criteria, and to provide construction recommendations for the proposed roadway alignment.

The scope of this investigation included research of available geologic literature pertaining to the site, site reconnaissance, subsurface exploration, soil sampling, laboratory testing, analysis of field and laboratory data, and report preparation. The geotechnical report for this project is presented in two separate volumes, Volume I and Volume II. Volume I titled, "Geotechnical Design Report" is comprised of detailed geotechnical investigation and analysis including:

- The site geologic profile
- Physical and geotechnical properties of the site geologic profile
- Potential geologic hazards
- Foundation type and design criteria
- Settlement of the structures
- Lateral earth pressures on retaining walls
- Design criteria for hydraulic structures
- Earthwork
- Stability of embankment fill-slopes
- Stability of excavation cut-slopes

Volume II titled, "Geotechnical Data Presentation" is comprised of the boring logs, laboratory test results, geophysical surveys, and rock slope stability analysis results.

II. GEOTECHNICAL EXPLORATION

The following geotechnical exploration procedures, including soil drilling, rock coring, and geophysical surveys, were developed for the entire project:

2.1 Drilling Procedure

The Geotechnical Section of the Materials Division of NDOT conducted a subsurface investigation along the proposed roadway alignment by drilling intermittently from December 2005 to April 2011. The exploratory borings for bridges were located at approximate bridge abutments and pier foundation unless access conditions dictated otherwise. Also, four exploratory borings were conducted along the proposed roadway cuts near the existing railroad track crossing. The subsurface soil conditions were explored by drilling thirty-six borings to a maximum depth of 80 feet. The boring logs are presented in Volume II-Appendix A.

Drilling was performed by truck-mounted auger drill rigs (NDOT Units # 1627 and # 1082) using six-inch Hollow Stem Continuous Flight Auger (HSA). These rigs also were used for rotary wet drilling using bentonite slurry as a drilling fluid. Logs of the subsurface conditions, as encountered during the field investigation, were recorded by NDOT geotechnical engineers. Drive samples were obtained using a Standard Penetration Testing (SPT-ASTM D1586) sampler. The sampler was advanced using a 140-lb mass (hammer) falling free from a height of 30 inches. Sampler driving resistance (N-value), expressed as blows per one foot of penetration, is presented on the boring logs at the respective sampling depth. The N-values is an indication of the apparent density of coarse-grained soils and the consistency of fine-grained soils. The blow counts presented on the boring logs have not been corrected for sampler type, overburden pressure, hammer type, rod length, etc. The Energy Transfer Ratio (ETR) of the hammers is provided on the Key to Boring Log sheet and on the boring logs in Volume II - Appendix A.

Due to the presence of gravel and cobbles at the site, the recovered soil samples in the SPT sampler may be indicative of the matrix component of the in-situ materials and not represent the entire geological profiles of the areas. The soil classifications presented on the boring logs are based on the field and laboratory classifications of the SPT recovered samples and the auger cuttings.

The number of blow counts per one foot of penetration (N-values) from the Standard Penetration Testing (SPT) as presented on the boring logs may not be dependable in gravelly soil layers. Since the SPT sampler (standard split-spoon) inside diameter is 1-3/8 inches, gravel sizes larger than 1-3/8 inches will not enter the spoon. Therefore, the soil classifications on the boring logs may not represent the actual soil classifications of the in-situ materials such as coarse gravels, cobbles or boulders. Also, some of the high SPT blow counts (N-values) may be due to the jammed sampler shoe by gravel pieces.

NDOT geotechnical engineers examined and identified the soils encountered during the field exploration in general accordance with ASTM D-2488. Additional soil classification was subsequently conducted on selected soil samples obtained from SPT sampler and augered samples from the auger cuttings in accordance with ASTM D-2487 at the NDOT headquarters' laboratory facilities.

2.2 Rock Coring Procedure

Rock coring was conducted in exploratory borings where bedrock was encountered. Core drilling was conducted in accordance to ASTM D 2113. Rock coring was performed using rotary drilling method with potable water as a drilling fluid. The drilling fluid prevented overheating of the bit and carried the drill cuttings to the surface and provided lateral pressure to support the sides of the boreholes. Christensen Double-Tube core barrel was used to recover rock cores. The rock coring was accomplished with wireline coring system with a diamond-impregnated coring bit. N-size cores were collected at specific depths in certain boreholes. The core recovery, Rock Quality Designation (RQD), and core-drilling rate are recorded on the boring logs (Volume II). The cores were photographed after being placed in the core boxes. Selected representative rock cores were tested for Unconfined Compressive Strength tests and X-

Ray Diffraction tests at the University of Nevada-Reno (UNR), Rock Mechanics Laboratory and the Geochemistry Department respectively. The summary of the test results is presented in Volume II - Appendix G.

2.3 Geophysical Survey Procedure

In March 2007, September 2010, and March 2011, the NDOT Geotechnical Section conducted surface Seismic Refraction and Refraction Microtremor (ReMi) surveys along the selected sections of the roadway alignment and the proposed rock cut areas. The Seismic Refraction Surveys were performed according to ASTM D 5777 procedures to map the primary wave (P-wave) arrivals.

Volume 2 - Appendix F presents the detailed information in regards to the P-wave and ReMi surveys and the findings. The purpose of these surveys was to provide information regarding the rippability of the bedrock and the site average shear wave velocities.

Geophysical Survey West of Station "WF" 58+40

Geophysical Survey at Station "P"151+50

III. SITE CONDITIONS

3.1 Site Location and Access

The subject site is located within Township 23 South, Range 63 East, M. D. B. & M. The approximate ground elevation along the proposed alignment is 2500 feet above Mean Sea Level (MSL). The proposed alignment is located within a variable terrain. The access to the site is obtained from the north through an un-paved narrow utility access road and from the south through Silverline Road, off of US 95.

Site topography slopes downward toward the center of the site. There are natural drainage paths that run from the north of the site toward south direction. Existing maximum vertical relief across the site is approximately 360 feet.

3.2 Local Geology

The extent of geologic units has been derived primarily from the geologic map prepared by C. R. Longwell, E. H. Pampeyan, and Ben Bowyer of Nevada Bureau of Mines and Geology with the scale of 1:250,000; 1965. The geological map (Appendix B) indicates that the proposed roadway alignment is located between River Mountain and McCollough Range. A Quaternary age formation of alluvium deposits underlies southern section of the roadway alignment. To the southwest, Tertiary age volcanic rocks such as granite, rhyolite, and trachydolerite underlie the alignment. The roadway alignment will pass through bedrock outcrops between Station "P" 150 and Station "P" 152. The material present in this area consists of rhyolite/andesite.

3.3 Local Faults

Geological mapping of the site shows no mapped faults within the Quaternary aged alluvial deposit at the proposed site. There are several mapped normal and reverse faults of Tertiary age volcanic rocks located within 4 miles northeast of the alignment within River Mountains called Sunrise Mountain-Frenchman-River Mountain Fault Zone (SFRFZ). This is widely recognized active tectonic fault which extends southward along the eastern edge of Las Vegas Basin (LVB) to Lake Mead Fault System (LMFS) near Railroad Pass.

No evidence of faulting was observed during exploration of the site, including the area in and around the roadway alignment. As a result, no additional fault hazard evaluation is considered necessary for the proposed Phase I alignment.

3.4 Ground Motion

The bedrock acceleration generated by earthquake at the site is based on AASHTO LRFD Bridge Design Specifications-Figure 3.10.2.1-1. The site is located in Region 1. Based on this figure, the estimation of the horizontal peak ground acceleration coefficient (PGA) with 7 percent probability of exceedance in 75 years (approximately 1000-year return period) is 0.15g. In accordance with provisions presented in AASHTO LRFD Table 3.10.33.1-1, Site Class Definitions, the site generally can be classified as Site Class C. According to the NDOT Bridge Design and Procedures Manual – Figure 12.3-H, the minimum seismic coefficients for Short-Period Spectral Acceleration Coefficient (S_s) = 0.40 and the Long-Period Spectral Acceleration Coefficient (S_1) = 0.15 at this site.

3.5 Liquefaction

Where boreholes were advanced along the alignment, granular soil deposits typically were dry, and groundwater was deeper than the maximum depth of exploration. The site is underlain at depth by dense to very dense granular deposits. Therefore, the liquefaction potential is considered non-existent at this location.

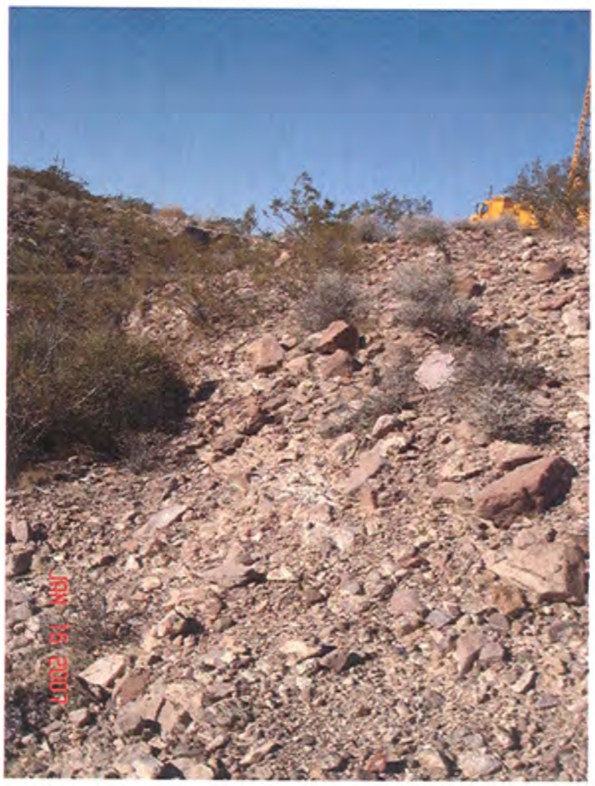
3.6 Surface Drainage Distribution

Terrain information obtained from aerial photographs shows several slopes with different steepnesses and escarpments created drainage paths along the proposed alignment. According to the proposed construction plans, the major drainage paths along the proposed alignment will be perpetuated by installing culverts and energy dissipaters.

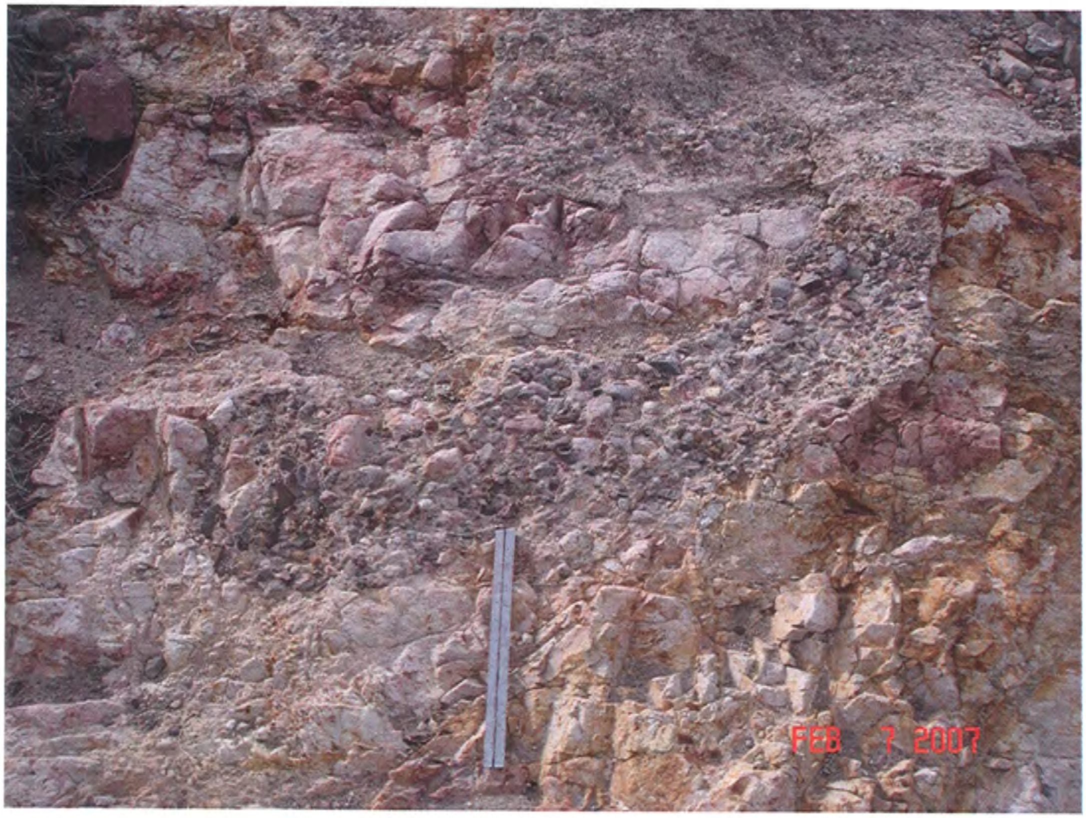
3.7 Site Geological Profile and Soil Condition

The following section presents a generalized description of the geological profile encountered during our subsurface site investigation. The boring logs should be reviewed for more detail.

Alluvium is the predominant surficial deposit over the project site. As encountered in our borings and depicted in the photos below, the alluvial deposit is characterized as dense to very dense mixtures of sand, gravel, cobbles, and boulders. The presence of coarse gravel, cobbles, and occasional boulders caused SPT blow count refusal in almost all the boreholes.



Cemented materials called caliche also was encountered during the subsurface investigation. The caliche is formed by lithification of fine grained sediment from evaporation of lime-rich groundwater introduced through capillary action or precipitation. The fluctuation of the groundwater level and intrusion of surface water into the soil has resulted in caliche layers at various depths and thickness as addressed in the Geological Profile table on page 16. The photo below shows caliche encountered at the project site.



In areas of little topographic relief in the southwestern section of the alignment, occasional bedrock outcrops, such as between Station "P" 150 and Station "P" 152, (photo next page), have been exposed.

Bedrock Outcrop

Bridge I-2869 - Center Pier Location

Approximate Location: Station "RR" 108+86 / Station "P" 152

BRIDGE STRUCTURES	GEOLOGICAL PROFILE
I-2868	**Boring Logs: SLA1, SLP1, SLP2, SLA2**
Abutment #1 (Station ≈ "SL" 14 + 12.8) Pier #1 (Station ≈ "SL" 15 + 52.8) Pier #2 (Station ≈ "SL" 17 + 32.8) Abutment #2 (Station ≈ "SL" 19 +14)	Alluvial deposits of silty gravel with sand and cobble size rock fragments with occasional cementation. Upper 8 feet at Boring Log SLA1 and Boring Log SLP2 locations, medium dense, otherwise, very dense.
I-2869	**Boring Logs: RRAP1, RRA2**
Abutment #1 (Station ≈ "RR" 107 + 88)	Highly weathered bedrock at Boring Log RRAP1 (between Abutment #1 and Center Pier).
Center Pier (Station ≈ "RR" 108 + 86)	Bedrock outcrops at the Center Pier location; no borehole at this location.
Abutment #2 (Station ≈ "RR" 109 + 82)	10 feet of loose well-graded gravel over weathered bedrock at Boring Log RRA2 (Abutment 2).
I-2870S	**Boring Logs: FA3, FA4**
Abutment #1 (Station ≈ "F" 27 +79)	5 feet of medium dense to dense sand with silt and gravel with occasional caliche (Alluvium) over weathered to highly fractured bedrock (Rhyolite). Rock cores were obtained at Abutment # 1, Boring Log FA3.
Abutment #2 (Station ≈ "F" 29 + 14)	15 feet of very dense rock fragments in a matrix of silty sand overlying highly fractured bedrock at Boring FA4.
I-2870N	**Boring Logs: DCA1, DCA2**
Abutment #1 (Station ≈ "DC" 22 +68)	10 feet of dense to very dense silty gravel with occasional caliche layer over weathered bedrock (Rhyolite); Boring Log DCA1.
Abutment #2 (Station ≈ "DC" 23 + 97)	7 feet of dense to very dense silty gravel with occasional caliche layer over weathered bedrocks (Rhyolite), (Rock cores were obtained at Abutment # 2); Boring Log DCA2.

I-2871	Boring Logs: FA1, FP1, FP2, FP3, FA2
Abutment #1 (Station ≈ "F" 14 + 47)	5 feet of very dense sand and gravel is overlying hydro-thermally weathered Rhyolite/Granite-Diorite (Boring Log FA1)
Pier #1 (Station ≈ "F" 16 + 07)	13 feet of very dense sand and gravel is overlying hydro-thermally weathered Rhyolite/Granite-Diorite (Boring Log FP1).
Pier #2 (Station ≈ "F" 17 + 80.5)	12 feet of very dense sand and gravel is overlying cemented sand and gravel in clay matrix to the depth of 35 feet. At 35 feet of depth, hydro-thermally weathered Rhyolite/Granite-Diorite (rock cores were obtained at depth 35 feet), (Boring Log FP2).
Pier #3 (Station ≈ "F" 19 + 67)	17 feet of very dense silty sand, clayey sand, and clayey gravel is overlying hydro-thermally altered Rhyolite/Granite-Diorite bedrock (Boring Log FP3).
Abutment #2 (Station ≈ "F" 21 +27)	24 feet of dense to very dense sand and gravel is overlying hydro-thermally altered Rhyolite/Granite-Diorite (Boring Log FA2).
G-2872	**Boring Logs RRBA1, RRBP1, RRBA2**
Abutment 1 (Station ≈ "P" 98+96) Pier 1 (Station ≈ "P" 98+54) Abutment 2 (Station ≈ "P"98+73)	The proposed footing elevations are approximately 30 feet below the existing ground surface. The general soil condition at this site is mainly composed of strong cemented silt-sand-fractured gravel (breccias/caliche).

H-2972N (Simple Span)	Boring Logs NBA1, NBA2
North Bridge - Abutment 1 (Station ≈ "P" 208+56) North Bridge - Abutment 2 (Station ≈ "P" 210+00)	5 to 7 feet of medium dense cohesionless soil is overlying the dense to very dense cohesionless soil.
H-2972S (simple Span)	**Boring Logs SBA1, SBA2**
South Bridge -Abutment 1 (Station ≈ "P" 209+58.32)	Boring Log SBA1: Approximately 6 feet of loose cohesionless soil is overlying the dense to very dense cohesionless soil.
South Bridge -Abutment 2 (Station ≈ "P" 210+59.25)	Boring Log SBA2: Approximately 20 feet of medium dense cohesionless soil is overlying the dense to very dense cohesionless soil.
Native Soil under Roadway Embankment Fill alignments	Variable soil conditions from dense silty/clayey gravel with sand and cobble size rock fragments, moderate to strong cementation to shallow hydro-thermally weathered Rhyolite/Granite-Diorite bedrock.
Roadway Cut Slopes (deepest roadway cut at Station ≈ "P" 98+54)	**Boring Logs: RRC1, RRC2, RRC3, RRC4** Materials are alluvial deposits of dense silty/clayey gravel with sand and cobble size rock fragments, moderate to strong cementation.
Frontage Road Retaining Wall (Station ≈ "WF" 32 to "WF" 42)	**Boring Logs: BRW1, BRW2, BRWC3, BRW4** Materials are alluvial deposits of dense silty/clayey gravel with sand and cobble size rock fragments, moderate to strong cementation.

3.8 Groundwater

Groundwater was not encountered in any borehole. Therefore, groundwater should not have any adverse effect on design, construction, or performance of the proposed structures. Seasonal fluctuation of the groundwater table should be expected.

3.9 Surface water

Surface water was not present in any drainage channel during the time of our exploration. Observations during site reconnaissance indicated the erosion gullies have formed on the partially cemented soils along the proposed alignment. These observations provide a good indication that the on-site material is erodible under concentrated storm water flow.

3.10 Abandoned Mine Shafts

Two abandoned exploratory mine shafts along and in close proximity of the roadway alignment (photos below) were seen that are hazardous if left unfilled. A field meeting was held with the State of Nevada-Division of Minerals representative on June of 2007 in this regard. It was permitted to fill these mine shafts during the construction of the alignment.

Abandoned mine shafts near the project loop ramps

IV. LABORATORY TESTING

Laboratory testing program for selected samples consisted of:

- Natural Moisture Contents (AASHTO T-265)
- Particle Size Gradations (AASHTO T-87, AASHTO T-27)
- Atterberg Limits (AASHTO T-89, T-90)
- Unconfined compression tests (ASTM D 7012-Method C)
- X-Ray Diffraction
- Point Load Tests (ASTM D 5731)
- Electro-Chemical analyses (AASHTO T-288 for determining soil resistivity, T-289 for determining soil pH, T-290 for determining water-soluble sulfate in soil, T-291 for determining water-soluble chloride in soil)
- Resistance Value (R-Value - Nevada T115), testing is a measure of subgrade strength and expansion potential, and is used in design of flexible pavements and Borrow qualification.

Individual laboratory test results can be found in Volume II of this report.

4.1 Soil Test Results

Laboratory test results, such as moisture content, particle size analysis, plasticity index, R-values, and chemical tests, on selected soil samples indicate that subsurface soil was dry, coarse-grained, sub-angular, and having less than 30% fine grains with non to low swelling potential. Therefore, it is expected that the on-site soil behavior to be mainly cohesionless. Yet, soils have enough fines to be compacted by conventional smooth-drum roller, rubber-tired rollers, or vibratory rollers. The R-value test results indicated that, in general, the on-site soils meet the requirements of NDOT Borrow.

Because of the difficulties in measuring the effective friction angle of the cohesionless materials in the laboratory, in-situ index tests such as the Standard Penetration Test (SPT) were used to estimate the friction angles, using Hatanaka and Uchida (1996) correlation of the blow-count measured in situ with the friction angle evaluated in the laboratory. The conservative estimate of design friction angle of native soil is 36 degrees.

Selected samples of the near-surface soils encountered at the site were subjected to chemical analysis for the purpose of corrosion assessment. The samples were tested for pH, resistivity, soluble sulfates, and soluble chlorides. Electro-Chemical analyses were performed on the subsurface soil samples to determine the concentration of corrosive chloride and sulfate salts. Soil pH values that represent the hydrogen concentration in the soil (referred to as the intensity factor), and soil resistivity which is an indirect measurement of the soluble salt content in the soil, were also measured. Results of these analyses are provided in Volume II - Appendix D.

4.2 Rock Test Results

Obtained cores from Boring FA3, DCA1, and FP3 show that the bedrock at some locations within the vicinity of I 515-US93 Interchange (Refer to Boring Logs) is hydro-thermally altered (chemically weathered) igneous rocks.

Hydro-Thermally Altered Bedrock

Cored Bedrock from Borehole FP1

Core Recovery, determined as the ratio of core recovered to the total drilled run length expressed as a percentage, was more than 90% in most locations.

The Rock Quality Designation (RQD), defines the fraction of solid core recovered greater than 4 inches in length, varies drastically across the site.

In the vicinity of Silverline Drive and to the south of the roadway alignment, the bedrock is highly fractured and the RQD = 0 (poor: 0 – 25%). Photo below shows the obtained core from Borehole BSL2 with RQD = 0:

Cored Bedrock from Borehole BSL2 with RQD = 0

In the vicinity of I515-US93 Interchange, the RQD were mostly in good range (75 – 90%). The following photos show the obtained bedrock cores from this area:

Cored Bedrock from Borehole FA3 with excellent RQD (90 – 100%)

Cored Bedrock from Borehole DCA1 with excellent RQD (90 – 100%)

The type and the degrees of bedrock weathering vary across the site. In the vicinity of I-515-US 93 Interchange, the type of the bedrock weathering is mostly chemical, which has resulted in discoloration and decomposition of the rock minerals, such as hydro-thermally altered bedrock.

In the vicinity of Silverline Drive and to the south of the roadway alignment, the type of the bedrock weathering is mostly mechanical (physical) which has resulted in highly fractured (High Fracture Index) bedrock without the mineral discoloration and decomposition.

Twelve core samples were tested to evaluate their Unit Weight and Unconfined Compressive Strength (q_u) tests under air-dried and soaked conditions. Tests showed that cores under soaked condition had substantially (up to 50%) lower compressive strengths than air-dried due to the effect of moisture on the gypsum intrusions.

Unconfined Compressive Strength testing of a rock core
Core shows gypsum healed joint

Considering the lower-bound compressive strength from test results, we assign:

$q_{u\,(\text{lab-dry condition})} = 3700$ psi (Medium Strong Rock: 3500 psi< q_u <7500 psi)

$q_{u\,(\text{lab-soaked condition})} = 1900$ psi (Weak Rock :750 psi< q_u <3500 psi)

The average core wet unit weight is = 155.7 pcf and the average dry unit weight is = 151.6 pcf.

V. EVALUATION & RECOMMENDATIONS

5.1 General

Based on the results of field investigation and laboratory testing, the site is suitable for construction of the proposed structures. There are at least two acceptable foundation alternatives for the structures on this project. The structures could be supported on conventional spread footing foundations or on drilled shaft foundations. Shallow depth to suitable bearing material was the factor for choosing shallow spread footings to support bridge structures. Therefore, the geotechnical analysis and design for spread footings will be addressed.

The NDOT Structures Division has provided the following applied un-factored loading at each structure:

STRUCTURES	DEAD LOAD (kips) (un-factored)	LIVE LOAD (kips) (un-factored)
STRUCTURE I-2868		
Abutment 1	1227	251
Pier 1	2307	525
Pier 2	2585	564
Abutment 2	1376	273
STRUCTURE I-2869		
Abutment 1	2150	800
Center Pier (4 columns)	1080 (each column)	130 (each column)
Abutment 2	2150	800

STRUCTURES	DEAD LOAD (kips) (un-factored)	LIVE LOAD (kips) (un-factored)
STRUCTURE I-2870S		
Abutment 1	1661	603
Abutment 2	1661	603
STRUCTURE I-2870N		
Abutment 1	1661	603
Abutment 2	1661	603
STRUCTURE I-2871		
Abutment 1	2090	336
Pier 1	3499	700
Pier 2	3317	690
Pier 3	3558	699
Abutment 2	2092	338
STRUCTURE G-2872		
Abutment 1 Pier 1 Abutment 2	Not Available	Not Available
STRUCTURE H-2972N		
Abutment 1 Abutment 2	Not Available	Not Available
STRUCTURE H-2972S		
Abutment 1 Abutment 2	Not Available	Not Available

5.2 Spread Footings

The proposed structures may be designed using conventional spread footing foundations supported on approved undisturbed native soil or properly compacted fill.

Partially to fully cemented soils with moderate to strong cementation and bedrock outcrops exist at various locations across this project site. To achieve design level stress-strain compatibility of footings in a single bridge structure, the footings should either be supported entirely on un-cemented soils or entirely on cemented soils. In case, if one or more of the footings are to be placed directly on cemented soils and the rest on un-cemented soils, we recommend that the cemented soil directly below the footings to be over-excavated to a minimum depth of 1.0 foot and replaced with a properly compacted approved fill.

Table I (next page) presents the presumptive soil bearing resistance for spread footings located within the roadway embankment fill with end-slopes of 2H:1V.

Table II (page 28) and the design charts on the subsequent related pages present the recommended LRFD soil bearing resistance for spread footings embedded in the native ground.

- Factored Net Bearing Resistance (q_{Rn}) curves at Strength I Limit State include bearing resistance factor $\varphi_b = 0.45$. Factored Net Bearing Resistance (q_{Rn}) can be derived from the Strength I Limit State curves based on the effective footing width for Strength I Limit State loading conditions. Nominal Net Bearing Resistance at Strength I Limit State (q_{nn}) can be obtained by this equation: $q_{nn} = q_{Rn} / 0.45$.

- Factored Net Bearing Resistance (q_{Rn}) curves at Service I Limit State for the family of settlement related curves includes bearing resistance factor $\varphi_b = 1$. Factored Net Bearing Resistance (q_{Rn}) can be derived from the Service I Limit State curves based on the effective footing width for Service I Limit State loading conditions. Since the resistance factor for service limit state is 1.0, the factored net bearing resistance (q_{Rn}) is equal to the nominal net bearing resistance (q_{nn}).

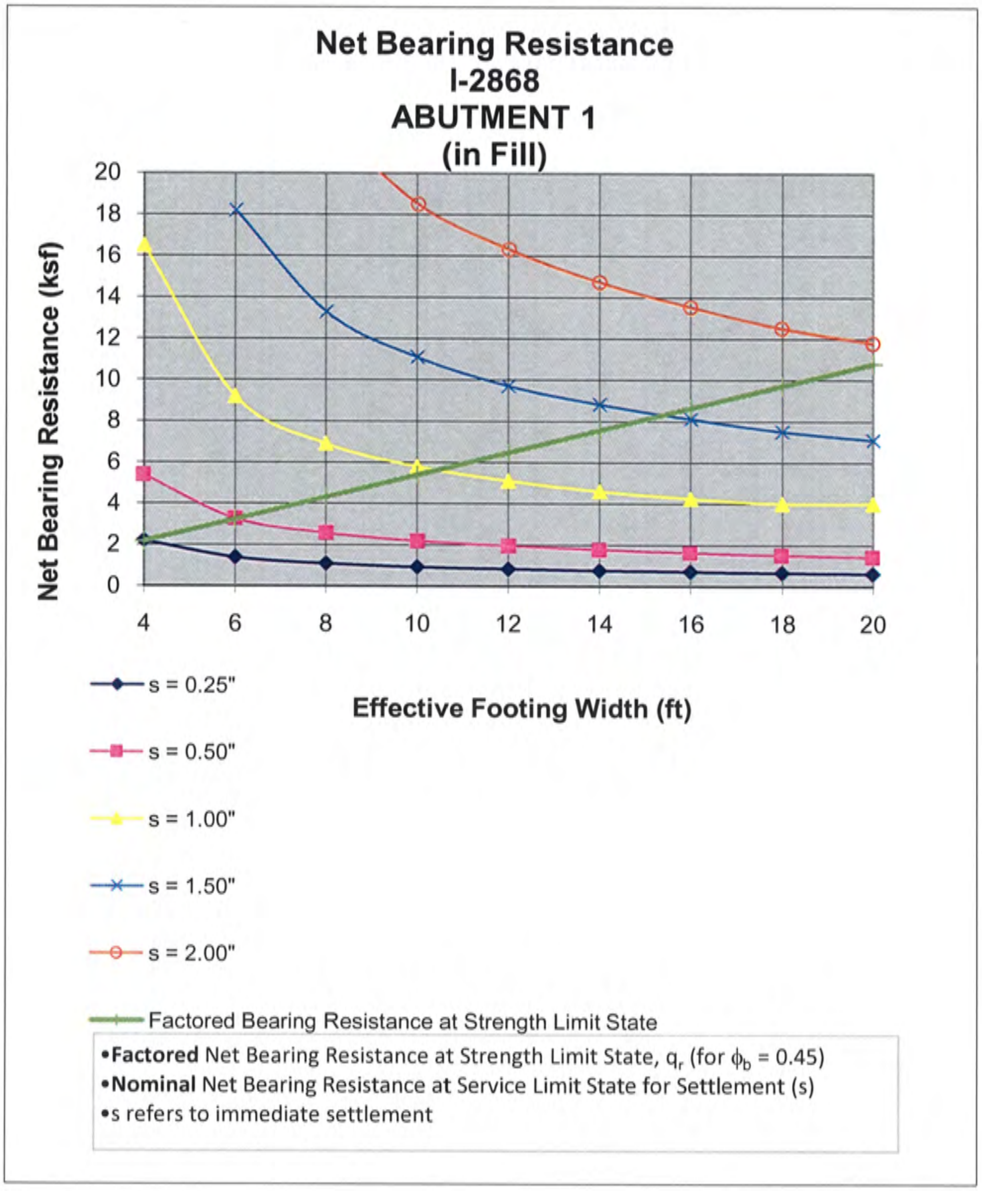
- Nominal Net Bearing Resistance at Extreme Event Limit State-Seismic (q_{nn}) can be obtained by this equation: $q_{nn} = q_{Rn} /0.45$. Factored Net Bearing Resistance (q_{Rn}) can be derived from the Strength I Limit State curves based on the effective footing width for Extreme Event Limit State-Seismic loading conditions. Since the resistance factor for extreme limit state is 1.0, the factored net bearing resistance (q_{Rn}) is equal to the nominal net bearing resistance (q_{nn}).

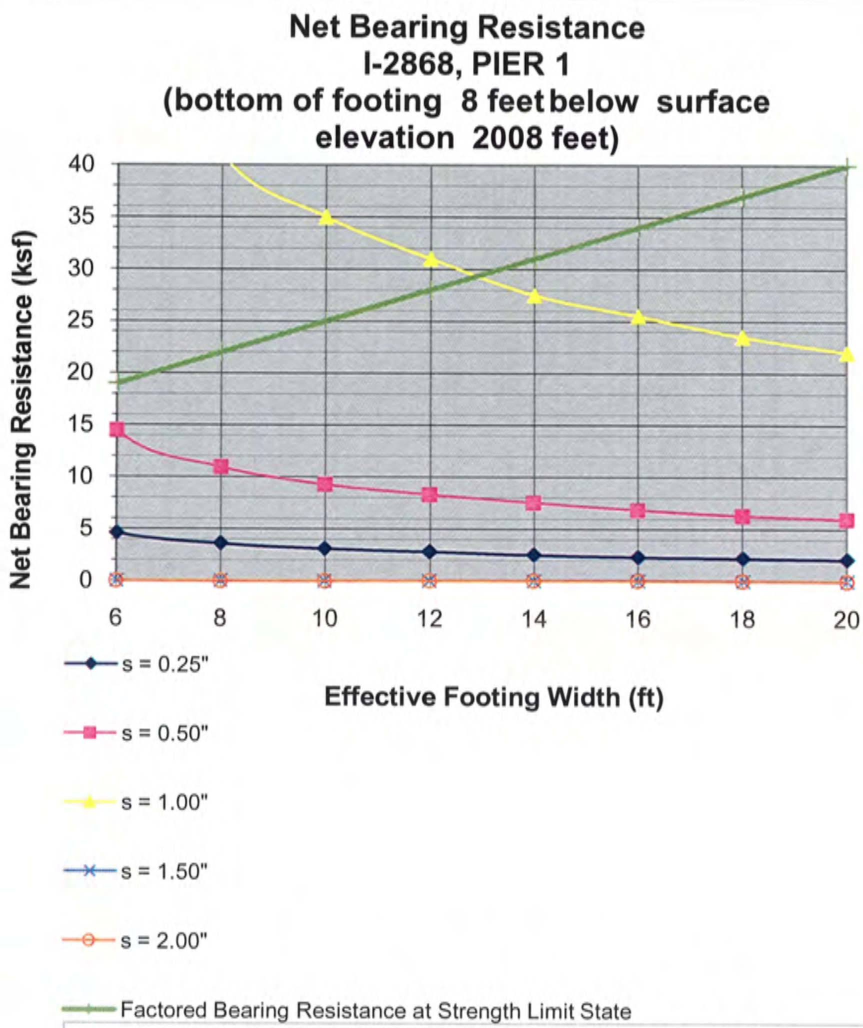
TABLE I

BEARING RESISTANCE OF SPREAD FOOTINGS WITHIN EMBANKMENT FILL WITH 2H:1V SLOPE

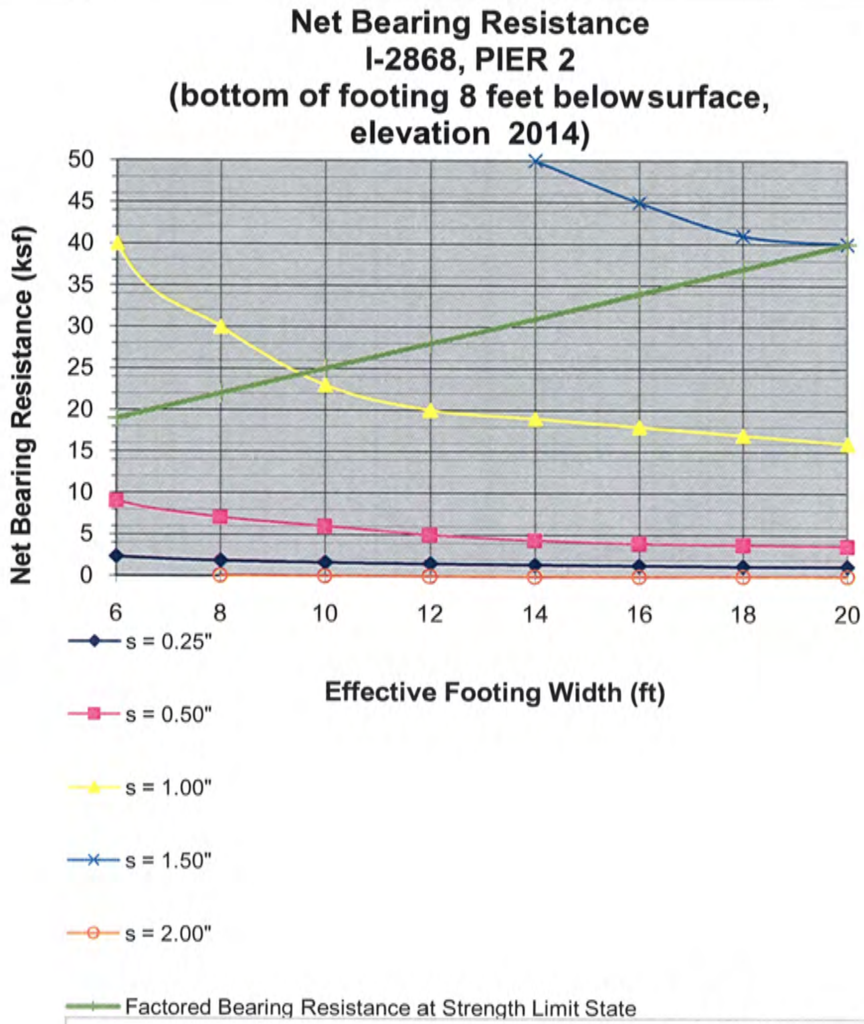
Limit State	**Presumptive Bearing Resistance (ksf)**	
	Nominal (q_n)	**Factored (q_R)**
Strength I	12.0 (AASHTO 10.6.3.1.2)	5.4 (resistance factor $\varphi_b = 0.45$) (AASHTO Table 10.5.5.2.2-1)
Service I (max. 1" settlement)	4.0 (AASHTO 10.5.5.1)	4.0 (resistance factor $\varphi_b = 1.0$) (AASHTO 10.5.5.1)
Extreme I-EQ	10.0 (AASHTO 10.6.4.1, 10.5.5.3.3)	10.0 (resistance factor $\varphi_b = 1.0$) (AASHTO 10.6.4.1, 10.5.5.3.3)

TABLE 2 BEARING RESISTANCE OF SPREAD FOOTINGS IN NATIVE GROUND (minimum footing embedment depth of 5 feet)		
Limit State	**Bearing Resistance (ksf)**	
	Nominal Net (q_{nn})	**Factored Net (q_{Rn})**
Strength I	Refer to the design charts for q_{Rn} $q_{nn} = q_{Rn} / 0.45$ (AASHTO 10.6.3.1.2)	Refer to the design charts for q_{Rn} (Resistance Factor $\varphi_b = 0.45$) (AASHTO Table 10.5.5.2.2-1)
Service I	Refer to the design charts for q_{Rn} $q_{nn} = q_{Rn}$ (AASHTO 10.5.5.1)	Refer to the design charts for q_{Rn} (Resistance Factor $\varphi_b = 1.0$) (AASHTO 10.5.5.1)
Extreme I-EQ	Refer to the design charts for q_{Rn} $q_{nn} = q_{Rn} / 0.45$ (AASHTO 10.6.41, 10.5.5.3.3)	Refer to the design charts for q_{Rn} $q_{nn} = q_{Rn} / 0.45$ (Resistance Factor $\varphi_b = 1.0$) (AASHTO 10.6.41, 10.5.5.3.3)

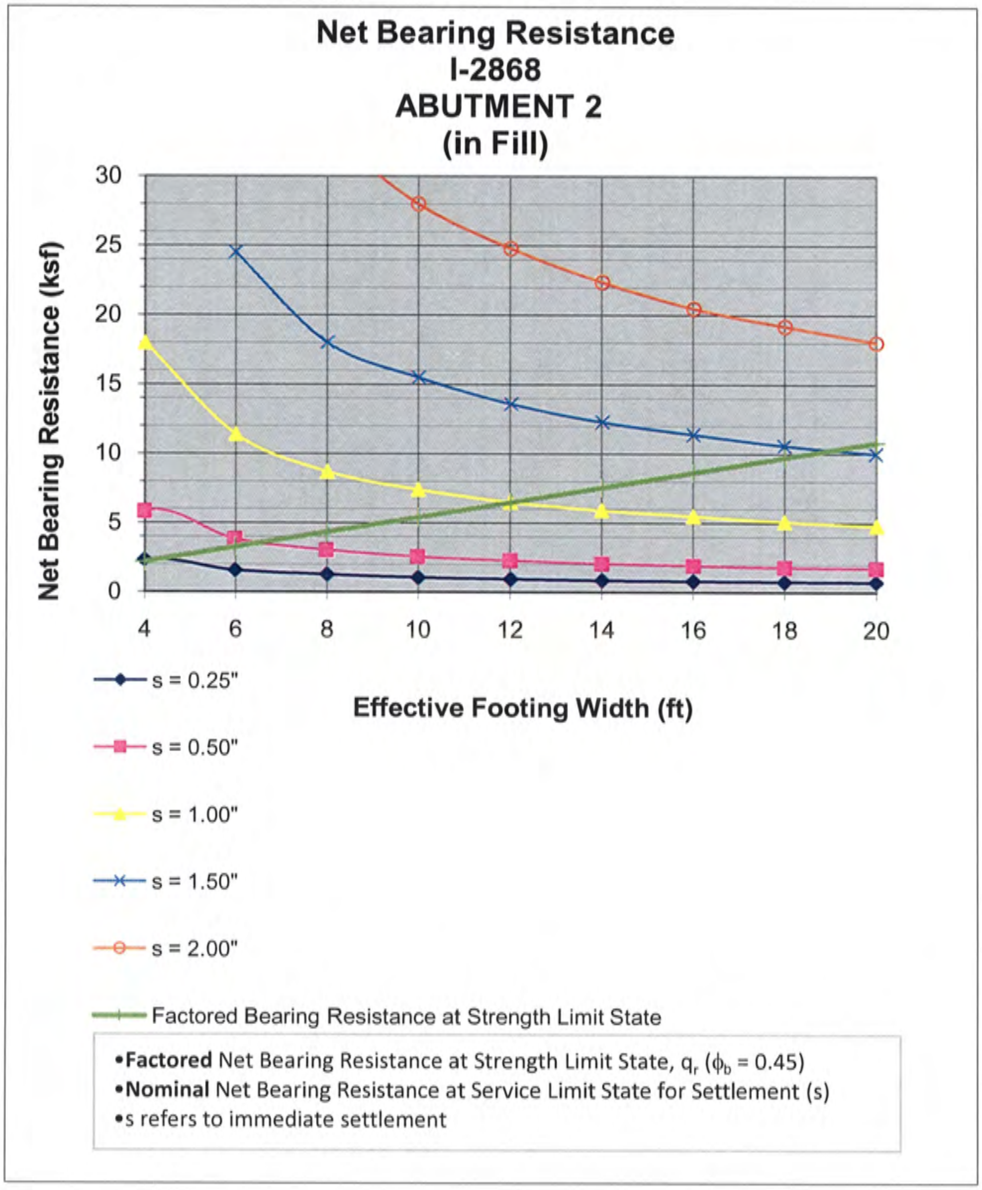

Net Bearing Resistance
I-2868
ABUTMENT 1
(in Fill)
Net Bearing Resistance (ksf)
20
18
16
14
12
10
8
6
4
2
0
4
6
8
10
12
14
16
18
20
Effective Footing Width (ft)
s = 0.25"
s = 0.50"
s = 1.00"
s = 1.50"
s = 2.00"
Factored Bearing Resistance at Strength Limit State
•Factored Net Bearing Resistance at Strength Limit State, q_r (for ϕ_b = 0.45)
•Nominal Net Bearing Resistance at Service Limit State for Settlement (s)
•s refers to immediate settlement

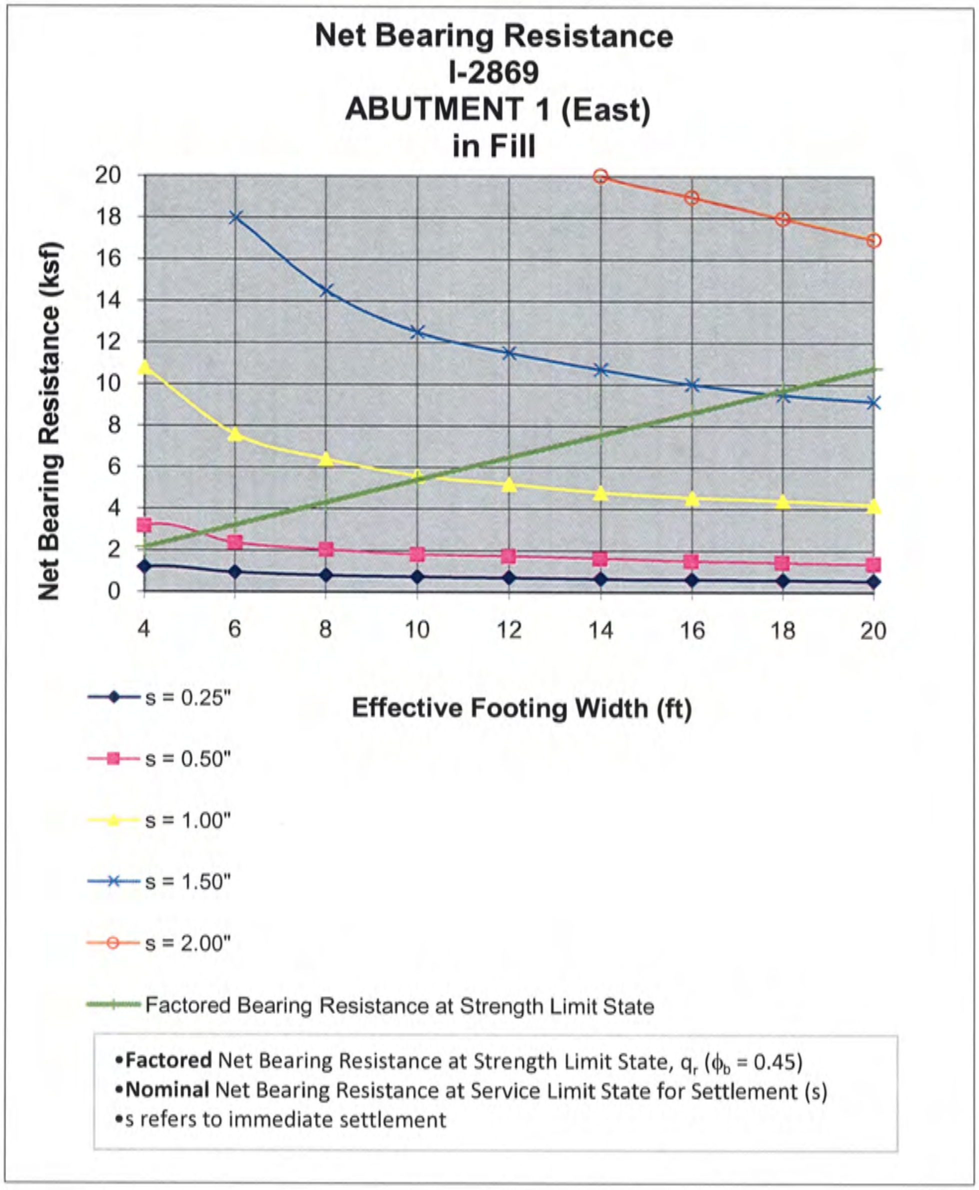


- **Factored** Net Bearing Resistance at Strength Limit State, q_r ($\phi_b = 0.45$)
- **Nominal** Net Bearing Resistance at Service Limit State for Settlement (s)
- s refers to immediate settlement



- **Factored** Net Bearing Resistance at Strength Limit State, q_r (ϕ_b = 0.45)
- **Nominal** Net Bearing Resistance at Service Limit State for Settlement (s)
- s refers to immediate settlement


Net Bearing Resistance
I-2868
ABUTMENT 2
(in Fill)
Net Bearing Resistance (ksf)
0
5
10
15
20
25
30
4
6
8
10
12
14
16
18
20
Effective Footing Width (ft)
s = 0.25"
s = 0.50"
s = 1.00"
s = 1.50"
s = 2.00"
Factored Bearing Resistance at Strength Limit State
•Factored Net Bearing Resistance at Strength Limit State, q_r (ϕ_b = 0.45)
•Nominal Net Bearing Resistance at Service Limit State for Settlement (s)
•s refers to immediate settlement

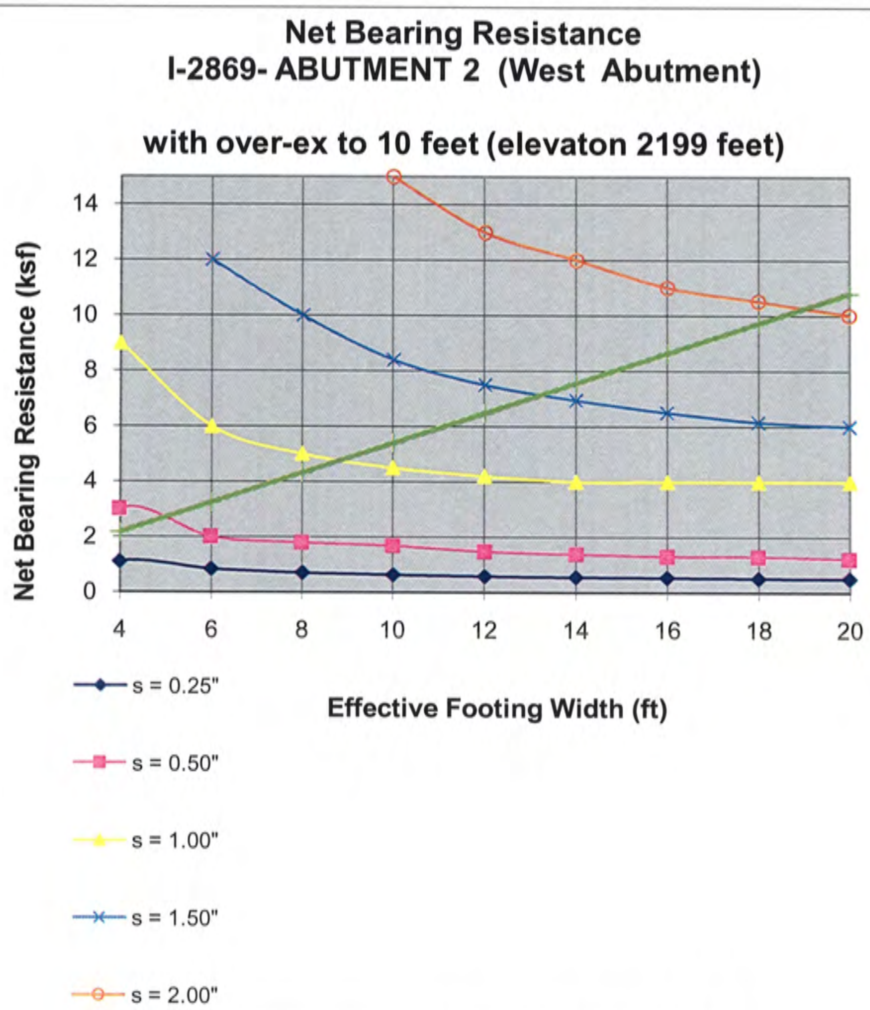


Net Bearing Resistance
I-2869
ABUTMENT 1 (East)
in Fill
Net Bearing Resistance (ksf)
20
18
16
14
12
10
8
6
4
2
0
4
6
8
10
12
14
16
18
20
Effective Footing Width (ft)
s = 0.25"
s = 0.50"
s = 1.00"
s = 1.50"
s = 2.00"
Factored Bearing Resistance at Strength Limit State
•Factored Net Bearing Resistance at Strength Limit State, q_r (ϕ_b = 0.45)
•Nominal Net Bearing Resistance at Service Limit State for Settlement (s)
•s refers to immediate settlement

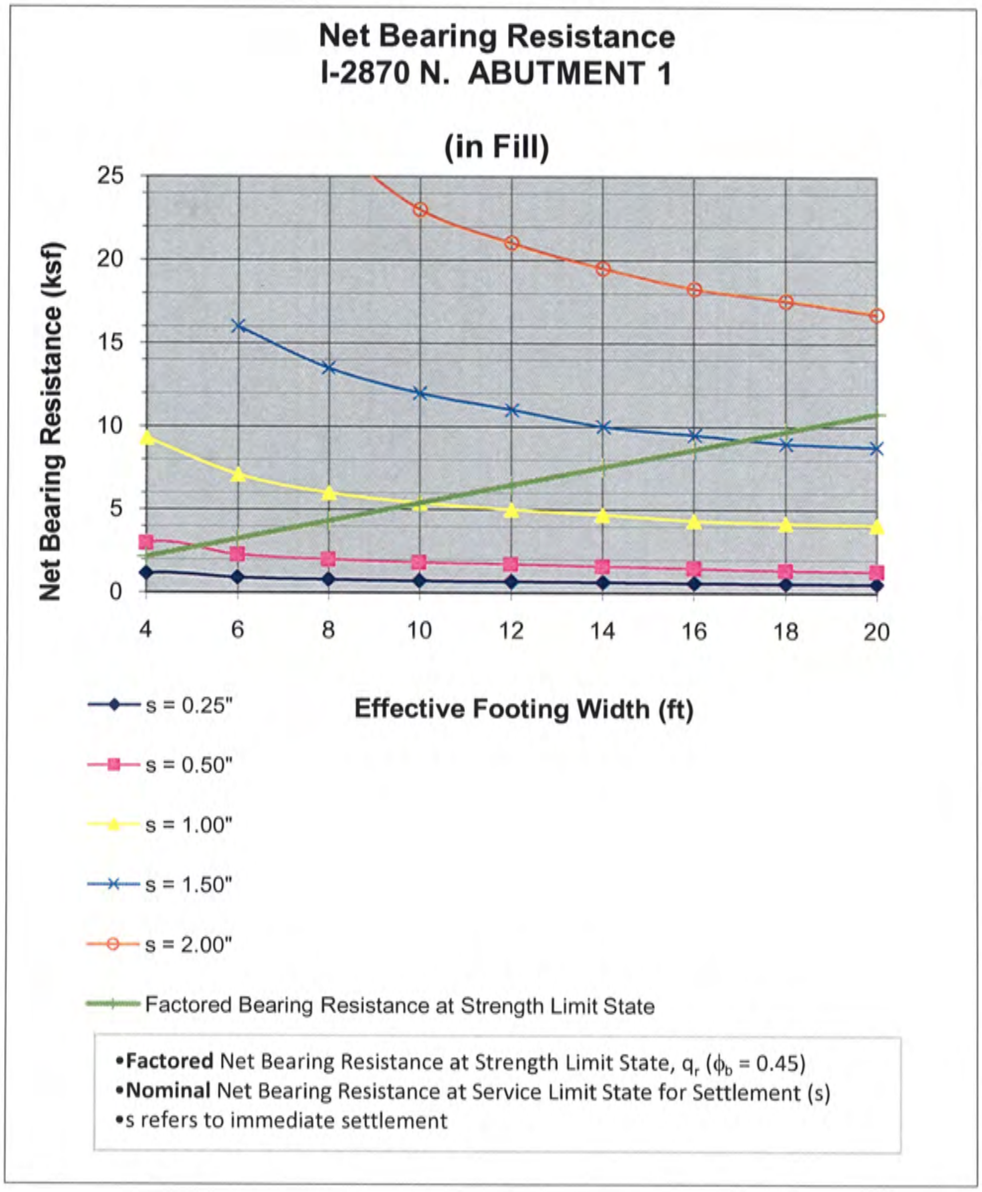
Bedrock Outcrop

Bridge I-2869, Center Pier Location

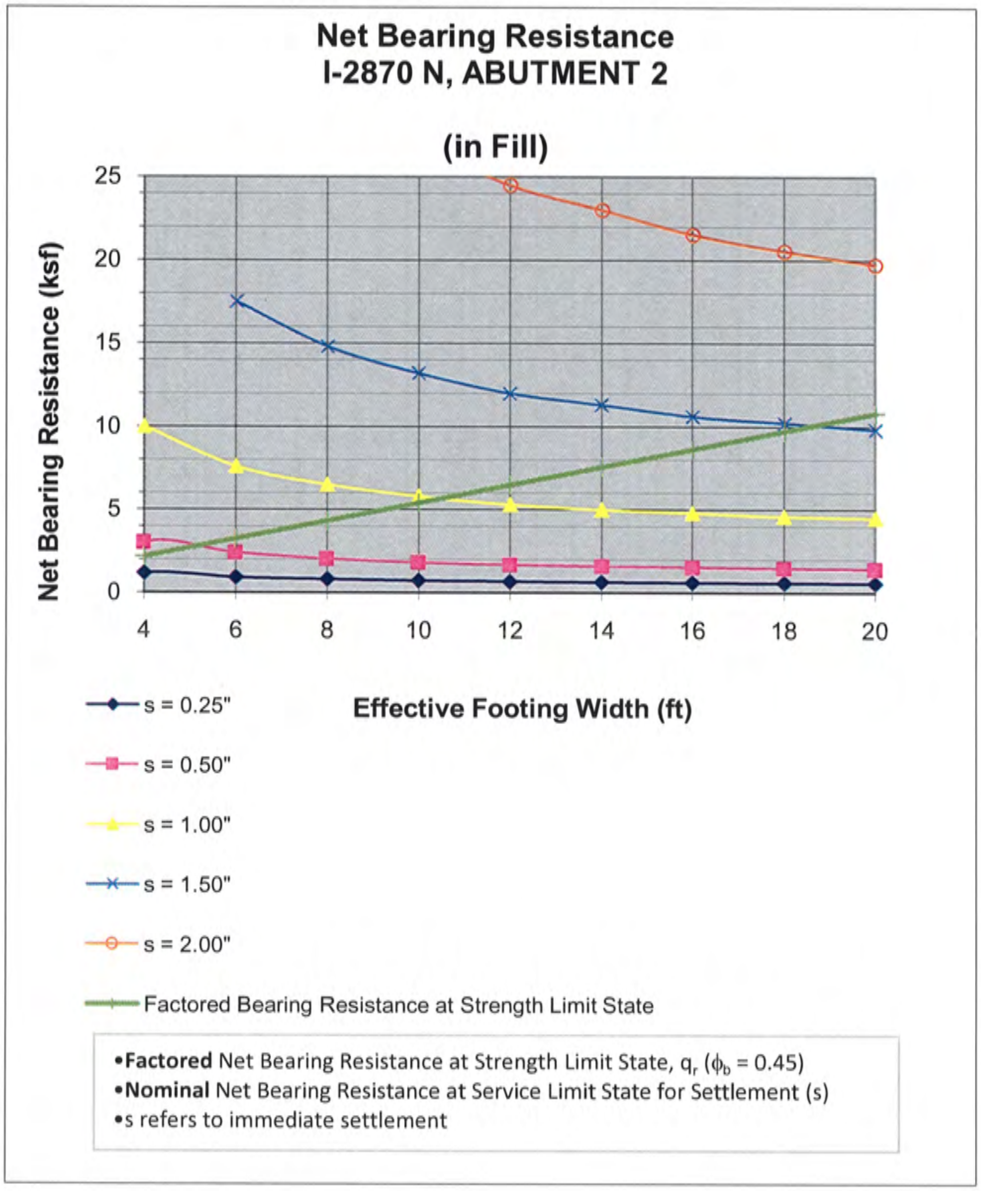
Approximate Location: Station "RR" 108+86 / Station "P" 152

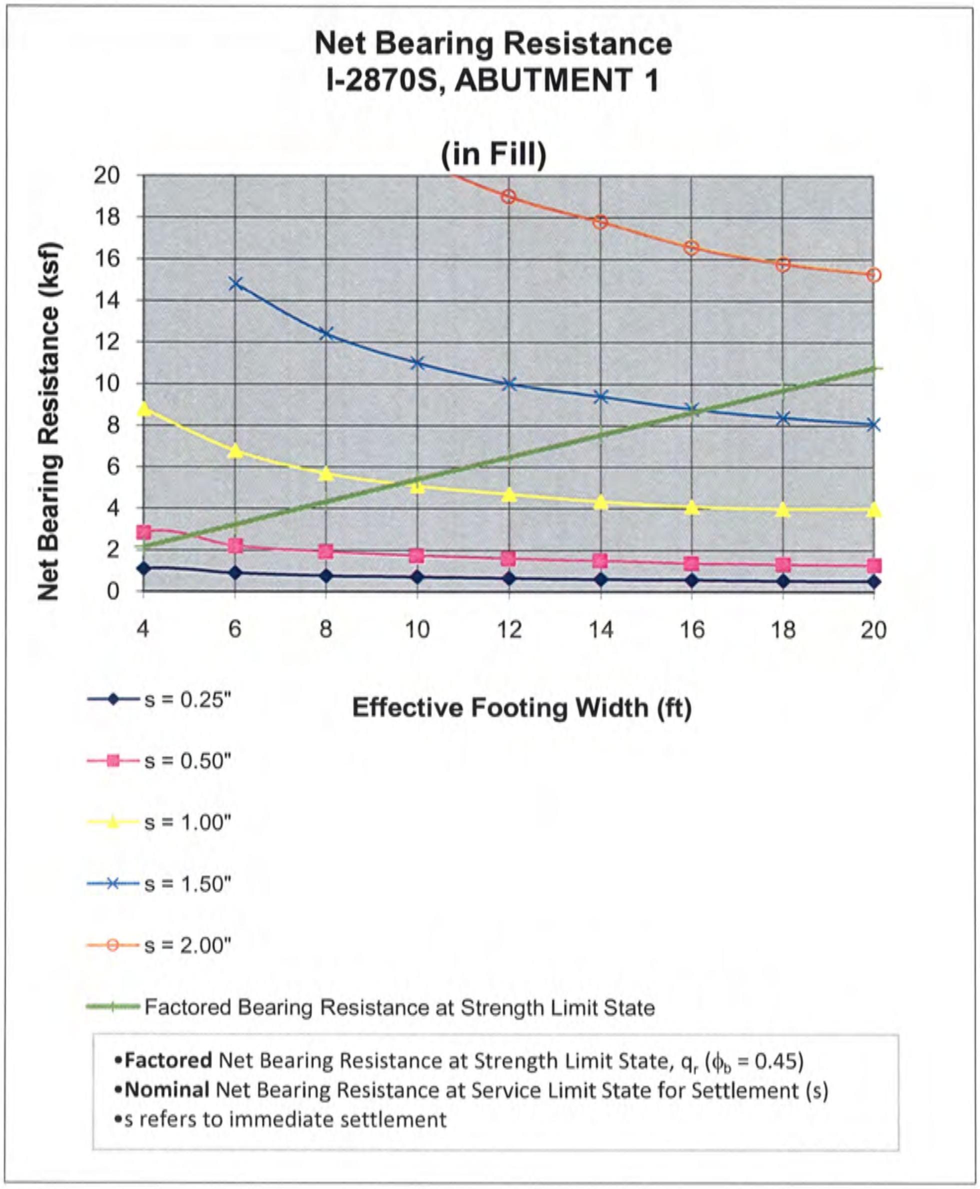


•**Factored** Net Bearing Resistance at Strength Limit State, q_r (ϕ_b = 0.45)
•**Nominal** Net Bearing Resistance at Service Limit State for Settlement (s)
•s refers to immediate settlement

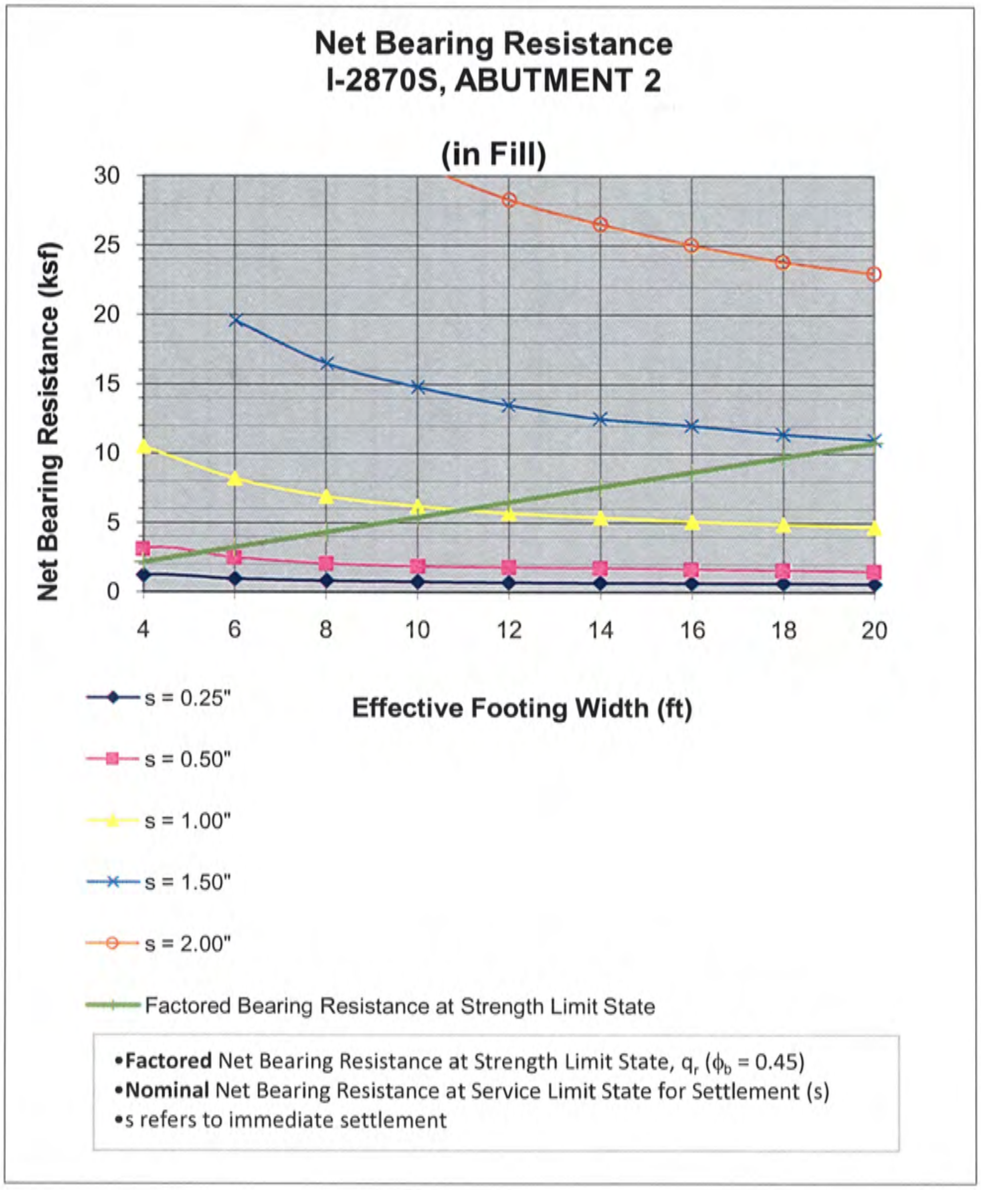


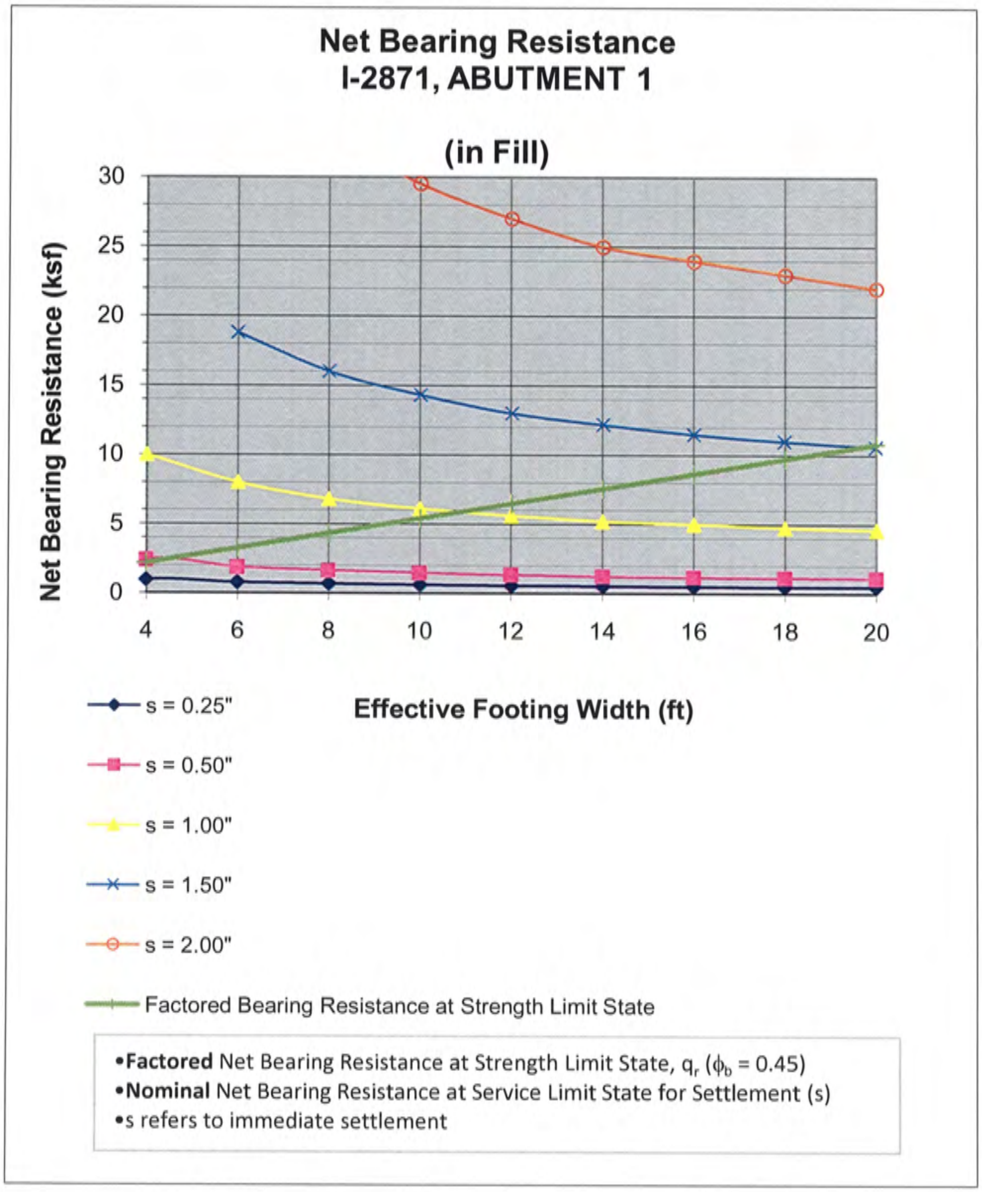
Net Bearing Resistance
I-2870 N. ABUTMENT 1
(in Fill)
Net Bearing Resistance (ksf)
25
20
15
10
5
0
4
6
8
10
12
14
16
18
20
Effective Footing Width (ft)
s = 0.25"
s = 0.50"
s = 1.00"
s = 1.50"
s = 2.00"
Factored Bearing Resistance at Strength Limit State
•Factored Net Bearing Resistance at Strength Limit State, q_r (ϕ_b = 0.45)
•Nominal Net Bearing Resistance at Service Limit State for Settlement (s)
•s refers to immediate settlement

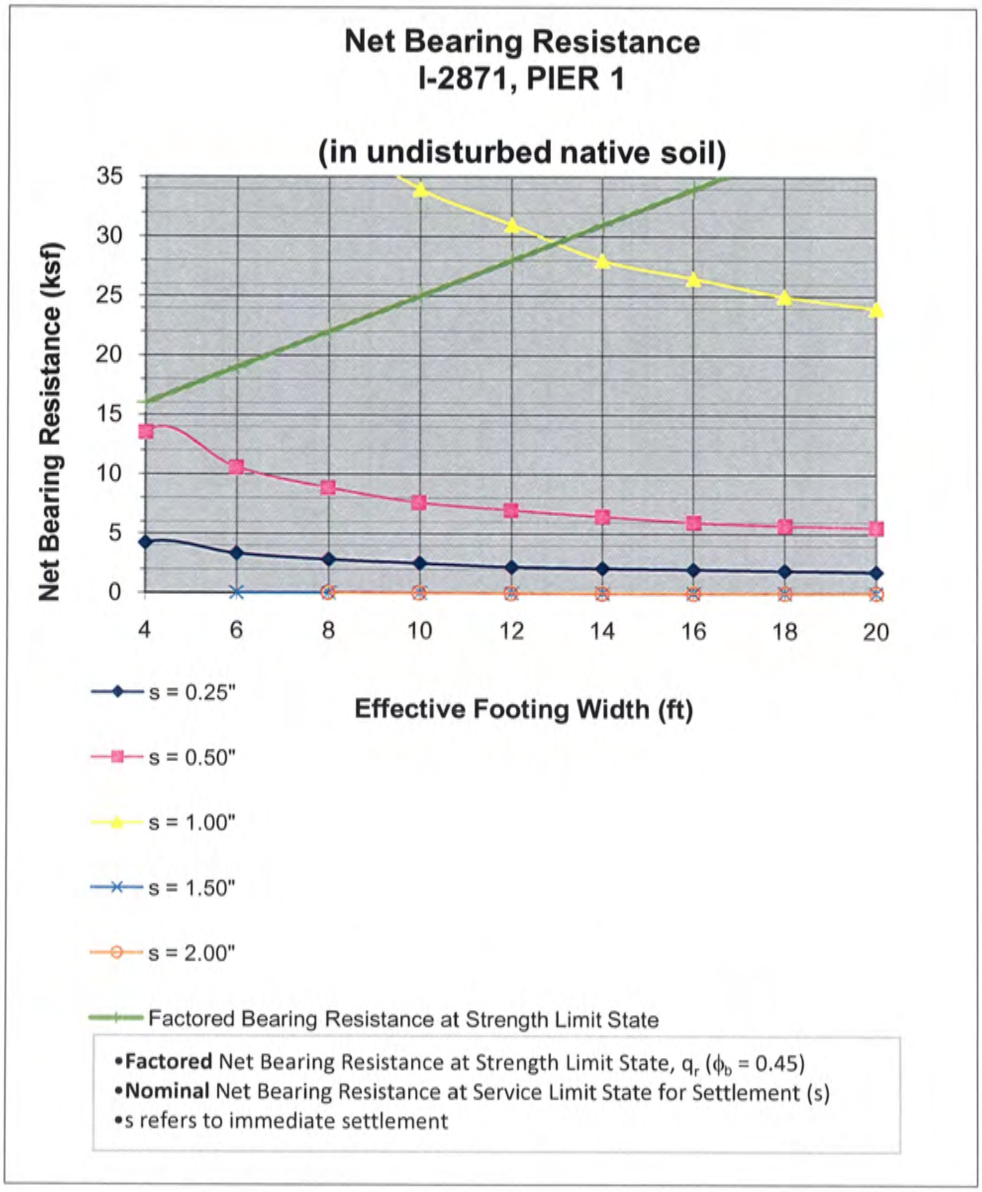

Net Bearing Resistance
I-2870 N, ABUTMENT 2
(in Fill)
Net Bearing Resistance (ksf)
Effective Footing Width (ft)
s = 0.25"
s = 0.50"
s = 1.00"
s = 1.50"
s = 2.00"
Factored Bearing Resistance at Strength Limit State
•Factored Net Bearing Resistance at Strength Limit State, q$_r$ (ϕ_b = 0.45)
•Nominal Net Bearing Resistance at Service Limit State for Settlement (s)
•s refers to immediate settlement



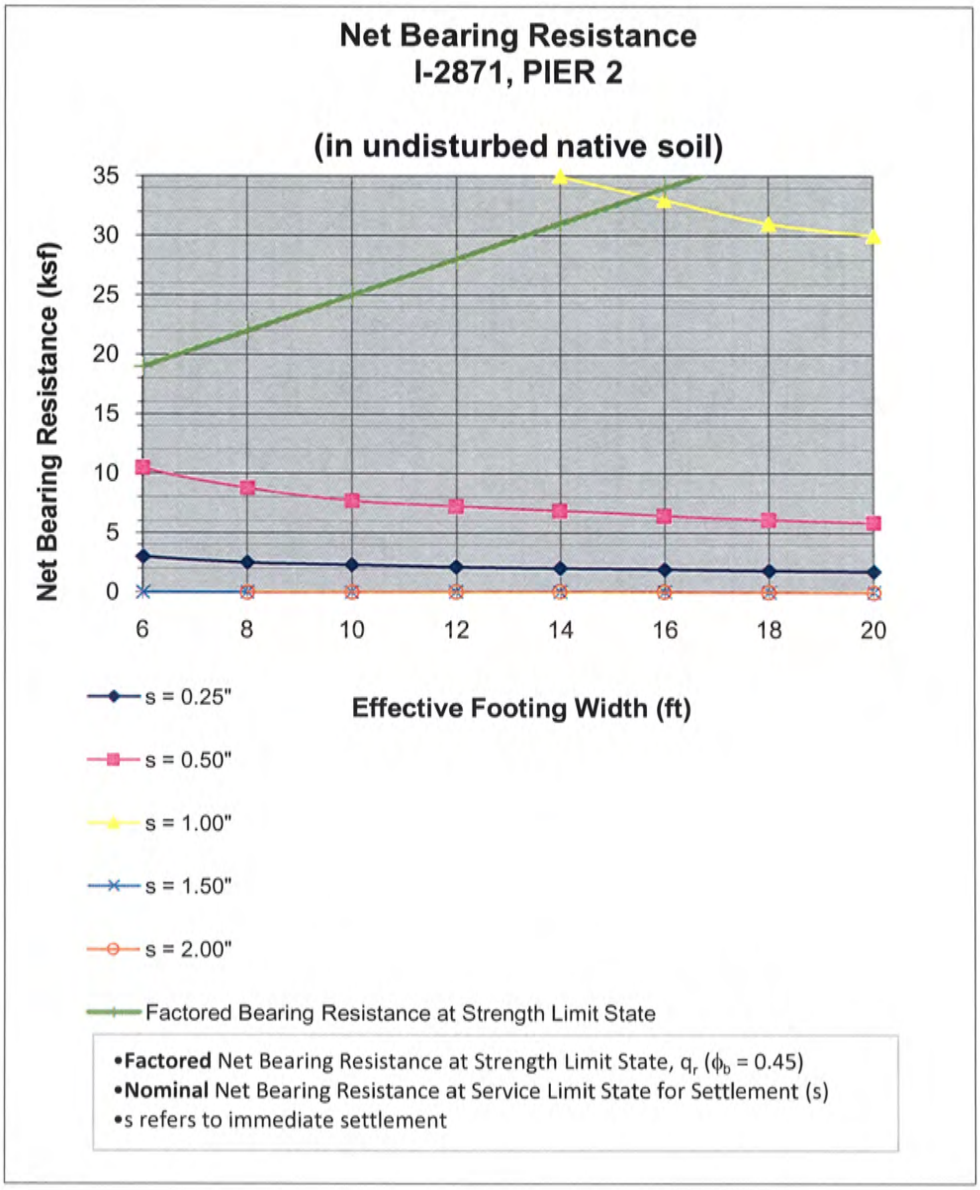
Net Bearing Resistance
I-2870S, ABUTMENT 1
(in Fill)
Net Bearing Resistance (ksf)
Effective Footing Width (ft)
s = 0.25"
s = 0.50"
s = 1.00"
s = 1.50"
s = 2.00"
Factored Bearing Resistance at Strength Limit State
•Factored Net Bearing Resistance at Strength Limit State, q_r (ϕ_b = 0.45)
•Nominal Net Bearing Resistance at Service Limit State for Settlement (s)
•s refers to immediate settlement


Net Bearing Resistance
I-2870S, ABUTMENT 2
(in Fill)
Net Bearing Resistance (ksf)
30
25
20
15
10
5
0
4
6
8
10
12
14
16
18
20
Effective Footing Width (ft)
s = 0.25"
s = 0.50"
s = 1.00"
s = 1.50"
s = 2.00"
Factored Bearing Resistance at Strength Limit State
•Factored Net Bearing Resistance at Strength Limit State, q_r (ϕ_b = 0.45)
•Nominal Net Bearing Resistance at Service Limit State for Settlement (s)
•s refers to immediate settlement

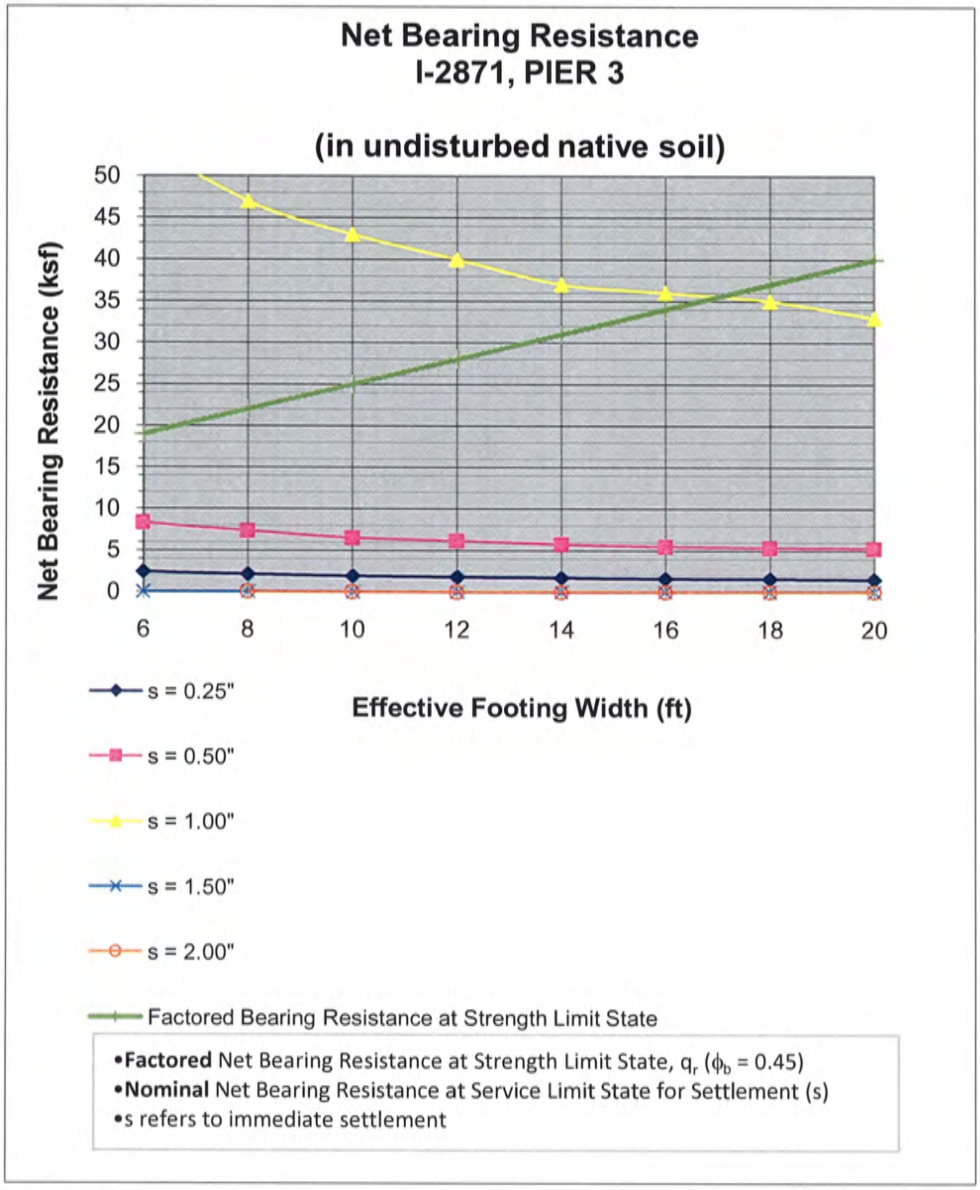

Net Bearing Resistance
I-2871, ABUTMENT 1
(in Fill)
Net Bearing Resistance (ksf)
30
25
20
15
10
5
0
4
6
8
10
12
14
16
18
20
Effective Footing Width (ft)
s = 0.25"
s = 0.50"
s = 1.00"
s = 1.50"
s = 2.00"
Factored Bearing Resistance at Strength Limit State
•Factored Net Bearing Resistance at Strength Limit State, q_r (ϕ_b = 0.45)
•Nominal Net Bearing Resistance at Service Limit State for Settlement (s)
•s refers to immediate settlement



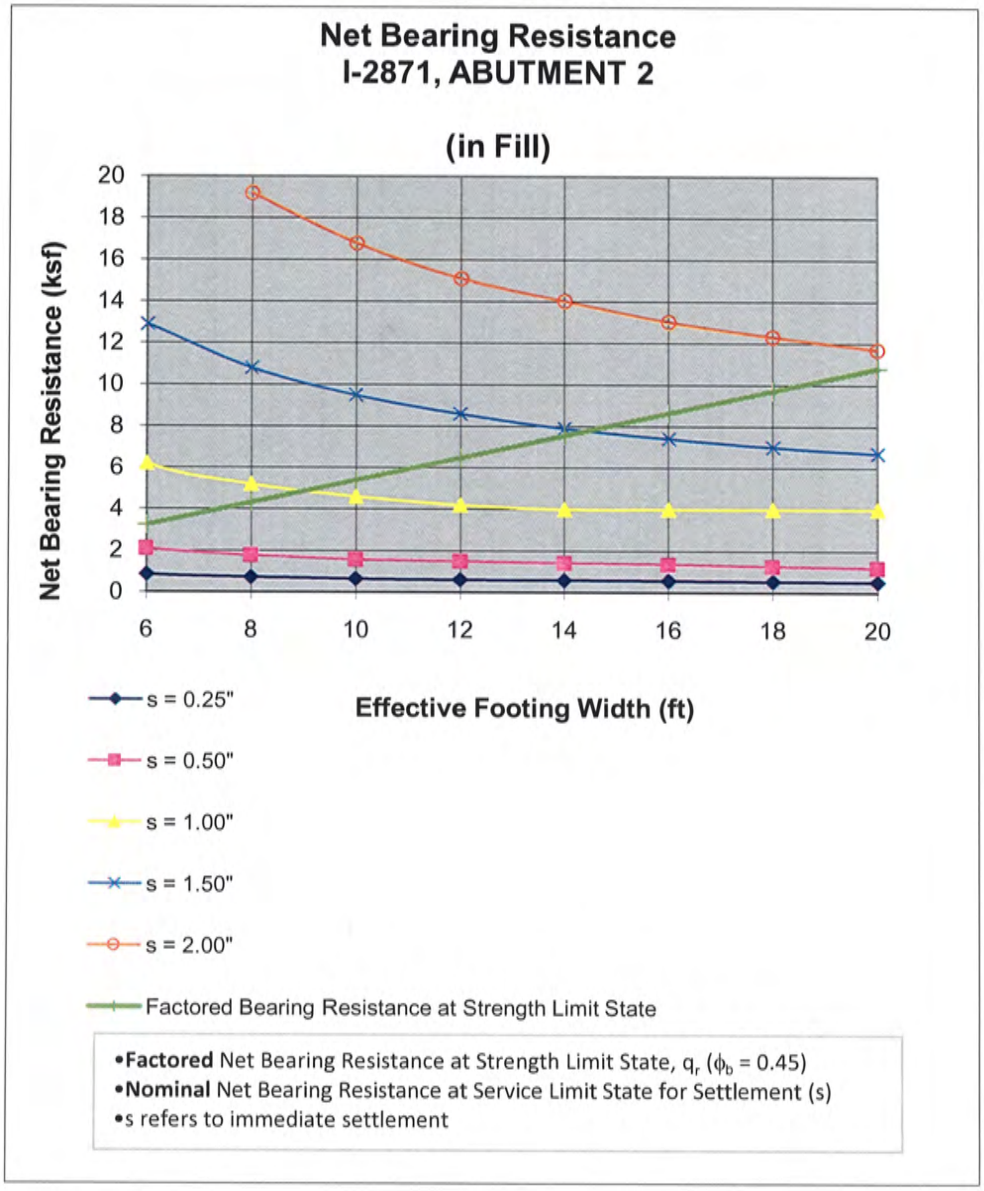
Net Bearing Resistance
I-2871, PIER 1
(in undisturbed native soil)
Net Bearing Resistance (ksf)
35
30
25
20
15
10
5
0
4
6
8
10
12
14
16
18
20
Effective Footing Width (ft)
s = 0.25"
s = 0.50"
s = 1.00"
s = 1.50"
s = 2.00"
Factored Bearing Resistance at Strength Limit State
•Factored Net Bearing Resistance at Strength Limit State, q_r (ϕ_b = 0.45)
•Nominal Net Bearing Resistance at Service Limit State for Settlement (s)
•s refers to immediate settlement



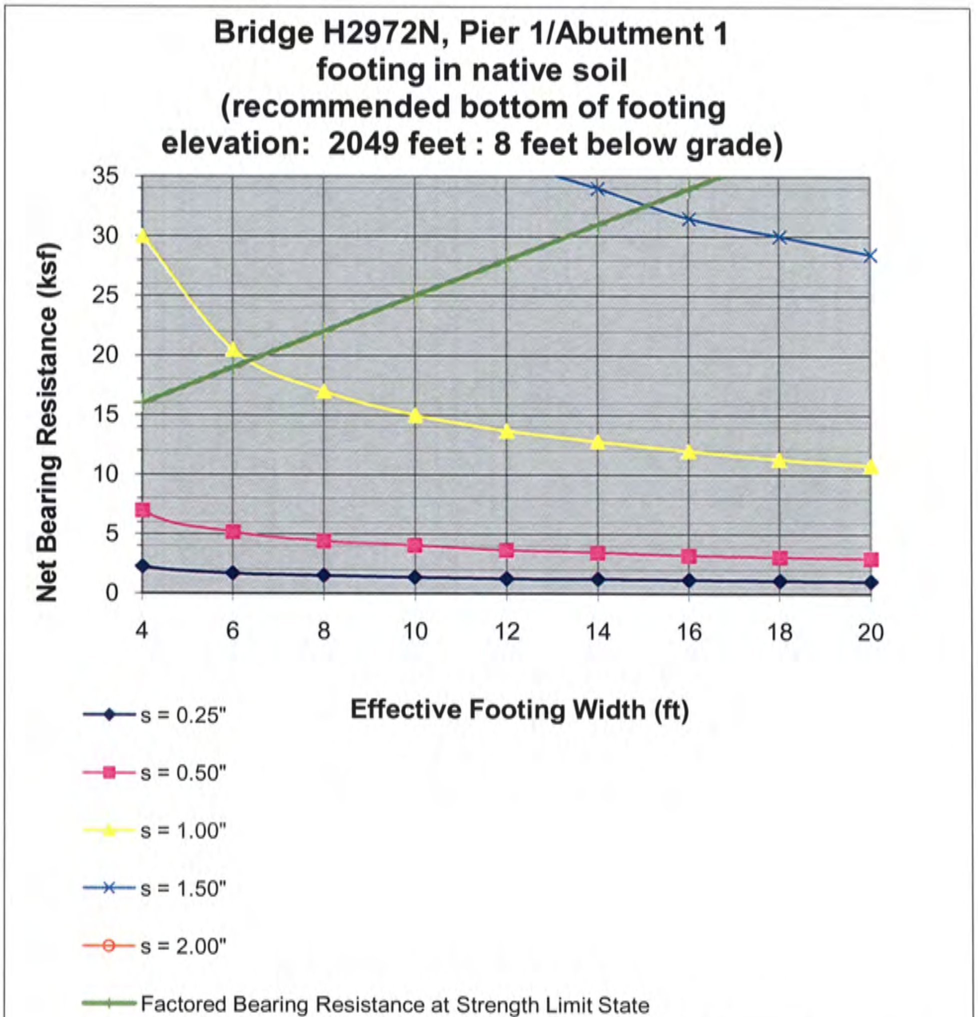
Net Bearing Resistance
I-2871, PIER 2
(in undisturbed native soil)
Net Bearing Resistance (ksf)
35
30
25
20
15
10
5
0
6
8
10
12
14
16
18
20
Effective Footing Width (ft)
s = 0.25"
s = 0.50"
s = 1.00"
s = 1.50"
s = 2.00"
Factored Bearing Resistance at Strength Limit State
•**Factored** Net Bearing Resistance at Strength Limit State, q_r (ϕ_b = 0.45)
•**Nominal** Net Bearing Resistance at Service Limit State for Settlement (s)
•s refers to immediate settlement



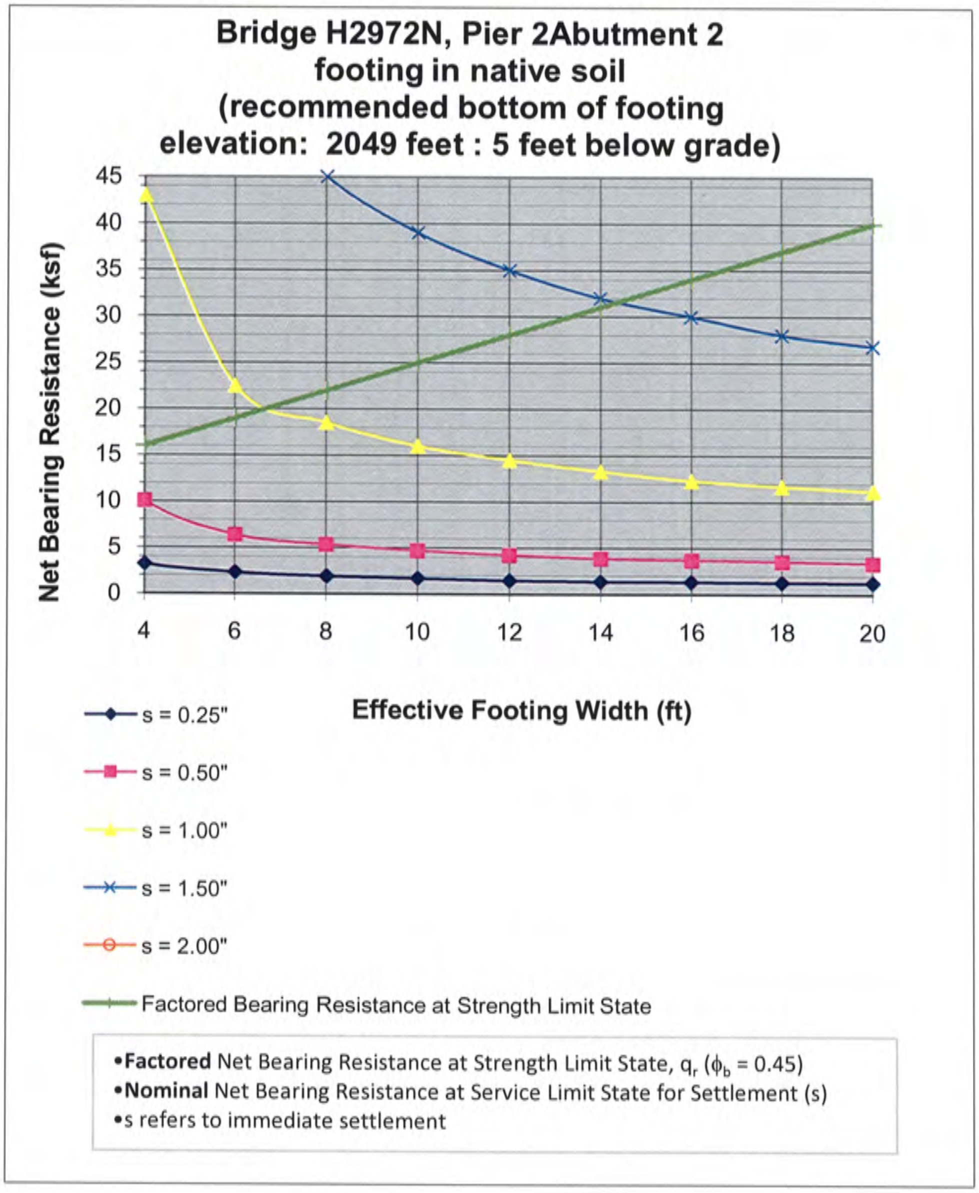
Net Bearing Resistance
I-2871, PIER 3
(in undisturbed native soil)
Net Bearing Resistance (ksf)
50
45
40
35
30
25
20
15
10
5
0
6
8
10
12
14
16
18
20
Effective Footing Width (ft)
s = 0.25"
s = 0.50"
s = 1.00"
s = 1.50"
s = 2.00"
Factored Bearing Resistance at Strength Limit State
•Factored Net Bearing Resistance at Strength Limit State, q_r (ϕ_b = 0.45)
•Nominal Net Bearing Resistance at Service Limit State for Settlement (s)
•s refers to immediate settlement

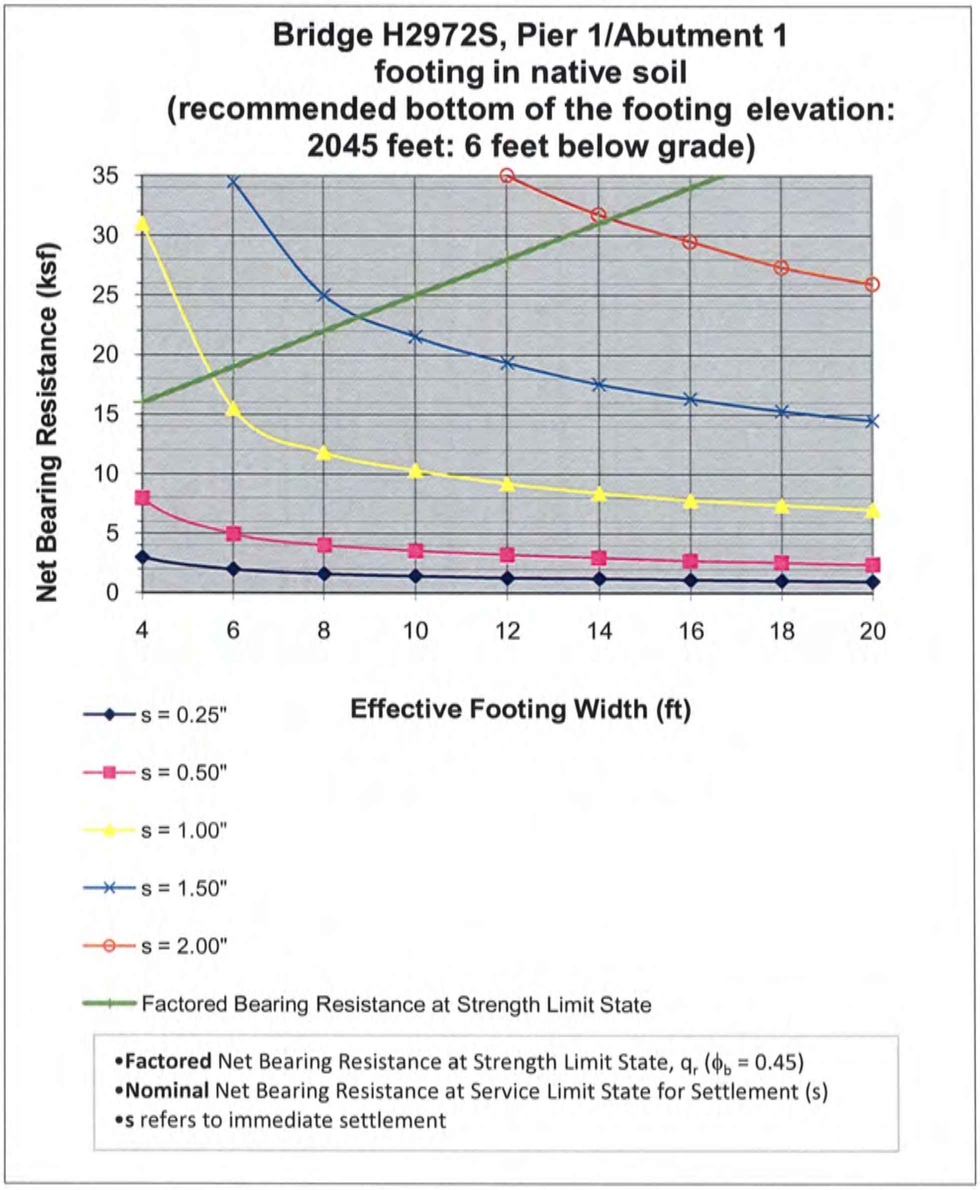


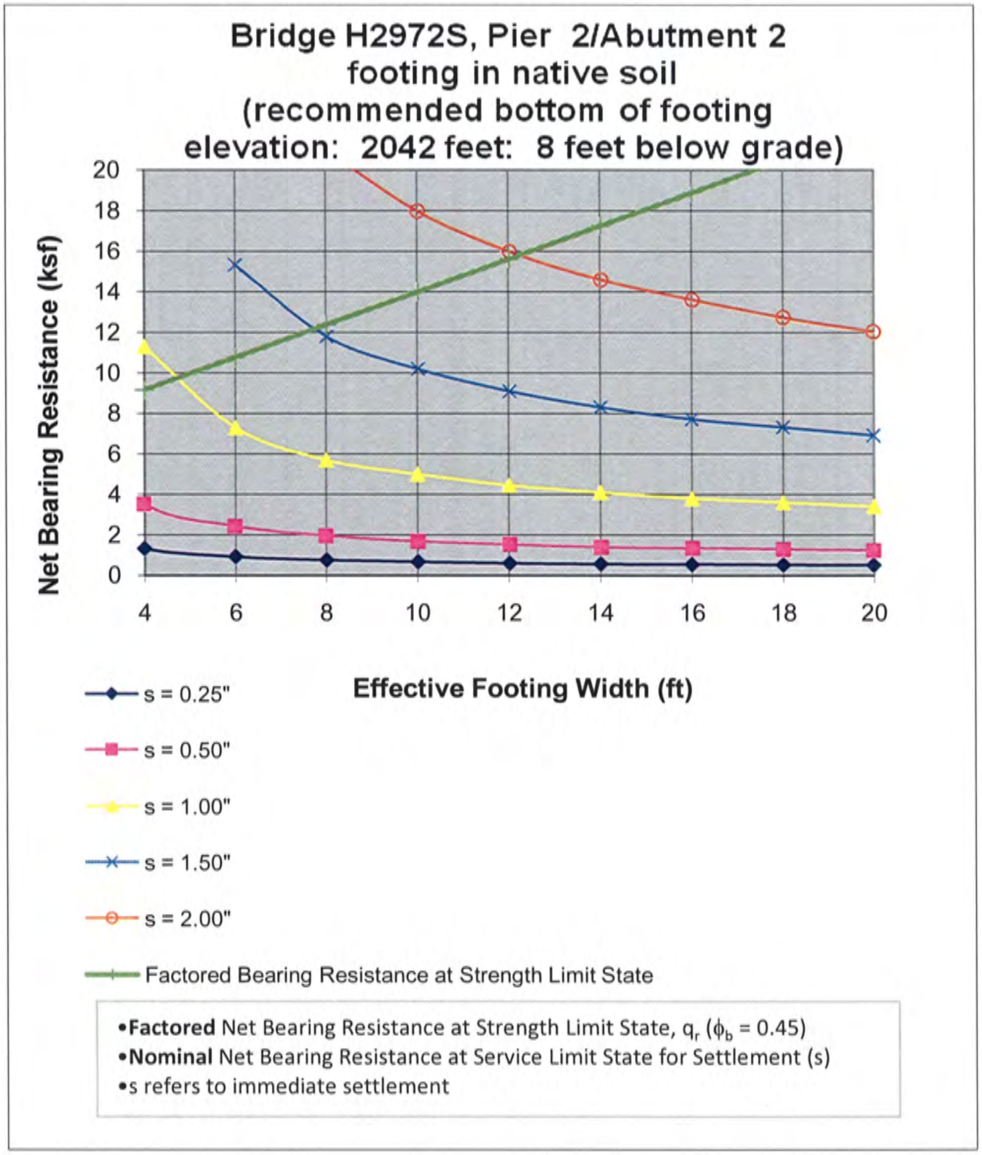
Net Bearing Resistance
I-2871, ABUTMENT 2
(in Fill)
Net Bearing Resistance (ksf)
20
18
16
14
12
10
8
6
4
2
0
6
8
10
12
14
16
18
20
Effective Footing Width (ft)
s = 0.25"
s = 0.50"
s = 1.00"
s = 1.50"
s = 2.00"
Factored Bearing Resistance at Strength Limit State
•**Factored** Net Bearing Resistance at Strength Limit State, q_r (ϕ_b = 0.45)
•**Nominal** Net Bearing Resistance at Service Limit State for Settlement (s)
•s refers to immediate settlement

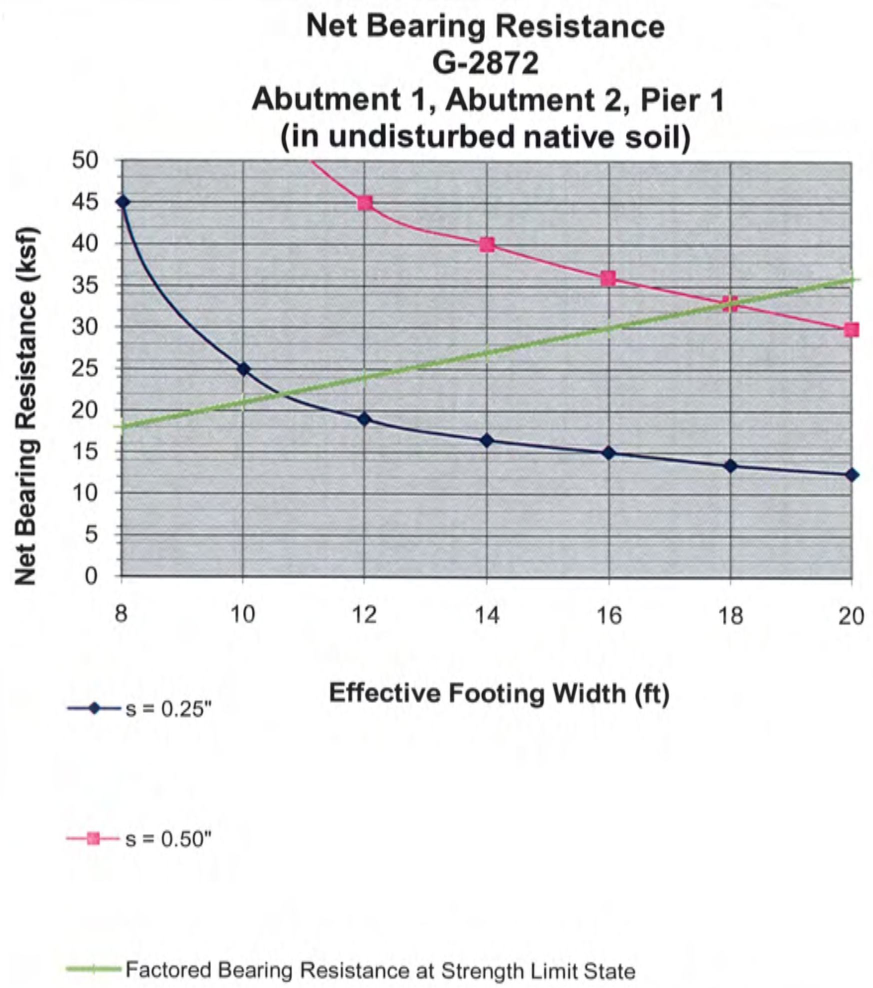


- **Factored** Net Bearing Resistance at Strength Limit State, q_r (ϕ_b = 0.45)
- **Nominal** Net Bearing Resistance at Service Limit State for Settlement (s)
- s refers to immediate settlement


Bridge H2972N, Pier 2Abutment 2
footing in native soil
(recommended bottom of footing
elevation: 2049 feet : 5 feet below grade)
Net Bearing Resistance (ksf)
45
40
35
30
25
20
15
10
5
0
4
6
8
10
12
14
16
18
20
Effective Footing Width (ft)
s = 0.25"
s = 0.50"
s = 1.00"
s = 1.50"
s = 2.00"
Factored Bearing Resistance at Strength Limit State
•Factored Net Bearing Resistance at Strength Limit State, q_r (ϕ_b = 0.45)
•Nominal Net Bearing Resistance at Service Limit State for Settlement (s)
•s refers to immediate settlement


Bridge H2972S, Pier 1/Abutment 1
footing in native soil
(recommended bottom of the footing elevation:
2045 feet: 6 feet below grade)
Net Bearing Resistance (ksf)
0
5
10
15
20
25
30
35
4
6
8
10
12
14
16
18
20
Effective Footing Width (ft)
s = 0.25"
s = 0.50"
s = 1.00"
s = 1.50"
s = 2.00"
Factored Bearing Resistance at Strength Limit State
•Factored Net Bearing Resistance at Strength Limit State, q_r (ϕ_b = 0.45)
•Nominal Net Bearing Resistance at Service Limit State for Settlement (s)
•s refers to immediate settlement


Bridge H2972S, Pier 2/Abutment 2
footing in native soil
(recommended bottom of footing
elevation: 2042 feet: 8 feet below grade)
Net Bearing Resistance (ksf)
0
2
4
6
8
10
12
14
16
18
20
4
6
8
10
12
14
16
18
20
Effective Footing Width (ft)
s = 0.25"
s = 0.50"
s = 1.00"
s = 1.50"
s = 2.00"
Factored Bearing Resistance at Strength Limit State
•Factored Net Bearing Resistance at Strength Limit State, q_r (ϕ_b = 0.45)
•Nominal Net Bearing Resistance at Service Limit State for Settlement (s)
•s refers to immediate settlement



- **Factored** Net Bearing Resistance at Strength Limit State, q_r (ϕ_b = 0.45)
- **Nominal** Net Bearing Resistance at Service Limit State for Settlement (s)
- s refers to immediate settlement

The estimates of the spread footing settlement bearing on cohesionless soil deposits are based on Hough method as recommended by the AASHTO LRFD Article 10.6.2.4.2. The location of the spread footings in fills was assumed to be at mid-height of the fill with end and side slopes of 2H:1V (horizontal: vertical).

Calculation of settlement within the structural fill due to the spread footings supported in fills requires an assumption about the compressibility of the approach embankment fill. To do so, an assumption was made about the representative SPT N-value ($N1_{60}$) of the compacted fill. SPT N-value ($N1_{60}$) of 32 blows per foot was assumed as a representative value for estimating settlement in the structural fill.

Based on successful NDOT experience, the Hough method tends to overestimate settlement of dense sands and gravels. For footings on sands and gravels with $N1_{60}$ greater than 30 blows per foot, a reduction of the estimated Hough settlement by a factor of 1.5 was considered.

The differential settlement between the bridge supports can be interpreted by using the appropriate provided design charts.

Most of the expected settlement due to dead loads will take place during construction.

Seismically-induced settlements of the pier footings on native soils will be negligible due to presence of dense to very dense soils. Seismically-induced settlements of the abutment footings on the properly compacted embankment fill are estimated to be less than ¼ of the total settlement.

The following table presents the summary of the recommended bottom of the footing depths or the over-excavation depths of the loose in-situ soils below certain structures footings and placing them back with proper compaction to minimize the settlement of the structures. Where no recommendation is made, there are no special geotechnical considerations and the Bridge Engineer will make the recommendations on the bottom of the footing elevations.

Structures	Recommended Over-Excavation Depths/ Bottom of footing Depths
STRUCTURE I-2868 Abutment #1 Pier 1 Pier 2 Abutment # 2	Areas under Abutment 1 (south) should be over-excavated to minimum depth of 8.0 feet (elevation 2001 feet) within the foot-print of the approach fill (due to presence of loose soil) and be re-compacted properly upon placement of backfill. Bottom of Pier 1 and Pier 2 footings should be placed at least 8.0 feet below the existing ground (elevations 2008 and 2014 feet respectively) due to presence of loose soil.
STRUCTURE I-2869 Abutment #1 Center Pier Abutment #2	Top of the pier footing should be at least 3 feet below the adjacent grade. Areas under Abutment No.2 should be over-excavated to a minimum depth of 10 feet (elevation 2199 feet) within the foot-print of the approach fill (due to presence of loose soil) and be re-compacted properly upon placement of backfill.
STRUCTURE I-2870S Abutment #1 Abutment #2	As recommended by the Bridge Engineer.
STRUCTURE I-2870N Abutment #1 Abutment #2	As recommended by the Bridge Engineer.

Structures	Recommended Over-Excavation Depths/ Bottom of footing Depths
STRUCTURE I-2871 Abutment #1 Pier 1 Pier 2 Pier 3 Abutment # 2	Top of the pier footings should be at least 3 feet below the adjacent grade.
Structure G-2872 Abutment #1 Pier #1 Abutment #2	Top of the pier footing should be at least 3 feet below the adjacent grade.
Structure H-2972N Abutment #1/Pier 1 Abutment #2/Pier 2	Abutment #1/Pier 1: Bottom of the footing at 8 feet (elevation 2049 feet) below the existing grade. Abutment #2/Pier 2: Bottom of the footing at 5 feet (elevation 2049 feet) below the existing grade.
Structure H-2972S Abutment #1/pier 1 Abutment #2/pier 2	Abutment #1/Pier 1: Bottom of the footing at 6 feet (elevation 2045 feet) below the existing grade. Abutment #2/Pier 2: Bottom of the footing at 8 feet (elevation 2042 feet) below the existing grade.

5.3 LATERAL EARTH PRESSURES

5.3.1 Abutments, Wingwalls, and Conventional Retaining Walls

Lateral earth pressures are determined based on the characteristics of a wall such as free-standing abutments and yielding (un-restrained) retaining walls (AASHTO A11.1.1), monolithic abutments (AASHTO A11.1.2), and non-yielding abutments and retaining walls which are restrained against lateral movement by tiebacks or batter piles (AASHTO A11.1.1.3). The different states of stress that should be considered include at-rest, active, passive, and seismic.

The design of walls requires consideration of both static and seismic conditions.

Compaction of backfill material within the vicinity of the walls by heavy equipment may result in development of lateral pressures greater than the design condition. Therefore, no heavy static or vibratory compaction equipment should be allowed within a distance of one-half of the wall height behind the walls during construction, unless the walls are designed structurally for this additional lateral loading.

The passive pressure is generally ignored in front of the walls in stability computations (AASHTO LRFD Article 11.6.3.5 - Passive Resistance).

The live load surcharge (LS) and its equivalent height (in feet) of soil for vehicular load (h_{eq}) shall be based on AASHTO LRFD Article 3.11.6.4. The unit weight for this imaginary surcharge is typically taken as 0.120 kcf. The pressure distribution from this equivalent soil height is uniform along the back of the wall, and the resultant is halfway up from the base of the wall.

Wingwalls may either be designed as monolithic with the abutments (non-yielding wall), or be separated from the abutment wall with an expansion joint and designed to be free-standing (yielding wall).

5.3.1.1 Free-Standing Abutments and Yielding Retaining Walls

The NDOT Structural Division has stated that bridges I-2868, I-2871, G-2872, and H-2972N/S will have free-standing (seat-type) abutments.

For external stability analysis of free-standing abutments and yielding retaining walls under static condition, lateral active earth pressure is recommended, provided the abutment/retaining wall can rotate or translates outward from the retained soil mass. The amount of movement (movement = deflection = Δ/H) to develop the minimum active pressure is a function of the wall height and the retained soil type (AASHTO Table C3.11.1-1). This required movement is very small, typically 0.001 to 0.01 times the exposed wall height. For NDOT granular backfill, this minimum required movement is about 0.002.

For external stability analysis under seismic condition, the pseudo-static Mononobe-Okabe (M-O) method of analysis is recommended for computing the combined static plus seismic lateral active soil pressures, addressed in the following page.

AASHTO LRFD A11.1.1.2 states that the horizontal acceleration coefficient $K_h = 0.5A_{max}$ can be used if an allowance is made for an outward displacement of the wall of up to 10 times the maximum earthquake acceleration A_{max} (1.5 inches for this project site).

Passive pressure resistance will develop behind the free-standing abutment when the seismic displacement is larger than the expansion gap between the bridge superstructure and the back wall. When the seismic displacement at the top of the free-standing abutment exceeds the gap between the superstructure and the abutment back wall, the analysis and design methods of monolithic abutments shall be used.

Following table presents the summary of the geotechnical design parameters that can be used in the external stability analyses of the free-standing abutments and yielding retaining walls under both static and seismic conditions:

BOULDER CITY BYPASS

NEVADA DOT GEOTECHNICAL ENGINEERING

LATERAL EARTH PRESSURE DESIGN PARAMETERS

FREE-STANDING ABUTMENTS & YIELDING RETAINING WALLS

(required minimum wall deflection: AASHTO Table C3.11.1-1)
(with no build-up of hydrostatic pressure)

Passive resistance in front of the wall should not be used in the analyses. Additional resistance to sliding (if required) may be achieved through the use of a shear key or increasing the size of the footing.

AASHTO A11.1.1.2 Design for Displacement states, "$K_h = ½ A_s$ is adequate for most design purposes, provided that allowance is made for an outward displacement of abutment of up to $10A_s$ (in inches) ."

Where heavy static and dynamic compaction equipment is used within a distance of one-half the wall height behind the wall, the effect of additional earth pressure that may be induced by compaction shall be taken into account.

Static Active Earth Pressure Coefficient = $K_A = 0.278$ (Coulomb's equation for $\delta/\phi_f = 0.5$)
Static Active Earth Pressure = $K_A \gamma H$
Static Active Earth Force by the Driving Wedge = $½ K_A \gamma H^2$; (located at 1/3 from the bottom of the wall)

Static Passive Earth Pressure Coefficient = $K_P = 8.02$ (Caquot and Kerisel for $\delta/\phi_f = 0.5$).
Static Passive Earth Pressure = $K_p \gamma D$; D = depth of wall embedment
Static Passive Force by the Resisting Wedge = Nominal Passive Resistance = $½ K_p \gamma D^2$

(Static + Seismic) Active Earth Pressure Coefficient = K_{AE} (Mononobe and Okabe):
$K_{AE} = 0.379$ for $K_h = A_s = 0.15g$
$K_{AE} = 0.325$ for $K_h = ½ A_s = 0.075g$
(Static + Seismic) Active Earth Pressure = $K_{AE} \gamma H$
(Static + Seismic) Active Earth Force by the Driving Wedge = $½ K_{AE} \gamma H^2$
Seismic Incremental Component of active earth Pressure Coefficient, $\Delta K_{AE} = K_{AE} - K_A$
Seismic incremental component of active earth Pressure = $\Delta K_{AE} \gamma H$ (inverted triangular distribution)
AASHTO A11.1.1- Free Standing Abutments states, "Static component of soil force acts at H/3 from bottom of the abutment, while the additional dynamic effect should be taken to act at a height of 0.6H. For most purposes it is sufficient to assume the location of resultant forces at H/2, with a uniform distributed pressure."

(Static + Seismic) Passive Earth Passive Coefficient (Mononobe and Okabe):
$K_{PE} = 7.1$ for $K_h = 0.15g$
$K_{PE} = 7.56$ for $K_h = 0.075g$
(Static + Seismic) Passive Earth Pressure = $K_{PE} \gamma D$, D=depth of footing embedment in native soil
(Static + Seismic) Passive Earth Force by the Resisting Wedge = $½ K_{PE} \gamma D^2$

Abutment Backfill Internal Friction Angle (ϕ_f)	32 degrees
Foundation Native Soil Internal Friction Angle (ϕ_s)	36 degrees
Backfill, Native Soil Unit Weight (γ_f)	0.120 kcf
Interface Friction Coefficient (tan δ) for concrete cast against soil Interface Friction Coefficient (tan δ) for precast concrete against soil	tan δ = 0.50 tan δ = 0.40
Resistance Factor for Sliding Resistance between Soil and Foundation (φ_τ) (AASHTO 10.5.5.2.2-1)	0.80
Resistance Factor for Passive Earth Pressure Component of Sliding Resistance (φ_{ep}) (AASHTO Table 10.5.5.2.2-1)	0.50
Resistance factor (φ) for Extreme Limit State (Seismic): (AASHTO 11.6.5)	1.0

5.3.1.2 Monolithic Abutments

The NDOT Structural Division has stated that bridges I-2869, I-2870S, and I-2870N will have monolithic abutment walls (also called integral or end-diaphragm abutments).

This type of abutment is cast monolithically with the superstructure and does not deflect sufficiently to develop an active wedge in the backfill soil. Therefore, At-Rest value of lateral earth pressure should be used for this type of abutment under static condition. Under seismic condition, the seismic passive earth pressure (P_{PE}) is controlling.

During a seismic event, higher longitudinal and transverse superstructure inertia forces are transmitted directly into the backfill. As a guide, abutment forces are considered excessive if the corresponding effective stress in the soil behind the abutment exceeds 5 ksf (FHWA Seismic Design: Report No. FHWA-IP-87-6). Also, it is recommended that abutments be proportioned to restrict displacements to 3.5 inches or less in order to minimize damage (AASHTO A11.1.2 – Monolithic Abutments).

The following table presents the summary of the geotechnical design parameters that can be used in the external stability analyses of the monolithic abutments:

LATERAL EARTH PRESSURE DESIGN PARAMETERS

MONOLITHIC ABUTMENTS

(with no build-up of hydrostatic pressure)

Passive resistance in front of the wall should not be used in the analyses. Additional resistance to sliding (if required) may be achieved through the use of a shear key or increasing the size of the footing.

AASHTO A11.1.2 - Monolithic Abutments states, "…In seismic design of monolithic abutments, provision must be made for adequate passive resistance to avoid excessive relative displacement." And "It is recommended that abutments be proportioned to restrict displacement to 3.5 in. or less in order to minimize damage.

During a seismic event, the monolithic abutment walls are pushed into the backfill. Therefore, the seismic passive earth pressure (P_{PE}) is controlling. As a guide, abutment forces are considered excessive if the corresponding effective stress in the soil behind the abutment exceeds 5 ksf (FHWA Seismic Design: Report No. FHWA-IP-87-6).

Where heavy static and dynamic compaction equipment is used within a distance of one-half the wall height behind the wall, the effect of additional earth pressure that may be induced by compaction shall be taken into account.

Static At-Rest Earth Pressure Coefficient, $K_0 = 1\text{-}\sin \phi_f = 0.47$
Static At-Rest Earth Pressure = $K_0 \gamma H$; H = wall height
Total Static At-Rest Earth Force = $\frac{1}{2} K_0 \gamma H^2$; (located at 1/3 from the bottom of the wall)

Static Passive Earth Pressure Coefficient, $K_P = 7.5$ (Caquot and Kerisel for $\delta/\phi_f = 0.5$).
Passive Earth Pressure = $K_p \gamma H$; wall height (H) if wall pushed into the soil or D = depth of embedment.
Total Static Passive Force = Nominal Passive Resistance = $\frac{1}{2} K_p \gamma H^2$

Seismic Design of Monolithic (Integral) Abutments
(Static + Seismic) Passive Earth Pressure Coefficient, $K_{PE} = 4.51$ for $K_h = A_s = 0.15g$.
Seismic Passive Earth Pressure (P_{PE}) = $K_{pE} \gamma H$

Abutment Backfill Internal Friction Angle (ϕ_f); assumed	32 degrees
Foundation Native Soil Internal Friction Angle (ϕ_s)	36 degrees
Backfill, Native Soil Unit Weight (γ_f)	0.120 kcf
Interface Friction Coefficient (tan δ) for concrete cast against soil Interface Friction Coefficient (tan δ) for precast concrete against soil	$\tan \delta = 0.50$ $\tan \delta = 0.40$
Resistance Factor for Sliding Resistance between Soil and Foundation (φ_τ) (AASHTO 10.5.5.2.2-1)	0.80
Resistance Factor for Passive Earth pressure Component of Sliding Resistance (φ_{ep}) (AASHTO Table 10.5.5.2.2-1	0.50
Resistance factor (φ) for Extreme Limit State (Seismic): (AASHTO 11.6.5)	1.0

General Seismic Design Parameters

Based on NDOT Bridge Structures Division policy:

- Peak Ground Acceleration Coefficient (PGA) = 0.15g (NDOT policy for Clark County)
- Short-Period Spectral Acceleration Coefficient (S_s) = 0.40
- Long-Period Spectral Acceleration Coefficient (S_1) = 0.15

AASHTO Table 3.10.33.1-1, Site Class Definitions: the site generally can be classified as Site Class C.

Response Modification Factor = R = varies (AASHTO Table 3.10.7.1-1)

Vertical Acceleration Coefficient = 0 (AASHTO Appendix A11)

Poisson's ratio for granular backfill material = μ = 0.30

Young Modulus for granular backfill material (E_s; AASHTO Table C10.4.6.3-1):

- $E_s = 0.139 N1_{60}$ (ksi) $\approx$ 4.448 ksi ; for $N1_{60} = 32$ (estimated)

Shear Modulus (G) for granular backfill material $= E_s / 2(1+\mu) \approx 1.7$ ksi

We recommend that the Structural Engineer check the abutment and retaining wall footing design at Strength and Extreme Limit States for Bearing Stress, Eccentricity, and Sliding:

Check the Stability of Retaining Walls at Strength Limit State:

- Against Soil Bearing Failure (AASHTO 11.6.3.2):

  Factored applied pressure = $\sigma_v = \frac{\sum V}{B-2e} \leq (\varphi_b = 0.55)\ (q_n)$

  $\sum V$ = Total Applied vertical Loads

  q_n = Soil Nominal Bearing Resistance

- Against Overturning Failure (AASHTO 11.6.3.3):

  $e_{B\text{-static}} \leq B/4$

- Against Sliding Failure (AASHTO 11.6.3.6):

  $\sum$ Static Horizontal Forces $\leq (\varphi_\tau = 0.8)\ (\sum V)\ (\tan\delta)$ + [factored passive resistance if shear key used].

Check the Stability of Retaining Walls at Extreme (Seismic) Limit State:

- Against Soil Bearing Failure (AASHTO 11.6.5):
  The eccentricity ($e_{B\text{-seismic}}$) of the resultant of the reaction forces shall be:

  $\sigma_v = \frac{\sum V}{B-2e} \leq (\varphi_b = 1)\ (q_n)$

- Against Overturning Failure (AASHTO 11.6.5):

  $e_{B\text{-Seismic}} \leq 1/3\ B$ for $\gamma_{EQ} = 0$

  $e_{B\text{-Seismic}} \leq 4/10\ B$ for $\gamma_{EQ} = 1.0$

- Against Sliding Failure (AASHTO 11.6.3.6):

  $\sum$ Static Horizontal Forces + Seismic Lateral force $\leq (\varphi_\tau = 1.0)\ (\sum V)\ (\tan\delta)$ + [factored passive resistance if shear key is used].

Check the Overall Stability of Retaining Walls at Service Limit State:

- Against overall failure (Global Stability: AASHTO 11.6.2.3)

The overall (global) stability of a retaining wall should be checked using limit equilibrium method as addressed in AASHTO 11.6.2.3.

Check the Structural Design of Retaining Wall Footings:

For checking the structural design of a retaining wall footing, use a linearly varying (trapezoidal) distribution (AASHTO 11.6.3.2) to ensure that the maximum factored bearing pressure (σ_{vmax}) is less than the factored bearing resistance:

$$\text{If } e < B/4: \quad \sigma_{vmax} = \frac{\Sigma V}{B} + [1 + 6\ \frac{e}{B}] \le (\varphi_b = 0.55)\ (q_n)$$

$$\text{If } e > B/4: \quad \sigma_{vmax} = \frac{2\,\Sigma V}{3\,[(\frac{B}{2}) - e)]} \le (\varphi_b = 0.55)\ (q_n)$$

5.3.2 "West Frontage Road" Retaining Wall

A retaining wall is proposed on the west side of the roadway alignment, approximately from station "P" 103+/- to "P" 114+/-, mainly within the cut section that will separate "P" line from "WF" line. The maximum height of the wall is about 31 feet. Approximately 10% of the wall length will be entirely in cut. The rest of the wall length will be partially in cut and partially in fill.

The existing ground is sloping approximately 10% from West to East. The geotechnical subsurface investigation shows that the native ground at this location is composed of very dense cohesionless materials of sand, gravel, and cobbles with occasional strong cementation (caliche) which will make excavation very difficult.

It is our understanding that some types of underground utility lines will be placed at shallow depths in the immediate vicinity of the back face of the wall.

Two retaining wall systems are presented here:

(1) Fill Wall (Bottom- Up Construction)
(2) Cut Wall (Top-Down Construction)

The following Section 5.3.2.1 provides the geotechnical design parameters for evaluating the external stability analysis of bottom-up construction of conventional cast-in-place cantilevered retaining wall. This wall system is preferred where majority of wall height is located in fills.

Section 5.3.2.2 provides the recommendations for top-down construction of a tangent pile wall. This wall system is preferred where majority of wall height is located in cuts.

5.3.2.1 Conventional Cast-in-Place Cantilevered Retaining Wall (Fill Wall)

Table below presents the geotechnical design parameters that can be used in External Stability Analysis of conventional cast-in-place cantilevered retaining wall such as NDOT Type I Cantilever Concrete Retaining Walls:

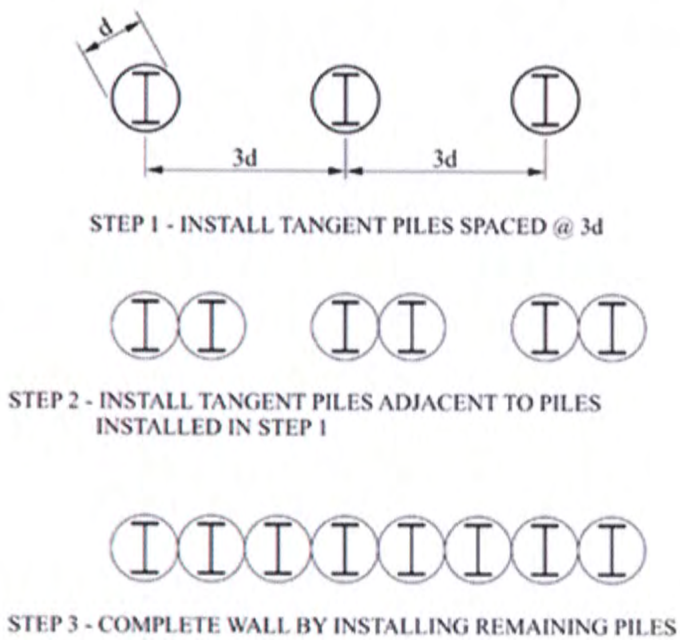
Conventional Cast-in-Place Cantilevered (Semi-Gravity) Retaining Wall	
In the absence of specific data: Internal Friction Angle of Driving Wedge (ϕ_f) = 30° (assumed) Driving Wedge Unit Weight (γ_f) = 0.120 kcf (assumed)	
Foundation Soils	Internal Friction Angle of Undisturbed Native Foundation Soil = ($\phi_{foundation}$) = 36° Foundation Soil Unit Weight = 0.120 kcf (assumed)
Overall Stability	AASHTO 11.6.2.3
Bearing Resistance at service limit state	6.0 ksf for walls embedded into native ground.
Overturning	AASHTO 11.6.3.3
Sliding Note: Passive resistance provided by soil at the toe of the wall by embedment is ignored (AASHTO Article 11.6.3.5).	Resistance Factor = 1.0 for sliding of semi-gravity walls on foundation (AASHTO Table 11.5.6-1). Sliding Coefficient = tan δ = 0.50 (AASHTO Table 3.11.5.3-1)
½ K_h = 0.075g is adequate for the design, provided that allowance is made for an outward displacement of up to 1.5 inches (10A_s in inches; AASHTO 11.6.5 and AASHTO A11.1.1.2) at this project site. K_{AE}: (Static + Seismic) Lateral Active Earth Pressure Coefficient K_A: Static Lateral Active Earth Pressure Coefficient	

BACKSLOPE	K_A (Coulomb)	K_{AE} (M-O equation)	
		K_h = 0.075g	K_h = 0.15g
FLAT	0.3014	0.35	0.407
4H:1V	0.366	0.443	0.547
3H:1V	0.40	0.497	0.654

5.3.2.2 Tangent Pile Wall (Cut Wall)

Tangent pile wall consists of a single row of tangentially touching drilled, reinforced-concrete piles to form a continuous wall. Each pile is independent from adjacent piles. Passive soil resistance is obtained by embedding the piles beneath the excavation grade.

Following is a schematic showing the sequence of construction in plan view of tangent pile wall:



Tangent Pile Wall Construction Schematic

Completed Tangent Pile Wall

If tangent walls to be constructed, we recommend 36-inch diameter drilled shafts extend below the final excavation grade to the depths as shown in the table below:

Tangent Pile Wall* (3-foot diameter reinforced drilled shafts)			
Wall Height (feet)	Minimum Embedment Depth Below excavation grade (feet)	Recommended Steel Beam*	Expected Top Lateral Deflection (in.)
31	18	W24X192	0.80
20	14	W24X76	0.44
15	13	W24X55	0.23

Tangent pile wall was designed using CivilTech Shoring Suite-Version 8. See Volume 2 for details.

* Preferred steel beams will be evaluated upon request.

Once all the structural shafts are installed, the wall face is excavated and an aesthetic cast-in-place concrete facing is constructed.

5.3.3 Mechanically Stabilized Earth Walls (MSEW)

MSEW has been proposed at the Railroad Bridge G-2872 approach embankment fill locations.

The MSEW should be designed based on AASHTO Article 11.10. Based on the results of preliminary analysis (AASHTO 11.10.2.1), the minimum reinforcement length of 8.0 feet or 0.80 times the wall height, whichever is greater, is sufficient for each wall up to 20 feet in height.

The Internal Stability Analysis of MSEW is recommended to be based on the Simplified Coherent Gravity Method (AASHTO 11.10.6.2.1).

The following table presents the geotechnical design parameters that can be used in the External Stability Analysis of the proposed MSE wall:

Mechanically Stabilized Earth Walls (MSEW)	
Retained Backfill	In the absence of specific data: Internal Friction Angle of Retained Backfill (ϕ_f) = 30° Retained Backfill Unit Weight (γ_f) = 0.120 kcf
Reinforced Soil Zone	In the absence of specific data: Internal Friction Angle of Reinforced Soil Zone (ϕ_r) = 34° Retained Backfill Unit Weight (γ_r) = 0.120 kcf A design friction angle greater than 40° should not be used, even if the measured angle is greater than 40°.
Foundation Soils	Internal Friction Angle of Native Foundation Soil ($\phi_{foundation}$) = 36° Foundation Soil Unit Weight ($\gamma_{foundation}$) = 0.120 kcf
Overall Stability	AASHTO 11.10.4.3
Bearing Resistance at service limit state	6.0 ksf for wall embedded into native soil
Overturning	AASHTO 11.10.5.5
Sliding Note: Passive resistance provided by soil at the toe of the wall is ignored.	Resistance Factor = 1.0 (AASHTO Table 11.5.6-1) In absence of specific data: (AASHTO 11.10.5.3) Sliding Coefficient = tan ϕ_r = tan 34° = 0.67 for wall with metallic strips Sliding Coefficient = tan ρ = tan (2/3 x 34°) = 0.42 for wall with grids and sheets
Seismic Design	Horizontal Ground Acceleration K_h = 0.15g ½ K_h = 0.075g is adequate for the design, provided that allowance is made for an outward displacement of up to 1.5 inches (Article 11.10.7). Static Lateral Active Earth Pressure Coefficient: K_A (Static + Seismic) Lateral Active Earth Pressure Coefficient: K_{AE}

BACKSLOPE	K_A (Rankin)	K_{AE} (M-O equation)	
		K_h = 0.075g	K_h = 0.15g
FLAT	0.333	0.380	0.433
4H:1V	0.367	0.443	0.546
3H:1V	0.40	0.50	0.662

5.4 Culvert Foundation

Buried structures and their foundations should be designed by the appropriate methods specified in AASHTO Section 12. Buried structures should be designed for force effects resulting from horizontal and vertical earth pressure, pavement load, live load, and vehicular dynamic load allowance. For vertical earth pressure, the maximum load factor from AASHTO Table 3.4.1-2 should be applied.

For trenches excavated in high-bearing native soils (SPT N-value > 50) such as cemented soils, decreased trench widths may be used up to the limits required for compaction. For these conditions, the use of a flowable backfill materials as specified in AASHTO Article 12.4.1.3, allows the envelope to be decreased to within 6.0 inches along each side of the pipe (AASHTO C12.6.6.1).

5.4.1 Soil Loads on Culverts - Buried Rigid Pipes (AASHTO 12.10.2-Loading)

The simplified formula for calculating the earth load is given in AASHTO, Article 12.10.2-Loading:

$W_E = F_e\, w\, B_c\, H$

W_E = unfactored earth load (kip/ft)
F_e = soil-structure interaction factor for the specified installation, AASHTO LRFD Table 12.10.2.1-3
B_c = out-to-out horizontal dimension of pipe (ft)
H = height of fill over pipe (ft)
w = Compacted fill on top of buried structures is assumed to have a unit weight of 0.130 kcf.

The product of $w\, B_c\, H$ is referred to as the Prism Load (PL) and is the weight of the column of earth over the outside diameter of the pipe.

Lateral Earth Pressure is assumed to have an equivalent fluid pressure (E.F.P) of 0.040 kcf (40 pounds per square foot per foot of depth).

For standard installations, the earth pressure distributions shall be the Heger pressure distribution shown in AASHTO Figure 12.10.2.1-1 and AASHTO Table 12.10.2.1-3 for each type of standard installation:

$$\text{Actual Total Vertical Load} = (\text{VAF})\ (\text{PL})$$
$$\text{Actual Total Horizontal Load} = (\text{HAF})\ (\text{PL})$$

VAF = Vertical Arching Factor

HAF = Horizontal Arching Factor

$w\ B_c\ H = PL =$ Prism Load

5.4.2 Culvert Headwalls and Wingwall Foundations

Headwalls will be constructed for culverts at various locations along the roadway alignment. Based on the general soil conditions in this area, the following design parameters are recommended for use in design of the culvert headwalls and wingwalls:

Culvert Headwalls and Wingwalls	
Nominal soil Bearing Resistance at Service Limit State	2.0 ksf
Passive Equivalent Fluid Pressure	0.280 kcf (Resistance Factor = 0.5 is included, (AASHTO Table 10.5.5.2.2-1)
Unit Weight of Soil –Compacted Fill Backfill Internal Friction Angle (ϕ_f)	0.120 kcf 32 degrees
Unit Weight of Soil –Undisturbed Native Internal Friction Angle for Undisturbed Native Soil (ϕ_s)	0.120 kcf 36 degrees
Interface Friction Coefficient (tan δ) for concrete cast on the undisturbed native soil Interface Friction Coefficient (tan δ) for precast concrete placed on native soil	tan δ = 0.50 tan δ = 0.40 (AASHTO Table 3.11.5.3-1)
Resistance Factor for Sliding Resistance between Soil and Foundation (ϕ_t) ; (AASHTO 10.5.5.2.2-1)	φ_τ = 0.80
Resistance Factor for Passive Earth pressure Component of Sliding Resistance (ϕ_{ep}) (AASHTO Table 10.5.5.2.2-1). The resistance due to passive earth pressure (P_P) in front of the headwalls shall be neglected.	φ_{ep} = 0.50

Equivalent Fluid Pressure (E.F.P.)			
Level Backfill		Backfill with 2H:1V	
At-Rest (kcf)	Active (kcf)	At-Rest (kcf)	Active (kcf)
0.056	0.037	0.081	0.055

5.5 Drainage System for Walls

Providing drainage systems for abutment walls and cast-in-place cantilevered walls to prevent the accumulation of surface runoff behind the walls, and subsequent hydrostatic pressure buildup is recommended. Drainage can be accomplished by providing weepholes behind the walls with outlets at or near the base of the walls. Refer to NDOT Standard Plans for Road and Bridge Construction, Sheet B-30.1 for details.

5.6 Modulus of Subgrade Reaction (k)

The modulus of subgrade reaction (k) is used as one of primary inputs for rigid slab/pavement design. It estimates the subgrade support below a concrete pavement slab.

The modulus of subgrade reaction (k-value) is a spring constant to model the support beneath the slab. The reactive pressure to resist a load is thus proportional to the spring deflection (which is a representation of slab deflection) and k:

$$P = k.\Delta$$

P = reactive pressure to support deflected slab

k = spring constant = modulus of subgrade reaction

Δ = slab deflection

A modulus of subgrade reaction of 500 pounds per cubic inch (pci) is recommended for foundation established on the very dense native granular soils. For foundation established on the embankment fill (with the minimum R-value of 45) a modulus of subgrade reaction of 200 pci may be used.

5.7 Miscellaneous Foundations

The NDOT Standard Plans for Road and Bridge Construction addresses the foundation requirements for various types of traffic structures (e.g., luminaires, sign bridges, traffic signals). There is no proposed soundwall on this project.

5.8 ROADWAY EMBANKMENTS

5.8.1 Overall Stability Analysis of Bridge Approach Embankment

The overall stability analyses of the bridge approach embankments were performed using the XSTABL™ computer program, employing Limit Equilibrium-Modified Bishop's Method.

The overall stability has been undertaken for the bridge with the maximum approach embankment height of 53 feet (Structure I-2871, Abutment 2) and adopting the subsurface profile at Borehole FA2.

The geotechnical parameters such as fill internal frictional angle of 32 degrees with apparent cohesion of 0.200 kips/ft^2, and fill unit weight of 0.120 kips/ft^3 were used in the analysis. Native soil internal friction angle of 36 degrees with apparent cohesion of 0.200 kips/ft^2 and soil unit weight of 0.120 kips/ft^3 were used for the native soils below the fill. A presumptive nominal bearing resistance of 4.0 kips/ft^2 at service limit state and a traffic surcharge load of 0.250 kips/ft^2 at the bridge abutments were used in the overall stability analysis.

It should be noted that presently most of the software used to analyze the overall global stability calculates a Factor of Safety (FOS). To comply with the AASHTO LRFD, Article 11.6.2.3, we invert the FOS (back calculated) to arrive at the required "Equivalent" Resistance Factor (φ). AASHTO ASD requires FOS = 1.3 (φ = 0.75 = 1/1.3) for slope that does not support or contain a structural element and FOS = 1.5 (φ = 0.65 = 1/1.5) for slope that contains a structural element under static condition. For the effect of earthquake loading, *AASHTO ASD requires FOS = 1.1 (φ = 0.9 = 1/1.1, AASHTO LRFD 11.6.5) for all slopes.*

The XSTABL™ computer program was used to perform a search procedure to locate the most critical failure surface identified as the one yielding the lowest factor of safety (Resistance Factor (φ) is the inverse of FOS). A minimum FOS of 1.5 (φ = 0.65) is used as acceptable criteria for the static load case. A minimum FOS of 1.1 (φ = 0.9, AASHTO LRFD 11.6.5) is used as acceptable criteria for the earthquake case. The horizontal acceleration used in the seismic stability analyses was based on peak ground acceleration of 0.15g. A traffic surcharge load of 0.250 ksf was included in each analysis. The results of these analyses are provided in Volume II - Appendix E.

The analyses indicate that the Resistance Factors (φ) under both static and seismic loading condition, for the proposed bridge approach embankments (estimated maximum height of 53 feet) constructed on 2H:1V (Horizontal:Vertical) meet the AASHTO LRFD requirements. Table below provides the summaries of the factors of safety and their "Equivalent" Resistance Factors (φ):

Overall Stability of Bridge Approach Embankment					
Soil Type	**Soil Properties**	**Static Loading**		**Static + Seismic Loading**	
		"Equivalent" Resistance Factor (φ)	**Factor of Safety**	**"Equivalent" Resistance Factor (φ)**	**Factor of Safety**
Fill	$\gamma = 120$ pcf $\phi = 32°$ C = 0.200 ksf	0.65	1.54	0.80	1.24
Native	$\gamma = 120$ pcf $\phi = 36°$ C = 0.200 ksf				

5.8.2 Overall Stability Analysis of Roadway Embankment

The overall stability of the roadway embankment has been undertaken for the embankment with the maximum height of 53 feet (station "F" 24+50), batter slopes of 2H:1V, and adopting the subsurface profile at Borehole FA2.

The LRFD method of evaluation of the overall stability of the roadway embankment is similar to the method addressed in "Overall Stability Analysis of Bridge Approach Embankment" page 50 of this report except the bridge foundation load is not included. The geotechnical design parameters, such as fill internal frictional angle of 32 degrees with apparent cohesion of 0.200 kips/ft^2 and fill unit weight of 0.120 kcf, were used in the analysis. Native soil internal friction angle of 36 degrees with apparent cohesion of 0.200 kips/ft^2 and unit weight of 0.120 kips/ft^3 was used for the native soils below the fill. The outputs of the slope stability analysis, using X-STABL computer software, are provided in Volume II - Appendix E.

The factors of safety for surface shear failure are estimated to be 1.67 (φ = 0.6) under static condition and 1.21 (φ = 0.826) under seismic condition. A minimum FOS of 1.3 (φ = 0.75) is used as acceptable criteria for the static load case (AASHTO ASD 5.2.2.3, AASHTO LRFD 11.6.2.3). A minimum FOS of 1.1 (φ = 0.90) is used as acceptable criteria for the seismic case (AASHTO ASD 5.2.2.3, ASSHTO LRFD 11.6.5).

The horizontal acceleration used in the seismic stability analyses was based on peak ground acceleration of 0.15g. On the basis of the slope stability analyses undertaken for the embankment, permanent batter slopes of two horizontal to one vertical are recommended.

5.8.3 Settlement Within Roadway Embankment Fill

It is expected that the immediate settlement of the roadway embankment fill would occur as the embankment is being constructed. Post construction wetting-induced collapse settlement of the embankment fill due to presence of void spaces in the compacted materials is estimated to be in the range of 1 to 2 inches (0.2 to 0.4 percent of the embankment height).

5.9 CUTS AND EXCAVATIONS

The following sections of 5.9.1 and 5.9.2 present the locations of the excavations and the recommended cut slopes:

5.9.1 Soil Cut Slopes

There will be roadway cuts into the native ground along "P" line. The proposed roadway cut will be from Station "P" 75 to Station "P" 125 with the maximum depth of 30 feet. Based on the field investigation at this location (Boring Logs: RRC1, RRC2, RRC3, RRC4), the materials consist of alluvial deposits of dense to very dense silty/clayey gravel with sand and cobble size rock fragments, moderate to strong cementation. The recommended cut slope along this line is 2H: 1V. Difficult excavation in this area should be expected.

5.9.2 Rock Cut Slopes

The proposed rock cuts will be located on the:

- West side of "WF" line (west frontage road) from Station "WF" 55 +30 to Station "WF" 62+50.

- East side of "R3" line (ramp 3 which connects to structure I-2869) from Station "R3" 1+50 to Station "R3" 15+30.

In September 2010 and March 2011, the Geotechnical Section of the Materials and Testing Division conducted mapping the critical geological discontinuity features of the above proposed rock cuts. The end product of this investigation was collecting structural geological data related to the discontinuities and strength of rocks which were needed for stability studies of the rock cuts. These data include:

* Mapping location
* Classification of discontinuities by type
* Orientation of discontinuities
* Infilling of the discontinuities
* Surface properties
* Spacing of discontinuities within sets
* Persistence of fractures
* Rock mass parameters

Stereographic analyses of the rock slopes were performed using RockPack III software to estimate the potential for rockslides and rockfalls based on the collected field outcrop discontinuity data and the proposed cut-slope configurations. Volume II - Appendix I presents the stereonet plots of the discontinuities using Dip Vector Plots.

Based on the site rock lithology, strength, weathering resistance, bedding characteristics, joint inclination, and discontinuity, we recommend the following:

- Rock Slope Cut from Station "WF" 55 +30 to Station "WF" 62+50:
  - The total height of the cut is approximately 140 feet. The recommended face cut slope along this line is 1H:1V.

- Rock Slope Cut from Station "R3" 1+50 to Station "R3" 15+30:
  - The total height of the cut is approximately 155 feet. The recommended face cut slope along this line is 1H:1V.

Rock Slope from Station "WF" 55 +30 to Station "WF" 62+50

"Ramp 3" Rock Slope from Station "R3" 1+50 to Station "R3" 15+30

It is essential that the excavation face be inspected by a geotechnical engineer during the excavation, to confirm where localized stabilization measures may be required.

Based on the field seismic survey, excavation in these slopes will require blasting.

5.10 Rockfall Catchment Ditch

Raveling of both coarse and fine debris from the rock cut slopes can reach the roadway and create hazards to traffic if not intercepted by a catchment ditch. Adequate rockfall catchment ditch at the toe of the slopes is recommended.

The following two pages (excerpted from the Reference No. 5) provide recommended minimum catchment ditch widths with foreslopes of 3H:1V, 4H:1V, and 6H:1V and two general catchment ditch configuration options:

GB3 Table C - Catchment Ditch Widths

3:1 Foreslope

	Cut Slope Height, H (ft)					
	16-40	**50**	**60**	**70**	**80-100**	***>100****
Cut Slope Angle	**Catchment Ditch Width, W (ft)**					
0.25:1	10	15	15	15	20	*25 min.*
0.5:1	10	15	20	20	20	*25 min.*
1.0:1	15	20	20	20/25**	25	*30 min.*

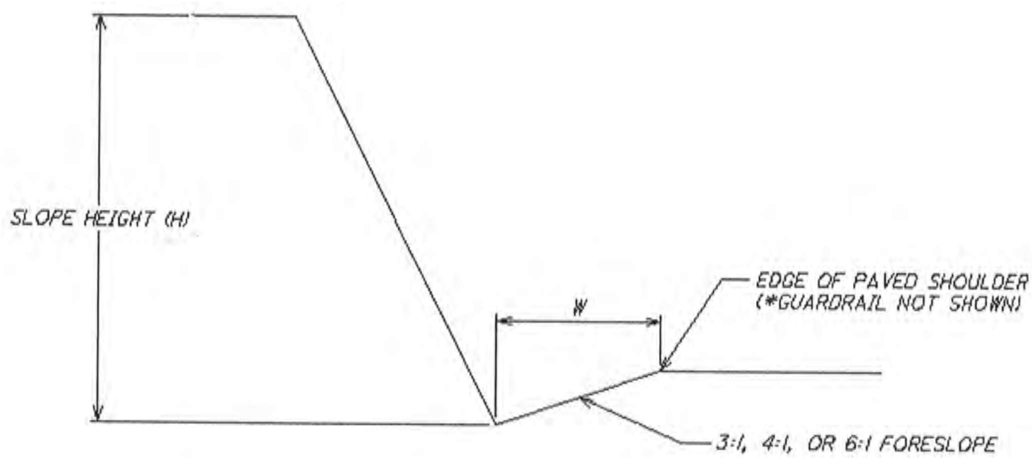
4:1 Foreslope

	Cut Slope Height, H (ft)					
	16-40	**50**	**60**	**70**	**80-100**	***>100****
Cut Slope Angle	**Catchment Ditch Width, W (ft)**					
0.25:1	10/15**	15	20	20	25	*30 min.*
0.5:1	15	15	20	20	25	*30 min.*
1.0:1	15/20**	20	20/25**	25/30**	30	*35 min.*

6:1 Foreslope

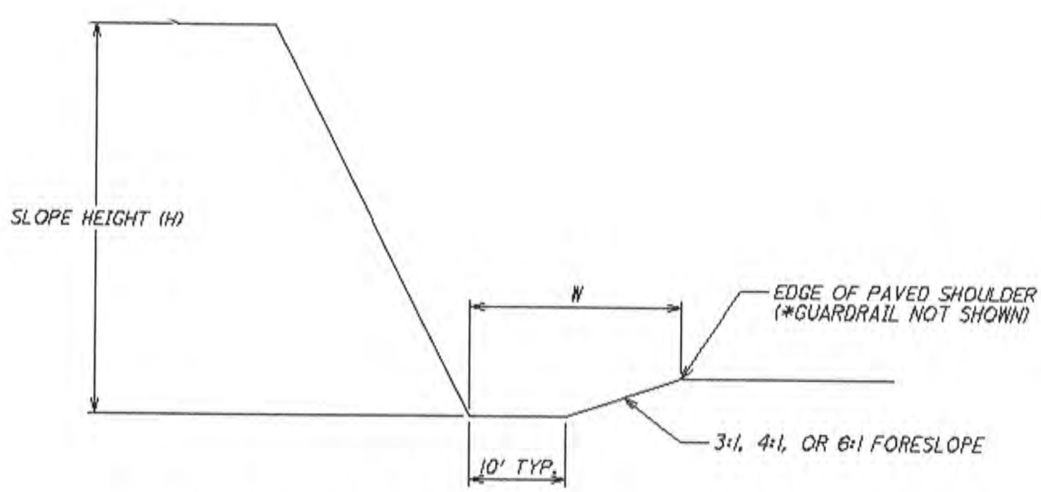
	Cut Slope Height, H (ft)					
	16-40	**50**	**60**	**70**	**80-100**	***>100****
Cut Slope Angle	**Catchment Ditch Width, W (ft)**					
0.25:1	15	20	25	30	35	*40 min.*
0.5:1	20	20	25	30	35	*40 min.*
1.0:1	25/30**	25/30**	30	35	40	*40 min.*

*Slopes with H greater than 100' should be designed with Table C width as minimum and adjusted according to specific site conditions



*Guardrail Required For All 3:1 Slopes and 4:1 Slopes With Safety Grading

Catchment Ditch Configuration Option 1



*Guardrail Required For All 3:1 Slopes and 4:1 Slopes With Safety Grading

Catchment Ditch Configuration Option 2

Guardrail is required for catchment ditches with foreslopes steeper than 6H:1V.

If the width of the catchment ditch is smaller than the recommended width (W), a lightweight chain link fence should be placed along the outside edge of the ditch to catch the rolling rocks before reaching the roadway.

5.11 Blasting and Rippability

The seismic refraction and ReMi surveys that were conducted at various locations (Refer to Volume II - Appendix F for details.) to measure the P-wave and S-wave velocities show that the excavation of the proposed rock slope on the west side of the "WF" line and the rock slope on the east side of the ramp "R3" is expected to require blasting. Also, the bedrock outcrop observed in the vicinity of structure I-2869 may require blasting.

P-wave velocity of up to 7500 feet/second was recorded along "P" line and P-wave velocity in excess of 12000 feet/second was recorded within the proposed rock cuts. This velocity may be compared with the following chart, provided by California Department of Transportation (Caltrans), which correlates seismic velocity with rippability.

P-Wave Velocity (feet/second): Rippability

< 3445: Easily Ripped

3445 –4921: Moderately Difficult

4921 – 6562: Difficult Ripping / Light Blasting

> 6562: Blasting Required

Blasting has already been used for excavation into the bedrock in the close vicinity of the project. We were told that conventional blasting was used on the north side of the Silverline Road to install fiber optic lines.

5.12 On-Site Borrow Source

The laboratory testing (R-Values) on soil samples obtained during the geotechnical subsurface investigation shows that these materials meet the NDOT Standard Specifications, Section 203 for Borrow. Please refer to the "Laboratory Testing Summary Sheets" in The Volume II - Appendix C for the details.

5.13 Riprap Source

The most promising potential sources for riprap are from the proposed rock slope cuts. The method of excavation (i.e., ripping, blasting, etc.) can have an effect on maximum rock size.

5.14 Estimation of Bulking Factors (Shrinkage and Swell Factors)

Based on our field pre-excavation compression seismic velocities, P-wave, (determined by refraction survey) of the site and correlating them with the AASHTO recommended procedure (AASHTO Manual On Subsurface Investigations, 1988) to estimate Bulking Factors (Shrinkage and Swell Factors), we recommend:

- Shrinkage/Swell factor = 1.0 for soils (one-to-one ratio between excavated soil and compacted fill soil).

- Swell factor = 1.15 for rocks excavated from the cut areas.

Note that the actual amount of shrinkage and swell are highly dependent on the accuracy of surface mapping as well as the difficulty of accurately estimating bulking factors of soil and rock materials. Other factors affecting shrinkage and swell include contractor's methods, wind losses, material waste, and under or over compaction.

On any project involving large amounts of earthwork there is always a risk of needing to import large amounts of borrows material from off-site locations or needing to dispose large amounts of excess excavated materials to off-site. Because of this, we recommend identifying the on-site areas where additional borrow can be mined and other on-site areas where excess material can be disposed.

5.15 On-Site Soil Corrosion Potential

Volume II - Appendix D of this report presents the on-site soil corrosion test results.

5.16 Soil Erosion Potential

Observations during site reconnaissance indicated the several erosion gullies have formed in the native materials along the proposed alignment. These observations provide a good indication that the on-site material could be erodable under concentrated storm water flow. If these velocities cannot be limited, some surface protection or even energy dissipation structures should be devised to minimize the erosion.

The cut and fill slopes on this project can be subjected to significant erosion during storm events and will therefore require erosion mitigation. Adequate surface drainage should be provided away from all structures. Ponding of water in and around the structure foundation areas should be prevented by proper grading.

If the drainage channels are to be lined with riprap, it is recommended that a layer of erosion control geotextile be placed on the prepared subgrade prior to riprap placement. The geotextile will prevent erosion of the channel and the mitigation of soil particles from the subgrade into the riprap.

Erosion control geotextiles such as nonwoven needle-punched polypropylene geotextile Class 1 AASHTO M288 or a woven monofilament geotextile Class 2 - AASHTO M288, meeting the following requirements is recommended:

PROPERTY	TEST METHOD	UNITS	REQUIREMENT
Permittivity	ASTM D4491	sec^{-1}	≥ 0.2
Apparent Opening Size (AOS)	ASTM D4751	inches (mm)	0.01 inch (0.25 mm) max.
Ultraviolet Stability at 500 hrs.	ASTM D4355	% Strength Retained	≥ 50

5.17 Trenching and Excavations

If temporary excavations are required, the unsupported excavations will require shoring or laying back the side walls of the excavations, conforming to OSHA, 29 CFR, 1926, Table B-1, Subpart P – Excavations as presented below, to maintain stability:

TABLE B-1 Maximum Allowable Slopes	
Soil or Rock Type	Maximum Allowable Slopes (H:V)[1] For Excavations Less than 20 feet deep[3]
Stable Rock	Vertical (90°)
Type A[2]	¾:1 (53°)
Type B	1:1 (45°)
Type C	1 ½:1 (34°)

Footnote [1]: Numbers shown in parentheses next to maximum allowable slopes are angles expressed in degrees from the horizontal. Angles have been rounded off.

Footnote [2]: A short-term maximum allowable slopes of 1/2H:1V (63 degrees) is allowed in excavations in Type A soil that is 12 feet or less in depth. Short-term maximum allowable slopes for excavations greater than 12 feet in depth shall be 3/4H:1V (53 degrees)

Footnote [3]: Sloping or benching for excavations greater than 20 feet deep shall be designed by a registered professional engineers.

If shoring and shielding options are considered, Appendix F to 1926 Subpart P must be followed.

Based on our field observation of the natural cuts and the laboratory test results, the on-site native soils are classified as Type B Soils. Where fill materials have been placed due to re-grading of the site, Type C Soils are classified. The suggested layback for the temporary excavation may require modification after the start of construction. The contractor is ultimately responsible for the safety of workers in the excavated trenches. Contractor must conform to the requirements of federal and local OSHA requirements for the safety of the workers.

Trenching and excavation in on-site materials will be difficult. Cobbles and boulders present in the sidewalls of trenches will have a tendency to spall if the trenches remain open for an extended period of time and the sidewalls are allowed to dry out.

Qualified geotechnical personnel should examine any area in question during construction.

5.18 Dust Generation

A moderate potential for dust generation is present, particularly in those areas underlain by cemented materials if roadway grading is performed in dry weather.

GEOTECHNICAL REPORT LIMITATIONS

Recommendations contained in this report are based on the information obtained from our field explorations, laboratory tests, and observations of our Project Engineer. The nature and extent of variations may not be evident until the construction takes place. If conditions are encountered during construction, which differ from those described in this report, or if the scope of construction is altered significantly, the Geotechnical Section must be notified in order that a review of our recommendations can be provided.

On-site aggregate potential quality studies were not performed. We did not evaluate the on-site materials as aggregate source. It is the sole responsibility of the contractor to perform studies on the on-site materials for aggregate production such as roadway base aggregate, asphaltic concrete mix, Portland concrete mix, and MSE wall backfill.

END

REFERENCES

1) C. R. Longwell, E. H. Pampeyan, Ben Boowyer, and R. J. Roberts, Geology and Mineralogy Deposits of Clark County, Nevada, Mackay School of Mines-University of Nevada-Reno, 1965.

2) AASHTO, AASHTO LRFD Bridge Design Specifications, 5th Edition, 2010, AASHTO, Washington D.C., 2010.

3) Wyman et al., Geology of Las Vegas, Bulletin of the Association of Engineering Geologists, March 1993, Volume XXX, Number 1.

4) FHWA, Geotechnical Engineering Circular No. 3- Geotechnical Earthquake Engineering For Highways, Federal Highway Administration Publication No. FHWA-SA-97-076, May 1997.

5) Ohio Department of Transportation, Geotechnical Bulletin GB3, January 13, 2006.

6) FHWA-NHI, Soil Slope and Embankment Design, Federal Highway Administration Publication No. FHWA-HI-01-026, Washington D.C., January 2002.

7) FHWA-NHI, Rock Slope, Publication No. FHWA NHI-99-007, October 1998.

8) FHWA, Seismic Design and Retrofit Manual for Highway Bridges, Federal Highway Administration Report No. FHWA-IP-87-6, May 1987.

APPENDIX A

SITE AERIAL PHOTO

Layout of the Roadway Alignment

APPENDIX B
GEOLOGICAL MAPS

FED. RD. REG. NO.	STATE	PROJECT NO.	COUNTY	SHEET NO.
	NEVADA			

Contact

Long-dashed where approximately located; short-dashed where sketched or inferred

D
65

Fault, showing dip or direction of dip

Long-dashed where approximately located; short-dashed where inferred; dotted where concealed. D, *downthrown side of reverse fault. No arrow on vertical or nearly vertical faults*

Thrust fault

Saw-teeth indicate thrust plate. Dashed where approximately located; dotted where concealed

Anticline

Showing trace of axial plane and direction of plunge of axis. Dashed where approximately located

Overturned anticline

Showing trace of axial plane, direction of dip of limbs, and plunge of axis

Syncline

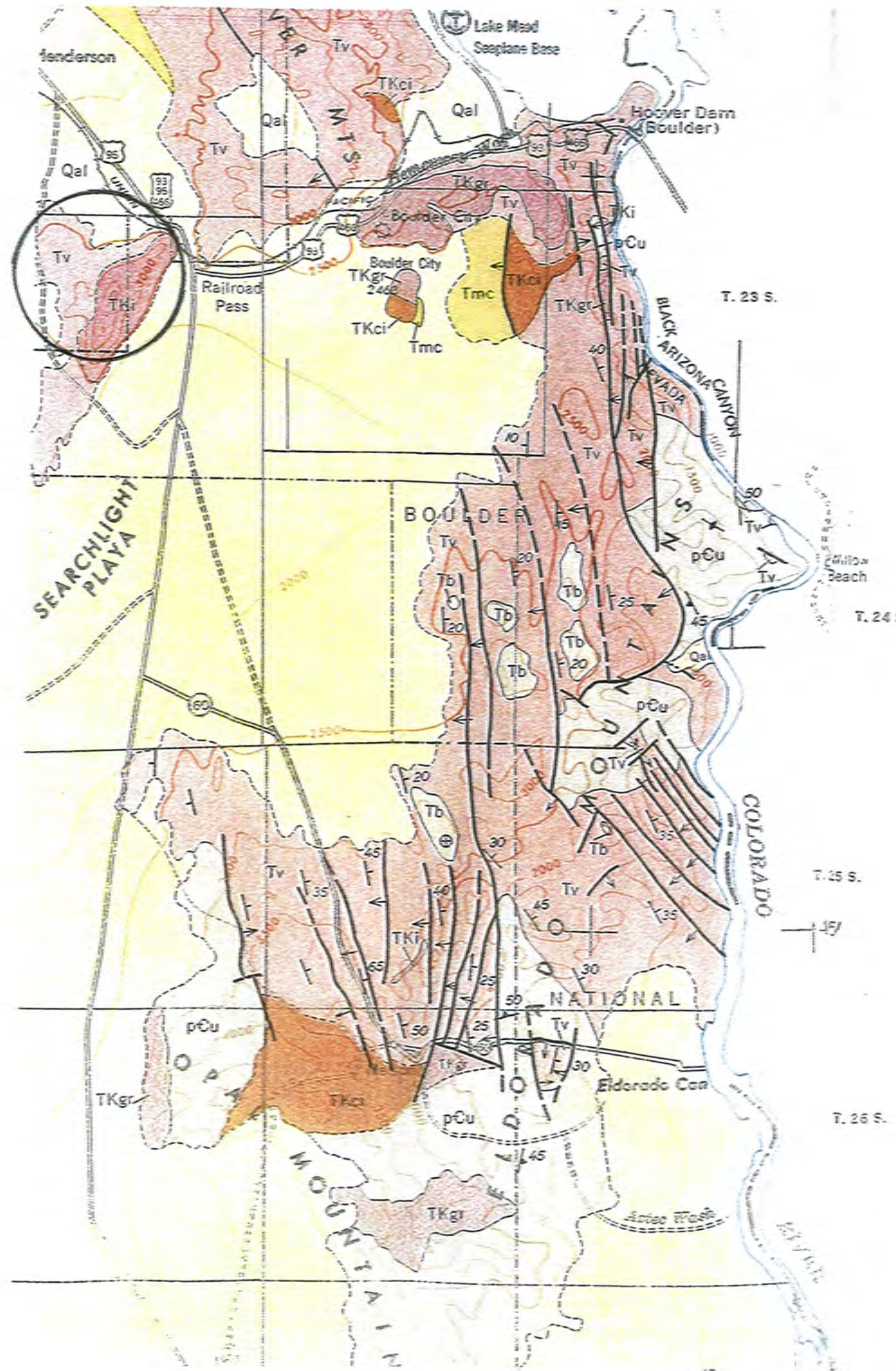
Showing trace of axial plane and direction of plunge of axis. Dashed where approximately located

45

Strike and dip of beds

30

Strike and dip of overturned beds



EXPLANATION

CLARK COUNTY

Qal

Alluvium

Ql

Las Vegas Formation

UNCONFORMITY

Tb
Tv

Volcanic rocks

Tv, *undifferentiated volcanic rocks.*
Tb, *Fortification Basalt Member of Muddy Creek Formation*

TKgr | TKi | TKci

Intrusive rocks

TKgr, *holocrystalline rock; mainly granite, quartz monzonite, granodiorite, and diorite*
TKi, *undivided porphyritic rocks. Includes granite porphyry, rhyolite, trachydolerite, and other intrusives ranging from basaltic to rhyolitic*
TKci, *undifferentiated intrusive rocks; mainly quartz monzonite and diorite containing roof pendants of volcanic rock, Paleozoic limestone and dolomite, and Precambrian rocks*

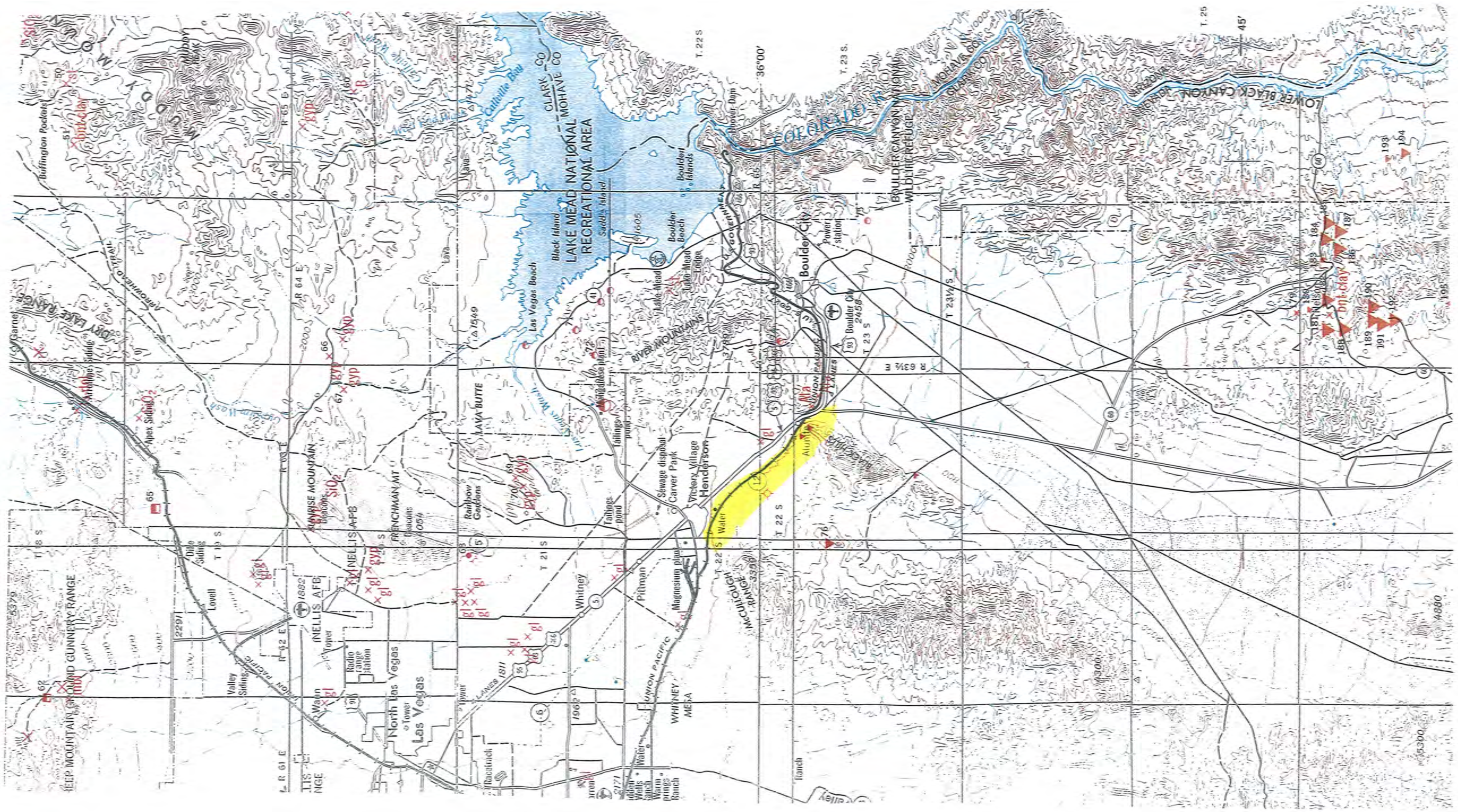
CRETACEOUS(?) AND TERTIARY

STATE OF NEVADA
DEPARTMENT OF TRANSPORTATION

Site

Geology Map

FED. RD. REG. NO.	STATE	PROJECT NO.	COUNTY	SHEET NO.
	NEVADA			



STATE OF NEVADA
DEPARTMENT OF TRANSPORTATION

Site

Topographic Map