MEMORANDUM

TO: Michael Carpenter, P.E.
Director
Division of Structural Design / Geotechnical Branch
Kentucky Transportation Cabinet
1236 Wilkinson Boulevard
Frankfort, Kentucky 40601

FROM: Christopher N. Farmer, P.E. (Consultant)
Principal Geotechnical Engineer
Bacon Farmer Workman Engineering & Testing, Inc. (BFW)
500 South 17th Street
Paducah, Kentucky 42001

DATE: November 29, 2023

SUBJECT: **Kenton County**
D6 059C00048N
Item No. 6-10046
Ernst Road
Bridge Over CSX Railroad
Supplemental Geotechnical Engineering Structure Foundation Report for Original Terracon Geotechnical Report – April 8, 2019

1.0 Original Geotechnical Report

This report is a supplement to the original Terracon "Geotechnical Engineering Report for Ernst Bridge Road Replacement over CSX Railroad, Kenton County, Kentucky" (Terracon Project No. N1185278) – April 8, 2019. This supplemental report was prepared for the KYTC SW Bridge Delivery Program for the addition of a driven H-Pile foundation support option that was not included in the original report. The H-Pile capacities for HP 12 x 53 and 14 x 89 were analyzed using the subsurface and laboratory information obtained and developed by Terracon in their original geotechnical report. Subsurface information included one boring (B-18-1) and one cone penetration testing (CPT) log (two additional CPT attempts were made near the first CPT location but were incomplete due to cone refusals).

2.0 Location and Description

The project is located on Ernst Road over CSX Railroad, approximately 0.5 miles southeast of the community of Ryland, Kentucky and immediately southeast and adjacent to the Ryland Lakes Country Club, Kenton County, Kentucky. The bridge is being replaced as part of the KYTC SW Bridge Delivery Program. The proposed bridge is a simple span structure using welded steel plate girders with a length of 98'-0" (out to out), a bridge width of 20'-5" on a 0˚ skew. The bridge will be supported by two pile supported vertical wall abutments with turned back wingwalls.

3.0 Site Geologic Conditions

The bridge is located within the Licking River valley and shown on the Alexandria, KY Geologic and Topographic Quadrangles (GQ #926). Geologic mapping of the bridge location shows the geologic strata composed of terrace deposits which contain lacustrine deposits eventually underlain by granular outwash deposits and then Ordovician Age bedrock. Depth to bedrock is estimated to be greater than 100 feet. The Natural Resources Conservation Service (NRCS) Soil map classifies the surface soils at the bridge location as Licking silt loams.

4.0 Field Investigation

Subsurface drilling was conducted by Terracon on February 22, 2019. One boring, B-18-1 (Station 105+47.14, 54.55' LT) was advanced in proximity to the eastern abutment (Abutment 2) and was advanced to an auger refusal depth of 102 feet below ground surface (bgs). Cone Penetrometer Testing (CPT) was also conducted by Terracon on February 21, 2019. A series of three (3) cone penetration tests, CPT-1, CPT-1A and CPT-1B 9 (near Station 104+25.03 29.53' RT) were advanced near the western abutment (Abutment 1). CPT-1 was advanced to a refusal depth of 64.2 feet. CPT-1A was offset a few feet from the previous location and was predrilled to a depth of 30 feet where the CPT testing commenced with a refusal depth of 36.8 feet. A final attempt was made with CPT-1B which was predrilled to a depth of 40 feet where CPT testing commended with a refusal depth of 65.2 feet.

Soil samples were collected during the drilling activities and were delivered to and analyzed by Terracon's soil laboratory. No rock outcroppings were observed near the existing bridge location.

5.0 Laboratory Testing

Terrace deposits comprised of lacustrine soils were encountered during drilling activities and consisted of intermixed inorganic lean to silty clays, sands with interbedded silts, silty sands, poorly graded sands with silt. Soil samples were collected during drilling activities and were taken to Terracon for laboratory testing and classification.

Based on laboratory results, soils were classified as CL, SM, and SP-SM using the Unified Soil Classification System and A-1-b, A-2-4, A-4, and A-6 using the AASHTO Classification Method.

6.0 Subsurface Conditions

Boring B-18-1 (near Abutment 2) and CPT log, CPT-1 (near Abutment 1) were roughly similar in subsurface soil makeup. Below the surface stratum and near surface fill, lean clays with sand and concretions were encountered to a depth of approximately 20 feet. Silt and sand partings and laminations were encountered and increased with depth. The sandy and silty clays extended to depths of approximately 72 feet bgs. Below this depth silty to clayey sands were encountered and extended to approximately 88 feet bgs where the soil transitioned to a poorly graded sand with silty and some gravel. Auger refusal was met within the sands at a depth of 102 feet bgs.

Soil consistencies in the upper 40 feet ranged from very soft to firm in the clays with sands and silt. Soils consistencies dropped significantly in boring B-18-1 between the depths of 48 to 62 feet bgs. Soil consistency between these depths ranged from weight of hammer (WHO) to very soft. The soil consistency increased rapidly at depths beyond 62 feet and ranged from soft to stiff. Soil consistencies within the clayey sands and poorly graded sands with silt and gravel ranged from firm to splitspoon refusal. Two of the standard Penetration Tests encountered splitspoon refusal at depths below 93 feet.

The groundwater level in boring B-18-1 after drilling activities were concluded was determined to be approximately 24 feet bgs (Elev 523 approx.). Approximate groundwater levels in the CPT test locations were approximately 12 feet bgs (Elev. 523).

7.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

7.1 Embankments and Settlement – Based on the available bridge and roadway plans, new approaches will be constructed for both the eastern and western abutments. The new eastern approach will be close to the existing grade with minimal fill; therefore, slope stability and settlement are not of geotechnical concern.

The new western approach will require the construction of a new embankment ranging in heights from 0 to 18 feet. Based on subsurface data and embankment heights estimated settlement of up to 2 inches are possible. As a result, the piles at the western abutment will be subject to down drag loads. The estimated total settlement values are based upon the soil conditions in soil borings, borehole test data, and one-dimensional consolidation testing conducted by Terracon and using Settle3 software. The estimated time-rate of settlement for 90% consolidation and time to reduce settlement remaining to 1 inch is approximately 12 weeks and 4 weeks, respectively. Time-rate of settlement assumes that silt layers encountered within the clay soils will act as intermediate drainage paths.

Based on Terracon's geotechnical slope analysis, and the on-site soils encountered, cut and fill slopes will need to be maintained at 2.5H:1V for safety of long-term maintenance requirements. Temporary fill slopes for construction roads can be constructed to 2H:1V.

7.2 Abutment 1 and 2 – The use of either HP 12x53 or HP 14x89 are recommended as friction piles at both abutment locations. According to the **KYTC Bridge Program Project Delivery Manual** the use of H-piles is preferred over pipe piles. LRFD Factored Pile Capacities are shown on the pile capacity tables included in the attachments to this report. Capacities may be linearly interpolated between the five-foot intervals presented in the tables. If the base of pile cap varies from the elevation used for the capacity tables base of pile cap by more than 5 feet, contact BFW Engineering for re-evaluation of the capacities. **H-piles used as friction piles should not include pile points as this will result in loss of side friction as the piles are being driven**.

Piles should be installed with a center-to-center spacing of three (3) times the pile diameter or greater in order to optimize group resistance and minimize installation problems. If spacing less than three diameters is needed, please contact BFW Engineering for capacity reduction factors.

Please note that the Total Factored Geotechnical Axial Resistance from the charts may not exceed the Maximum Nominal Geotechnical Axial Capacity of the pile. We recommend using a resistance factor (Φ_c) of 0.6 to determine the Maximum Nominal Geotechnical Axial Capacity of the pile, which results in a maximum of 465 kips and 783 kips for HP 12x53 and HP 14 x 89 piles, respectively.

7.3 Scour – The proposed bridge is a dry crossing over CSX railroad; therefore, a scour analysis is not required.

7.4 **Slope Protection** – Slope protection will be required for the soil berms in front of both vertical wall abutments meeting the requirements of Sections 703 & 805 of the Standard Specifications for Road and Bridge Construction, current edition. Place a Class 1, Geotextile Fabric, in accordance with Sections 214 & 843 of the Standard Specification for Road and Bridge Construction, current edition, between the embankment and the slope protections.

7.5 **Wave Equation Analysis –** Drivability analyses were performed for the piles at this location assuming either HP 12x53 or HP 14x89, 50-ksi steel H-piles. These analyses indicated that a sufficient range of single acting diesel hammers are available to install the piles to the required end bearing depths without excessive blow counts or overstressing the piles. Drivability studies were performed assuming continuous driving. If interruptions in driving individual piles should occur, difficulties in continuing the installation process will likely occur due to pile "set-up" characteristics.

7.6 **Verification of Piles Capacities** – Based on the **KYTC Bridge Program Project Delivery Manual** the construction control of friction piles will use the FHWA Modified Gates Formula. Therefore, it is recommended that field verification of pile capacity should be performed using the FHWA Modified Gates Formula instead of the formulas provided in the Standard Specifications. The field verification values for End of Driving (EOD) using the Modified Gates Formula are provided under the Static Analysis Method columns of the LRFD Pile Capacity Tables for friction piles located in the attachments to this report.

Due to the fine-grained nature of the cohesive soils, excess pore pressures will likely develop during driving. As a result, the pile resistance during driving will likely be less than the long-term static resistance of the piles. The resistance in the cohesive soils will likely increase with time (soil set up), once the excess pore pressures dissipate after driving the piles. The set up can only be determined by restriking the piles approximately 7 days (or longer) after the initial driving of the piles.

7.7 **Seismic Site Class Definition** – The seismic design procedures outlined in the current AASHTO LRFD Bridge Design Specification indicates that structural design loads are to be based on site class definitions developed from the subsurface condition encountered. Based on the results of the exploration and geology of the area, a site class of D, as per Table 3.10.3.1.1 – Site Class Definitions, should be used for design purposes.

7.8 **Minimum Pile Lengths** – It is recommended that the structural designer include minimum required pile lengths or tip elevations required to satisfy pile lateral stability on the project plans. It is also recommended that factored uplift design loads, if applicable, be included in the pile record table. Since final pile lengths or tip elevations will be adjusted in the field based on field verification of axial capacity, this information will be used during construction to help ensure that adequate pile embedment and capacities are obtained, and pile lengths are not based on compressive axial capacity alone.

7.9 **Lateral Loads** – Perform lateral load analysis as needed using the idealized geotechnical parameters provided in the original Terracon Geotechnical Engineering Report. These parameters may be used to perform analysis using LPILE or other similar software. Some of the parameters may not be required to input depending on the version of software used.

8.0 Plan Notes

The following notes should be included at the appropriate locations in the plans.

8.1 HAMMER CRITERIA: Single acting diesel hammers with rated energy of between 40 kip-ft and 48 kip-ft is recommended for HP 12 x 53 piles and a rated energy between 55 kip-ft and 66 kip-ft is recommended for HP 14x89 piles to adequately drive the piles at the end bents without encountering excessive blow counts or overstressing the piles. The use of hammers other than single acting diesel may require different rated energies. The Contractor shall submit the proposed pile driving system to the Department for approval prior to the installation of the first pile. Approval of the pile driving system by the Engineer will be subject to satisfactory field performance of the pile driving procedures.

8.2 Embankments at the bridge vertical abutment locations shall be constructed in accordance with Special Provision 69 Embankment at Bridge End Bent Structures.

8.3 Slope protection will be required at the bridge meeting the requirements of Sections 703 & 805 of the Standard Specifications for Road and Bridges Construction, current edition. Place Geotextile Fabric, in accordance with Section 843 of the Standard Specifications for Road and Bridge Construction, current edition, between the embankment and the slope protection.

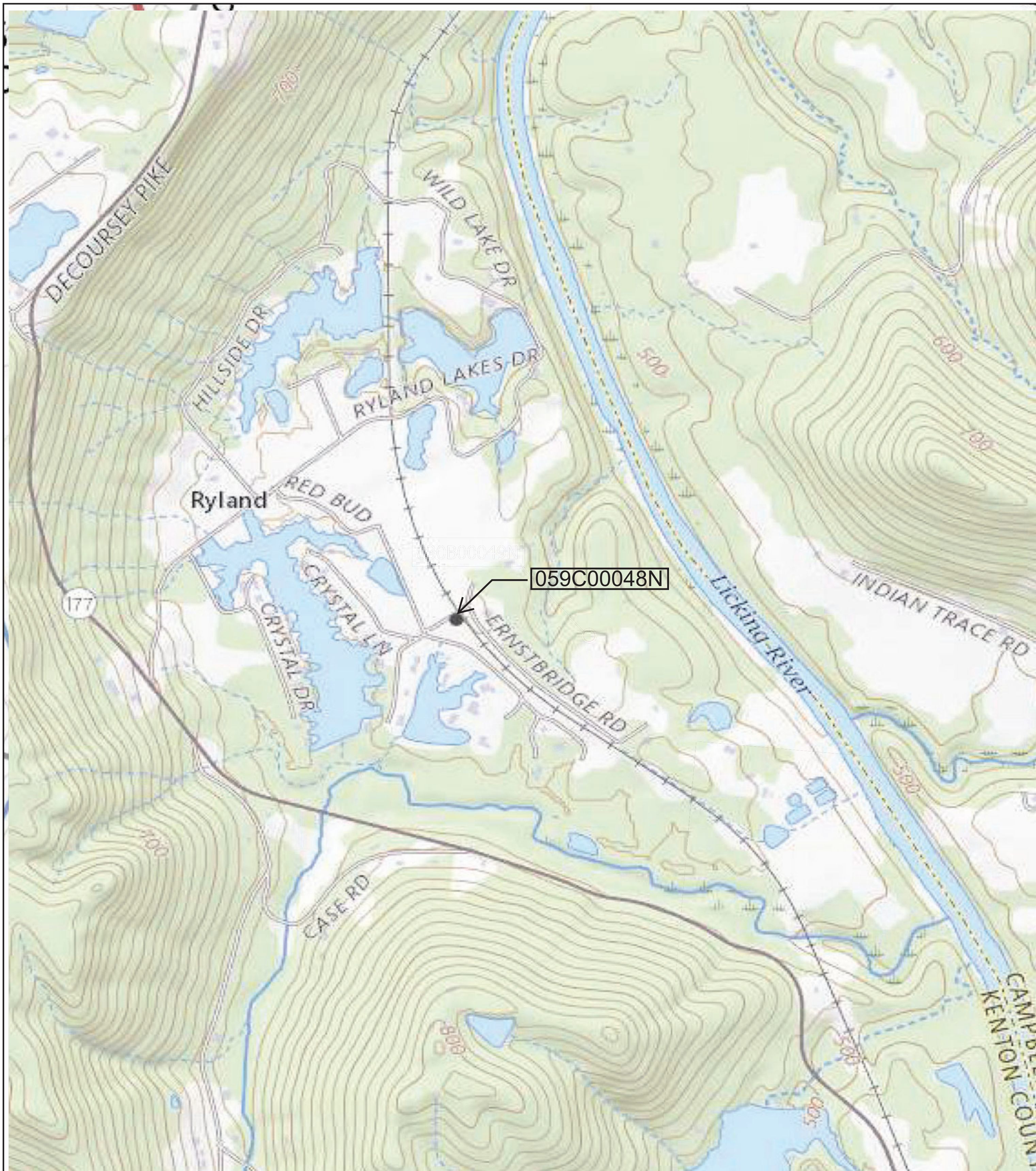
8.4 Temporary shoring or sheeting may be required to facilitate construction.

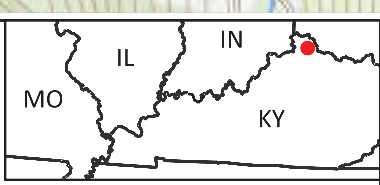
8.5 Field verification of pile capacity shall be performed using the FHWA Modified Gates Formula instead of the formulas provided in the Standard Specifications for Road and Bridge Construction.

Should there be any questions, please contact BFW at (270) 443-1995 for further recommendations.

Attachments:

- **Project Location Map**
- **Subsurface Data Sheet with Boring Locations**
- **Pile Capacity Tables**
- **Coordinate Data Sheet**
- **Original Terracon Geotechnical Engineering Report dated April 8, 2019**





0 600 1,200 ft

1 inch : 667 feet

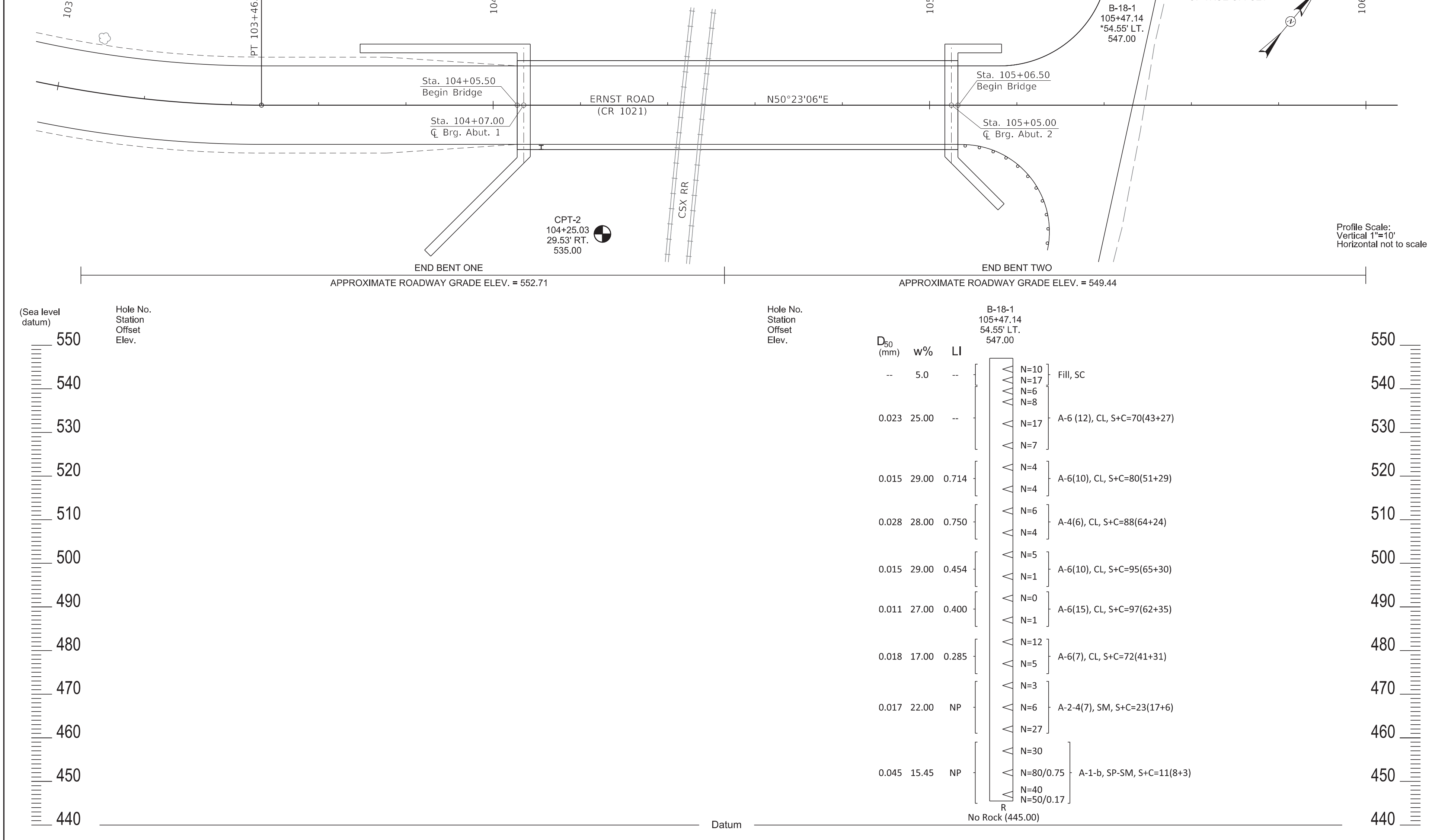
Ernst Road Bridge Over CSX Railroad

38.936389, -84.464722
Kenton County, Ky

Project Number:	Drafted/Checked:	Date:
22349 - 10081	HK/CF	2023-11-28

BFW
BACON | FARMER | WORKMAN
ENGINEERING & TESTING, INC.
500 SOUTH 17TH STREET
PADUCAH, KY 42003

SUBSURFACE DATA



COMMONWEALTH OF KENTUCKY
DEPARTMENT OF HIGHWAYS

DRAWING TITLE: SUBSURFACE DATA FOR ERNST RD. (CR 1021) OVER CSX RAILROAD

HORIZONTAL SCALE
SCALE: N.T.S.

ITEM NO. 6-10046
COUNTY OF KENTON
SHEET NO.

OpenRoads Designer v10.16.2.267 USER: bvollmer FILE NAME: C:\BMS\WSP-US-PW-02\D0732855\059C00048N_SUBSURFACE _DATA.DGN

LRFD Pile Capacities (For Friction Piles)

Abutment 1

County: Kenton
Location: Ernst Road Bridge
Item #: 6-10046

Date: 11/31/2023
Pile Size: HP 12X 53 Steel Piles (Friction)

Base of Pile Cap Assumed to be at approximate elevation*: **527.0 ft**
Finished Grade Elevation: **552.7 ft**
Original Groundline Elevation: **535.0 ft**

Depth Below Pile Cap (ft)	Approximate Elevation (ft)	Soil Type	Nominal Side Resistance		Nominal End Bearing		R_n Total Nominal Geotechnical Axial Resistance **		Static Analysis Method: ϕR_n for design: Total Factored Geotechnical Axial Resistance (Static Analysis Method)		Static Analysis Method: Field Verification Values: FHWA Modified Gates Formula Calculated Resistance		Dynamic Testing Method: ϕR_n for design: Total Factored Geotechnical Axial Resistance Dynamic Testing Method		Dynamic Testing Method: Field Verification Values (EOD) End of Driving Nominal Resistance		Dynamic Testing Method: Field Verification Values (BOR) Beginning of Restrike Nominal Resistance		Uplift: ϕR_n for design: Total Factored Geotechnical Uplift Resistance (Static Analysis Method)	
			Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons
0	527.0	cohesive	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0	0.0	0	0	0
60	467.0	cohesive	14	7	1	0	16	7	6	3	14	6	10	5	145.4	72	270.5	135	4	2
65	462.0	cohesive	48	23	8	3	56	27	19	10	49	24	36	18	185.1	92	310.2	155	12	6
70	457.0	cohesive	85	42	8	3	93	46	33	17	81	40	61	30	222.5	111	347.7	174	21	11
75	452.0	cohesionless	125	62	22	11	147	73	66	33	166	82	96	47	276.8	138	401.9	201	44	22
80	447.0	cohesionless	181	90	22	11	203	101	91	46	228	114	132	65	332.4	166	457.5	229	63	32
85	442.0	cohesionless	239	119	22	11	261	130	117	59	293	146	169	84	390.1	195	515.3	258	83	42

NOTE: Piles for Abutment 1 (Western Abutment) will be subject to downdrag loading. Downdrag effects have already been included in the capacities shown

Factors:

All Capacities are for a Single Pile

Axial Capacity	Static Analysis Method	Gates Analysis Method	Dynamic Analysis Method
Skin Friction and End Bearing in Clays, a-Method (Tomlinson/Skempton)	0.35	0.40	0.65
Skin Friction and End Bearing in Sands, Nordlund/Thurman Method	0.45	0.40	0.65

Uplift Resistance

Clays, a-Method (Tomlinson/Skempton)	0.25
Sands, Nordlund Method	0.35

Driving Resistance Reductions

Cohesive Soils	0.5
Cohesionless Soils	0.25

How to use this table:

Choose the total factored geotechnical axial resistance that equals or exceeds the total factored loads at the strength limit state ($\phi Rn \Rightarrow \Sigma \eta_i \Upsilon_i Q_i$) and use the corresponding depth below pile cap plus the required pile embedment into pile cap to estimate pile tip elevations and the lengths of pile required. The geotechnical report may recommend highest allowable pile tip elevations. Deeper pile tip elevations may be needed to address scour, lateral loads, seismic, and other loading conditions, If the total factored geotechnical axial resistance is chosen from the Static Analysis Method column, then field verification shall be conducted using the FHWA Modified Gates Formula. If the total factored geotecnical axial resistance is chosen from the Dynamic Testing Method column, then field verification by dynamic testing methods is required.

Side Friction Through Embankment Layers (kips): 0

Note: Reported nominal capacities have been adjusted. They are reducted to account or the effects of scour and side friction accumulated through embankment layers has been neglected

*If base of pile cap varies from plan elevation by more than five feet contact the geotechnical engineer for re-evaluation of capacities

** Value calculated using static method

LRFD Pile Capacities (For Friction Piles)

Abutment 2

County: Kenton
Location: Ernst Road Bridge
Item #: 6-10046

Date: 11/30/2023
Pile Size: HP 12 X 53 Steel Piles (Friction)

Base of Pile Cap Assumed to be at approximate elevation*: **528.0** ft
Finished Grade Elevation: **549.4** ft
Original Groundline Elevation: **547.0** ft

Depth Below Pile Cap (ft)	Approximate Elevation (ft)	Soil Type	Nominal Side Resistance		Nominal End Bearing		R_n Total Nominal Geotechnical Axial Resistance **		Static Analysis Method: ϕR_n for design: Total Factored Geotechnical Axial Resistance (Static Analysis Method)		Static Analysis Method: Field Verification Values: FHWA Modified Gates Formula Calculated Resistance		Dynamic Testing Method: ϕR_n for design: Total Factored Geotechnical Axial Resistance Dynamic Testing Method		Dynamic Testing Method: Field Verification Values (EOD) End of Driving Nominal Resistance		Dynamic Testing Method: Field Verification Values (BOR) Beginning of Restrike Nominal Resistance		Uplift: ϕR_n for design: Total Factored Geotechnical Uplift Resistance (Static Analysis Method)	
			Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons
0	528	cohesive	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0	0.0	0	0	0
25	503	cohesive	89	44	1	0	90	44	31	16	79	39	58	29	45.4	22	89.9	45	72	36
30	498	cohesive	106	52	2	0	107	53	38	19	94	46	70	34	54.5	27	107.4	54	88	44
35	493	cohesionless	133	66	2	0	135	67	61	31	152	75	88	43	68.2	34	134.8	67	106	53
40	488	cohesionless	161	80	2	0	162	81	73	37	183	91	106	52	82.0	40	162.4	81	124	62
45	483	cohesionless	188	93	1	0	189	94	85	43	213	106	123	61	95.2	47	189.0	95	141	71
50	478	cohesionless	213	106	1	0	215	107	97	49	242	120	140	69	108.1	54	214.7	107	158	79
55	238	cohesionless	239	119	1	0	240	120	108	54	270	135	156	78	125.8	62	240.1	120	186	93
60	468	cohesionless	270	135	8	3	278	139	125	63	313	156	181	90	163.7	81	278.1	139	213	107
65	463	cohesionless	312	155	8	3	320	159	144	72	360	179	208	103	205.3	102	319.6	160	240	120
70	458	cohesionless	356	177	8	3	364	181	164	82	409	204	236	118	249.2	124	363.5	182	266	133
75	453	cohesionless	411	205	22	11	433	216	195	98	488	243	282	140	319.1	159	433.4	217	292	146
80	448	cohesionless	476	238	22	11	498	249	224	112	560	280	324	161	383.9	191	498.2	249	330	165

Factors:

All Capacities are for a Single Pile

	Static Analysis Method	Gates Analysis Method	Dynamic Analysis Method
Axial Capacity			
Skin Friction and End Bearing in Clays, a-Method (Tomlinson/Skempton)	0.35	0.40	0.65
Skin Friction and End Bearing in Sands, Nordlund/Thurman Method	0.45	0.40	0.65

Uplift Resistance

Clays, a-Method (Tomlinson/Skempton)	0.25
Sands, Nordlund Method	0.35

Driving Resistance Reductions

Cohesive Soils	0.5
Cohesionless Soils	0.25

How to use this table:

Choose the total factored geotechnical axial resistance that equals or exceeds the total factored loads at the strength limit state ($\phi Rn \Rightarrow \Sigma\eta_i\gamma_i Q_i$) and use the corresponding depth below pile cap plus the required pile embedment into pile cap to estimate pile tip elevations and the lengths of pile required. The geotechnical report may recommend highest allowable pile tip elevations. Deeper pile tip elevations may be needed to address scour, lateral loads, seismic, and other loading conditions, If the total factored geotechnical axial resistance is chosen from the Static Analysis Method column, then field verification shall be conducted using the FHWA Modified Gates Formula. If the total factored geotecnical axial resistance is chosen from the Dynamic Testing Method column, then field verification by dynamic testing methods is required.

Side Friction Through Embankment Layers (kips): 0

Note: Reported nominal capacities have been adjusted. They are reducted to account or the effects of scour and side friction accumulated through embankment layers has been neglected

*If base of pile cap varies from plan elevation by more than five feet contact the geotechnical engineer for re-evaluation of capacities

** Value calculated using static method

LRFD Pile Capacities (For Friction Piles)

Abutment 1

County: Kenton
Location: Ernst Road Bridge
Item #: 6-10046

Date: 11/31/2023
Pile Size: HP 14 X 89 Steel Piles (Friction)

Base of Pile Cap Assumed to be at approximate elevation*: **527.0** ft
Middle Row Backfill side)
...ned Grade Elevation: **552.7** ft
...roundline Elevation: **535.0** ft

Depth Below Pile Cap (ft)	Approximate Elevation (ft)	Soil Type	Nominal Side Resistance		Nominal End Bearing		R_n Total Nominal Geotechnical Axial Resistance **		Static Analysis Method: ϕR_n for design: Total Factored Geotechnical Axial Resistance (Static Analysis Method)		Static Analysis Method: Field Verification Values: FHWA Modified Gates Formula Calculated Resistance		Dynamic Testing Method: ϕR_n for design: Total Factored Geotechnical Axial Resistance Dynamic Testing Method		Dynamic Testing Method: Field Verification Values (EOD) End of Driving Nominal Resistance		Dynamic Testing Method: Field Verification Values (BOR) Beginning of Restrike Nominal Resistance		Uplift: ϕR_n for design: Total Factored Geotechnical Uplift Resistance (Static Analysis Method)	
			Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons
0	527.0	cohesive	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0	0.0	0	0	0
60	467.0	cohesive	26	12	2	1	28	14	10	5	25	12	18	9	177.3	88	325.2	163	6	3
65	462.0	cohesive	75	37	13	6	88	44	31	16	77	38	57	28	237.4	118	385.3	193	19	10
70	457.0	cohesive	131	65	13	6	144	72	50	25	126	63	94	46	293.3	146	441.2	221	33	17
75	452.0	cohesionless	190	95	37	18	228	113	102	51	256	128	148	74	376.8	188	524.7	262	67	34
80	447.0	cohesionless	277	138	37	18	314	157	141	71	354	176	204	102	463.5	231	611.4	306	97	49
85	442.0	cohesionless	369	184	37	18	406	202	183	92	457	228	264	131	555.0	277	702.8	351	129	65

Outer Row and backfill side (Train side)

NOTE: Piles for Abutment 1 (Western Abutment) will be subject to downdrag loading. Downdrag effects have already been included in the capacities shown

Factors:

All Capacities are for a Single Pile

Axial Capacity	Static Analysis Method	Gates Analysis Method	Dynamic Analysis Method
Skin Friction and End Bearing in Clays, a-Method (Tomlinson/Skempton)	0.35	0.40	0.65
Skin Friction and End Bearing in Sands, Nordlund/Thurman Method	0.45	0.40	0.65

Uplift Resistance

Clays, a-Method (Tomlinson/Skempton)	0.25
Sands, Nordlund Method	0.35

Driving Resistance Reductions

Cohesive Soils	0.5
Cohesionless Soils	0.25

How to use this table:

Choose the total factored geotechnical axial resistance that equals or exceeds the total factored loads at the strength limit state ($\phi Rn => \Sigma \eta_i \Upsilon_i Q_i$) and use the corresponding depth below pile cap plus the required pile embedment into pile cap to estimate pile tip elevations and the lengths of pile required. The geotechnical report may recommend highest allowable pile tip elevations. Deeper pile tip elevations may be needed to address scour, lateral loads, seismic, and other loading conditions, If the total factored geotechnical axial resistance is chosen from the Static Analysis Method column, then field verification shall be conducted using the FHWA Modified Gates Formula. If the total factored geotecnical axial resistance is chosen from the Dynamic Testing Method column, then field verification by dynamic testing methods is required.

Side Friction Through Embankment Layers (kips): 0

Note: Reported nominal capacities have been adjusted. They are reducted to account or the effects of scour and side friction accumulated through embankment layers has been neglected

*If base of pile cap varies from plan elevation by more than five feet contact the geotechnical engineer for re-evaluation of capacities

** Value calculated using static method

LRFD Pile Capacities (For Friction Piles)

Abutment 2

County: Kenton
Location: Ernst Road Bridge
Item #: 6-10046

Date: 11/30/2023
Pile Size: HP 14 X 89 Steel Piles (Friction)

Base of Pile Cap Assumed to be at approximate elevation*: **528.0 ft**
Finished Grade Elevation: **549.4 ft**
Original Groundline Elevation: **547.0 ft**

for uplift piles

(ft)	te (ft)	Soil Type	Nominal Side Resistance		Nominal End Bearing		R_n Total Nominal Geotechnical Axial Resistance **		Static Analysis Method: ϕR_n for design: Total Factored Geotechnical Axial Resistance (Static Analysis Method)		Static Analysis Method: Field Verification Values: FHWA Modified Gates Formula Calculated Resistance		Dynamic Testing Method: ϕR_n for design: Total Factored Geotechnical Axial Resistance Dynamic Testing Method		Dynamic Testing Method: Field Verification Values (EOD) End of Driving Nominal Resistance		Dynamic Testing Method: Field Verification Values (BOR) Beginning of Restrike Nominal Resistance		Uplift: ϕR_n for design: Total Factored Geotechnical Uplift Resistance (Static Analysis Method)	
			Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons	Kips	Tons
0	528	cohesive	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0	0.0	0	0	0
25	503	cohesive	104	51	1	0	105	52	37	19	92	45	68	34	53.2	26	105.0	52	72	36
30	498	cohesive	124	61	3	1	127	63	44	22	111	55	82	41	64.6	32	126.5	63	88	44
35	493	cohesionless	157	78	3	1	159	79	72	36	179	89	104	51	81.1	40	159.4	80	106	53
40	488	cohesionless	190	94	3	1	192	96	87	44	216	108	125	62	97.6	48	192.4	96	124	62
45	483	cohesionless	222	110	2	1	224	112	101	51	252	126	146	72	113.3	56	224.2	112	141	71
50	478	cohesionless	253	126	2	1	255	127	115	58	287	143	166	82	128.7	64	255.0	127	158	79
55	238	cohesionless	285	142	2	1	288	143	129	65	324	161	187	93	152.2	76	287.7	144	186	93
60	468	cohesionless	331	165	13	6	345	172	155	78	388	193	224	111	209.0	104	344.5	172	213	107
65	463	cohesionless	393	196	13	6	406	203	183	92	457	228	264	132	271.0	135	406.4	203	240	120
70	458	cohesionless	459	229	13	6	472	235	212	106	531	265	307	153	336.4	168	471.9	236	266	133
75	453	cohesionless	543	271	37	18	581	290	261	131	653	326	377	188	445.2	222	580.7	290	292	146
80	448	cohesionless	643	321	37	18	681	340	306	153	766	382	442	221	545.3	272	680.7	340	330	165

Factors:

All Capacities are for a Single Pile

Axial Capacity	Static Analysis Method	Gates Analysis Method	Dynamic Analysis Method
Skin Friction and End Bearing in Clays, a-Method (Tomlinson/Skempton)	0.35	0.40	0.65
Skin Friction and End Bearing in Sands, Nordlund/Thurman Method	0.45	0.40	0.65

Uplift Resistance

Clays, a-Method (Tomlinson/Skempton)	0.25
Sands, Nordlund Method	0.35

Driving Resistance Reductions

Cohesive Soils	0.5
Cohesionless Soils	0.25

How to use this table:

Choose the total factored geotechnical axial resistance that equals or exceeds the total factored loads at the strength limit state ($\phi Rn => \Sigma\eta_i\gamma_iQ_i$) and use the corresponding depth below pile cap plus the required pile embedment into pile cap to estimate pile tip elevations and the lengths of pile required. The geotechnical report may recommend highest allowable pile tip elevations. Deeper pile tip elevations may be needed to address scour, lateral loads, seismic, and other loading conditions, If the total factored geotechnical axial resistance is chosen from the Static Analysis Method column, then field verification shall be conducted using the FHWA Modified Gates Formula. If the total factored geotecnical axial resistance is chosen from the Dynamic Testing Method column, then field verification by dynamic testing methods is required.

Side Friction Through Embankment Layers (kips): 0

Note: Reported nominal capacities have been adjusted. They are reducted to account or the effects of scour and side friction accumulated through embankment layers has been neglected

*If base of pile cap varies from plan elevation by more than five feet contact the geotechnical engineer for re-evaluation of capacities

** Value calculated using static method

COORDINATE DATA SUBMISSION FORM
KYTC DIVISION OF STRUCTURAL DESIGN - GEOTECHNICAL BRANCH

County Kenton

Date 11/29/2023

Road Number Ernst Road

Survey Crew / Consultant BFW

Contact Person Chris Farmer

Item # 06-10046

Mars#

Project #

Notes:

(circle one)
Elevation Datum NAVD88 **Assumed**

HOLE NUMBER	LATITUDE (Decimal Degrees)	LONGITUDE (Decimal Degrees)	HOLE NUMBER	STATION	OFFSET	ELEVATION (FT)
1 - SPAN BRIDGE - ERNST ROAD OVER CSX RAILROAD						
B-18-1	38.9367	-84.4645	B-18-1	105+47.14	54.55' LT	547.00
CPT -1	38.9363	-84.4649	CPT-1	104+25.03	29.53' RT	535

Geotechnical Engineering Report

Ernstbridge Road Bridge Replacement over CSX Railroad
Kenton County, Kentucky
April 8, 2019
Terracon Project No. N1185278

Prepared for:
WSP USA, Inc.
Cincinnati, Ohio

Prepared by:
Terracon Consultants, Inc.
Cincinnati, Ohio

April 8, 2019

WSP USA, Inc.
312 Elm Street, Suite 2500
Cincinnati, Ohio 45202

Attn: Mr. Michael Zwick – Bridge Design Practice Leader
P: (513) 639 2112
E: michael.zwick@wsp.com

Re: Geotechnical Engineering Report
Ernstbridge Road Bridge Replacement over CSX Railroad
Ernstbridge Road
Kenton County, Kentucky
Terracon Project No. N1185278

Dear Mr. Zwick:

We have completed the Geotechnical Engineering services for the above-referenced project. This study was performed in general accordance with Terracon Proposal No. PN1185278 dated July 18, 2018. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork, the design and construction of foundations and MSE walls for the proposed project.

Terracon will provide geotechnical drawing sheets in a separate submittal. WSP and Terracon will need to discuss the sheets that will be required

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

Jeffrey D. Dunlap, P.E.
Senior Geotechnical Engineer

Ronald J. Ebelhar, P.E
Senior Consultant

REPORT TOPICS

Note: This report was originally delivered in a web-based format. **Orange Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the ***GeoReport*** logo will bring you back to this page. For more interactive features, please view your project online at client.terracon.com.

ATTACHMENTS

Note: Refer to each individual Attachment for a listing of contents.

Geotechnical Engineering Report

Ernstbridge Road Bridge Replacement over CSX Railroad
Ernstbridge Road
Kenton County, Kentucky
Terracon Project No. N1185278
April 8, 2019

INTRODUCTION

This report presents the results of our subsurface exploration and geotechnical engineering services performed for the proposed Bridge Replacement over the existing CSX Railroad to be located on Ernstbridge Road in Kenton County, Kentucky. The purpose of these services is to provide information and geotechnical engineering recommendations relative to:

- Subsurface soil conditions
- Groundwater conditions
- Site preparation and earthwork
- Pavement design and construction
- Foundation design and construction
- MSE wall design and construction
- Seismic site classification per IBC

The geotechnical engineering Scope of Services for this project included the advancement of one test boring and one cone penetration test (CPT) sounding to depths ranging from approximately 64 to 102 feet below existing site grades. An offset CPT sounding was also performed in an attempt to obtain CPT data at greater depths, but the offset CPT encountered refusal at approximately 65 feet below existing site grade.

Maps showing the site and boring locations are shown in the **Site Location** and **Exploration Plan** sections, respectively. The results of the laboratory testing performed on soil samples obtained from the site during the field exploration are included on the boring logs and/or as separate graphs in the **Exploration Results** section.

SITE CONDITIONS

The following description of site conditions is derived from our site visit in association with the field exploration and our review of publicly-available geologic and topographic maps.

Item	Description
Parcel Information	The project is located at Ernstbridge Road in Kenton County, Kentucky. See **Site Location**

Item	Description
Existing Improvements	Existing wood deck bridge with wood and steel girders. The bridge is approximately 18 feet in width. The bridge is supported on four piers constructed of wood and on concrete abutment walls. Several segmental block retaining walls have been constructed around the existing abutments and act as wingwalls.
Current Ground Cover	Ground cover below and adjacent to the bridge consists of grass and weed vegetation with sparse trees and some brush. There are also areas of gravel (ballast) around the existing railroad tracks. The approaches at the east and west abutments consist of asphalt pavement and the bridge has a wood deck.
Existing Topography (from GoogleEarth™)	Road grades at the existing bridge approaches are about Elevation 548 feet. The west road approach gently slopes downward to about Elevation 540 feet. The east road approach gently slopes to the northeast and south to between about Elevation 539 feet. The existing grade at the existing railroad tracks beneath the bridge is about Elevation 527 feet. The grade between the railroad tracks and the bridge approaches are supported by a series of tiered retaining walls on the west side and a single retaining wall on the east side of the existing railroad tracks.
Geology	Based on published topographic and geologic maps the site lies in the Licking River valley and is mapped as terrace deposits which contain lacustrine deposits eventually underlain by granular outwash deposits and then Ordovician Age bedrock. The encountered soil conditions in the recent test boring and CPT soundings are consistent with this geologic setting, except that man-placed existing fill soils were encountered immediately below the ground surface that are associated with the existing bridge construction and surrounding site development. The depth to bedrock at the site is estimated to be greater than 100 feet.

PROJECT DESCRIPTION

Our initial understanding of the project was provided in our proposal and was discussed during project planning. A period of collaboration has transpired since the project was initiated, and our final understanding of the project conditions is as follows:

Item	Description
Information Provided	Information regarding the proposed bridge replacement project has been provided on the Plan and Elevation drawing and foundation load information from WSP received via e-mails on March 29, and April 3, 2019. The original plate arch culver structure has been replaced with a steel bridge structure.
Project Description	Replace the existing bridge using a 2-span steel bridge structure with integral abutments. A spill-through type abutment is proposed at the west abutment and an MSE wingwall and abutment wall is proposed at the east abutment.
Proposed Structure	A planned 2-span steel plate girder bridge with concrete decking is proposed. The total bridge length is 120 feet. Span 1 from the west integral abutment to the interior pier is 45 feet in length. Span 2 from the interior pier to the west integral abutment has a proposed length of 75 feet. The planned width of the bridge is 23 feet. A 2.5H:1V spill-through abutment slope is proposed below the west abutment. The height of the proposed slope is about 20 feet. An MSE retaining wall will act as the abutment wall at the east abutment. The height of the proposed abutment wall is about 25 feet and the abutment will extend about 5 feet above the top of the MSE abutment wall. Wingwalls will support the fill required to construct the east bridge abutment areas. The maximum height of these wingwalls will be about 25 feet tall (30 feet with wall embedment). The south abutment wall has a proposed length of about 25 feet and the north wingwall curves into the existing slope and has a length of about 45 feet.
Crossing Construction	Steel plate girder bridge structure with cast-in-place decking. Concrete integral abutments and concrete interior pier will be pile supported
Maximum Loads (Need to be confirmed	■ West abutment piles have a maximum factored axial load of 100 kips each. ■ Interior bridge pier piles have a maximum factored axial load of 220 kips each. ■ East abutment piles have a maximum factored axial load of 120 kips each.
Grading/Slopes	Up to 25 feet of fill will be required to develop final grade at the east bridge abutment. Up to 4 feet of fill will be added to the existing road profile. At the proposed west abutment, the proposed abutment 2.5H:1V spill-through slope will require up to 14 feet of cut. Up to about 3 feet of fill will be added to the existing road profile at the abutment location. Final slope angles of as steep as 2.5H:1V (Horizontal: Vertical) are expected beyond the east MSE wingwalls and around the west abutment.

Item	Description
Below Grade Structures	None are anticipated
Free-Standing Retaining Walls	MSE abutment and wingwalls are proposed to support the fill between the existing road and the proposed east bridge abutment. Maximum planned wall height is on the order of about 30 feet (including wall embedment below toe grade). The south abutment wall has a proposed length of about 25 feet and the north wingwall curves into the existing slope and has a length of about 45 feet. Crest slopes are anticipated to be level with traffic loads above the abutment wall. Crest slopes are anticipated to be between 2.5H:1V and 3H:1V above the proposed wingwalls. Toe slopes are anticipated to be nearly level in the vicinity of the rail road alignment and transition to 2.5H:1V slopes to the east along the north wingwall.
Pavements	New asphalt pavement is proposed above approaches to the new bridge. Concrete approach slabs are anticipated behind each abutment.
Estimated Start of Construction	Late 2019 or 2020

GEOTECHNICAL CHARACTERIZATION

Subsurface Profile

We have developed a general characterization of the subsurface soil and groundwater conditions based upon our review of the data and our understanding of the geologic setting and planned construction. The following table provides our geotechnical characterization.

The geotechnical characterization forms the basis of our geotechnical calculations and evaluation of site preparation, foundation options and pavement subgrade options. Due to site access and railroad right-of-way, borings and soundings were located as near as practical, but not at the proposed abutments or interior pier. As noted in **General Comments**, the characterization is based mainly upon Boring B-18-1, and variations across the site are likely.

Stratum	Approximate Depth to Bottom of Stratum (feet)	Material Description	Consistency/Density
Surface	0.4	Asphalt Pavement	N/A

Stratum	Approximate Depth to Bottom of Stratum (feet)	Material Description	Consistency/Density
Surface	0.8	Granular based, crushed stone	N/A
1	5.5	Existing fill – lean clay with sand and sand seams, trace fine gravel, brown, A-6(12)	Not reported
2	23	Lean clay to silty clay, trace sand and concretions, occasionally weakly laminated structure, mottled brown and gray	Stiff to very stiff
3	35	Lean clay to silty clay trace concretions and silt laminations, brown trace gray to brownish-gray	Soft to medium stiff
4	42	Lean clay, trace concretions, sand and interbedded silt and sand partings to seams[1], gray, A-6(6)	Soft to stiff
5	63	Lean clay, trace concretions, sand and interbedded silt and sand partings to seams[1], gray, A-6(10) and A-6(15)	Medium stiff to stiff
6	73	Lean clay with sand, trace sand pockets and gravel, bluish-gray, A-6(7)	Stiff
7	78	Clayey sand with interbedded silty sand and silt lenses, fine to coarse grained, gray	Very loose to loose
8	92[2]	Silty Sand, trace clayey sand seams and gravel, fine to medium grained, gray, A-2-4(0)	Loose to medium dense

Stratum	Approximate Depth to Bottom of Stratum (feet)	Material Description	Consistency/Density
2	Undetermined: Boring 18-1 terminated within this stratum at approximately 102 feet	Poorly graded sand with silt and gravel, trace cobbles, fine to medium grained, gray, A-1-b(0)	Dense

1. CPT-1, CPT-1A and CPT-1B confirm the presence of interbedded silt or sand seams to partings.
2. It is believed that CPT-1 and CPT-1B met refusal on this stratum due to gravel in the soil stratum

Conditions encountered at the boring and CPT locations are indicated on the individual boring log and CPT sounding logs shown in the **Exploration Results** section and are attached to this report. Stratification boundaries on the boring log represents the approximate location of changes in native soil types; in situ, the transition between materials may be gradual.

Groundwater Conditions

The borehole was observed while drilling and after completion for the presence and level of groundwater. In addition, pore pressure dissipation tests were performed in CPT-1 at three depths, where only one dissipation test (performed about 25 feet deep) was able to completely dissipate in the time the test was performed. One of the dissipation tests at around 40 feet deep began to dissipate, but did not completely dissipate. The third test performed around 60 feet deep lost connection with the computer and the test data was not complete. The water levels observed in the boreholes and CPT soundings can be found on the boring and sounding logs in **Exploration Results**, and are summarized below.

Boring/CPT Sounding Number	Approximate Depth to Groundwater while Drilling (feet) [1]	Approximate Depth to Groundwater after Drilling (feet) [1]
B-18-1	30	24 (0 hr. reading)
CPT-1	12[2]	--

1. Below ground surface
2. Pore pressure dissipation tests were performed in CPT-1 in an attempt to estimate groundwater levels. Only one of the tests completely dissipated and indicated a water level at 5.2 feet. However, observation of the pore pressure readings indicates that the static water level in CPT-1 is approximately 25 feet using the pore pressures encountered in the interbedded granular seams within the soil profile.

Groundwater was observed in the boring while drilling, for the short duration the boring could remain open. However, this does not necessarily mean the water levels summarized above are stable groundwater levels. Due to the low permeability of the soils encountered in the boring and CPT soundings, a relatively long period may be necessary for a groundwater level to develop and stabilize in a borehole. Long-term observations in piezometers or observation wells sealed from the influence of surface water are often required to define groundwater levels in materials of this type.

The CPT soundings do indicate that seams and partings of silt and sand are present within the lacustrine fine-grained soil profile. As a result, perched water should be anticipated within silt or sand seams or partings sandwiched between less permeable cohesive soils. These seams and partings could be encountered at any depth within the soil profile. In addition, seams and partings may or may not be hydraulically connected to seams and partings of silt and sand across the site and these seams and partings may or may not be subject to recharge.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borings were performed. Therefore, groundwater levels during construction or at other times in the life of the structure may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

GEOTECHNICAL OVERVIEW

Encountered soils in the test boring and the CPT sounding consisted mainly of medium stiff to stiff natural cohesive lean clay and silty clay soils (A-6 and A-4 soils), likely of lacustrine origin. The depth of the cohesive soils ranged from about 65 to 73 feet in the explorations. A transitional clayey sand soil was encountered in Boring B-18-1 to a depth of about 78 feet below grade. Loose to medium-dense silty sand (A-2-4) soil was encountered to a depth of 92 feet in Boring B-18-1, which was underlain by dense poorly-graded sand with silt and gravel with occasional cobbles (A-1-b) to the termination depth of 102 feet in Boring B-18-1.

The near-surface fine-grained lean clay to silty clay soils could become unstable with typical earthwork and construction traffic, especially after precipitation events. Effective drainage should be completed early in the construction sequence and maintained after construction to avoid potential issues. If possible, the grading should be performed during the warmer and drier times of the year. If grading is performed during the winter months, an increased risk for possible undercutting and replacement of unstable subgrade will persist. Additional site preparation recommendations, fill placement, are provided in the **Plan Notes** section.

The soils which form the bearing stratum for are considered compressible and are not considered suitable for shallow footing support of the bridge abutments and interior pier. Driven steel closed-end pipe piles bearing in either the encountered silty sand (A-2-4) or poorly graded sand with silt and gravel with occasional cobbles (A-1-b) are recommended for support of the bridge structure. Due to the approximate 25 feet of new MSE fill required at the east abutment, the piles supporting the east abutment will be subjected to down drag loads that need to be considered in the pile foundation design at the east abutment. The **Deep Foundations** section addresses support of the bridge on driven, closed-end pipe piles.

The load added to the foundation soils from the proposed MSE retaining wall construction and new fill placement at the east abutment will result in foundation soil settlement. Using the undrained soil shear strengths from the CPT sounding, it appears that the foundation soils at the MSE retaining wall locations are suitable for support of the MSE wall with regard to bearing resistance. Further discussion with regard to the MSE wall design and construction are included in the **MSE Structures** section

Both rigid pavement Portland cement concrete approach slabs and flexible asphalt pavements are proposed as part of the new bridge project. The **Pavements** section addresses the recommended pavement support parameter for design of the proposed pavement systems.

Geotechnical plans will need to be provided at a later date. We will discuss with WSP the sheets that Terracon will provide for the project, prior to providing plan sheets.

The **General Comments** section provides an understanding of the report limitations.

DEEP FOUNDATIONS

Driven Pile Design Parameters

The following tables can be used to estimate resistances for individual, closed-end pipe piles. The values are nominal resistance values carrying capacity for driven piles having pile tip Elevations ranging from Elevation 475 to 460 feet. Driven piles should be spaced at least three pile widths apart (center-to-center) if side friction is used for compressive loads. The abutment piles can bear within the stiff overburden soils or within the underlying silty sand (A-2-4) soils, provided the pile factored resistances are greater than the factored loads. At the interior pier we recommend the piles bear within the granular silty sand (A-2-4) soils. Parameters for both design cases are provided in the following tables.

West Bridge Abutment (CPT-1/B-18-1) Driven Pile Design Summary [1, 2]			
Bearing Material	**Pile Type/Size**	**Nominal Resistance (kips)**	**Anticipated Bearing Elevation (feet)**
Silt and Clay (A-4 and A-6)	12-inch-diameter Pipe Pile	120	475

West Bridge Abutment (CPT-1/B-18-1) Driven Pile Design Summary [1, 2]			
Bearing Material	**Pile Type/Size**	**Nominal Resistance (kips)**	**Anticipated Bearing Elevation (feet)**
Silty Sand (A-2-4)	12-inch-diameter Pipe Pile	270	465

1. Nominal resistances are will need to be factored. Resistance factors are dependent upon the method of installation, and quality control parameters. Assuming dynamic load testing will be performed, a resistance factor of 0.7 should be applied to the nominal resistance.
2. See test boring logs and CPT logs for more details on Stratigraphy. Boring B-18-1 was used for soil stratigraphy below about Elevation 470 feet.

Interior Pier (CPT-1/B-18-1) Driven Pile Design Summary [1, 2]			
Bearing Material	**Pile Type/Size**	**Nominal Resistance (kips)**	**Anticipated Bearing Elevation (feet)**
Silty Sand (A-2-4)	12-inch-diameter Pipe Pile	330	460

1. Nominal resistances are will need to be factored. Resistance factors are dependent upon the method of installation, and quality control parameters. Assuming dynamic load testing will be performed, a resistance factor of 0.7 should be applied to the nominal resistance.
2. See test boring log and CPT logs for more details on Stratigraphy. Boring B-18-1 was used for soil stratigraphy below about Elevation 470 feet.

East Bridge Abutment (B-18-1) Driven Pile Design Summary [1, 2]			
Bearing Material	**Pile Type/Size**	**Nominal Resistance (kips)**	**Anticipated Bearing Elevation (feet)**
Silt and Clay (A-4 and A-6)	12-inch-diameter Pipe Pile	150	475
Silty Sand (A-2-4)	12-inch-diameter Pipe Pile	270	465
Silty Sand (A-2-4)	12-inch-diameter Pipe Pile	330	460

East Bridge Abutment (B-18-1) Driven Pile Design Summary [1, 2]			
Bearing Material	**Pile Type/Size**	**Nominal Resistance (kips)**	**Anticipated Bearing Elevation (feet)**

1. Nominal resistances are will need to be factored. Resistance factors are dependent upon the method of installation, and quality control parameters. Assuming dynamic load testing will be performed, a resistance factor of 0.7 should be applied to the nominal resistance.
2. See test boring logs for more details on Stratigraphy.

At the east bridge abutment, up to 3 inches of foundation soil settlement is estimated due to construction of the proposed MSE fill. As a result, the piles at the east abutment will be subject to down drag loads. The neutral axis along the pile, where ¼ inches of relative movement between the pile and the foundation soil occurs, is at about Elevation 485 feet. As a result, each of the east abutment piles will be subjected to a nominal down drag load of 135 kips. A load factor of 1.4 should be applied since the alpha method (Tomlinson method) was used to estimate the static pile capacity. The estimated down drag load assumes the piles are sleeved through the granular MSE fill soil, thus no down drag load over the portion of the pile within the MSE fill zone.

Driven Pile Lateral Loading

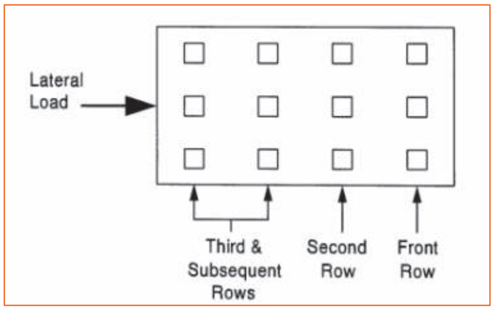
The following table lists input values for use in LPILE analyses. LPILE will estimate values of k_h and ε_{50} based on strength; however, non-default values of k_h should be used where provided. Since deflection or a service limit criterion will likely control lateral capacity design, no safety/resistance factor is included with the parameters.

Stratigraphy [1] Material	L-Pile Soil Model	S_u (psf) [2]	γ (pcf) [2,3]	ε_{50} [2]	k (pci)	Internal Angle of Friction (degrees)
Stiff to very stiff soil above Elevation 525 feet,	Stiff Clay w/o Free Water	2,500	125	0.005	--	--
Stiff to very stiff soil with some medium stiff zones below Elevation 525 feet,	Stiff Clay w/o Free Water	1,500	124	0.007	--	--
Silty Sand	Sand	--	128	--	100	35

Stratigraphy [1] / Material	L-Pile Soil Model	S_u (psf) [2]	γ (pcf) [2,3]	ε_{50} [2]	k (pci)	Internal Angle of Friction (degrees)
Poorly-Graded Sand	Sand	--	130	--	125	38

1. See test boring log and CPT logs more details on Stratigraphy.
2. Definition of Terms:
   S_u: Undrained shear strengthγ: Moist unit weight
   ε_{50}: Non-default E50 strain
   k: Lateral subgrade modulus
3. Buoyant unit weight values should be used below water table.

When piles are used in groups, the lateral capacities of the piles in the second, third, and subsequent rows of the group should be reduced as compared to the capacity of a single, independent pile. Alternatively, the piles could be battered to provide additional lateral support. Guidance for applying p-multiplier factors to the p values in the p-y curves for each row of pile foundations within a pile group are as follows:



- Front row: P_m = 0.8;
- Second row: P_m = 0.4
- Third and subsequent row: P_m = 0.3.

The load capacities provided herein are based on the stresses induced in the supporting soil strata. The structural capacity of the piles should be checked to assure they can safely accommodate the combined stresses induced by axial and lateral forces. Lateral deflections of piles should be evaluated using an appropriate analysis method, and will depend upon the pile's diameter, length, configuration, stiffness and "fixed head" or "free head" condition. We can provide additional analyses and estimates of lateral deflections for specific loading conditions upon request. The load-carrying capacity of piles may be increased by increasing the diameter or wall thickness (for pipe piles) and/or length.

Driven Pile Construction Considerations

We have performed preliminary WAVE equation analyses for the recommended driven Grade 50 12-inch-diameter closed-end pipe piles with 3/8-inch wall thickness. The analyses indicate that a Delmag D16-32 or D22 pile hammer (40 kip-ft rated energy) is capable of driving the piles to the anticipated tip elevation without overstressing the piles and keeping the hammer blows to less than 10 blows/per inch. A Delmag D15 (27 kip-ft rated energy) appears capable of also driving the piles, but the necessary blows to reach the anticipated tip elevation begins to approach 15 blows/inch. We recommend that the piling contractor subcontract an independent pile testing subcontractor to perform WAVE equation analyses prior to driving the piles and submit the results to the engineer for approval, prior to driving any piles.

Due to the fine-grained nature of the cohesive soils, excess pore pressures will likely develop during driving. As a result, the pile resistance during driving will likely be less than the long-term static resistance of the piles. The resistance in the cohesive soils will likely increase with time (soil set up), once the excess pore pressures dissipate after driving the piles. Driving resistance within the cohesive soils could be as low as 50% of the static resistance. The set up can only be determined by restriking the piles approximately 7 days (or longer) after the initial driving of the pile. If restrikes of the piles with PDA monitoring is not performed, it is anticipated that the resistance during driving will be much less than the long-term static resistance of the driven piles, which could lead to overruns of pile length during pile driving operations.

If practical refusal is experienced above the design tip elevation, the pile may be on a boulder or other obstruction and a replacement pile should be driven. If this occurs, the situation should be evaluated by Terracon during the pile driving operations.

The contractor should be prepared to cut or splice piles, as necessary. Splicing of piles should be in accordance with specifications provided by the project Structural Engineer.

Pile driving conditions, hammer efficiency, and stress on the pile during driving could be better evaluated during installation using a Pile Driving Analyzer (PDA). A Terracon representative should observe pile driving operations. Each pile should be observed and checked for buckling, crimping and alignment in addition to recording penetration resistance, depth of embedment, and general pile driving operations. We recommend that at least 1 test pile be designated at each substructure location (west abutment, interior pier and east abutment) for Pile Driving Analyzer testing per KYTC procedures.

The existing facility (structures and subsurface utilities) should be observed prior to pile installation to document their condition. Structures should also be observed during pile installation for indications of movement. Pile driving should be stopped and Terracon contacted if movement or cracking of the existing structures is observed. Monitoring vibration levels during pile driving should be considered. Although vibrations from pile driving may be below levels that will cause structural damage, they may be felt by occupants of the adjacent buildings. The potential impact of driving piles at this site should be considered when evaluating this alternative.

The pile driving process should be performed under the direction of the Geotechnical Engineer. The Geotechnical Engineer should document the pile installation process including soil/rock and groundwater conditions encountered, consistency with expected conditions, and details of the installed pile.

SEISMIC CONSIDERATIONS

The seismic design requirements for buildings and other structures are based on Seismic Design Category. Site Classification is required to determine the Seismic Design Category for a structure. The Site Classification is based on the upper 100 feet of the site profile defined by a weighted average value of either shear wave velocity, standard penetration resistance, or undrained shear strength in accordance with AASHTO Code. Based on the soil properties encountered at the site and as described on the exploration logs and results, it is our professional opinion that the **Seismic Site Classification is D**. Subsurface explorations at this site were extended to a maximum depth of 102 feet. Additional geophysical testing may be performed to confirm the conditions estimated using the conditions at Boring B-18-1 and CPT-1 boring depth.

MSE STRUCTURES

The provided plans indicate that the maximum MSE retaining wall height is on the order of 20 feet, with an abutment wall having a height of around 9 feet above the crest of the MSE wall (29 feet total height). For the purposes of our analyses, we have assumed a maximum wall height of 30 feet (measured from the top of leveling pad to the top of the pavement). We recommend that any vegetation and near-surface topsoil, soft soil or soils containing organics be completely removed prior to the MSE wall and leveling pad construction. The existing natural soils are considered suitable bearing materials, provided they are in an at least stiff condition when exposed during excavation.

The prepared subgrade for the MSE wall reinforced zone should extend a minimum of 3 feet beyond the outer edges of the MSE wall and across the entire reinforced zone. Following excavation, the exposed surface should be inspected. The natural cohesive materials will likely become disturbed during construction activities; therefore, a minimum 12-inch-thick layer of compacted DGA crushed stone should be placed across the reinforced zone. This layer of compacted DGA will help provide a stable working surface during the initial wall construction.

Per typical KYTC practice, the MSE wall construction will involve the use of granular backfill soil (reinforced zone) and thin metallic strips to form a gravity mass capable of supporting or restraining imposed loads. The backfill material should consist of compacted select granular in the reinforced zone, behind the MSE panel facing. The MSE wall should be designed to satisfy internal and external stability. For external stability, a vertical reinforced soil structure must satisfy the same external design criteria as a conventional retaining wall. Terracon performed geotechnical analyses for external stability, which include sliding as a rigid body at or below the base, eccentricity, bearing capacity failure, and rotation slip-surface failure (global stability). The design of the wall structure for internal stability is typically performed by the contractor/manufacturer. Terracon did not perform internal stability and compound stability analyses for this project. Please refer to the **Slope Stability** section for results of the global stability analyses

FHWA criteria indicate that reinforcement lengths in mechanically stabilized earth walls should have a minimum length of 70 percent of the total wall height or a minimum value of 8 feet, whichever is greater. The vertical MSE retaining structures must be designed to resist lateral earth pressures and surcharge pressures transferred from the traffic surcharge (a minimum of 250 psf traffic loading should be applied).

The design of this type of system requires that the interface friction should resist the soil pressure from the backfill layer between reinforcements, that the reinforcement length is long enough to support the interface friction and provide a stable mass, and that the reinforcement is strong enough to resist the tensile forces that develop. The length of reinforcements must be extended beyond the zone of Rankine failure. We recommend select granular backfill be placed behind and within the vertical reinforced soil structure in accordance with KYTC Standard Specifications Item 805.12. The following values are recommended for the design parameters for the MSE wall.

1. MSE Reinforced Zone Backfill (select granular backfill)

   γ_s = 120 pcf
   ϕ = 34°
   K_a = 0.28
   (Note that a free-draining granular zone immediately behind the wall should be a minimum of 2 feet thick.)

2. Retained Soils [natural cohesive soil, new embankment fill or reinforced fill (at the wing wall locations the reinforced fill for the abutment MSE wall will likely be the retained soil)] are based upon consolidated-undrained triaxial compression tests with pore pressure measurement performed on undisturbed samples of natural cohesive soil, or compacted samples of new embankment fill

   Natural Cohesive Soil
   γ_s = 125 pcf
   ϕ = 28°
   K_a = 0.36
   Reinforced Fill
   γ_s = 120 pcf
   ϕ = 24°
   K_a = 0.28

3. Foundation Soils (natural cohesive soils)

   γ_s = 124 to 125 pcf
   ϕ' = 28°
   c' = 100 psf
   c_u = 2000 psf, based on undrained shear strength from CPT correlations

The external stability of the MSE walls were evaluated with the MSEW 3.0 software using the minimum reinforcement lengths of 75% to 100% of the wall height (please note the minimum reinforcement lengths were controlled by global stability), which is defined as the height from the top of the proposed leveling pad to the proposed road grade. The capacity demand ratios (CDR) were calculated for the bearing capacity and the sliding resistance of the MSE walls using LRFD methods outlined by AASHTO. The CDR value is defined as the factored resistance divided by the factored loads; thus a CDR value greater than 1.0 indicates the factored resistance is greater than the factored loads. The calculated CDR values for sliding and bearing failure were greater than 1.0. Bearing and sliding resistance factors of 0.65 and 1.0, respectively, were used in the calculations per FHWA recommendations. The calculated eccentricity was within the middle third of reinforcement length which is considered acceptable. The factor of safety values against global stability failures were considered acceptable.

A summary is listed below. The results of the MSEW analyses are attached with this report.

Failure Mode	Reinforcing Length (feet)	Sliding	Bearing Failure	Eccentricity
Minimum value	**8 ft. or L/H≤0.7**	**CDR≥1.0**	**CDR≥1.0**	**e/L≤0.25**
East Abutment MSE Abutment Wall (L/H=0.75) (MSE and Abutment H=30 ft)	22.5 (L/H=0.75)	1.68	1.01	0.13
East Abutment MSE Abutment Wall (L/H=0.8) (MSE and Abutment H=30 ft)	24 (L/H=0.8)	1.79	1.04	0.11
East Abutment MSE Abutment Wall (L/H=0.9) (MSE and Abutment H=30 ft)	27 (L/H=0.9)	2.49	1.08	0.08
MSE Wingwall L/H=1.0 (MSE H=23 feet)	23 (L/H=1.0)	2.62	1.15	0.09

Failure Mode	Reinforcing Length (feet)	Sliding	Bearing Failure	Eccentricity
Minimum value	**8 ft. or L/H≤0.7**	**CDR≥1.0**	**CDR≥1.0**	**e/L≤0.25**
MSE Wingwall L/H=1.0 (MSE H=18 feet)	18 (L/H=1.0)	2.50	1.35	0.10

*Surcharge loads Load L = 250 psf for traffic were considered. Soil shear strength and unit weight values were based upon laboratory testing results, test boring and CPT sounding results, and engineering judgment.

Based on the MSE wall analyses, the recommended minimum MSE wall reinforcement strap lengths are provided in the following table. Also, the nominal bearing capacity changes along the length of the retaining walls due to geometry and soil conditions and are reported in the following table.

Wall and Stations	Recommended Minimum L/H Ratio	Recommended Nominal Bearing Capacity (psf)
MSE Abutment Retaining Wall	0.9	8,000
MSE Wingwalls	1.0	7,500

The following table outlines the estimated total settlement of the MSE walls at the analyzed stations, near the MSE wall face. The estimated settlements take into account the preloading condition of the existing embankment soils. The estimated total settlement values are based upon the soil conditions in the test borings, test boring data, one-dimensional consolidation tests performed on relatively undisturbed soil samples from the test borings and using the EMBANK software developed by FHWA. The differential settlement along the MSE wall face is estimated to be less than 1/100. The estimated time-rate of settlement for 90% consolidation and the time to reduce the total remaining settlement to 1 inch or less is also reported in the following table. The time-rate of consolidation assumes that the silt layers interbedded in the lakebed clay soils will act as intermediate drainage paths. To account for differential settlement as a result of variation of the foundation soils, slip joints can be considered in the design of the facing and connections, and located by the MSE designer.

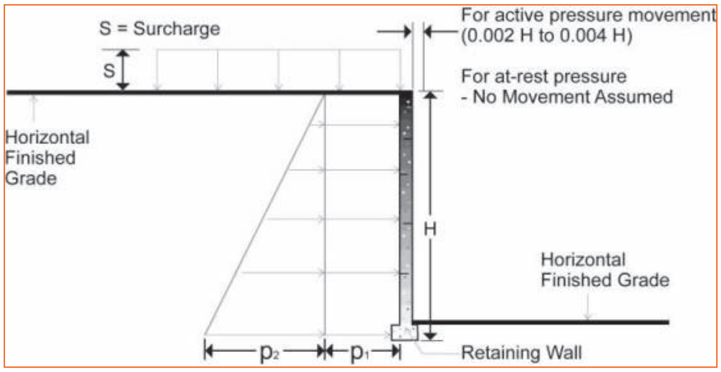
Wall and Station	Estimated Total Settlement (inches)	Estimated Time for 90% Consolidation (weeks)	Estimated Time for 1 inch or less Settlement (weeks)
Abutment Wall	4.5	12-14	3-5
Wingwall H=23 feet	3.5		
Wingwall H=18 feet	2.5		

Due to the estimated total and differential settlement along the MSE wall face, the MSE designer may want to consider a 2-stage facing system. A 2-stage facing system is where welded wire facing is initially used for the MSE wall facing. After the facing settlement has slowed to an acceptable rate, then the permanent concrete facing panels are attached, which reduces potential of cracking of the concrete wall panels. The MSE wall designer could also use control joints in the MSE concrete wall facing to help control damage of the wall panels due to differential settlement.

LATERAL EARTH PRESSURES

Design Parameters

Structures with unbalanced backfill levels (concrete abutment walls) on opposite sides should be designed for earth pressures at least equal to values indicated in the following table. Earth pressures will be influenced by structural design of the walls, conditions of wall restraint, methods of construction and/or compaction and the strength of the materials being restrained. Two wall restraint conditions are shown in the diagram below. Active earth pressure is commonly used for design of free-standing cantilever retaining walls and assumes wall movement. The “at-rest” condition assumes no wall movement, such as walls restrained at the top. The recommended design lateral earth pressures do not include a factor of safety and do not provide for possible hydrostatic pressure on the walls (unless stated).



Lateral Earth Pressure Design Parameters				
Earth Pressure Condition [1]	**Coefficient for Backfill Type** [2]	**Surcharge Pressure** [3, 4, 5] **p_1 (psf)**	**Effective Fluid Pressures (psf)** [2, 4, 5]	
			Unsaturated [6]	**Submerged** [6]
Active (Ka)	Granular - 0.31	(0.31)S	(40)H	(80)H
	Fine Grained - 0.41	(0.41)S	(50)H	(85)H
At-Rest (Ko)	Granular - 0.47	0.47)S	(55)H	(90)H
	Fine Grained - 0.58	(0.58)S	(70)H	(95)H

1. For active earth pressure, wall must rotate about base, with top lateral movements 0.002 H to 0.004 H, where H is wall height. For passive earth pressure, wall must move horizontally to mobilize resistance.
2. Uniform, horizontal backfill, compacted to at least 95% of the AASHTO T-99 maximum dry density, rendering a maximum unit weight of 120 pcf. Parameters assume ϕ=32 degrees for granular material, ϕ=25 degrees for cohesive soil, parameters will vary if materials with different properties are used
3. Uniform surcharge, where S is surcharge pressure.
4. Loading from heavy compaction equipment is not included.
5. No safety factor is included in these values.
6. To achieve “Unsaturated” conditions, permanent drainage of the wall backfill needs to be provided. “Submerged” conditions are recommended when drainage behind walls is not incorporated into the design.

Backfill placed against structures should consist of granular soils or low plasticity cohesive soils. For the granular values to be valid, the granular backfill must extend out and up from the base of the wall at an angle of at least 45 and 60 degrees from vertical for the active and passive cases, respectively.

PAVEMENTS

Pavement Design Parameters

California Bearing Ratio (CBR) testing was performed on a remolded bulk sample of soil from about 1 to 5 feet deep in Boring B-18-1. The sample was remolded to about 100%of maximum dry density and near optimum moisture content per AASHTO T-99. The tested soil sample was classified per AASHTO as A-6(12). It should be noted that the tested sample had about 30% sand and gravel, which was higher than most of the tested natural cohesive soil samples. Results of the soaked CBR test per KYTC methods indicated a CBR value of 9.6 at 0.1-inch penetration and 11.5 at 0.2inch penetration. The measured swell of the sample was 0.1%.

Due to the relatively high sand and gravel content of the tested CBR sample, it is our opinion that the sand and gravel likely increased the CBR value as compared to a sample that would have a much smaller fraction of sand and gravel. In addition, subgrade soils during construction may not receive as much compaction or have a higher or lower moisture content than the sample tested in the laboratory. Therefore, we recommend that the pavement design be based on a lower CBR value, such as 5 or 6, which accounts for variability in soil types, soil compaction and soil moisture content.

Pavement Drainage

Pavements should be sloped to provide rapid drainage of surface water. Water allowed to pond on or adjacent to the pavements could saturate the subgrade and contribute to premature pavement deterioration. In addition, the pavement subgrade should be graded to provide positive drainage within the granular base section. Appropriate sub-drainage or connection to a suitable daylight outlet should be provided to remove water from the granular subbase.

SLOPE STABILITY

Mechanics of Stability

Slope stability analyses take into consideration material strength, presence and orientation of weak layers, water (piezometric) pressures, surcharge loads, and the slope geometry. Mathematical computations are performed using computer-assisted simulations to calculate a Factor of Safety (FS). Minor changes to slope geometry, surface water flow and/or groundwater levels could result in slope instability. Reasonable FS values are dependent upon the confidence in the parameters utilized in the analyses performed, among other factors related to the project itself.

Geometric Analysis Results

Slope stability analyses were performed for the cross-section geometries obtained from the provided plan and profile drawing on March 29, 2109. Parameters for the analyses were derived from our exploratory borings, CPT soundings, laboratory shear testing and experience. Stability analyses were conducted using the computer program ReSSA for the MSE retaining wall structures at the east abutment and STABLE 6H developed originally at Purdue University for the spill-through slope at the proposed west abutment.

Unstable or Potentially Unstable Slopes

Based on the results of our field exploration, laboratory testing program, and geotechnical analysis, development of the site is considered feasible from a geotechnical viewpoint provided the conclusions and considerations provided herein are incorporated into the design and construction of the project.

The stability of the slopes at the cross-section locations shown on the **Global Stability Section Plan** were analyzed based on the provided topography, proposed grading, soil properties derived from our geotechnical exploration, laboratory test results and our experience with similar soil conditions. Soil properties used in the analyses are shown below:

Material	Moist Unit Weight (pcf)	Drained Cohesion (psf)	Drained Friction Angle (degrees)
Embankment Fill	125	50 (0 for MSE analyses)	28
Natural Soil above Elevation 525 feet (A-4/A-6)	125	100	28
Natural Soil below Elevation 525 feet (A-4/A-6)	124	100	28
Silty Sand (A-2-4)	128	0	34

Note: Deeper soils were not considered in the global stability analyses, since failures with minimum safety factors occurred about Elevation 500 feet.

Material	Moist Unit Weight (pcf)	Undrained Cohesion (psf)	Undrained Friction Angle (degrees)
Embankment Fill	125	2500	0
Natural Soil above Elevation 525 feet (A-4/A-6)	125	2000	0
Natural Soil below Elevation 525 feet (A-4/A-6)	124	1500	0
Silty Sand (A-2-4)	128	0	34

Material	Moist Unit Weight (pcf)	Undrained Cohesion (psf)	Undrained Friction Angle (degrees)
Note: Deeper soils were not considered in the global stability analyses, since failures with minimum safety factors occurred about Elevation 500 feet.			

Based on the analyses, the calculated FS for the critical surface identified in each section is shown below. The typically accepted minimum FS for long-term slope stability supporting bridge abutments and MSE wing walls is 1.5. For the MSE analyses, the length of the reinforcement straps (L) to the total wall height (H) is indicated in parentheses in the table. The slope stability results are included in the Appendix of this report.

Cross-Section	Minimum Calculated Factor-of-Safety for Slopes/MSE Walls	
	Long-term Circular Failure Surface	Short-term Circular Failure Surface
Centerline through West Abutment	1.67	2.32
Centerline through MSE Abutment Wall East Abutment	1.42 (L=0.75H)	2.19 (L=0.75H)
Centerline through MSE Abutment Wall East Abutment	1.45 (L=0.8H)	2.19 (L=0.75H)
Centerline through MSE Abutment Wall East Abutment	1.53 (L=0.9H)	2.19 (L=0.75H)
MSE Wing Wall H=18 ft	1.48 (L=1.0H)	2.54 (L=1.0H)

The minimum factor-of-safety for global stability at the cross sections analyzed is greater than 1.5. Cut and fill slopes should be re-vegetated as soon as possible after grading and protected from erosion until vegetation is established or other forms of erosion control installed.

PLAN NOTES

Roadway/Earthwork

1. Clearing and grubbing of roadway areas shall be completed in accordance with the requirements of Section 202 of Standard Specifications for Road and Bridge Construction, current edition, before embankment placement.

2. Removal of existing structures and other obstructions shall be completed in accordance with Section 203 of the Standard Specifications for Road and Bridge Construction, current edition.
3. All soils, whether from roadway or borrow, may require manipulation to obtain proper moisture content prior to compaction. Direct payment shall not be permitted for rehandling, hauling, stockpiling, and/or manipulating soils.
4. In accordance with Section 206 of the Standard Specifications for Road and Bridge Construction, current edition, the moisture content of embankment material shall not vary from optimum moisture content as determined by KM 64-511 by more than +2 or less than -2 percent. This moisture content requirement shall have equal weight with the density requirements when determining the acceptability of the embankment construction. Refer to the Family of Curves for moisture/density correlation.
5. The contractor is responsible for conducting any operations necessary to excavate the cut areas to the required typical section. These operations shall be incidental to Roadway Excavation or Embankment-in-Place and no additional compensation shall be made for this work.
6. Some soil horizons and slopes on the project are subject to erosion. Necessary procedures in accordance with Sections 212 and 213 of the current Standard Specifications shall be followed on construction.
7. Cut and fill slopes will need to be flatter than 2H:1V to maintain minimum factor of safety requirements for slope stability. Cut and fill slopes will need to be maintained at 2.5H:1V or flatter. Flatter slopes are recommended for safety of long-term maintenance requirements.
8. Existing bituminous concrete that is not being overlaid and is located at distance less than three feet below the proposed subgrade elevation within the limits of new roadway embankments, shall be removed entirely. This shall be performed in compliance with Section 206 of the Standard Specifications for Road and Bridge Construction.
9. As directed by the Engineer, existing bituminous concrete located at a distance greater than three feet below the proposed subgrade elevation within the limits of new roadway embankments, shall be scarified or broken until all cleavage planes are destroyed, or the pavement shall be removed entirely as conditions demand. This shall be performed in compliance with Section 206 of the Standard Specifications for Road and Bridge Construction.

Structure plan notes for foundations and MSE walls will be provided when geotechnical plan sheets are provided.

GENERAL COMMENTS

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Natural variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence or collaboration through this system are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client, and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly impact excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety, and cost estimating including, excavation support, and dewatering requirements/design are the responsibility of others. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.

ATTACHMENTS

EXPLORATION AND TESTING PROCEDURES

Field Exploration

Our field exploration work included drilling and sampling one exploratory soil boring and cone penetration test (CPT) soundings consistent with the following schedule. We performed 1 standard penetration test (SPT) boring and 2 CPT soundings in the vicinity of the proposed bridge replacement. One of the CPT soundings (CPT-1A) was offset about 5 feet from CPT-1 and augered to 30 feet below grade and then pushed in an effort to push the sounding deeper. During the push of CPT-1A, refusal was encountered at about 36.8 feet below grade. The soil was augered to about 40 feet and then pushed to a depth of about 65.2 feet below grade where excessive cone inclination was encountered.

Boring or CPT	Boring or CPT Depth (feet)	Notes
Boring B-18-1	102	East approach
CPT-1	64.2	West approach – Refusal due to excessive cone inclination.
CPT-1A	36.8	West approach – Augered to 30 feet prior to initiating push. Refusal due to excessive cone inclination.
CPT-1B	65.2	West approach – Augered to 40 feet prior to initiating push. Refusal due to excessive cone inclination.

The locations of field exploration points were established in the field by Terracon's exploration team using a NetRover survey grade GPS unit to establish boring locations. During staking of the exploration points, a cell phone signal could not be obtained at the site, and the boring locations or the ground surface elevations could not be accurately measured. The project surveyors surveyed the exploration locations after the boring and the CPT soundings were performed.

We advanced the soil boring with a track-mounted drill rig using continuous hollow-stem augers. We obtained representative samples primarily by the split-barrel sampling procedure. In the split-barrel sampling procedure, a standard, 2-inch O.D., split-barrel sampling spoon is driven into the boring with a 140-pound automatic SPT (Standard Penetration Test) hammer falling 30 inches. We recorded the number of blows required to advance the sampling spoon the last 12 inches of an 18-inch sampling interval as the standard penetration resistance value, N. Split-barrel samples were obtained at 2.5-foot-depth intervals to 10 feet and then at 5-foot-depth intervals thereafter in the borings. We also obtained a near-surface bag sample in the borehole. Several offset borings were required to collect adequate volume of soil in the bag sample. Auger refusal was encountered in the borehole within a cobble zone.

We have reported the sampling depths, penetration distances, hand penetrometer test values, and the standard penetration resistance values on the boring log. In the field we placed the samples into containers, sealed them, and returned them to the laboratory for observation, testing and classification.

Our exploration team prepared a field SPT boring log as part of the drilling operations. The field log included visual classifications of the materials encountered during drilling and our interpretation of the subsurface conditions between samples. Ground water observations were also recorded. Since the borings were located in an active roadway area, the groundwater readings were performed during drilling and immediately after drilling, since the borings were backfilled immediately upon completion for safety reasons. A final boring log was prepared from the field logs. The final boring log represent the engineer's interpretation of the field log and includes modifications based on observations and tests of selected samples in the laboratory.

Two cone penetration test (CPT) soundings were completed as part of our field exploration program. The CPT soundings were extended to about 64.2 and 65.2 feet below existing grade. Sounding CPT-1 encountered refusal due to excessive cone tilt at a depth of 64.2 feet below existing grade. CPT-1A was offset about 5 feet from CPT-1 and augered to 30 feet prior to initiating the push. CPT-1A encountered refusal at a depth of 36.8 feet due to excessive cone tilt. We then augered to a depth of 40 feet below grade and initiated pushing CPT-1B, which encountered refusal at a depth of 65.2 feet below existing grade due to excessive cone tilt. In an effort to estimate the groundwater level at the CPT-1 sounding location, we performed pore pressure dissipation tests at this location.

Cone Penetration Test (CPT) soundings were performed in general accordance with industry-standard procedures with continuous data collection. CPT soundings were performed with a penetrometer device consisting of a cone-shaped sounding tip attached to steel rods with flush-joint couplings. The cone contains transducers to measure cone tip penetration resistance, sleeve friction resistance and excess pore pressure. The tilt angle of the penetrometer was also measured by an inclinometer located within the sounding tip. The CPT was logged electronically in the field. The data collected from the CPT was reduced and presented graphically, including the tip resistance, sleeve resistance, a ratio of sleeve to tip resistance, pore pressure and interpreted soil classifications (based upon published correlations) with depth.

Laboratory Testing

The project engineer reviewed the field data and assigned laboratory tests to understand the engineering properties of the various soil strata, as necessary, for this project. Procedural standards noted below are for reference to methodology in general. The laboratory tests, when available, were performed per Kentucky Transportation Standards. The remaining tests were performed per the appropriate AASHTO standard. Standards noted below include reference to other, related standards. Such references are not necessarily applicable to describe the specific test performed.

- ASTM D2216 Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
- ASTM D4318 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
- ASTM D422 Standard Test Method for Particle-Size Analysis of Soils
- ASTM D2850 Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils
- ASTM D4767-11 Standard Test Method for Consolidated-Undrained Triaxial Compression Test for Cohesive Soils
- ASTM D2435/D2435M Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading
- AASHTO T-99-18 Standard Method of Test for Moisture-Density Relations of Soils Using a 2.5-kg (5.5-lb) Rammer and a 305-mm (12-in.) Drop
- KM 64-501 08 Determining the California Bearing Ratio of Laboratory Compacted Soils and Soil - Aggregate Mixtures

The laboratory testing program included visual examination of soil samples by the project engineer. Based on the material's texture and plasticity, we described and classified the soil samples in accordance with the Unified Soil Classification System and the AASHTO classification system when classification laboratory data was available.

SITE LOCATION AND EXPLORATION PLANS

Contents:

Note: All attachments are one page unless noted above.

SITE LOCATION

Ernstbridge Road Bridge Replacement over CSX Railroad ■ Kenton County, Kentucky
April 8, 2019 ■ Terracon Project No. N1185278

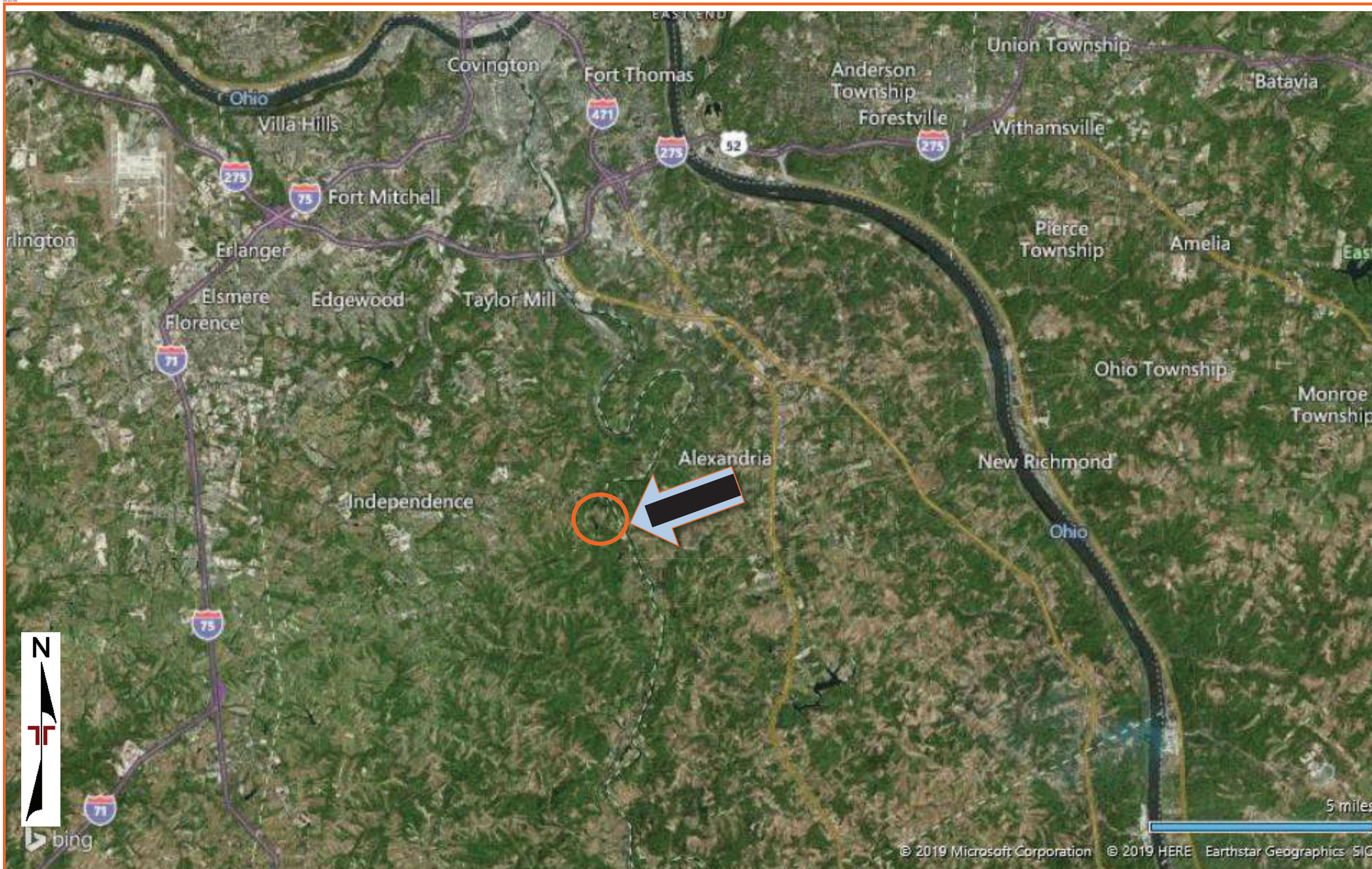


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

EXPLORATION PLAN

Ernstbridge Road Bridge Replacement over CSX Railroad ■ Kenton County, Kentucky
April 8, 2019 ■ Terracon Project No. N1185278

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

GLOBAL STABILITY SECTION PLAN

Ernstbridge Road Bridge Replacement over CSX Railroad ■ Kenton County, Kentucky

April 8, 2019 ■ Terracon Project No. N1185278

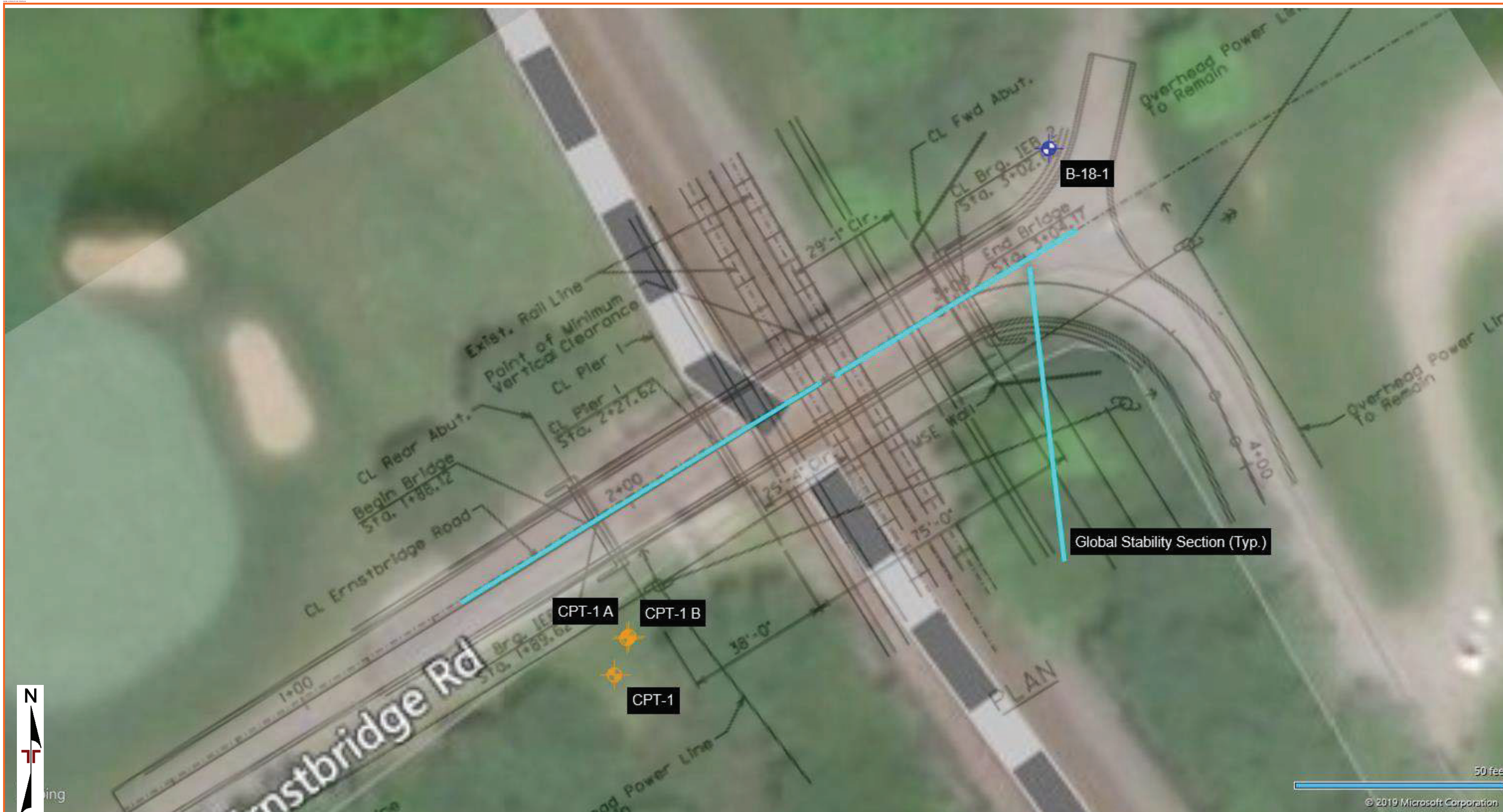


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

EXPLORATION RESULTS

Contents:

Note: All attachments are one page unless noted above.

BORING LOG NO. B-18-1

PROJECT: Ernstbridge Road Bridge Replacement

CLIENT: WSP USA Inc.
Cincinnati, OH

SITE: Ernstbridge Road
Ryland Heights, KY

LOCATION See Exploration Plan

Latitude: 38.9367° Longitude: -84.4645°

Approximate Surface Elev.: 547 (Ft.) +/-

DEPTH (Ft.)	Description	ELEVATION (Ft.)
0.4	**ASPHALT PAVEMENT (5")**	546.5+/-
0.9	**GRANULAR BASE, crushed stone (6")**	546+/-
5.5	**FILL - LEAN CLAY WITH SAND (CL)**, trace fine gravel, brown, (A-6(12))	541.5+/-
12.0	**LEAN CLAY (CL)**, trace sand and concretions, mottled brown and gray, stiff	535+/-
18.0	**LEAN CLAY (CL)**, trace sand and concretions, mottled brown and gray, very stiff	529+/-
23.0	**SILTY CLAY (CL-ML)**, trace concretions, weak laminated structure, mottled brown and gray, stiff	524+/-
33.0	**LEAN CLAY WITH SAND (CL)**, trace concretions, trace silt laminations, brown trace gray, soft, (A-6(10))	514+/-
35.0	**SILTY CLAY (CL-ML)**, with reddish brown concretions, brownish gray, medium stiff	512+/-

Sample	RECOVERY (%)	FIELD TEST RESULTS	LABORATORY HP (tsf)	TEST TYPE	COMPRESSIVE STRENGTH (tsf)	STRAIN (%)	CONFINING PRESSURE (psi)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI
Split spoon	78	3-5-5 N=10						5		
Bag	100									38-19-19
Split spoon	89	27-10-7 N=17						4		
Split spoon	67	3-2-4 N=6	2.75 (HP)					25		
Split spoon	67	3-4-4 N=8	2.75 (HP)					23		
Split spoon	89	4-8-9 N=17	3.5 (HP)					20		
Split spoon	100	3-3-4 N=7	2.0 (HP)					28		
Split spoon	100	2-2-2 N=4	0.5 (HP)					46		
Shelby tube	100		0.75 (HP)	CU				29		33-19-14
Split spoon	100	3-2-2 N=4	0.5 (HP)					31		
Split spoon	100	0-2-4 N=6	1.0 (HP)					28		

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
3.25-inch Continuous-Flight Hollow-Stem Augers
2-inch Split-Barrel Sampler

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were interpolated from a topographic site plan.

Notes:

WATER LEVEL OBSERVATIONS

Water observed at 30' during drilling

Water observed at 24' after drilling

Terracon
611 Lunken Park Dr
Cincinnati, OH

Boring Started: 02-22-2019	Boring Completed: 02-22-2019
Drill Rig: CME-55X	Driller: Hayslip
Project No.: N1185278	

BORING LOG NO. B-18-1

PROJECT: Ernstbridge Road Bridge Replacement

CLIENT: WSP USA Inc. Cincinnati, OH

SITE: Ernstbridge Road Ryland Heights, KY

LOCATION See Exploration Plan

Latitude: 38.9367° Longitude: -84.4645°

Approximate Surface Elev.: 547 (Ft.) +/-

DEPTH / Description	ELEVATION (Ft.)	DEPTH (Ft.)	SAMPLE TYPE	RECOVERY (%)	FIELD TEST RESULTS	LABORATORY HP (tsf)	TEST TYPE	COMPRESSIVE STRENGTH (tsf)	STRAIN (%)	CONFINING PRESSURE (psi)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI
LEAN CLAY (CL), trace concretions, sand and interbedded silt partings to seams, laminated structures, gray with brown, soft to medium stiff, (A-4(6))			Shelby tube	100		0.5 (HP)	UU	1.56	11.4	35	28	98	30-22-8
		40	Split spoon	100	2-2-2 N=4	0.5 (HP)					27		
42.0	505+/-												
LEAN CLAY (CL), weakly laminated, trace sand and silt partings and seams, gray, medium stiff, (A-6(10-15))			Split spoon	100	3-2-3 N=5	0.75 (HP)					29		
		45	Shelby tube	100		0.75 (HP)	UU	1.50	9.2	45	26	97	32-21-11
		50	Split spoon	100	0-0-1 N=1	0.75 (HP)					29		
		55	Split spoon	100	0-0-0 N=0	0.75 (HP)					29		
			Shelby tube	100		1.0 (HP)	UU	1.67	10.8	55	27	97	36-21-15
		60	Split spoon	89	0-0-1 N=1	0.75 (HP)					26		
63.0	484+/-												
LEAN CLAY WITH SAND (CL), trace sand pockets and gravel, bluish-gray, stiff, (A-6(7))		65	Split spoon	100	3-5-7 N=12	1.5 (HP)					18		28-14-14
--medium stiff below 68 feet		70	Split spoon	100	1-3-2 N=5	1.0 (HP)					17		

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
3.25-inch Continuous-Flight Hollow-Stem Augers
2-inch Split-Barrel Sampler

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were interpolated from a topographic site plan.

Notes:

WATER LEVEL OBSERVATIONS

Water observed at 30' during drilling

Water observed at 24' after drilling

Terracon
611 Lunken Park Dr
Cincinnati, OH

Boring Started: 02-22-2019	Boring Completed: 02-22-2019
Drill Rig: CME-55X	Driller: Hayslip
Project No.: N1185278	

BORING LOG NO. B-18-1

PROJECT: Ernstbridge Road Bridge Replacement

**CLIENT: WSP USA Inc.
Cincinnati, OH**

**SITE: Ernstbridge Road
Ryland Heights, KY**

LOCATION See Exploration Plan

Latitude: 38.9367° Longitude: -84.4645°

Approximate Surface Elev.: 547 (Ft.) +/-

DEPTH	Material Description	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (%)	FIELD TEST RESULTS	LABORATORY HP (tsf)	TEST TYPE	COMPRESSIVE STRENGTH (tsf)	STRAIN (%)	CONFINING PRESSURE (psi)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI
	LEAN CLAY WITH SAND (CL), trace sand pockets and gravel, bluish-gray, stiff, (A-6(7)) *(continued)*														
73.0		474+/-													
	CLAYEY SAND (SC), with interbedded silty sand and silt lenses, trace gravel, fine to medium coarse, gray, very looses to loose		75		X	100	0-0-3 N=3						16		
78.0		469+/-													
	SILTY SAND (SM), trace clayey sand seams and gravel, fine to medium grained, gray, loose, (A-2-4(0))		80		X	100	3-3-3 N=6						22		
	--medium dense to dense below 83 feet		85		X	89	8-12-15 N=27						22		16-15-1
			90		X	67	30-15-15 N=30						21		
92.0		455+/-													
	POORLY GRADED SAND WITH SILT AND GRAVEL (SP-SM), trace cobbles, fine to medium grained, gray, dense, (A-1-b(0))		95		X	66	24-30-50/3"						14		NP
					X	78	15-15-25 N=40						8		
			100		X	100	50/2"								
102.0		445+/-													
	Auger Refusal at 102 Feet														

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
3.25-inch Continuous-Flight Hollow-Stem Augers
2-inch Split-Barrel Sampler

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were interpolated from a topographic site plan.

Notes:

WATER LEVEL OBSERVATIONS

Water observed at 30' during drilling

Water observed at 24' after drilling

Terracon
611 Lunken Park Dr
Cincinnati, OH

Boring Started: 02-22-2019	Boring Completed: 02-22-2019
Drill Rig: CME-55X	Driller: Hayslip
Project No.: N1185278	

CPT LOG NO. CPT-1

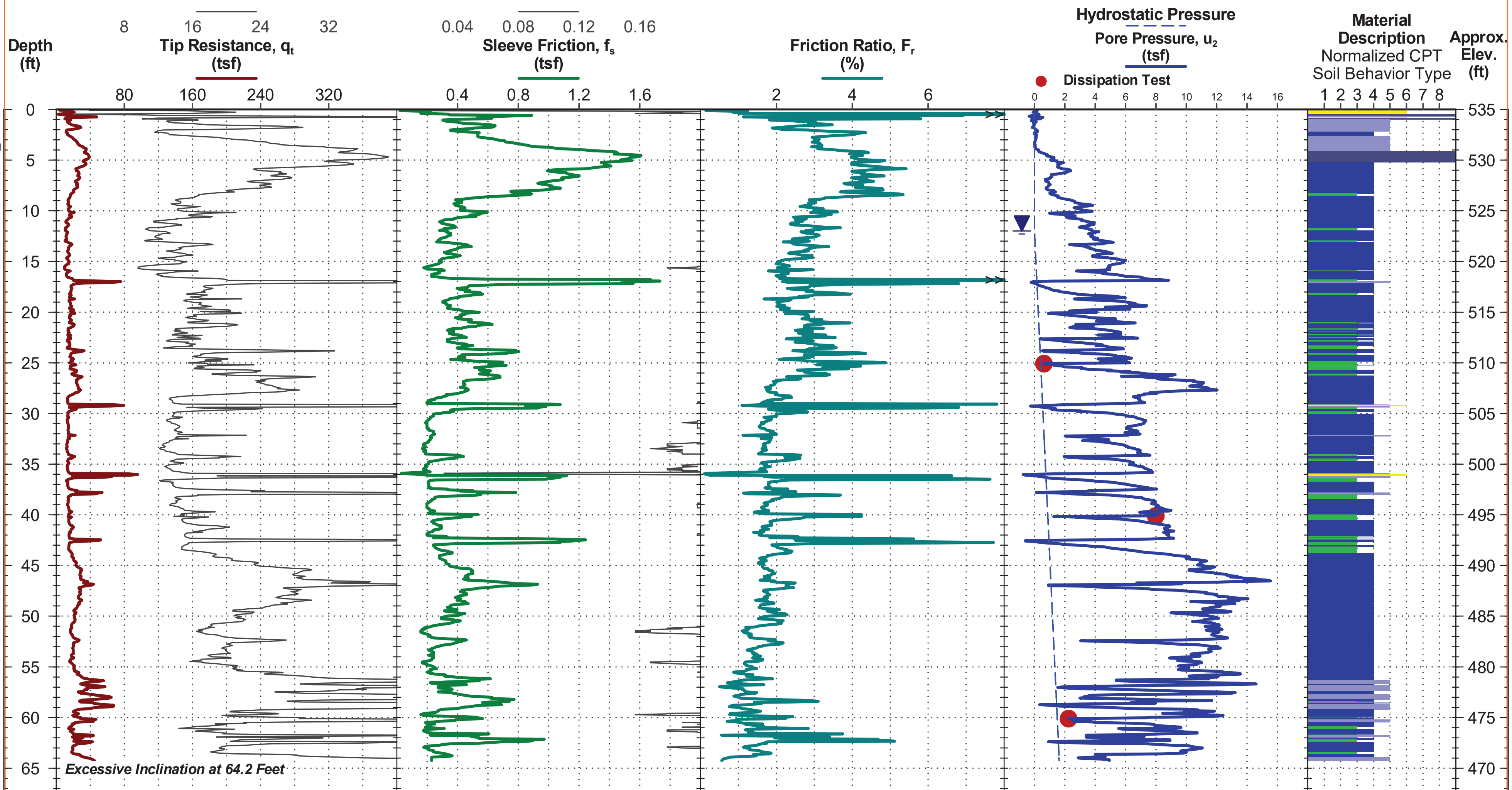
PROJECT: Ernstbridge Road Bridge Replacement

CLIENT: WSP USA Inc.
Cincinnati, OH

TEST LOCATION: See Exploration Plan

Approx. Surface Elev: 535 ft +/-
Latitude: 38.9363°
Longitude: -84.4649°

SITE: Ernstbridge Road
Ryland Heights, KY



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
Elevations were interpolated from a topographic site plan.

CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION

12 ft estimated water depth
(used in normalizations and correlations; See Supporting Information)

Probe no. 5356 with net area ratio of .861
U2 pore pressure transducer location
Manufactured by Nova Cone
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.875 in

611 Lunken Park Dr
Cincinnati, OH

CPT Started: 2/21/2019	CPT Completed: 2/21/2019
Rig: CME-55X	Operator: Hayslip
Project No.: N1185278	

CPT LOG NO. CPT-1 A

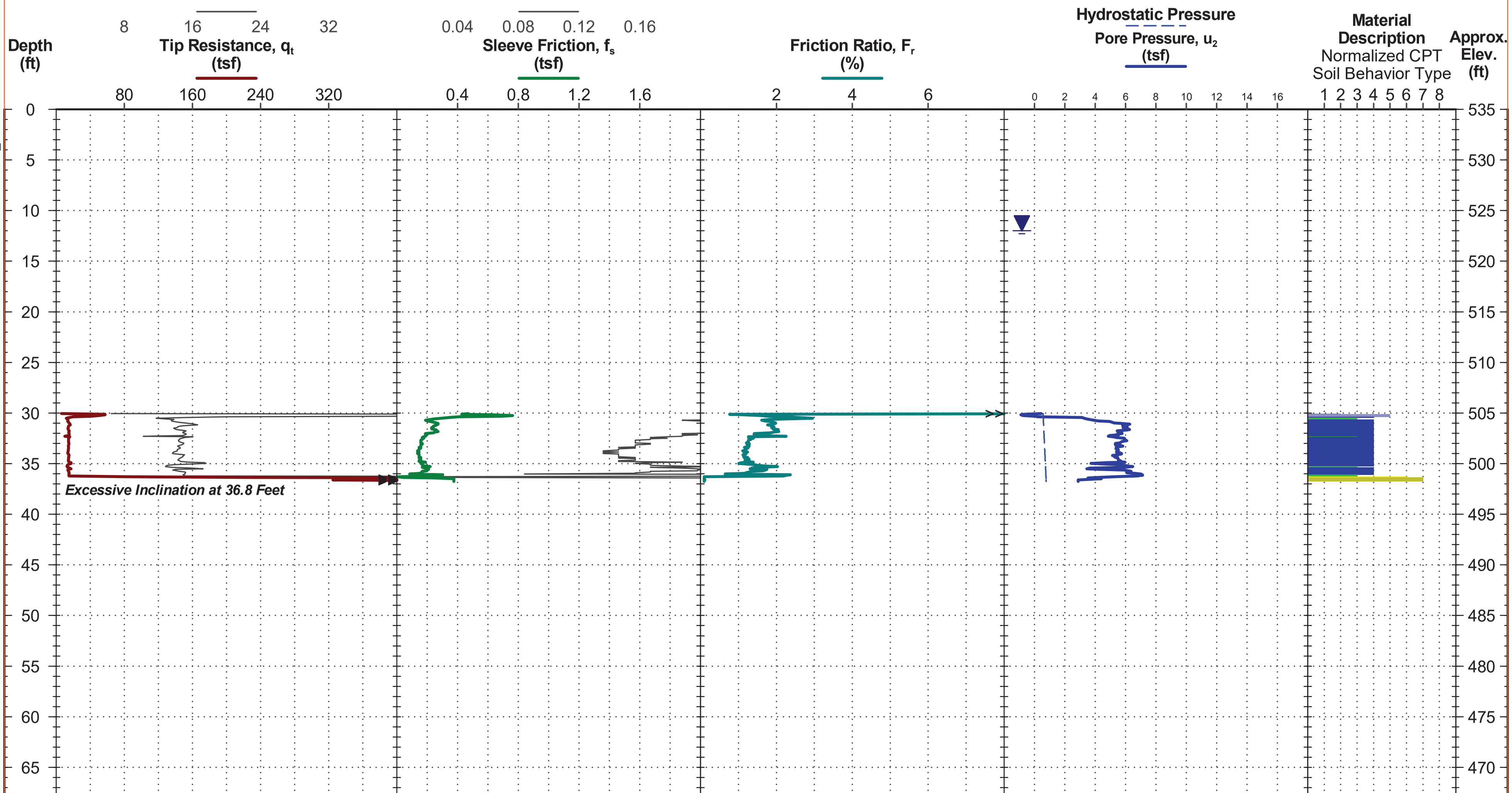
PROJECT: Ernstbridge Road Bridge Replacement

CLIENT: WSP USA Inc.
Cincinnati, OH

SITE: Ernstbridge Road
Ryland Heights, KY

TEST LOCATION: See Exploration Plan

Approx. Surface Elev: 535 ft +/-
Latitude: 38.93631°
Longitude: -84.46491°



Offset 8 feet from CPT-1and augered to 30 feet prior to push

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Elevations were interpolated from a topographic site plan.

CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION 12 ft estimated water depth (used in normalizations and correlations; See **Supporting Information**)	Probe no. 5356 with net area ratio of .861 U2 pore pressure transducer location Manufactured by Nova Cone Tip and sleeve areas of 10 cm^2 and 150 cm^2 Ring friction reducer with O.D. of 1.875 in	Terracon 611 Lunken Park Dr Cincinnati, OH	CPT Started: 2/21/2019 Rig: CME-55X Project No.: N1185278	CPT Completed: 2/21/2019 Operator: Hayslip

CPT LOG NO. CPT-1 B

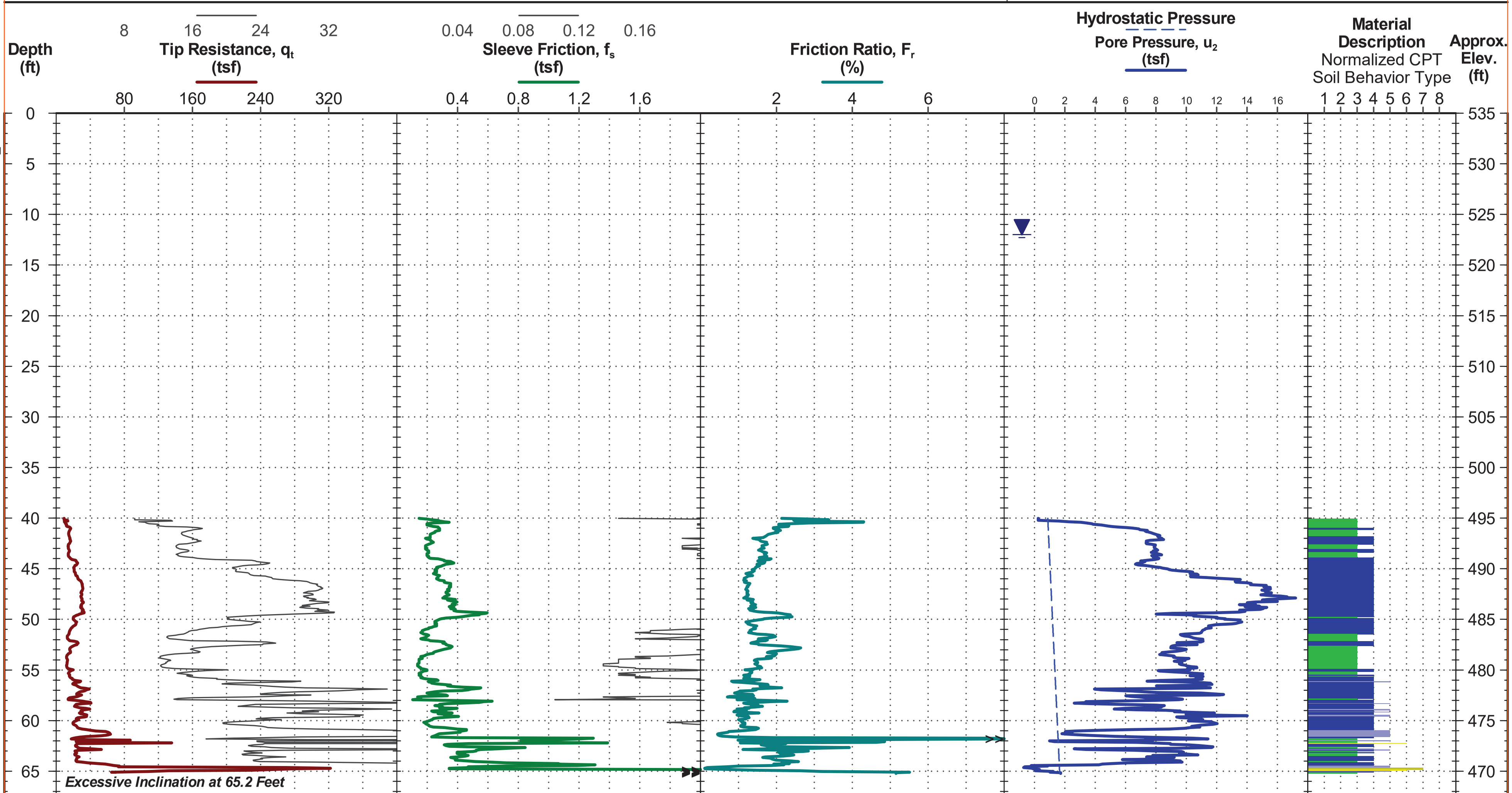
PROJECT: Ernstbridge Road Bridge Replacement

CLIENT: WSP USA Inc.
Cincinnati, OH

SITE: Ernstbridge Road
Ryland Heights, KY

TEST LOCATION: See Exploration Plan

Approx. Surface Elev: 535 ft +/-
Latitude: 38.93631°
Longitude: -84.46491°



Augered to 40 feet and began push in same location as CPT-1A

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Elevations were interpolated from a topographic site plan.

CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION

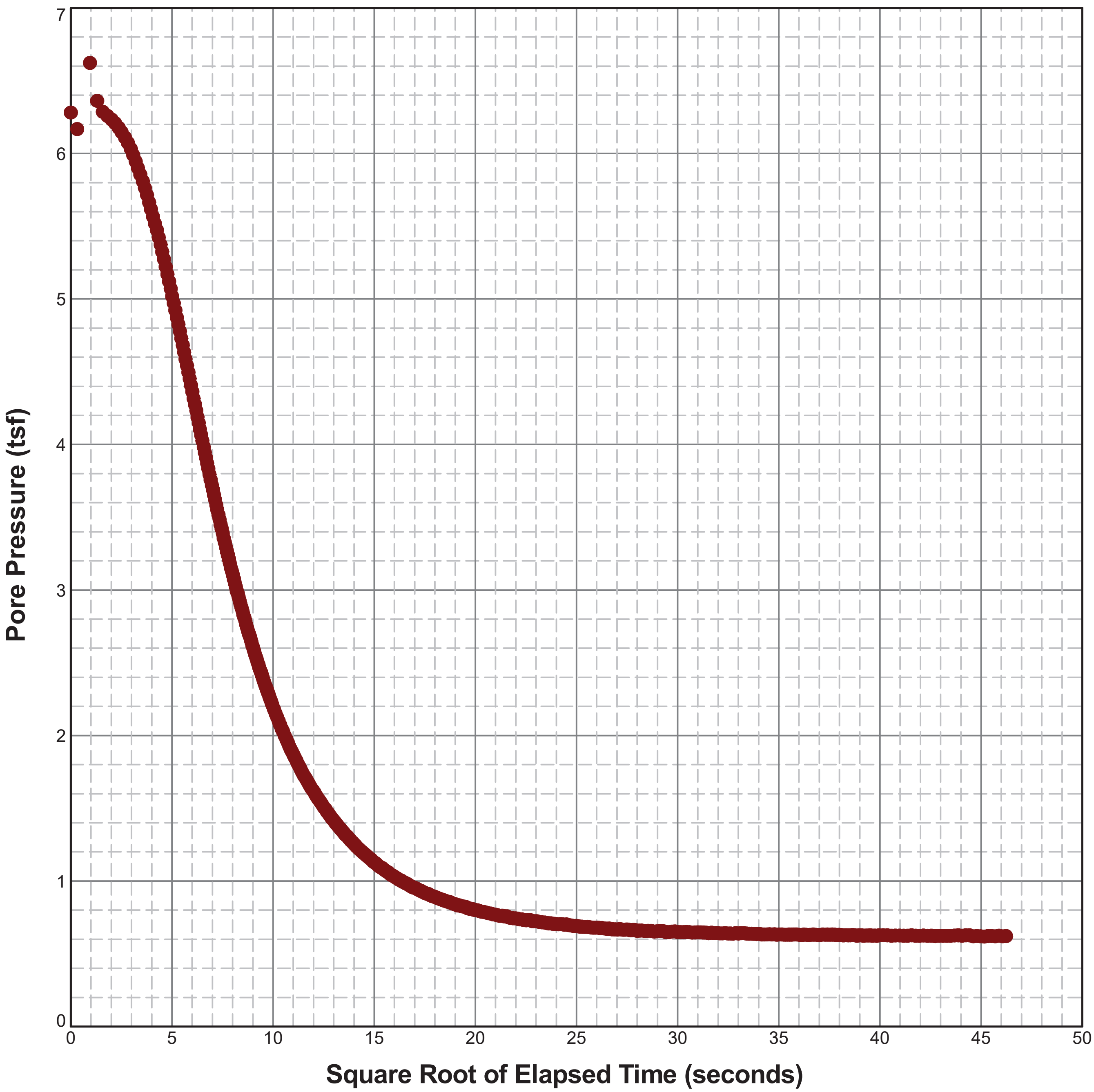
12 ft estimated water depth
(used in normalizations and correlations; See Supporting Information)

Probe no. 5356 with net area ratio of .861
U2 pore pressure transducer location
Manufactured by Nova Cone
Tip and sleeve areas of 10 cm^2 and 150 cm^2
Ring friction reducer with O.D. of 1.875 in

611 Lunken Park Dr
Cincinnati, OH

CPT Started: 2/21/2019	CPT Completed: 2/21/2019
Rig: CME-55X	Operator: Hayslip
Project No.: N1185278	

PORE PRESSURE DISSIPATION TEST RESULTS



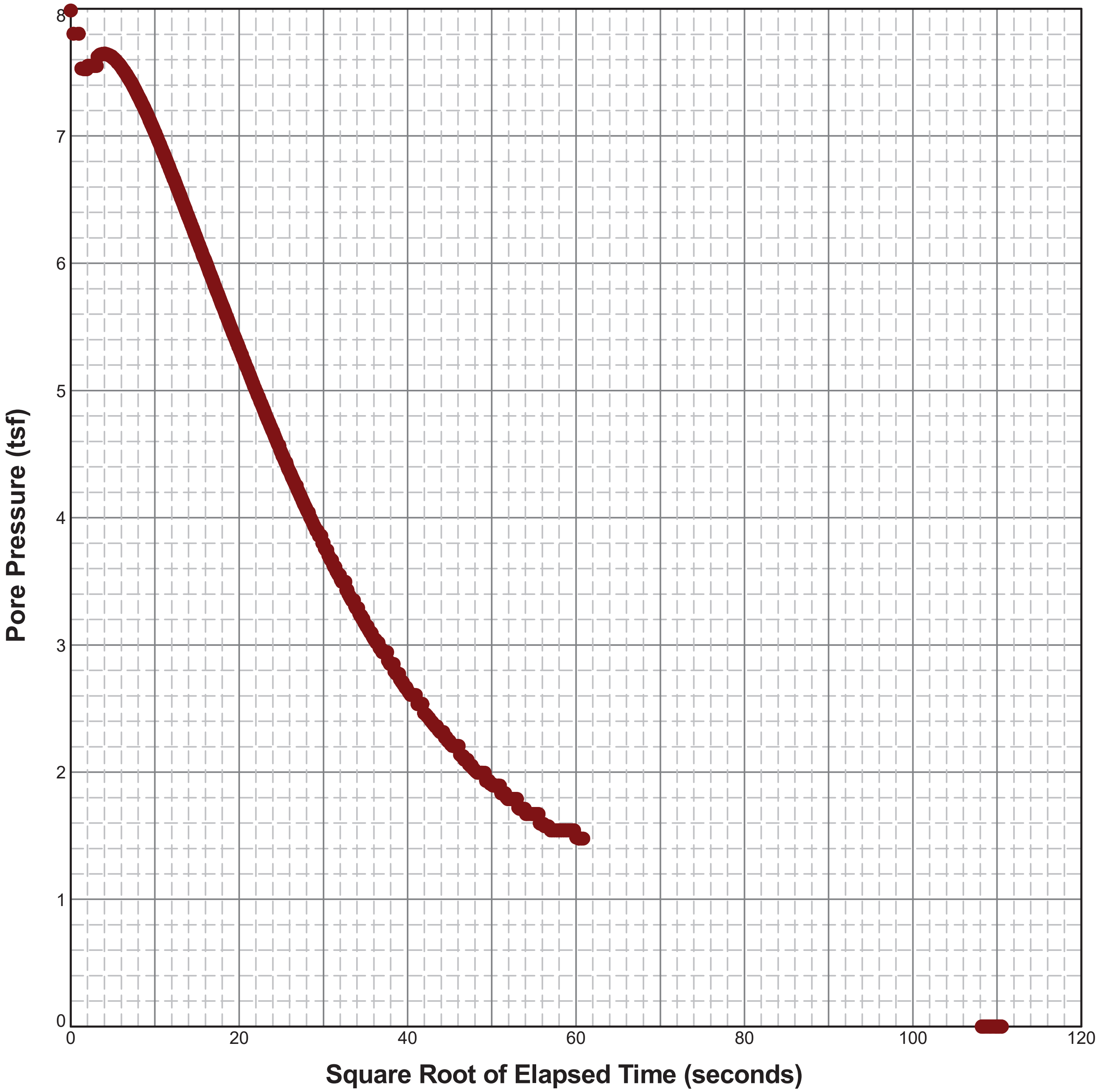
TEST: CPT-1

TEST DEPTH: 25.082 ft

TEST DURATION: 2137.6 sec

PROJECT: Ernstbridge Road Bridge Replacement	Terracon 611 Lunken Park Dr Cincinnati, OH	PROJECT NUMBER: N1185278
SITE: Ernstbridge Road Ryland Heights, KY		CLIENT: WSP USA Inc.

PORE PRESSURE DISSIPATION TEST RESULTS



TEST: CPT-1

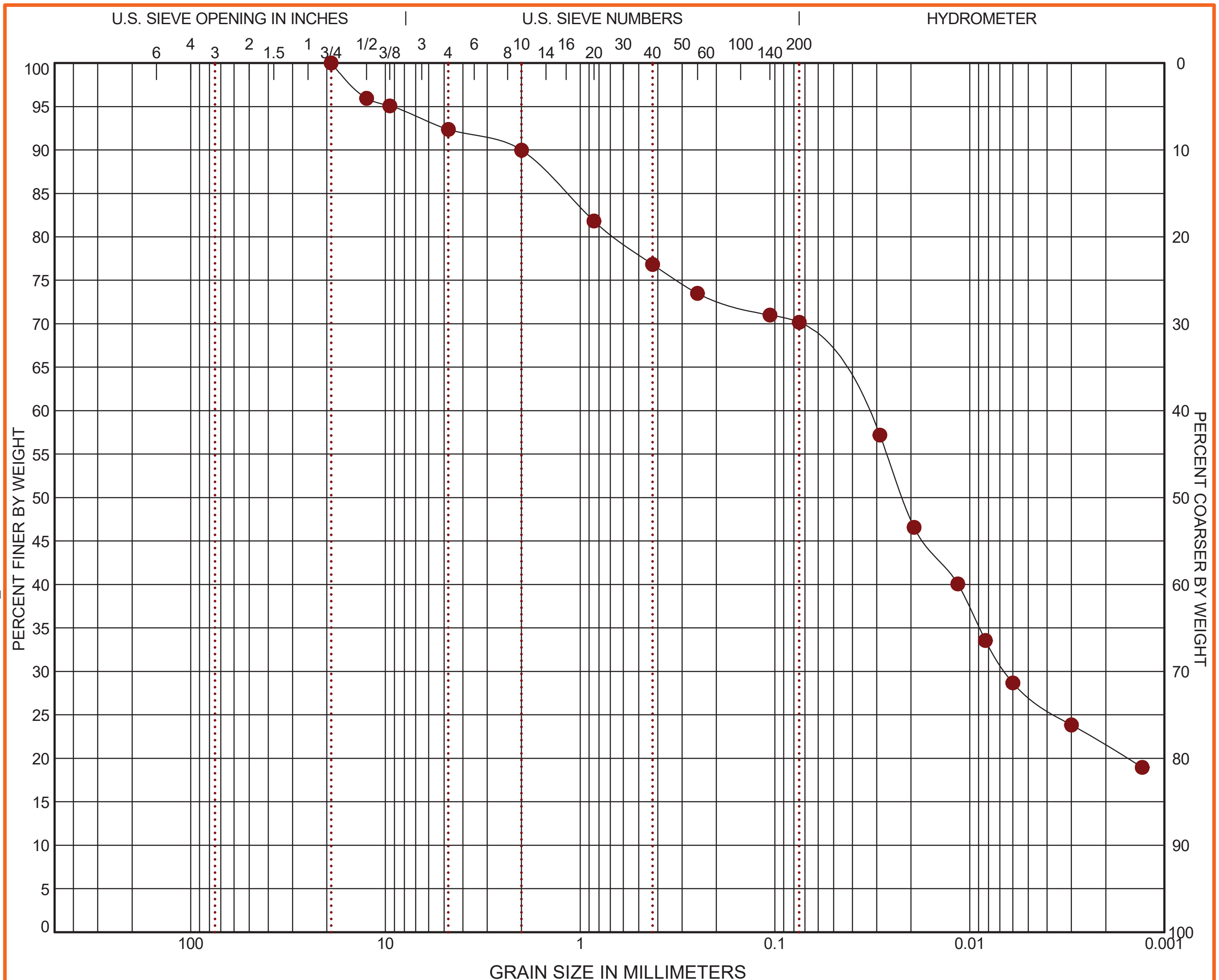
TEST DEPTH: 40.115 ft

TEST DURATION: 12220 sec

PROJECT: Ernstbridge Road Bridge Replacement	Terracon 611 Lunken Park Dr Cincinnati, OH	PROJECT NUMBER: N1185278
SITE: Ernstbridge Road Ryland Heights, KY		CLIENT: WSP USA Inc.

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	B-18-1	0.9 - 4.9	0.0	7.6	22.2	42.8		27.4	CL

GRAIN SIZE	●		
D_{60}	0.036		
D_{30}	0.007		
D_{10}			

COEFFICIENTS	●		
C_c			
C_u			

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
3/4"	100.0				
1/2"	95.94				
3/8"	95.07				
#4	92.36				
#10	89.97				
#20	81.82				
#40	76.82				
#60	73.5				
#140	71.0				
#200	70.17				

	SOIL DESCRIPTION
●	A-6(12)

	REMARKS
●	LAB NO. 1499

PROJECT: Ernstbridge Road Bridge Replacement

SITE: Ernstbridge Road
Ryland Heights, KY

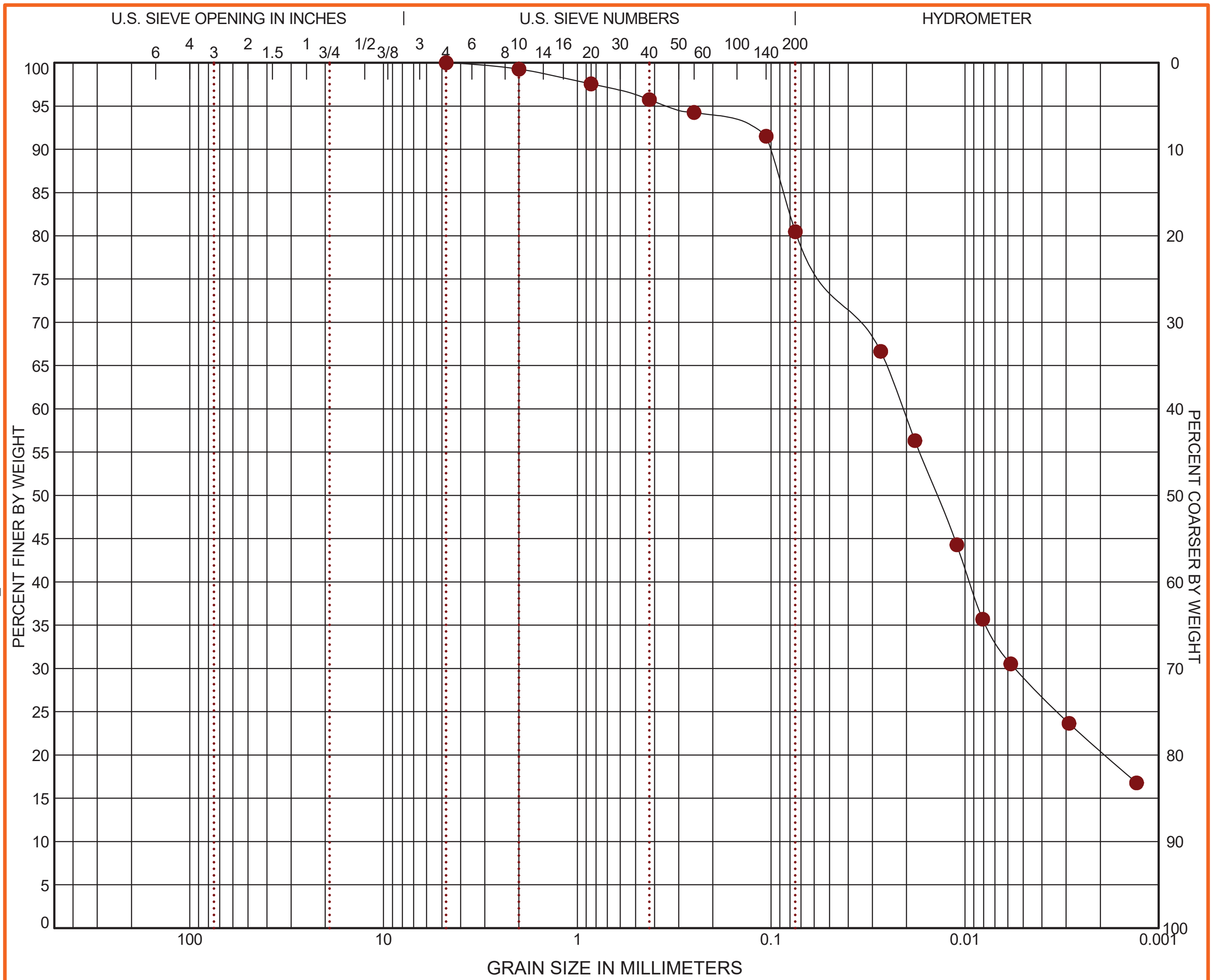
Terracon
611 Lunken Park Dr
Cincinnati, OH

PROJECT NUMBER: N1185278

CLIENT: WSP USA Inc.
Cincinnati, OH

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**B-18-1**	25 - 27	0.0	0.0	19.5	51.4		29.1	CL

GRAIN SIZE			
	●		
D_{60}	0.021		
D_{30}	0.005		
D_{10}			

COEFFICIENTS			
	●		
C_C			
C_U			

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
#4	100.0				
#10	99.26				
#20	97.55				
#40	95.73				
#60	94.26				
#140	91.5				
#200	80.47				

	SOIL DESCRIPTION
●	A-6 (10)

	REMARKS
●	1507

PROJECT: Ernstbridge Road Bridge Replacement

SITE: Ernstbridge Road
Ryland Heights, KY

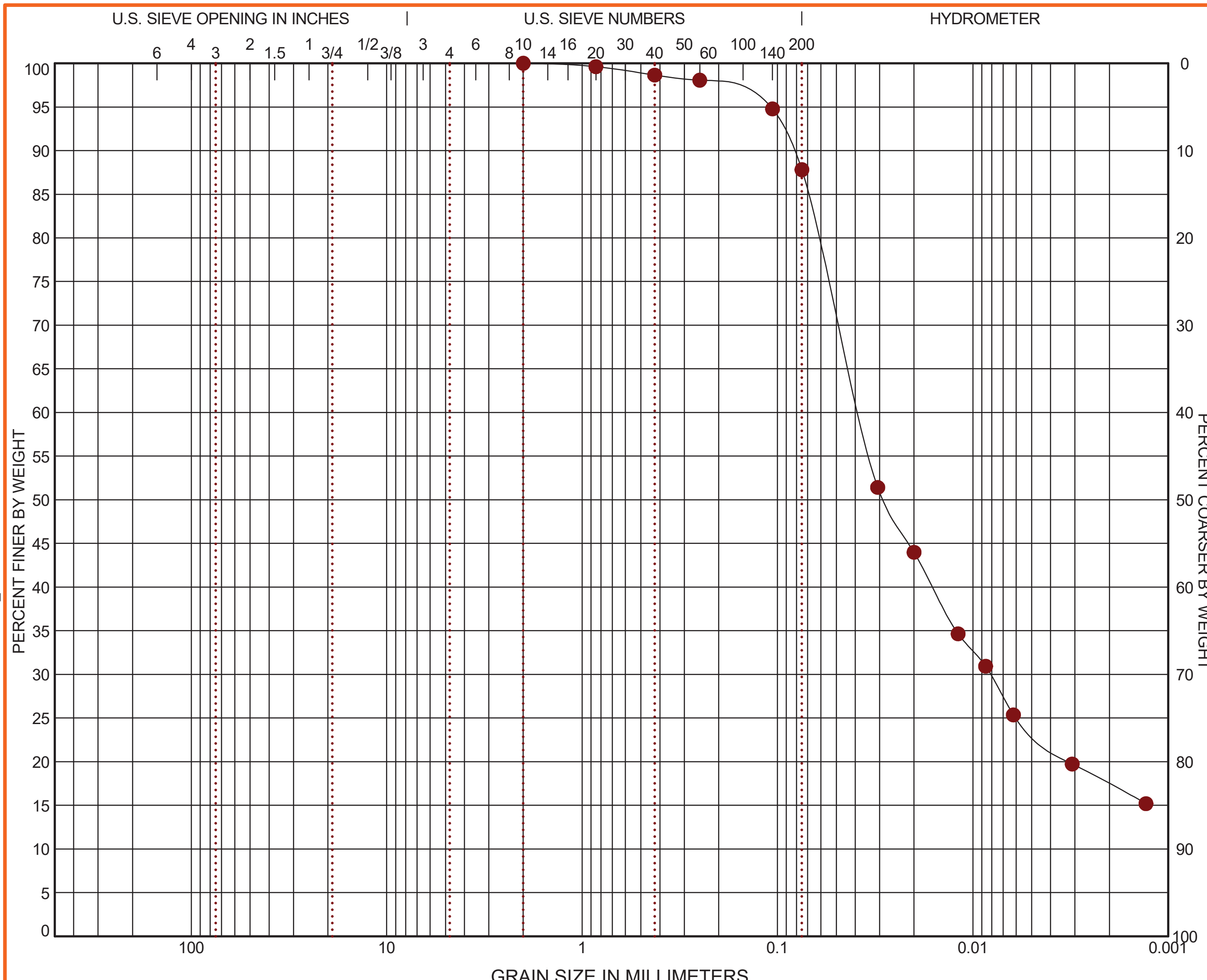
Terracon
611 Lunken Park Dr
Cincinnati, OH

PROJECT NUMBER: N1185278

CLIENT: WSP USA Inc.
Cincinnati, OH

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	B-18-1	35 - 37	0.0	0.0	12.2	64.2		23.6	CL

GRAIN SIZE	●		
D_{60}	0.038		
D_{30}	0.008		
D_{10}			

COEFFICIENTS	●		
C_C			
C_U			

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
#10	100.0				
#20	99.61				
#40	98.65				
#60	98.07				
#140	94.78				
#200	87.81				

SOIL DESCRIPTION	
●	A-4 (6)

REMARKS	
●	1510

PROJECT: Ernstbridge Road Bridge Replacement

SITE: Ernstbridge Road
Ryland Heights, KY

Terracon
611 Lunken Park Dr
Cincinnati, OH

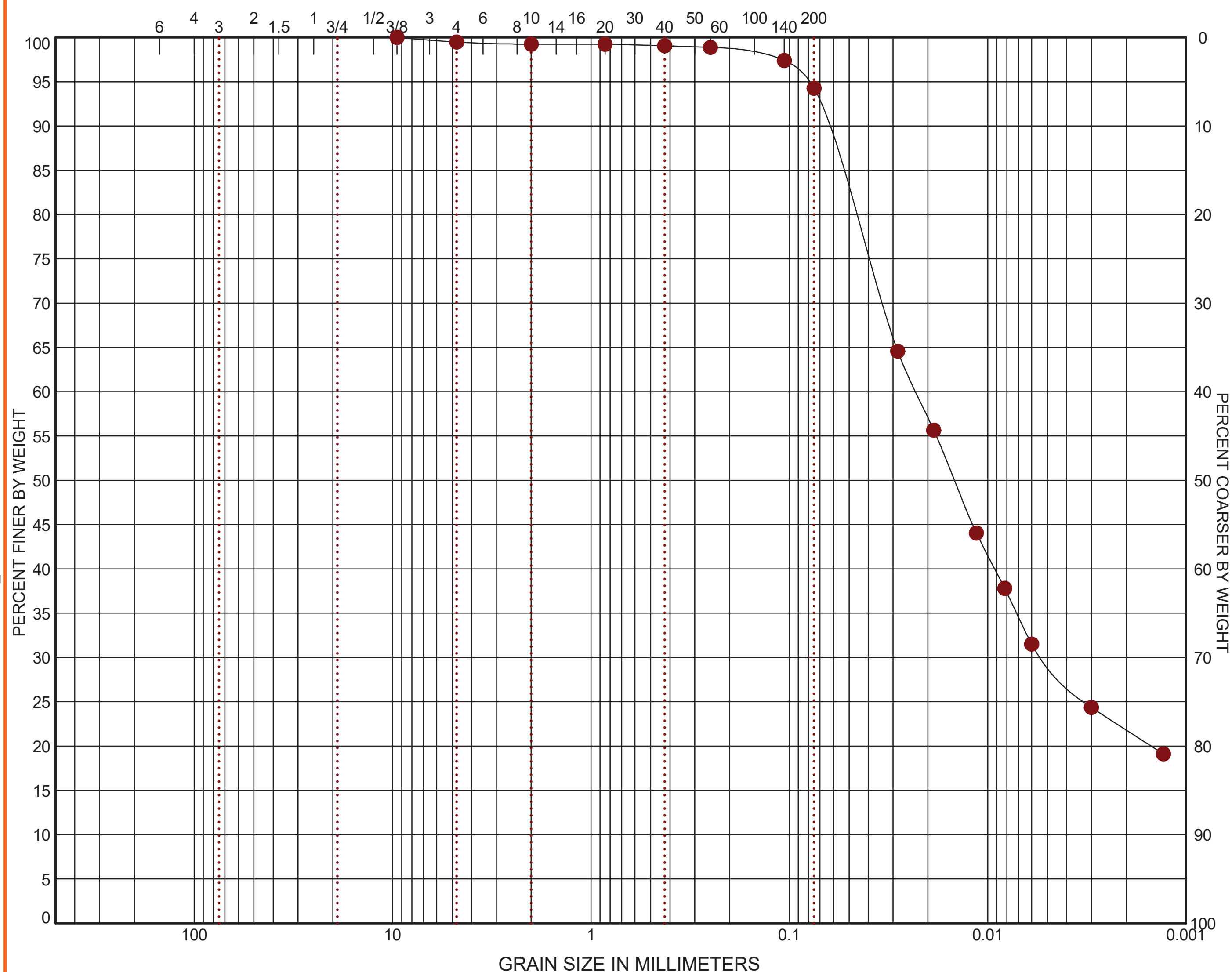
PROJECT NUMBER: N1185278

CLIENT: WSP USA Inc.
Cincinnati, OH

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: AASHTO DESC-1 N1185278 ERNSTBRIDGE ROAD .GPJ TERRACON_DATATEMPLATE.GDT 3/26/19

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	B-18-1	45 - 47	0.0	0.5	5.2	64.6		29.6	CL

GRAIN SIZE	●		
D_{60}	0.023		
D_{30}	0.005		
D_{10}			

COEFFICIENTS	●		
C_C			
C_U			

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
3/8"	100.0				
#4	99.48				
#10	99.24				
#20	99.24				
#40	99.05				
#60	98.87				
#140	97.39				
#200	94.26				

SOIL DESCRIPTION	
●	A-6 (10)

REMARKS	
●	1513

PROJECT: Ernstbridge Road Bridge Replacement

SITE: Ernstbridge Road
Ryland Heights, KY

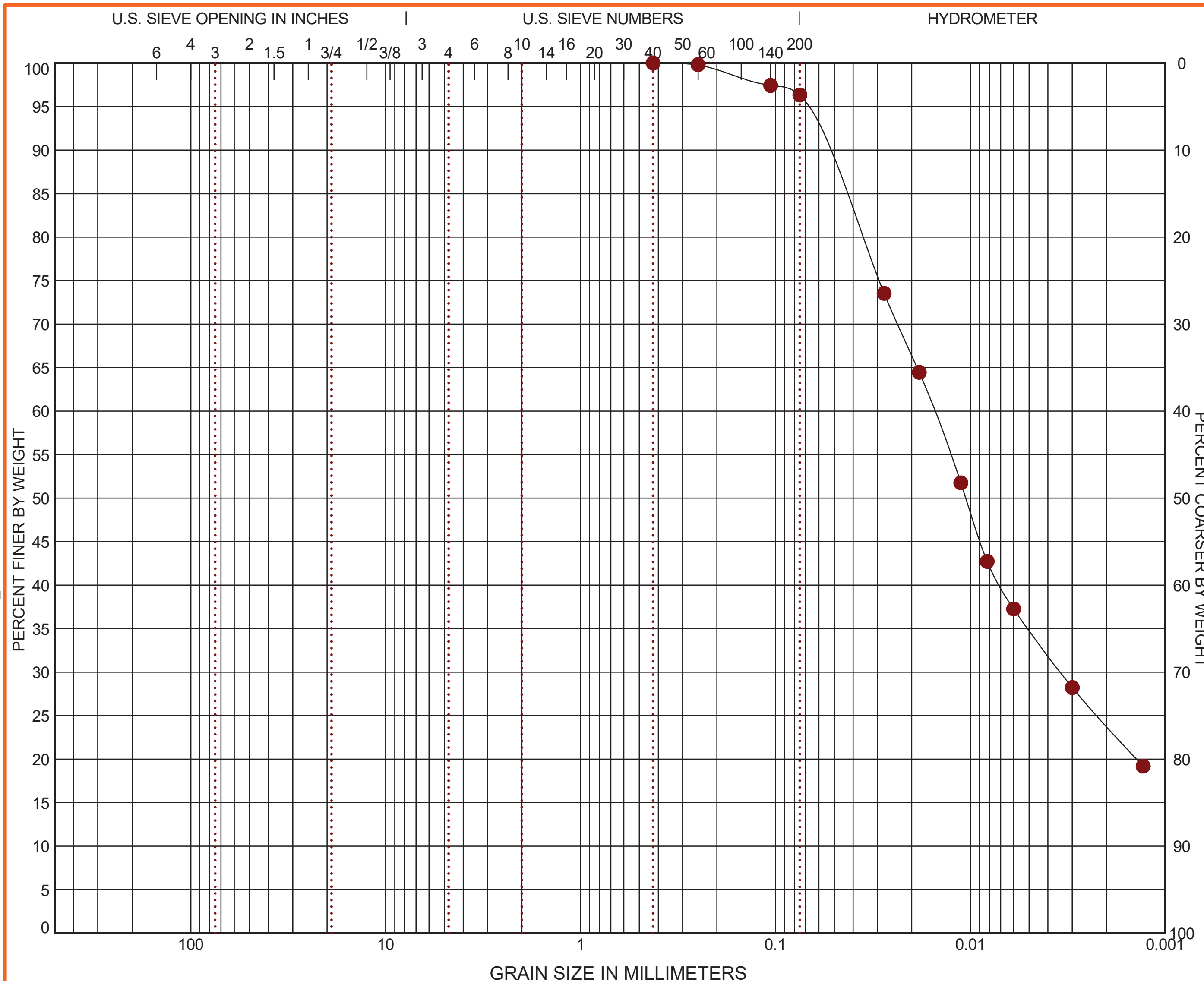
Terracon
611 Lunken Park Dr
Cincinnati, OH

PROJECT NUMBER: N1185278

CLIENT: WSP USA Inc.
Cincinnati, OH

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	B-18-1	55 - 57	0.0	0.0	3.7	61.4		34.9	CL

GRAIN SIZE			
	●		
D_{60}	0.015		
D_{30}	0.003		
D_{10}			

COEFFICIENTS			
	●		
C_C			
C_U			

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
#40	100.0				
#60	99.82				
#140	97.43				
#200	96.33				

SOIL DESCRIPTION	
●	A-6 (15)

REMARKS	
●	1516

PROJECT: Ernstbridge Road Bridge Replacement

SITE: Ernstbridge Road
Ryland Heights, KY

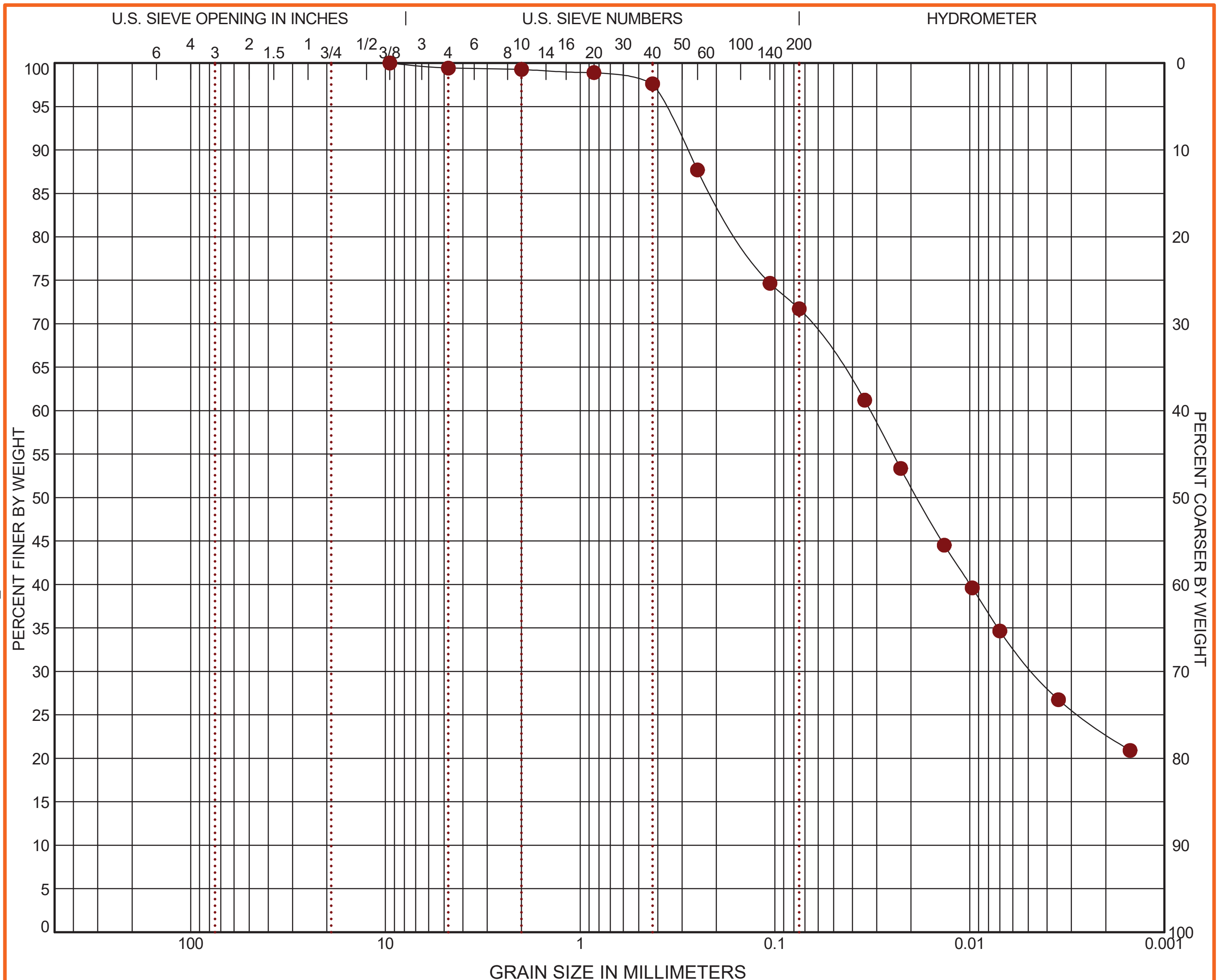
Terracon
611 Lunken Park Dr
Cincinnati, OH

PROJECT NUMBER: N1185278

CLIENT: WSP USA Inc.
Cincinnati, OH

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	B-18-1	63.5 - 65	0.0	0.6	27.7	40.9		30.8	CL

GRAIN SIZE	●		
D_{60}	0.032		
D_{30}	0.005		
D_{10}			

COEFFICIENTS	●		
C_c			
C_u			

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
3/8"	100.0				
#4	99.42				
#10	99.26				
#20	98.9				
#40	97.61				
#60	87.7				
#140	74.66				
#200	71.72				

	SOIL DESCRIPTION
●	A-6 (7)

	REMARKS
●	1518

PROJECT: Ernstbridge Road Bridge Replacement

SITE: Ernstbridge Road
Ryland Heights, KY

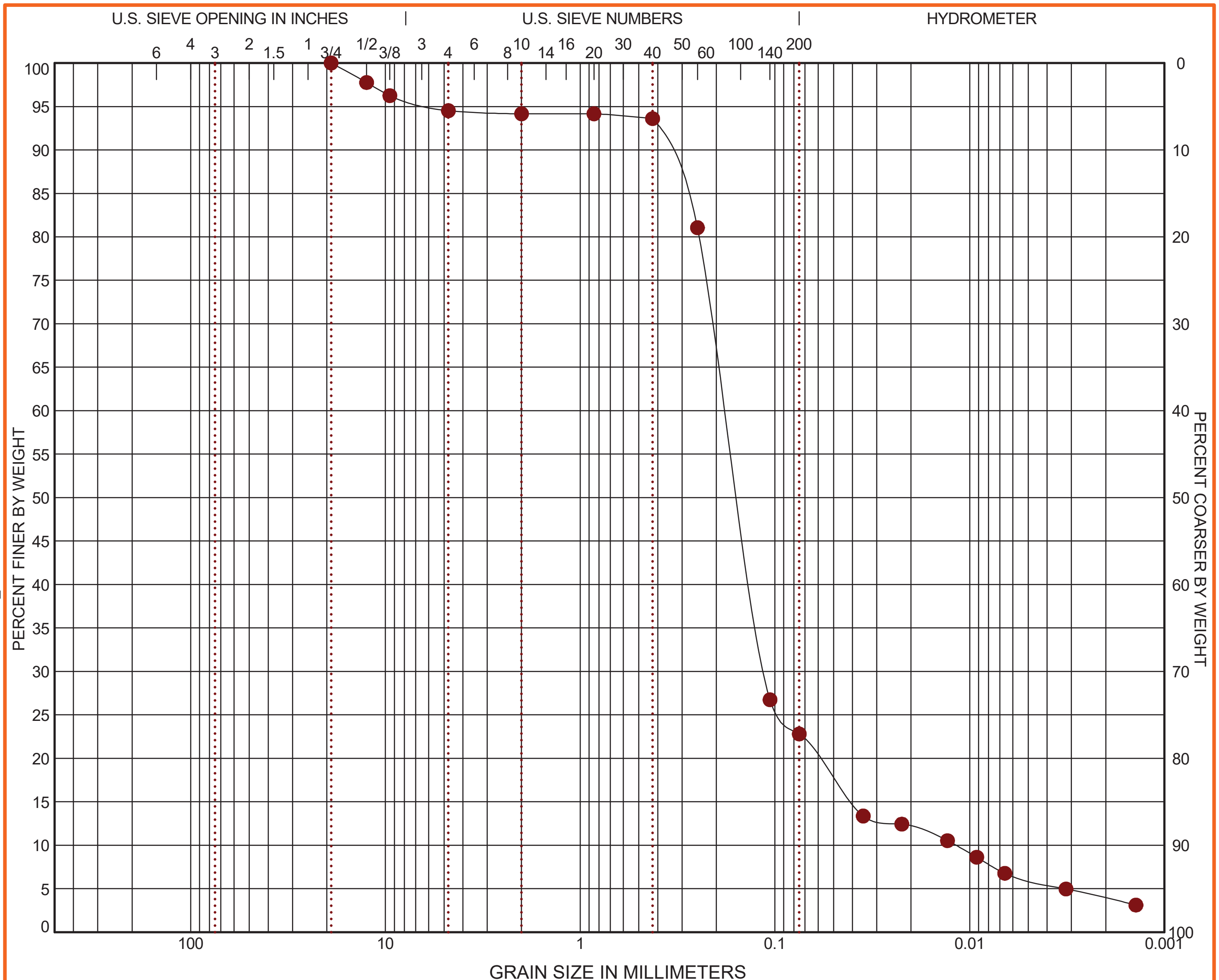
Terracon
611 Lunken Park Dr
Cincinnati, OH

PROJECT NUMBER: N1185278

CLIENT: WSP USA Inc.
Cincinnati, OH

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	B-18-1	83.5 - 85	0.0	5.5	71.7	16.7		6.1	SM

GRAIN SIZE			
	●		
D_{60}	0.179		
D_{30}	0.112		
D_{10}	0.012		

COEFFICIENTS			
	●		
C_C	5.89		
C_U	15.19		

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
3/4"	100.0				
1/2"	97.75				
3/8"	96.25				
#4	94.51				
#10	94.16				
#20	94.16				
#40	93.6				
#60	81.05				
#140	26.74				
#200	22.81				

	SOIL DESCRIPTION
●	A-2-4 (0)

	REMARKS
●	1522

PROJECT: Ernstbridge Road Bridge Replacement

SITE: Ernstbridge Road
Ryland Heights, KY

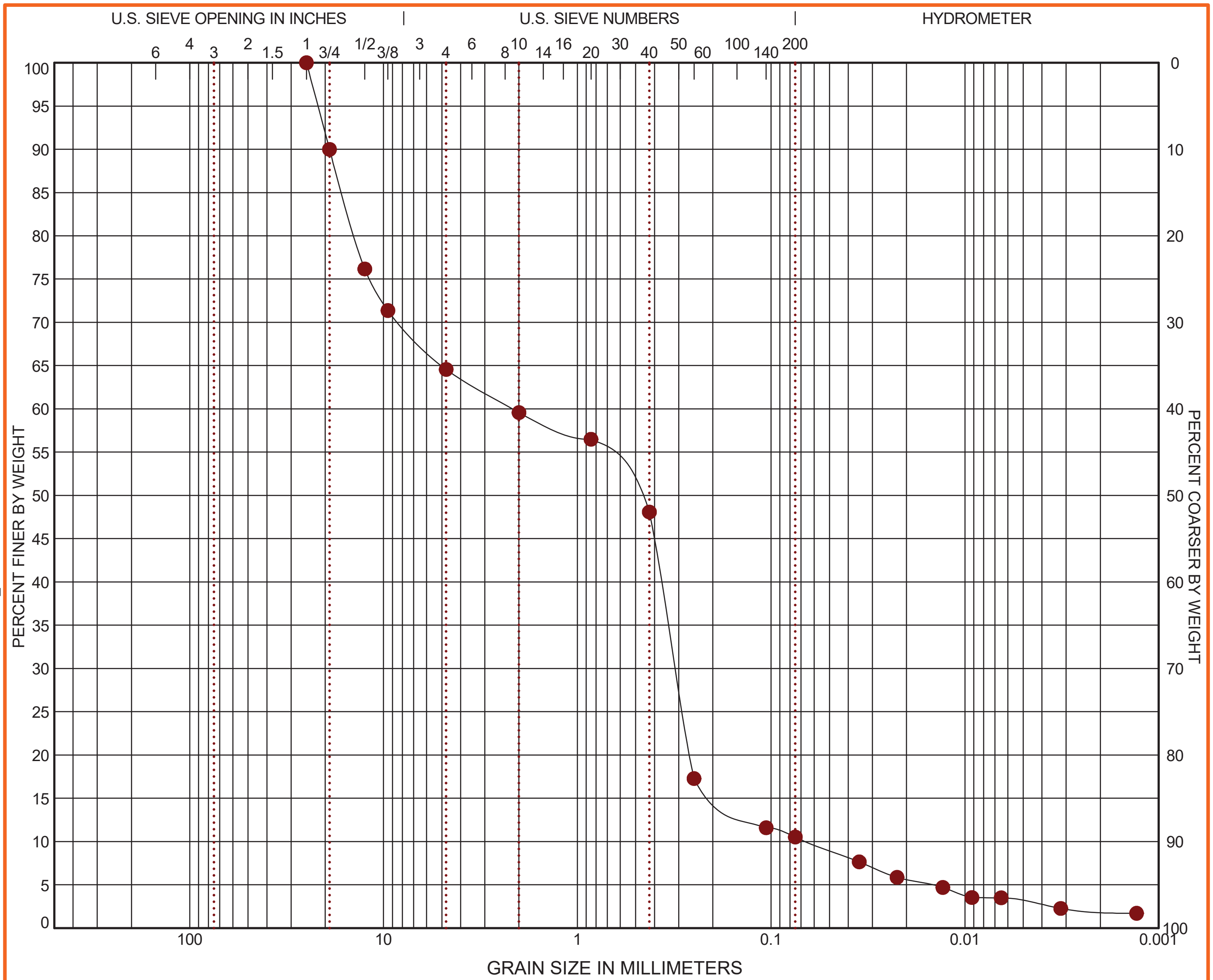
Terracon
611 Lunken Park Dr
Cincinnati, OH

PROJECT NUMBER: N1185278

CLIENT: WSP USA Inc.
Cincinnati, OH

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	B-18-1	93.5 - 94.8	0.0	35.4	54.0	7.5		3.0	SP-SM

GRAIN SIZE

	●		
D_{60}	2.154		
D_{30}	0.311		
D_{10}	0.065		

COEFFICIENTS

	●		
C_C	0.69		
C_U	33.03		

●					
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
1"	100.0				
3/4"	89.99				
1/2"	76.17				
3/8"	71.36				
#4	64.56				
#10	59.57				
#20	56.49				
#40	48.08				
#60	17.28				
#140	11.6				
#200	10.53				

SOIL DESCRIPTION

●	A-1-b (0)

REMARKS

●	1524

PROJECT: Ernstbridge Road Bridge Replacement

SITE: Ernstbridge Road
Ryland Heights, KY

Terracon
611 Lunken Park Dr
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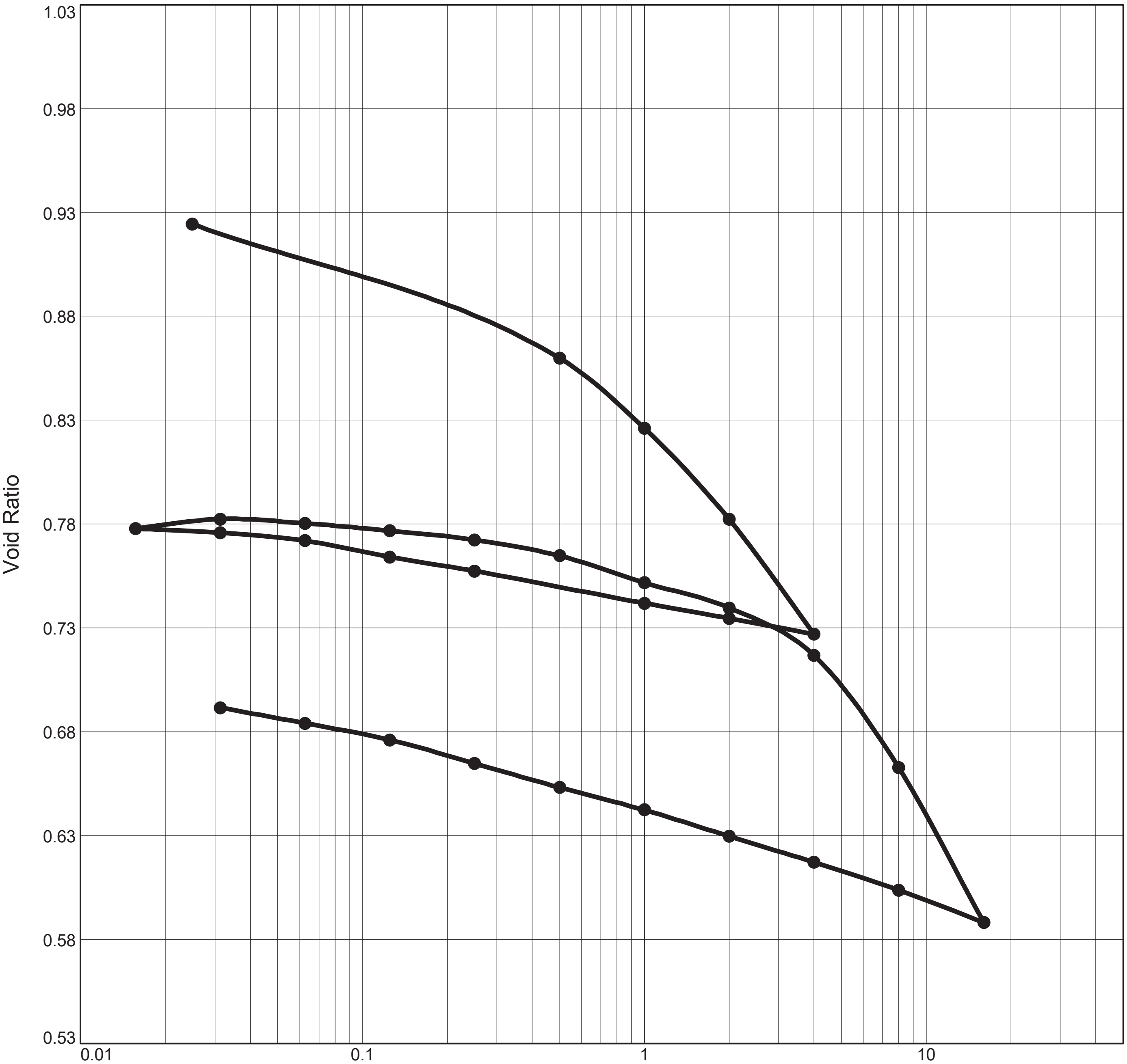
PROJECT NUMBER: N1185278

CLIENT: WSP USA Inc.
Cincinnati, OH

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
91.1 %	30.1 %	90.8	33	14	2.798	1.55	1.95	0.31	0.02	0.924

MATERIAL DESCRIPTION	USCS	AASHTO
Lean Clay with Sand	CL	A-6(10)

Project No. N1185278 **Client:** WSP USA Inc.

Project: Ernstbridge Road Bridge

Source of Sample: B-18-1 **Depth:** 25.0-27.0 ft **Sample Number:**

Terracon Consultants, Inc.

Chattanooga, TN

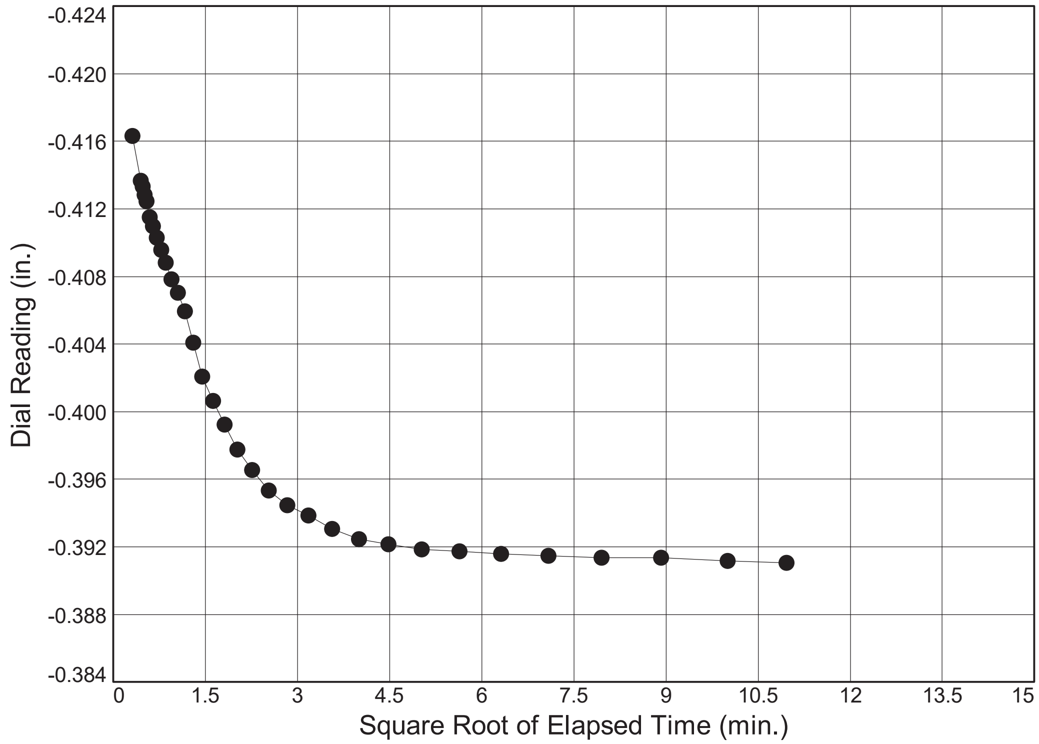
Remarks:

Swell pressure of 49.8psf.

Dial Reading vs. Time

Project No.: N1185278
Project: Ernstbridge Road Bridge

Source of Sample: B-18-1 Depth: 25.0-27.0 ft Sample Number: N/



Load No.= 2

Load= 0.50 tsf

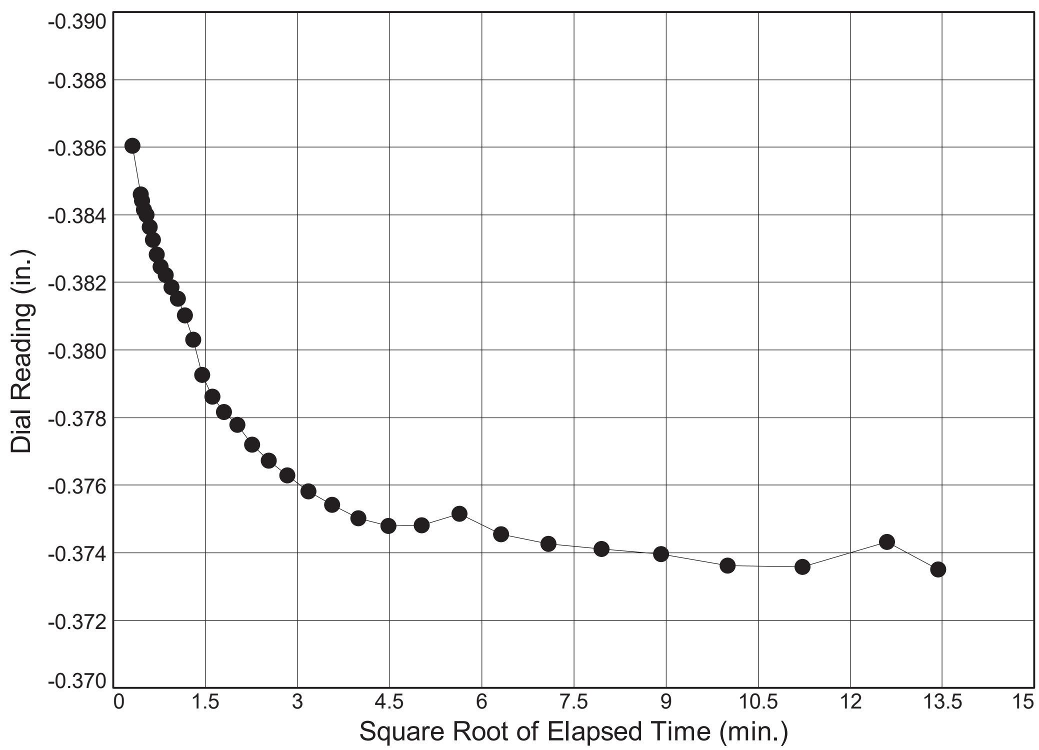
D_0 = -0.4185

D_{90} = -0.3963

D_{100} = -0.3938

T_{90} = 5.38 min.

C_v @ T_{90}
0.381 ft.^{2}/day



Load No.= 3

Load= 1.00 tsf

D_0 = -0.3868

D_{90} = -0.3778

D_{100} = -0.3768

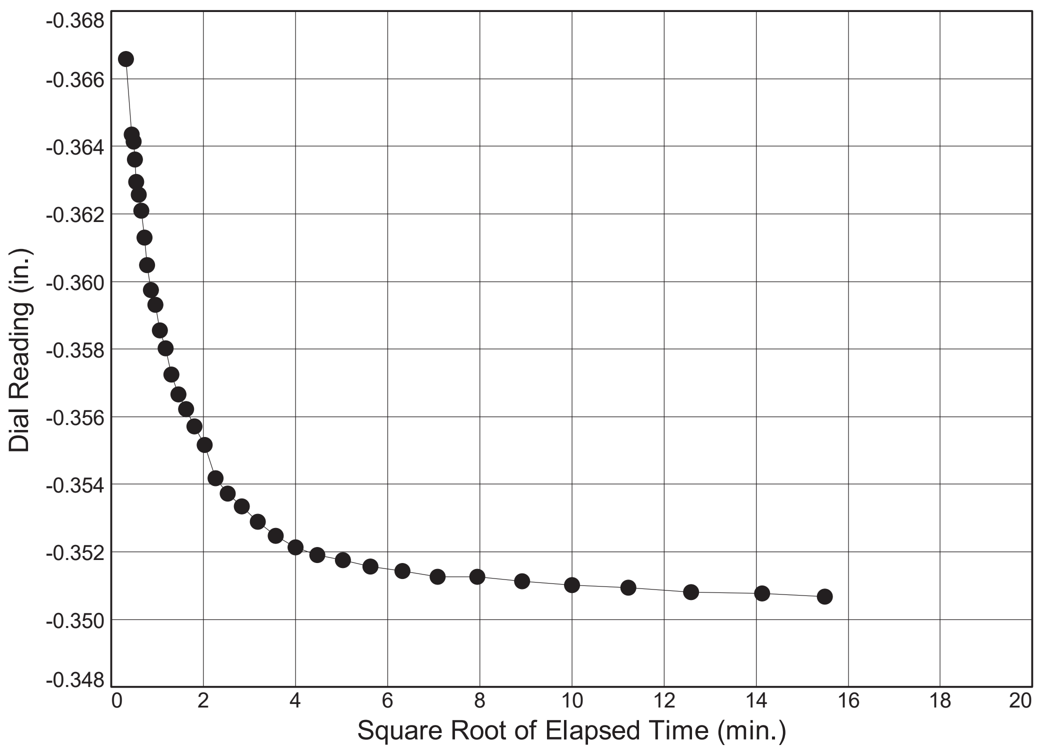
T_{90} = 4.01 min.

C_v @ T_{90}
0.485 ft.^{2}/day

Dial Reading vs. Time

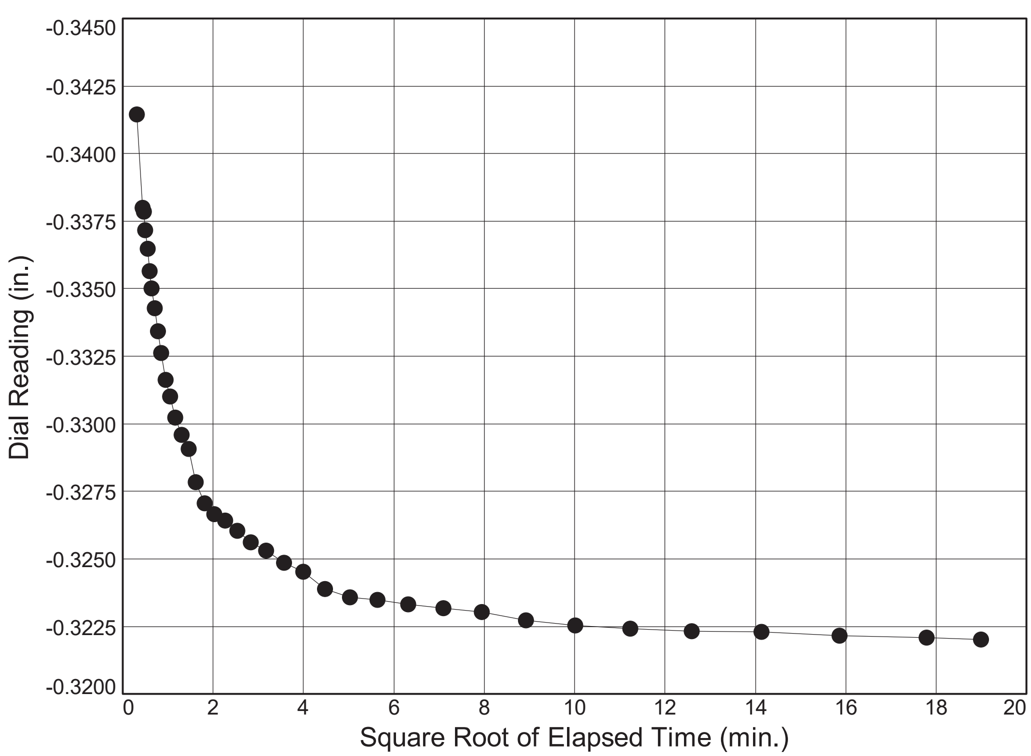
Project No.: N1185278
Project: Ernstbridge Road Bridge

Source of Sample: B-18-1 Depth: 25.0-27.0 ft Sample Number: N/



Load No.= 4
Load= 2.00 tsf
D_0 = -0.3691
D_{90} = -0.3574
D_{100} = -0.3561
T_{90} = 1.63 min.

C_v @ T_{90}
1.142 ft.^{2}/day



Load No.= 5
Load= 4.00 tsf
D_0 = -0.3448
D_{90} = -0.3305
D_{100} = -0.3289
T_{90} = 1.27 min.

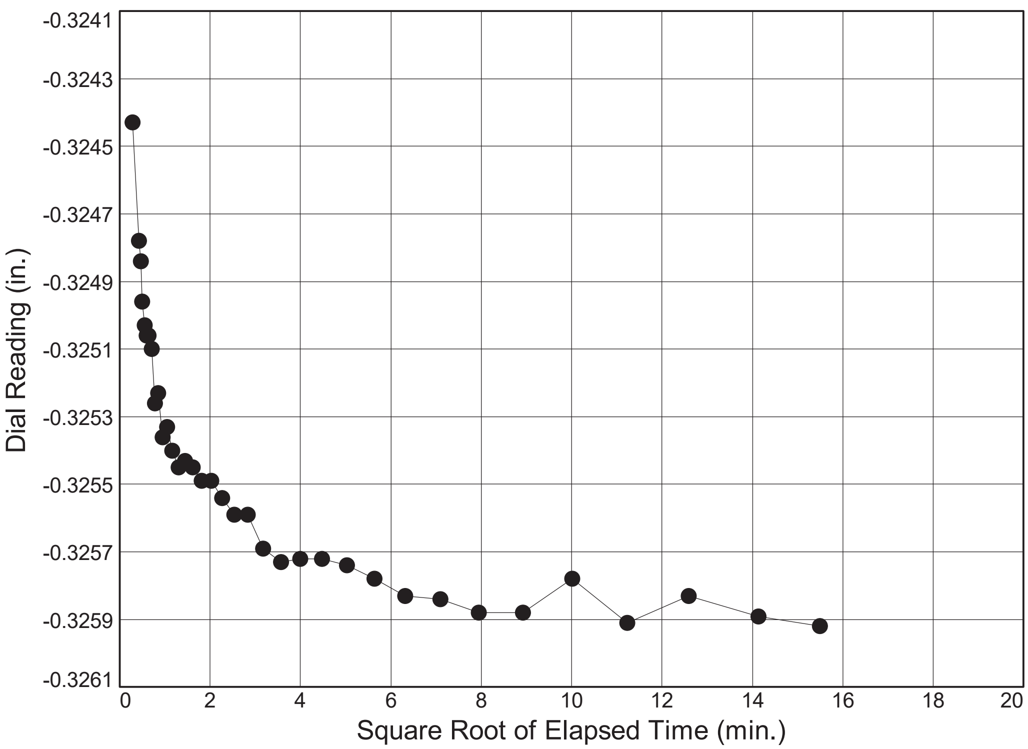
C_v @ T_{90}
1.389 ft.^{2}/day

Terracon Consultants, Inc.

Dial Reading vs. Time

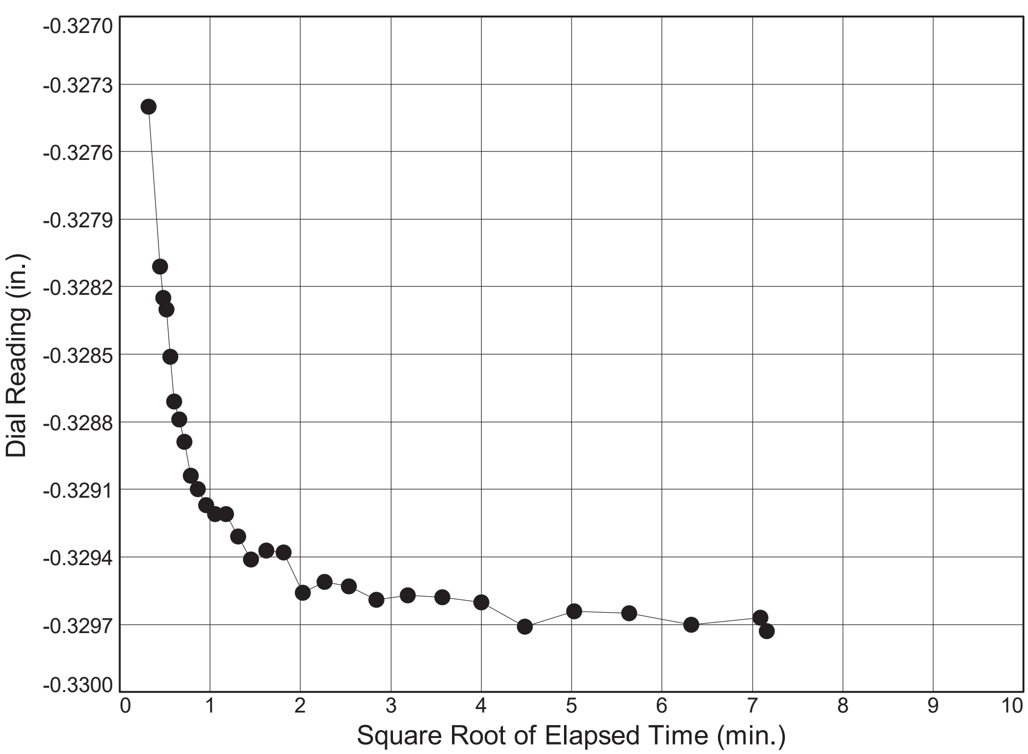
Project No.: N1185278
Project: Ernstbridge Road Bridge

Source of Sample: B-18-1 Depth: 25.0-27.0 ft Sample Number: N/



Load No.= 6
Load= 2.00 tsf
D_0 = -0.3241
D_{90} = -0.3252
D_{100} = -0.3254
T_{90} = 0.72 min.

C_v @ T_{90}
2.388 ft.^{2}/day



Load No.= 7
Load= 1.00 tsf
D_0 = -0.3262
D_{90} = -0.3291
D_{100} = -0.3294
T_{90} = 0.65 min.

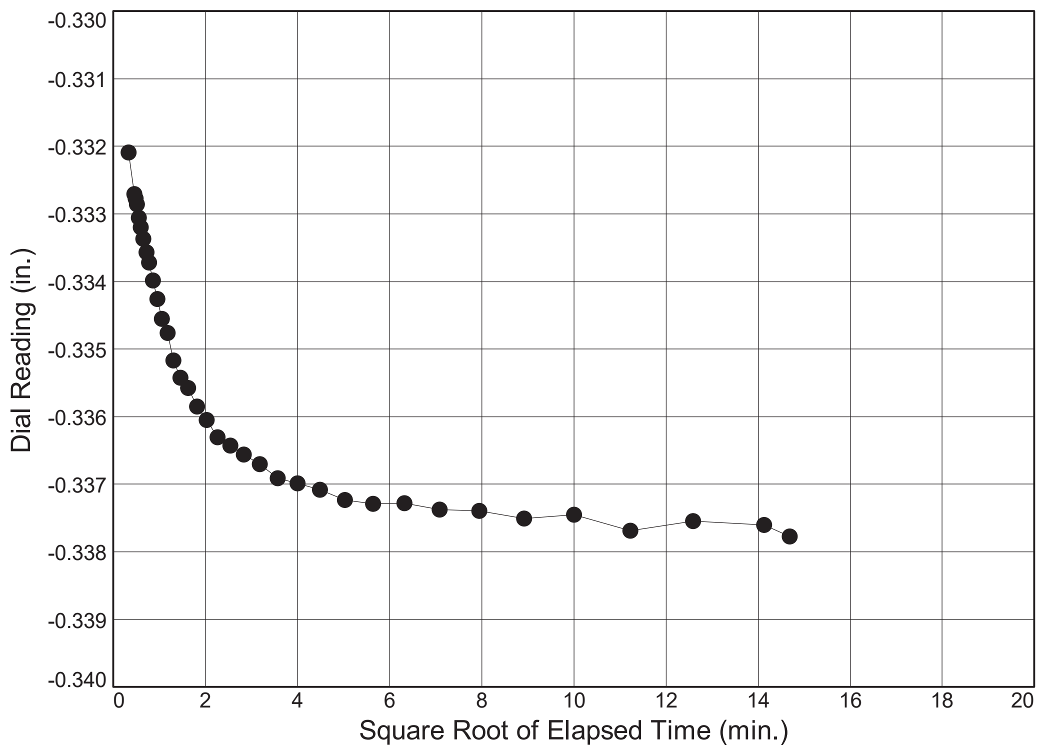
C_v @ T_{90}
2.681 ft.^{2}/day

Terracon Consultants, Inc.

Dial Reading vs. Time

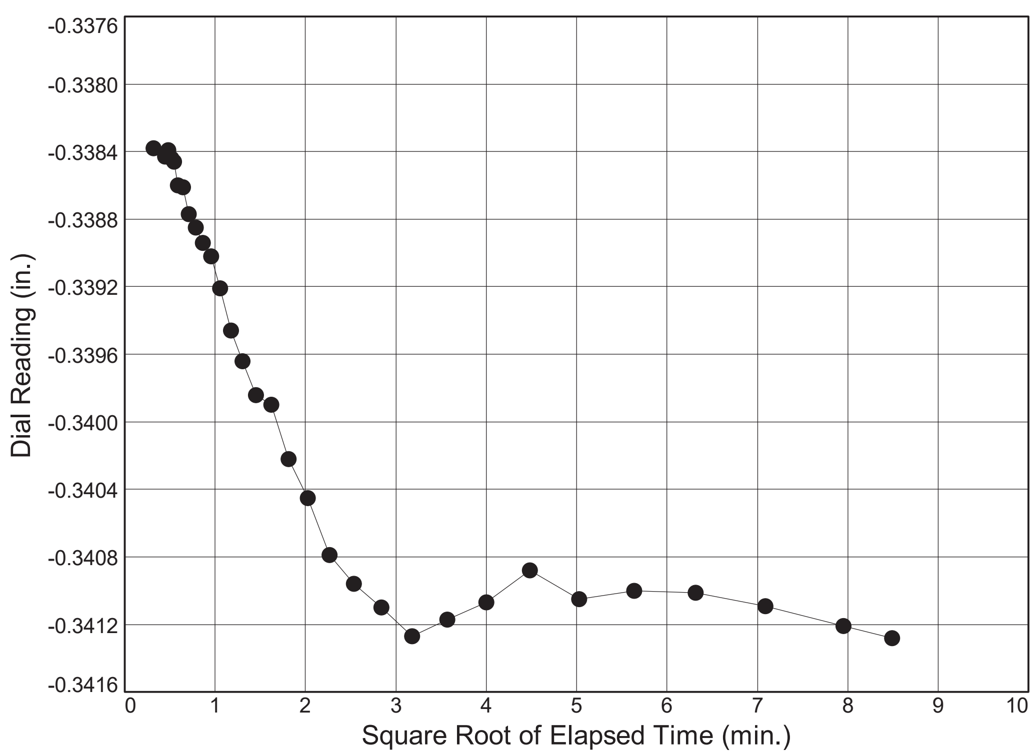
Project No.: N1185278
Project: Ernstbridge Road Bridge

Source of Sample: B-18-1 Depth: 25.0-27.0 ft Sample Number: N/



Load No.= 8
Load= 0.25 tsf
D_0 = -0.3314
D_{90} = -0.3357
D_{100} = -0.3362
T_{90} = 2.91 min.

C_v @ T_{90}
0.603 ft.^{2}/day



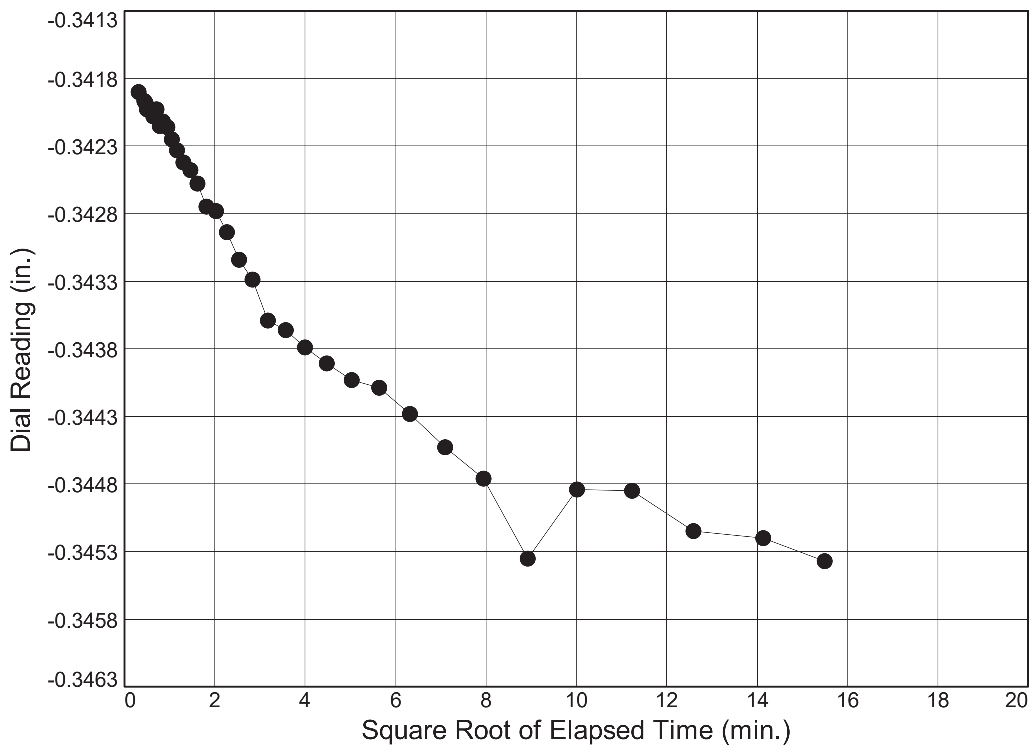
Load No.= 9
Load= 0.13 tsf
D_0 = -0.3378
D_{90} = -0.3412
D_{100} = -0.3415
T_{90} = 8.82 min.

C_v @ T_{90}
0.201 ft.^{2}/day

Dial Reading vs. Time

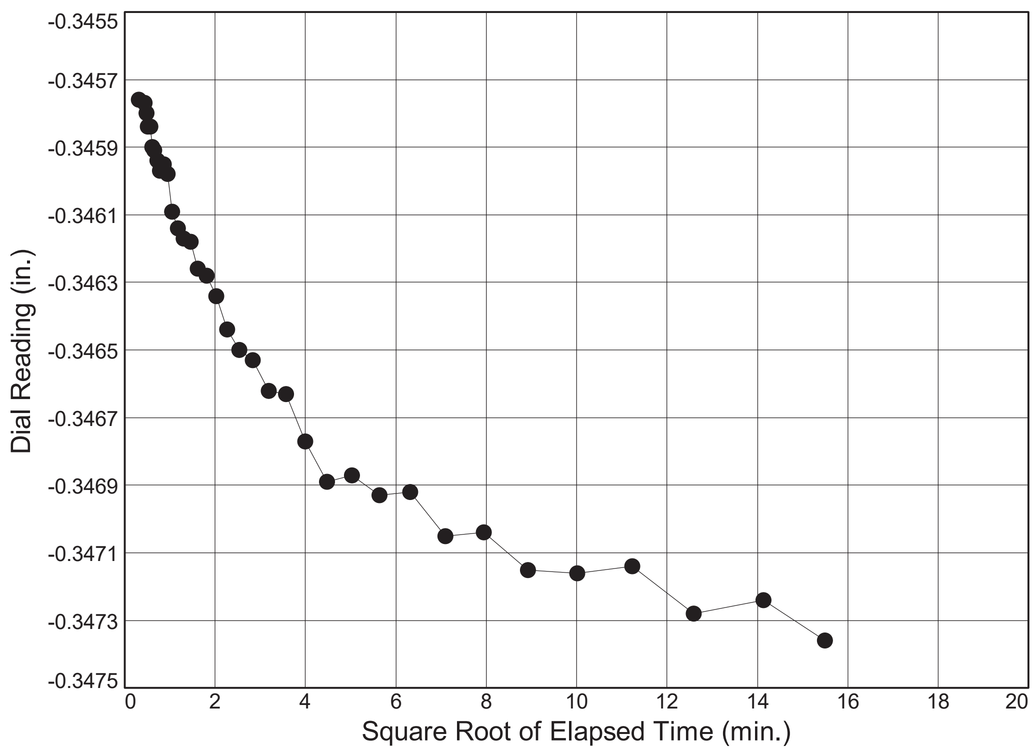
Project No.: N1185278
Project: Ernstbridge Road Bridge

Source of Sample: B-18-1 Depth: 25.0-27.0 ft Sample Number: N/



Load No.= 10
Load= 0.06 tsf
D_0 = -0.3417
D_{90} = -0.3439
D_{100} = -0.3442
T_{90} = 20.42 min.

C_v @ T_{90}
0.088 ft.^{2}/day



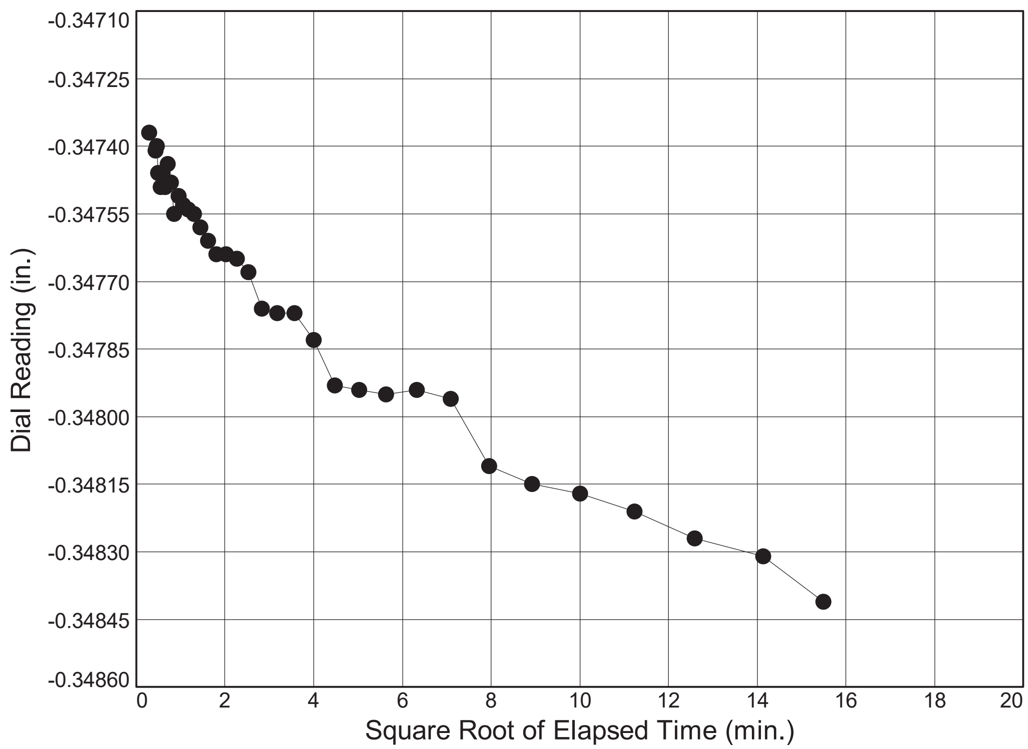
Load No.= 11
Load= 0.03 tsf
D_0 = -0.3456
D_{90} = -0.3463
D_{100} = -0.3463
T_{90} = 3.08 min.

C_v @ T_{90}
0.586 ft.^{2}/day

Dial Reading vs. Time

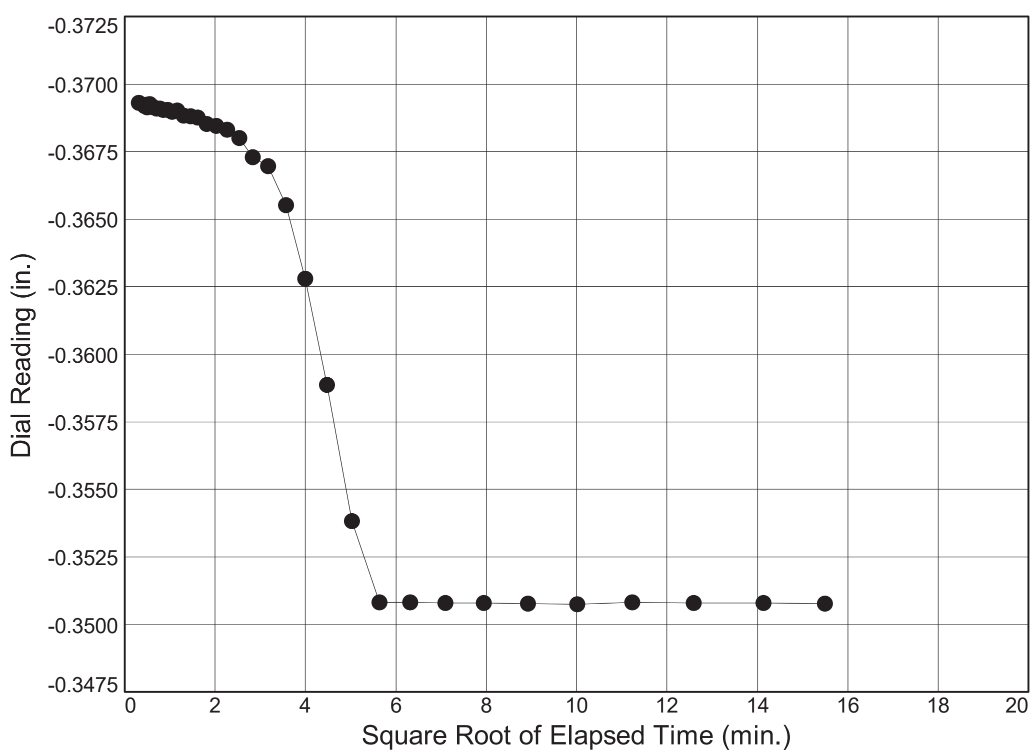
Project No.: N1185278
Project: Ernstbridge Road Bridge

Source of Sample: B-18-1 Depth: 25.0-27.0 ft Sample Number: N/



Load No.= 12
Load= 0.02 tsf
D_0 = -0.3473
D_{90} = -0.3475
D_{100} = -0.3475
T_{90} = 0.46 min.

C_v @ T_{90}
3.971 ft.^{2}/day



Load No.= 13
Load= 0.03 tsf
D_0 = -0.3910
D_{90} = -0.3508
D_{100} = -0.3464
T_{90} = 40.86 min.

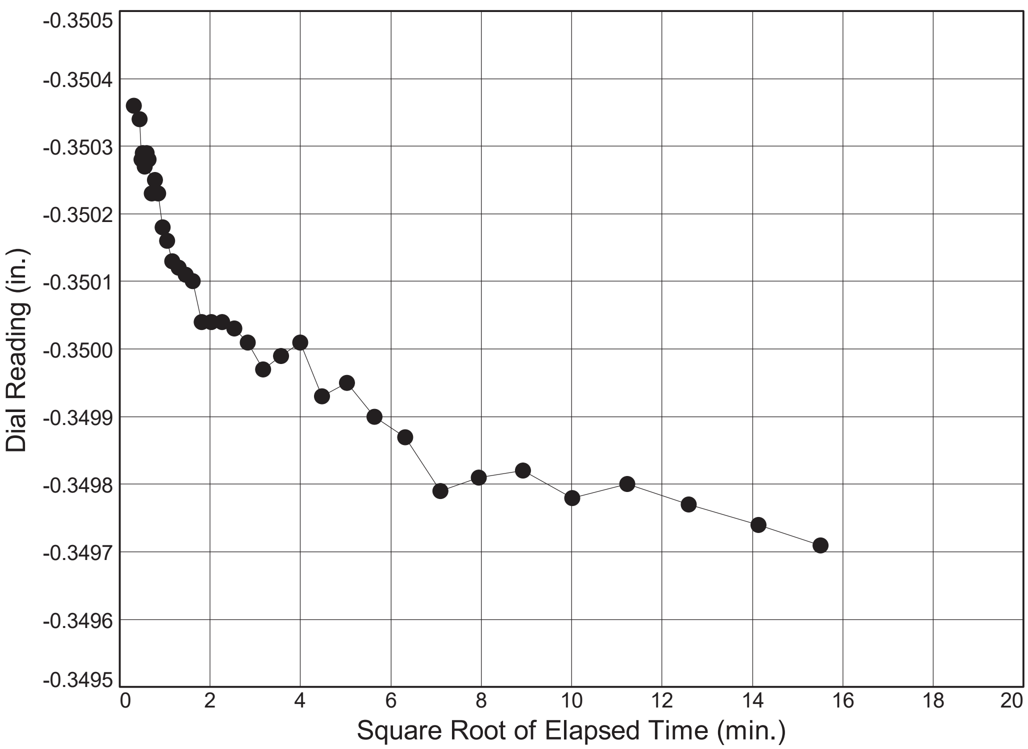
C_v @ T_{90}
0.045 ft.^{2}/day

Terracon Consultants, Inc.

Dial Reading vs. Time

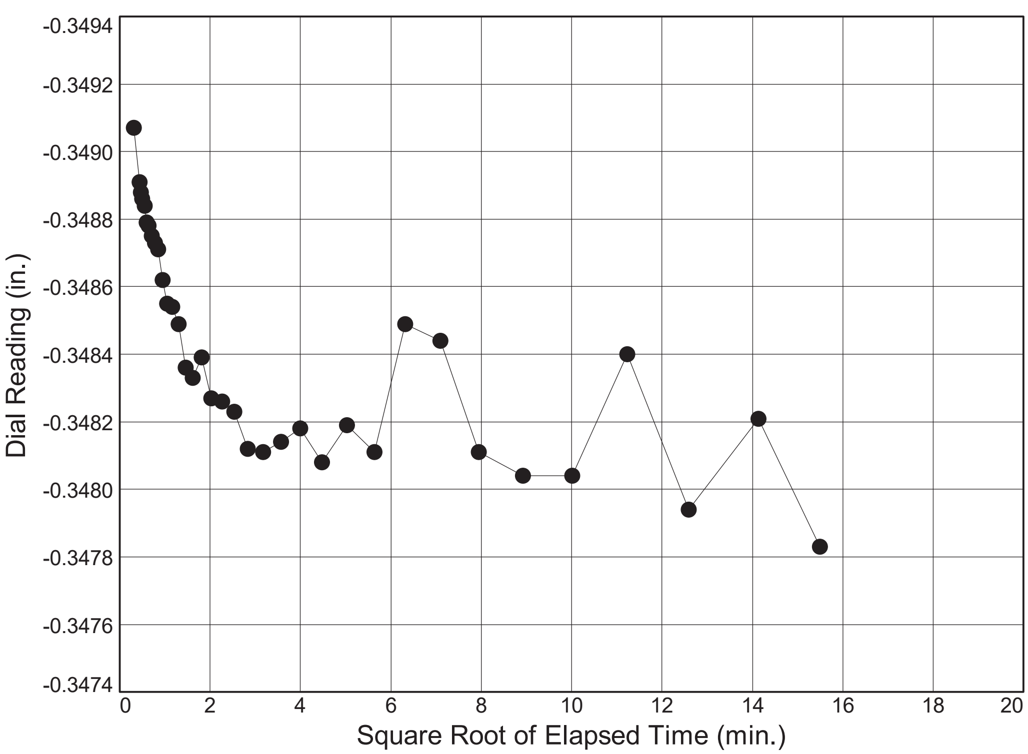
Project No.: N1185278
Project: Ernstbridge Road Bridge

Source of Sample: B-18-1 Depth: 25.0-27.0 ft Sample Number: N/



Load No.= 14
Load= 0.06 tsf
D_0 = -0.3504
D_{90} = -0.3500
D_{100} = -0.3500
T_{90} = 4.10 min.

C_v @ T_{90}
0.444 ft.^{2}/day



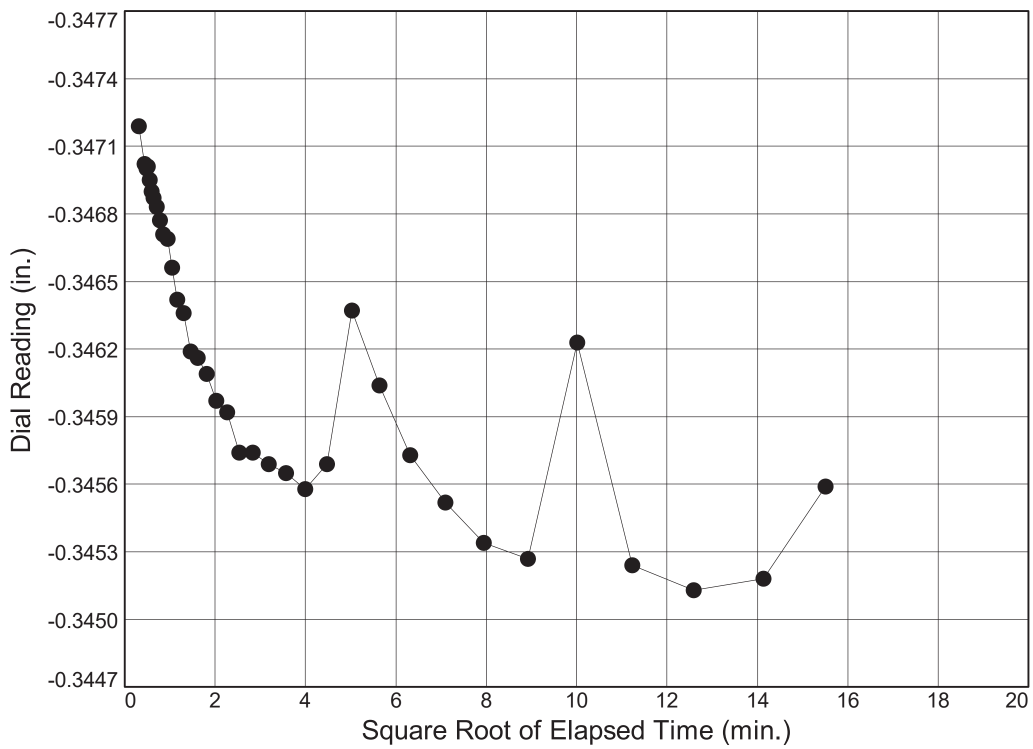
Load No.= 15
Load= 0.13 tsf
D_0 = -0.3491
D_{90} = -0.3484
D_{100} = -0.3483
T_{90} = 2.95 min.

C_v @ T_{90}
0.613 ft.^{2}/day

Dial Reading vs. Time

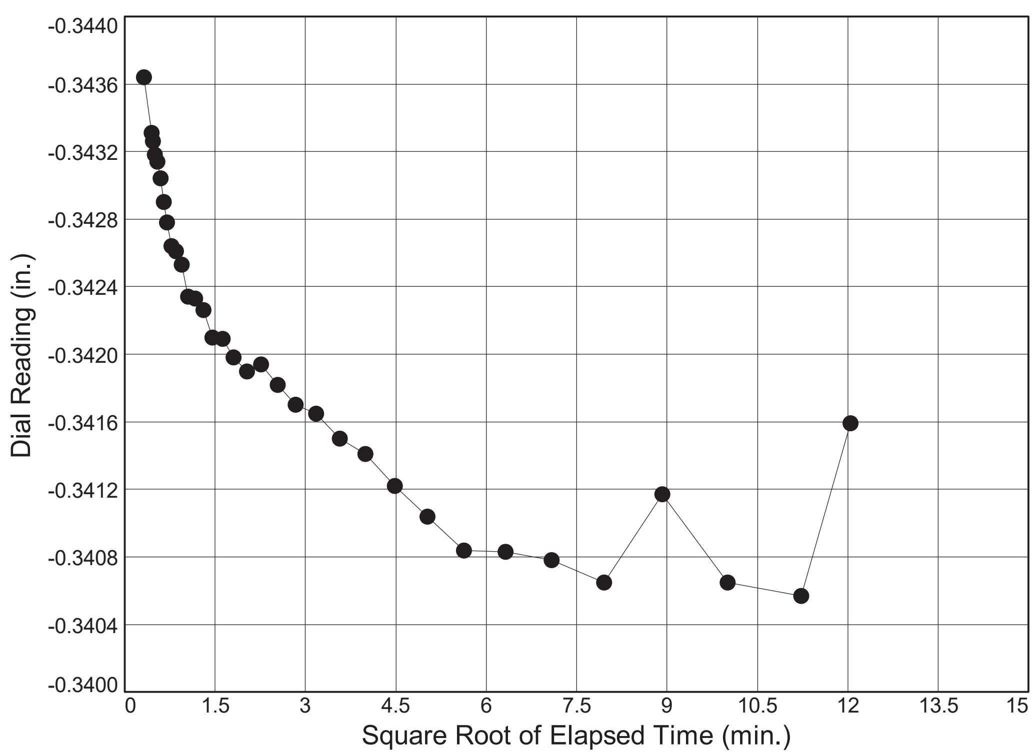
Project No.: N1185278
Project: Ernstbridge Road Bridge

Source of Sample: B-18-1 Depth: 25.0-27.0 ft Sample Number: N/



Load No.= 16
Load= 0.25 tsf
D_0 = -0.3473
D_{90} = -0.3457
D_{100} = -0.3456
T_{90} = 7.46 min.

C_v @ T_{90}
0.242 ft.^{2}/day



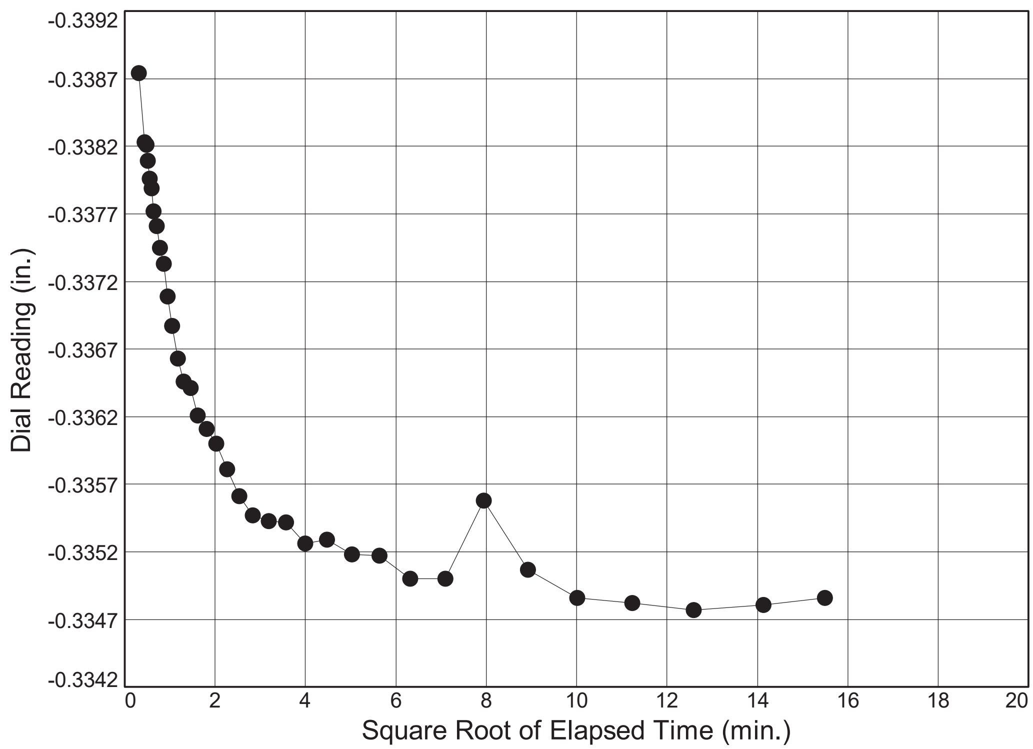
Load No.= 17
Load= 0.50 tsf
D_0 = -0.3441
D_{90} = -0.3423
D_{100} = -0.3421
T_{90} = 1.37 min.

C_v @ T_{90}
1.310 ft.^{2}/day

Dial Reading vs. Time

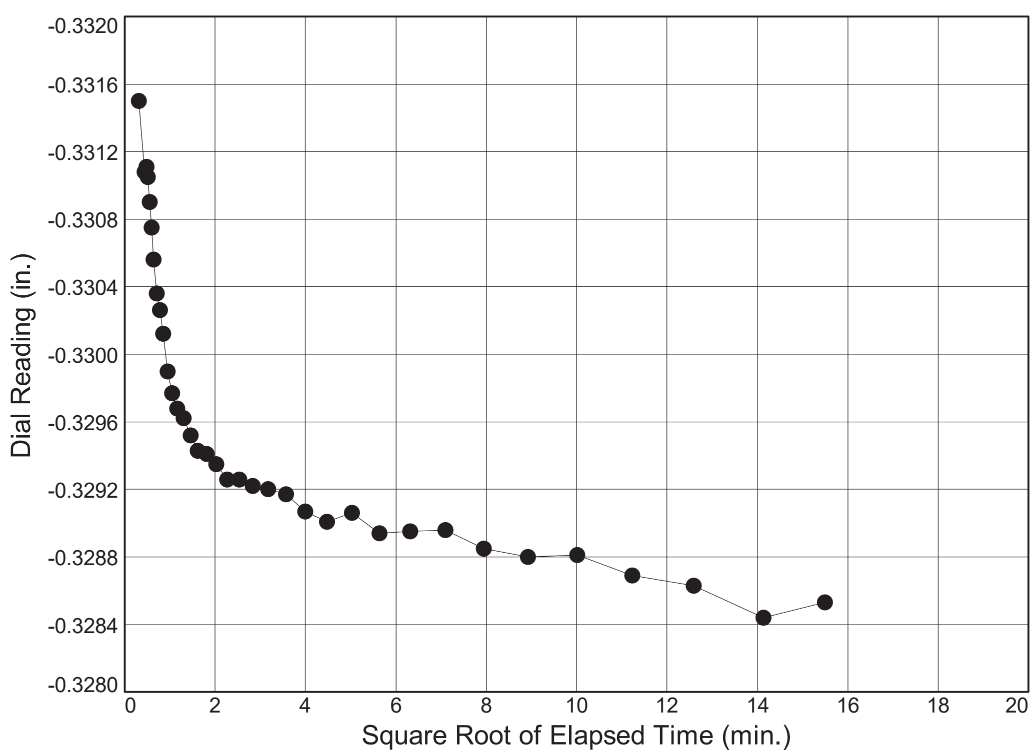
Project No.: N1185278
Project: Ernstbridge Road Bridge

Source of Sample: B-18-1   Depth: 25.0-27.0 ft   Sample Number: N/



Load No.= 18
Load= 1.00 tsf
D_0 = -0.3393
D_{90} = -0.3364
D_{100} = -0.3361
T_{90} = 2.03 min.

C_v @ T_{90}
0.871 ft.^{2}/day



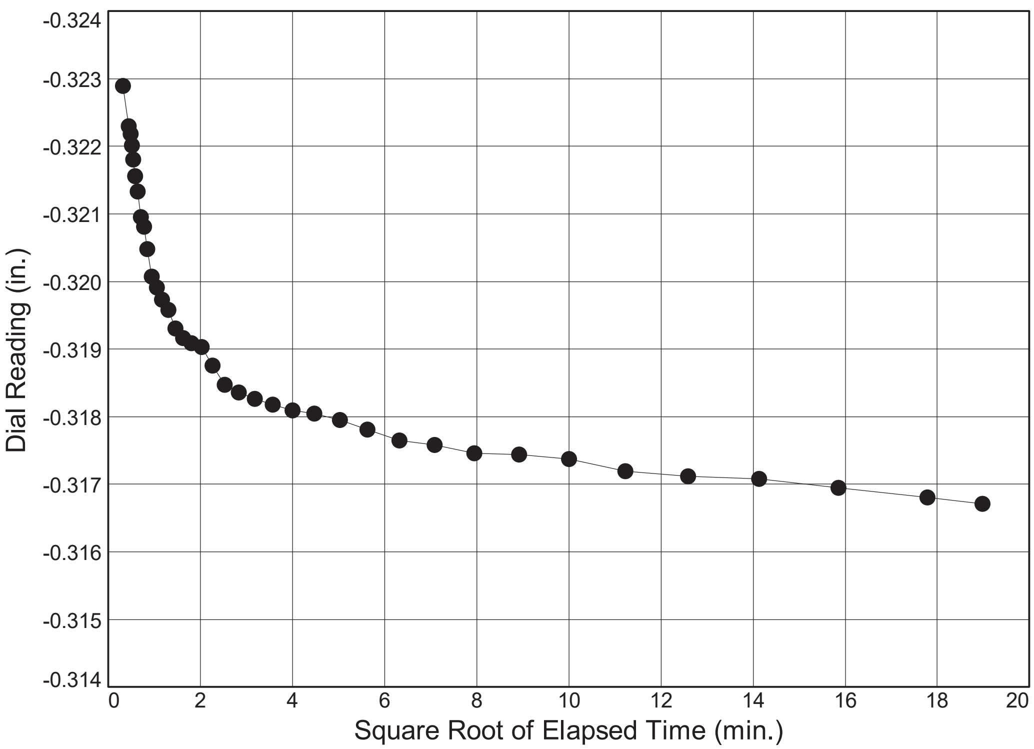
Load No.= 19
Load= 2.00 tsf
D_0 = -0.3323
D_{90} = -0.3297
D_{100} = -0.3294
T_{90} = 1.34 min.

C_v @ T_{90}
1.303 ft.^{2}/day

Dial Reading vs. Time

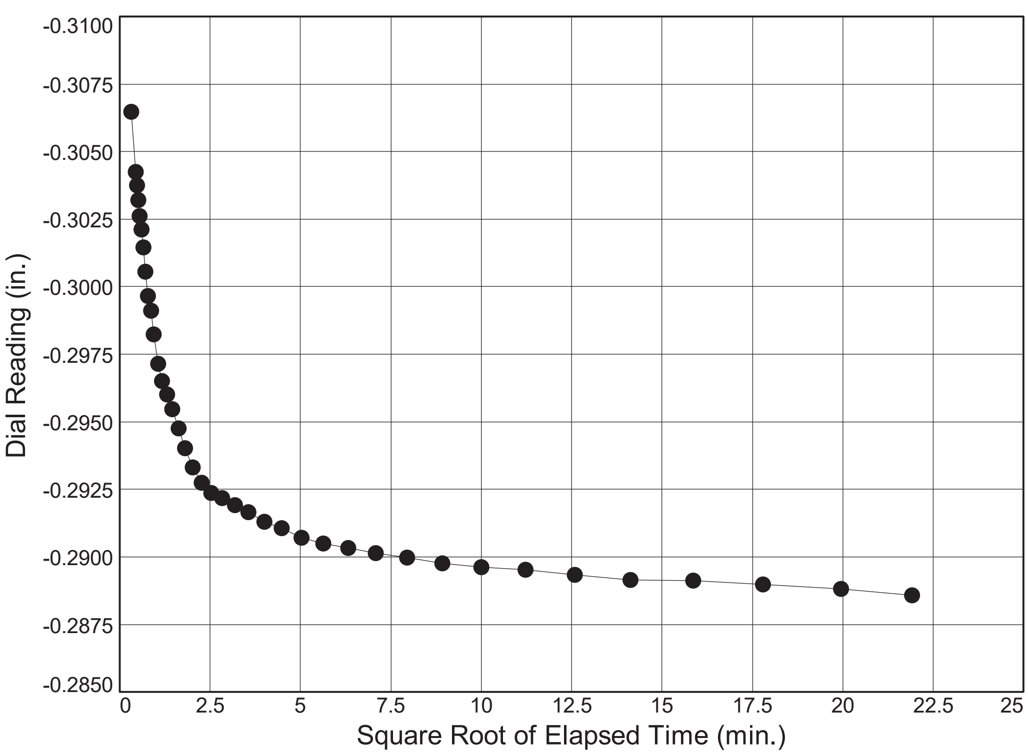
Project No.: N1185278
Project: Ernstbridge Road Bridge

Source of Sample: B-18-1 Depth: 25.0-27.0 ft Sample Number: N/



Load No.= 20
Load= 4.00 tsf
D_0 = -0.3243
D_{90} = -0.3197
D_{100} = -0.3192
T_{90} = 1.35 min.

C_v @ T_{90}
1.265 ft.^{2}/day



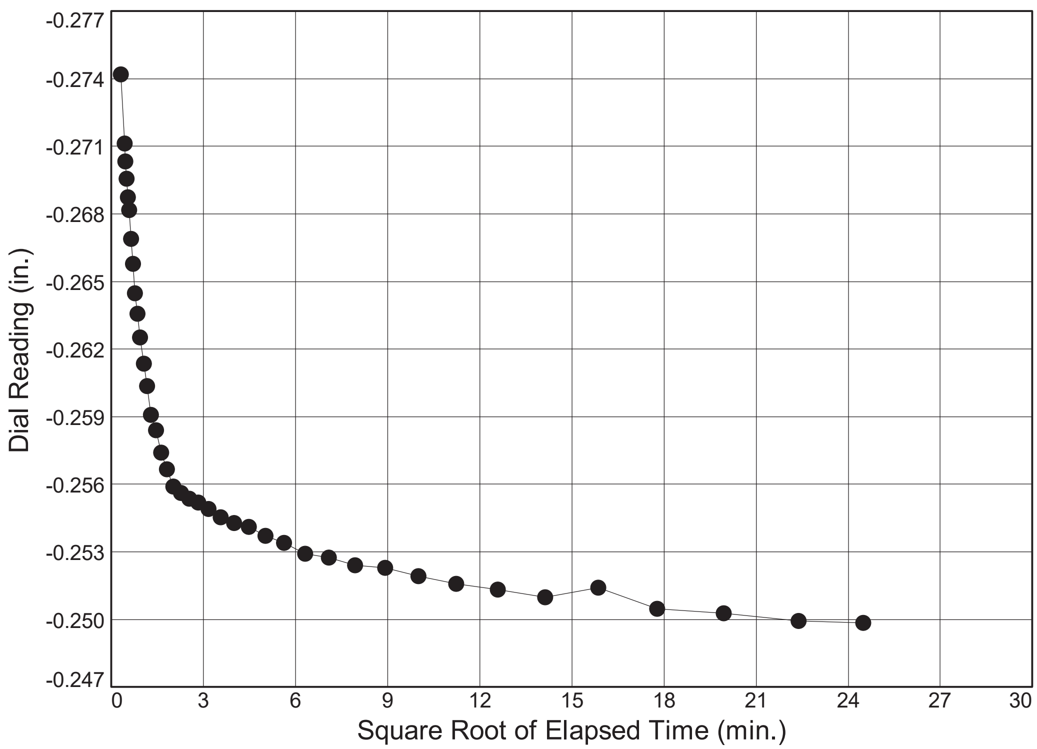
Load No.= 21
Load= 8.00 tsf
D_0 = -0.3097
D_{90} = -0.2961
D_{100} = -0.2946
T_{90} = 1.61 min.

C_v @ T_{90}
1.018 ft.^{2}/day

Dial Reading vs. Time

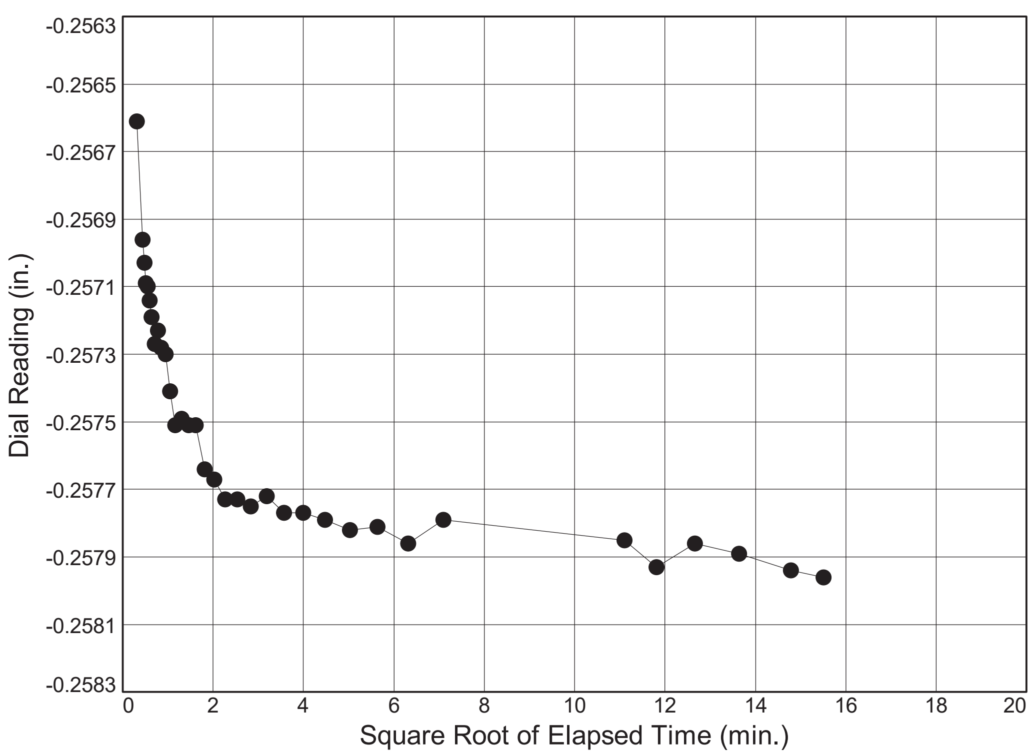
Project No.: N1185278
Project: Ernstbridge Road Bridge

Source of Sample: B-18-1 Depth: 25.0-27.0 ft Sample Number: N/



Load No.= 22
Load= 16.00 tsf
D_0 = -0.2791
D_{90} = -0.2602
D_{100} = -0.2581
T_{90} = 1.40 min.

C_v @ T_{90}
1.080 ft.^{2}/day



Load No.= 23
Load= 8.00 tsf
D_0 = -0.2560
D_{90} = -0.2573
D_{100} = -0.2574
T_{90} = 0.52 min.

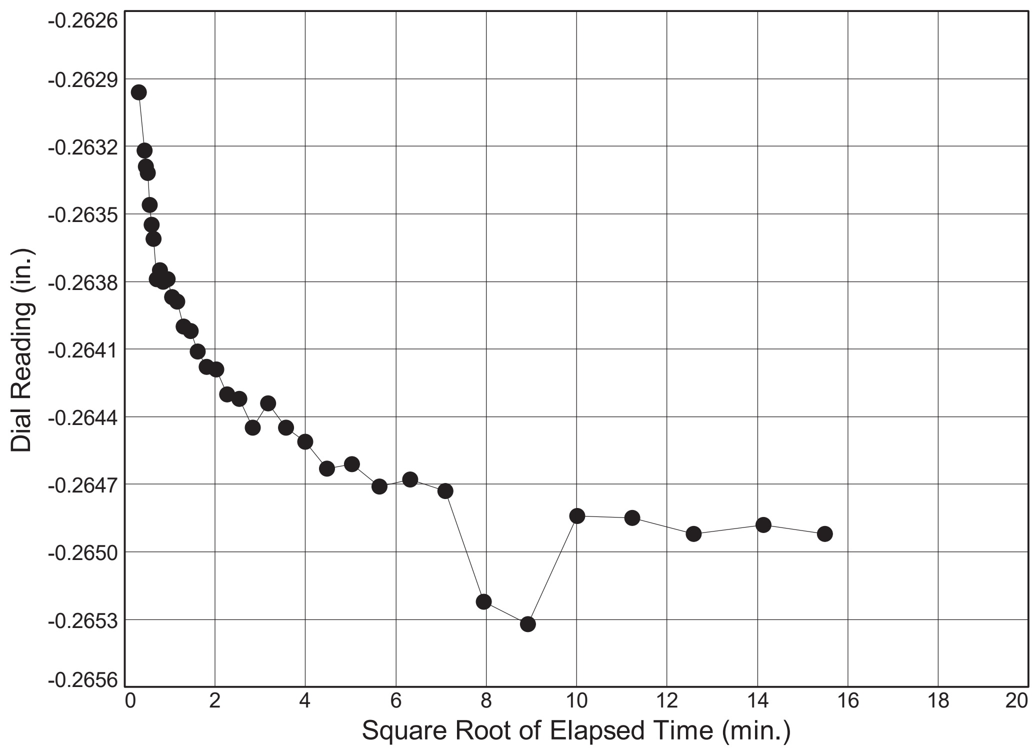
C_v @ T_{90}
2.832 ft.^{2}/day

Terracon Consultants, Inc.

Dial Reading vs. Time

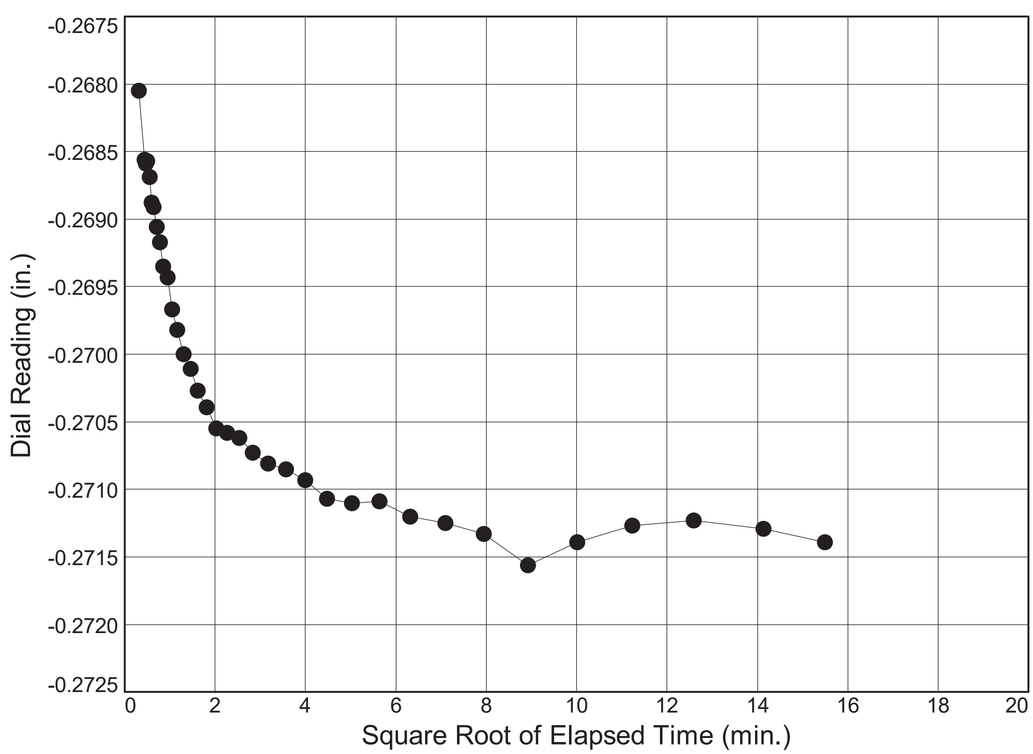
Project No.: N1185278
Project: Ernstbridge Road Bridge

Source of Sample: B-18-1 Depth: 25.0-27.0 ft Sample Number: N/



Load No.= 24
Load= 4.00 tsf
D_0 = -0.2623
D_{90} = -0.2638
D_{100} = -0.2639
T_{90} = 0.67 min.

C_v @ T_{90}
2.228 ft.^{2}/day



Load No.= 25
Load= 2.00 tsf
D_0 = -0.2677
D_{90} = -0.2702
D_{100} = -0.2705
T_{90} = 2.46 min.

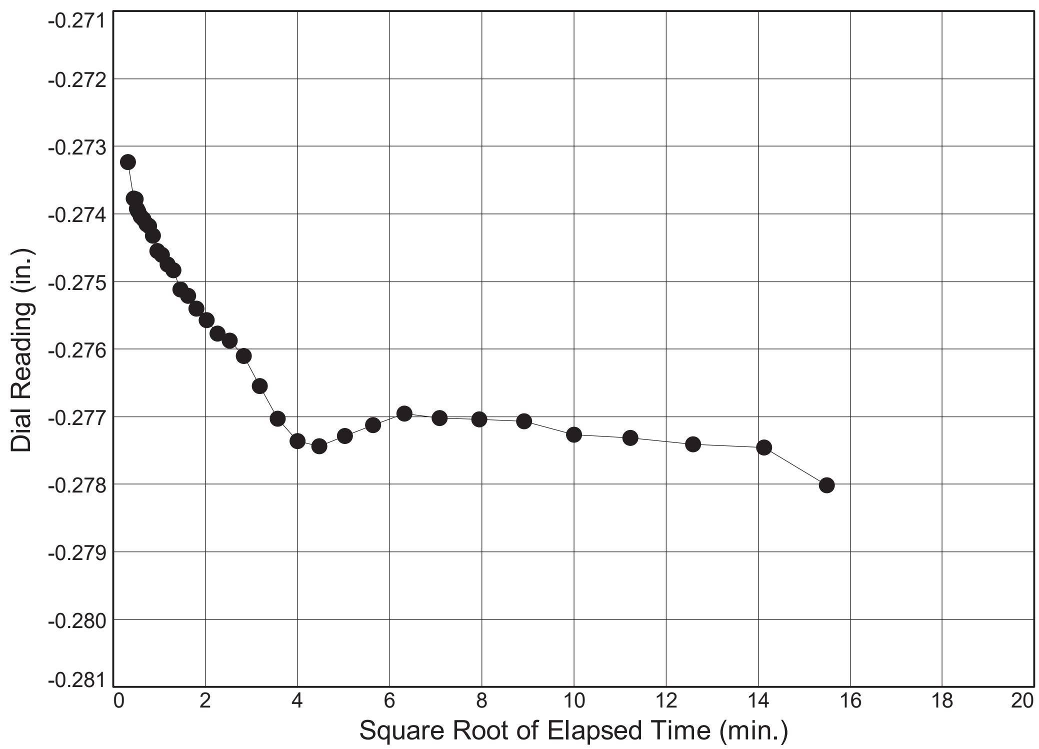
C_v @ T_{90}
0.613 ft.^{2}/day

Terracon Consultants, Inc.

Dial Reading vs. Time

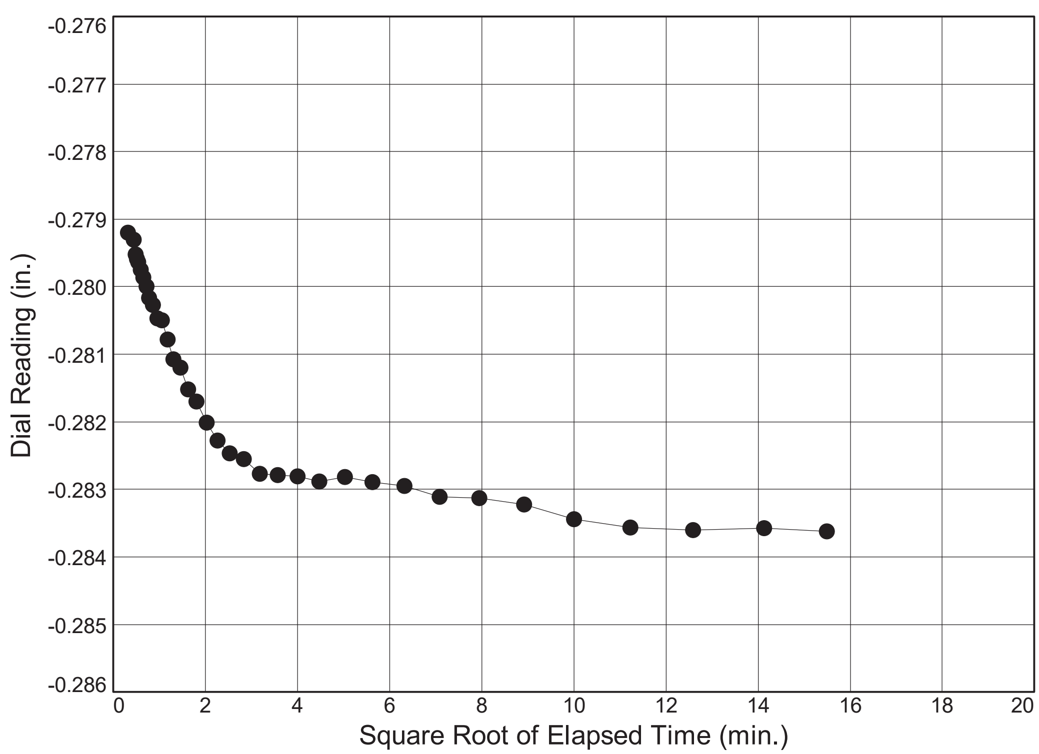
Project No.: N1185278
Project: Ernstbridge Road Bridge

Source of Sample: B-18-1 Depth: 25.0-27.0 ft Sample Number: N/



Load No.= 26
Load= 1.00 tsf
D_0 = -0.2734
D_{90} = -0.2774
D_{100} = -0.2779
T_{90} = 20.70 min.

C_v @ T_{90}
0.074 ft.^{2}/day



Load No.= 27
Load= 0.50 tsf
D_0 = -0.2786
D_{90} = -0.2822
D_{100} = -0.2826
T_{90} = 4.70 min.

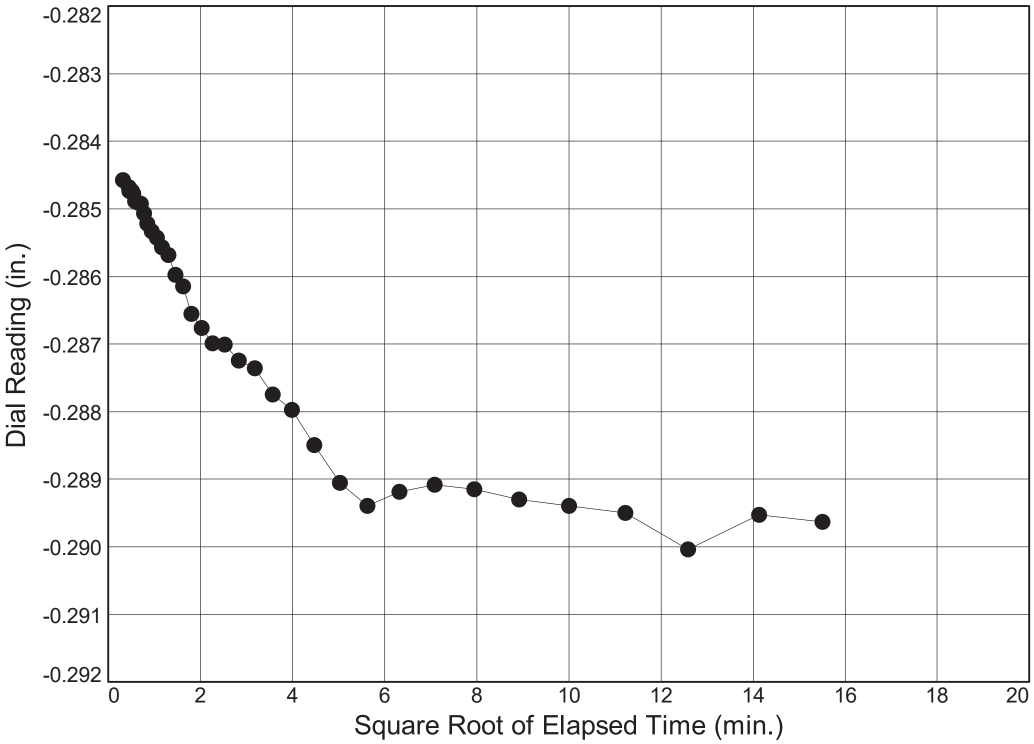
C_v @ T_{90}
0.330 ft.^{2}/day

Terracon Consultants, Inc.

Dial Reading vs. Time

Project No.: N1185278
Project: Ernstbridge Road Bridge

Source of Sample: B-18-1 Depth: 25.0-27.0 ft Sample Number: N/



Load No.= 28

Load= 0.25 tsf

D_0 = -0.2841

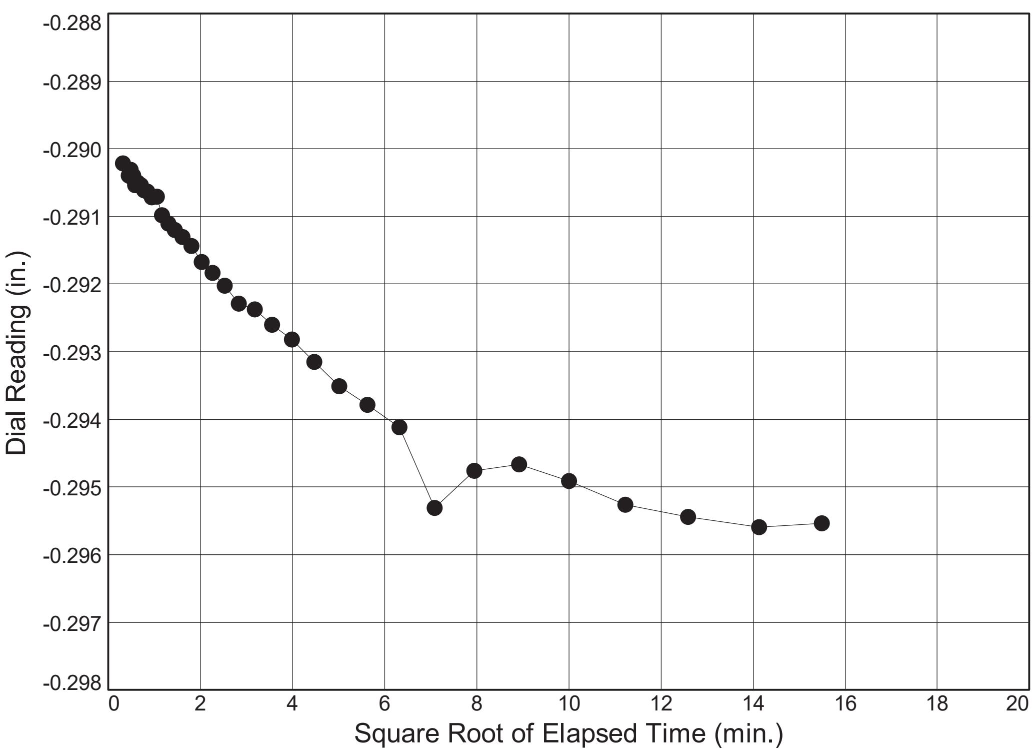
D_{90} = -0.2871

D_{100} = -0.2875

T_{90} = 7.40 min.

C_v @ T_{90}

0.213 ft.^{2}/day



Load No.= 29

Load= 0.13 tsf

D_0 = -0.2901

D_{90} = -0.2948

D_{100} = -0.2953

T_{90} = 61.61 min.

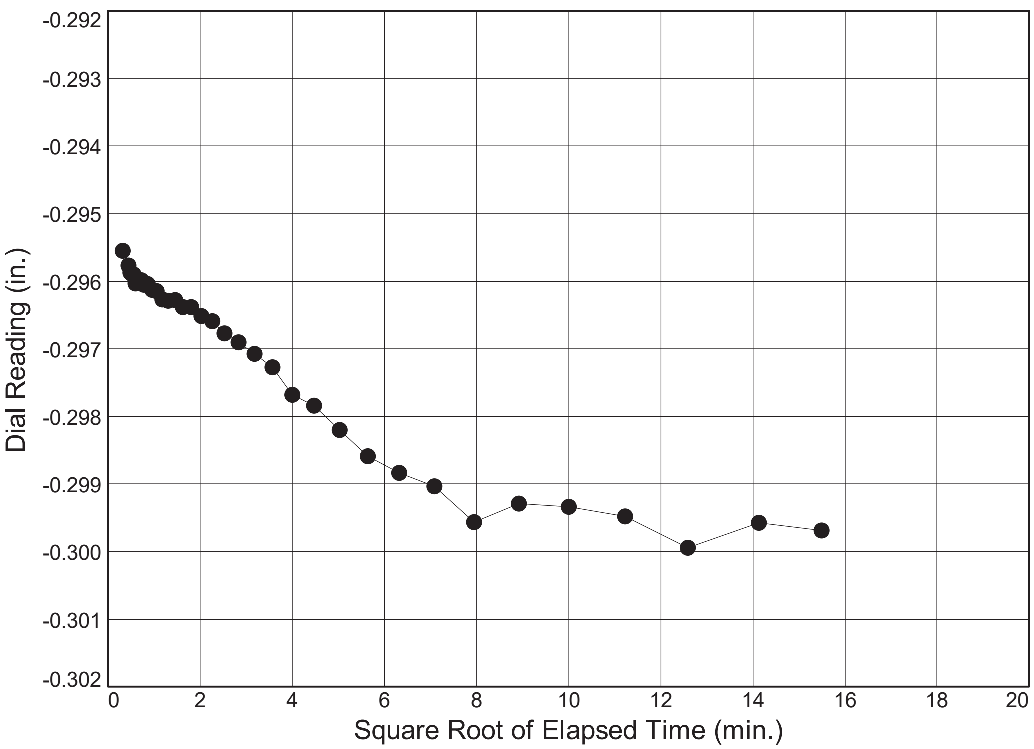
C_v @ T_{90}

0.026 ft.^{2}/day

Dial Reading vs. Time

Project No.: N1185278
Project: Ernstbridge Road Bridge

Source of Sample: B-18-1 Depth: 25.0-27.0 ft Sample Number: N/



Load No.= 30

Load= 0.06 tsf

D_0 = -0.2956

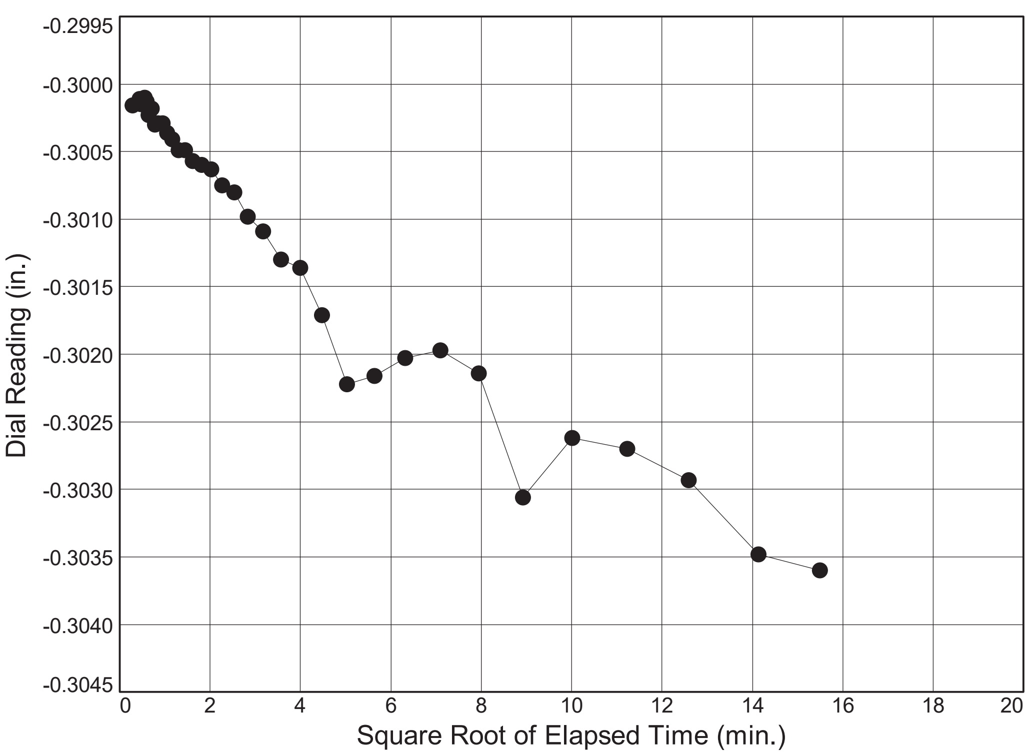
D_{90} = -0.2994

D_{100} = -0.2998

T_{90} = 74.21 min.

C_v @ T_{90}

0.022 ft.^{2}/day



Load No.= 31

Load= 0.03 tsf

D_0 = -0.3000

D_{90} = -0.3020

D_{100} = -0.3022

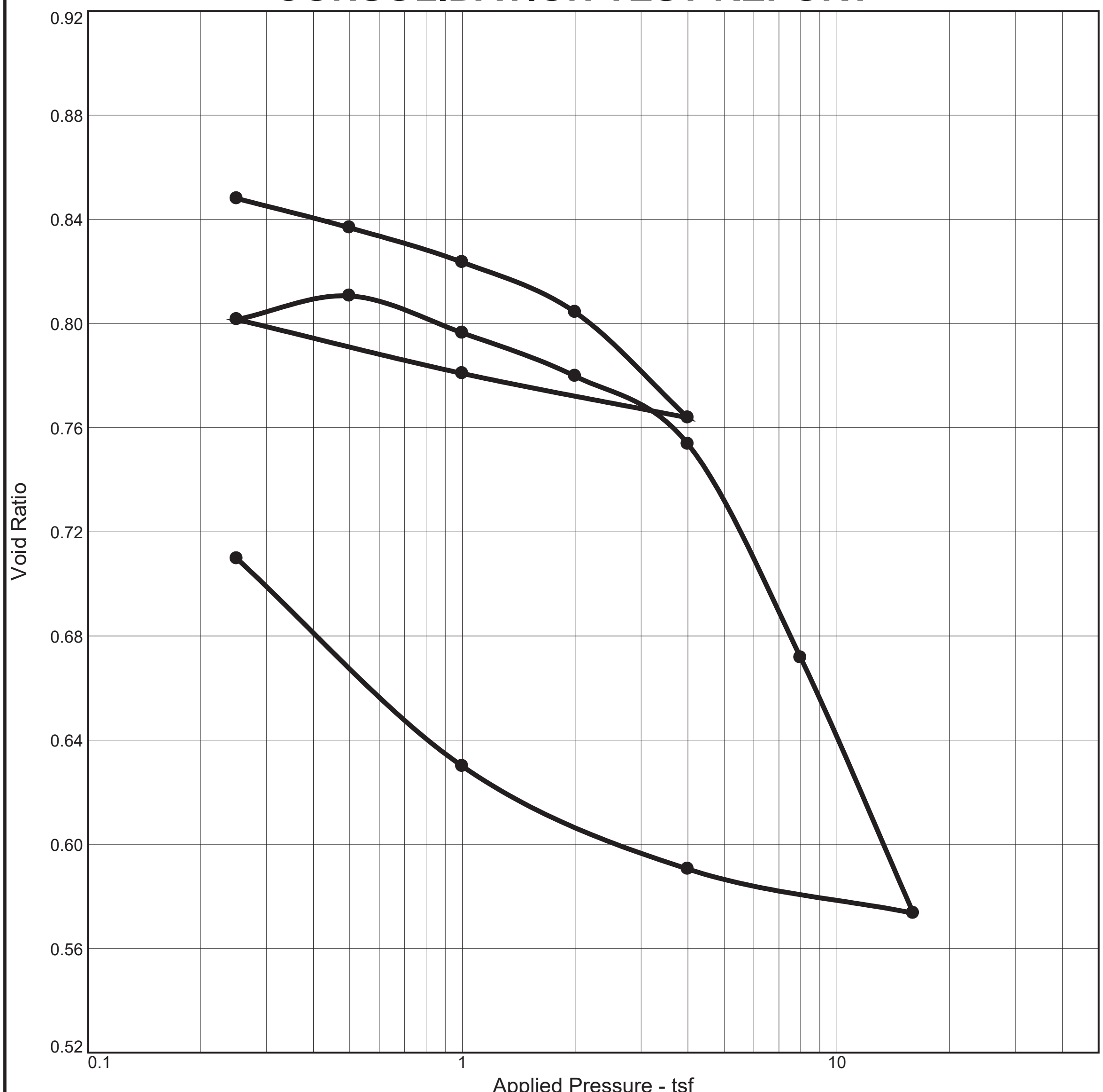
T_{90} = 42.20 min.

C_v @ T_{90}

0.039 ft.^{2}/day

Terracon Consultants, Inc.

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
94.7 %	29.8 %	91.1	32	11	2.697	2.8	3.8	0.38	0.03	0.848

MATERIAL DESCRIPTION	USCS	AASHTO
LEAN CLAY	CL	A-6(10)

Project No. N1185278 **Client:** WSP USE INC

Project: ERNSTBRIDGE ROAD BRIDGE REPLACEMENT

Source of Sample: B-18-1 **Depth:** 45-47'

Terracon, Inc.

Cincinnati, Ohio

Remarks:

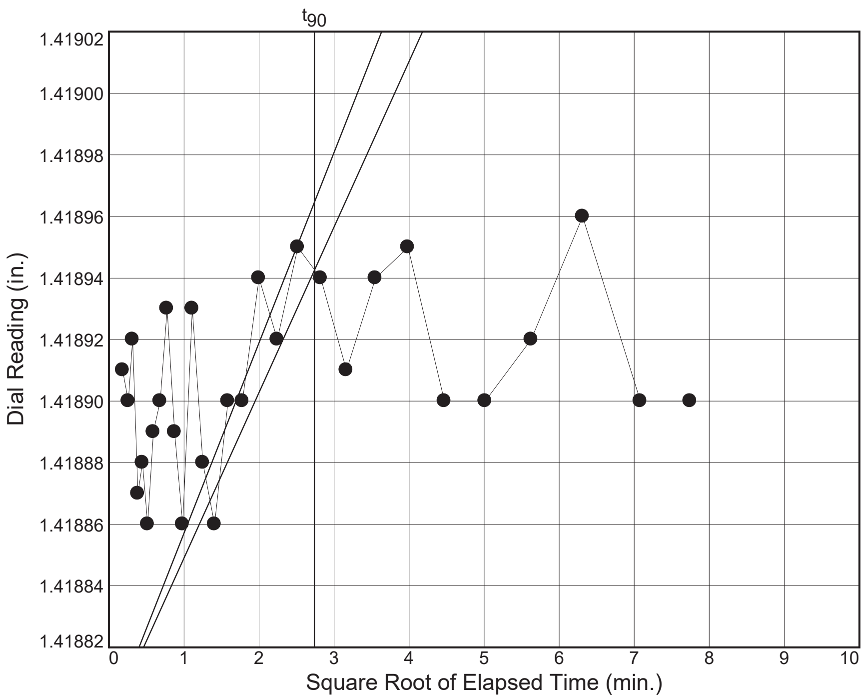
Exhibit 1513

Tested By: DR **Checked By:** GS

Dial Reading vs. Time

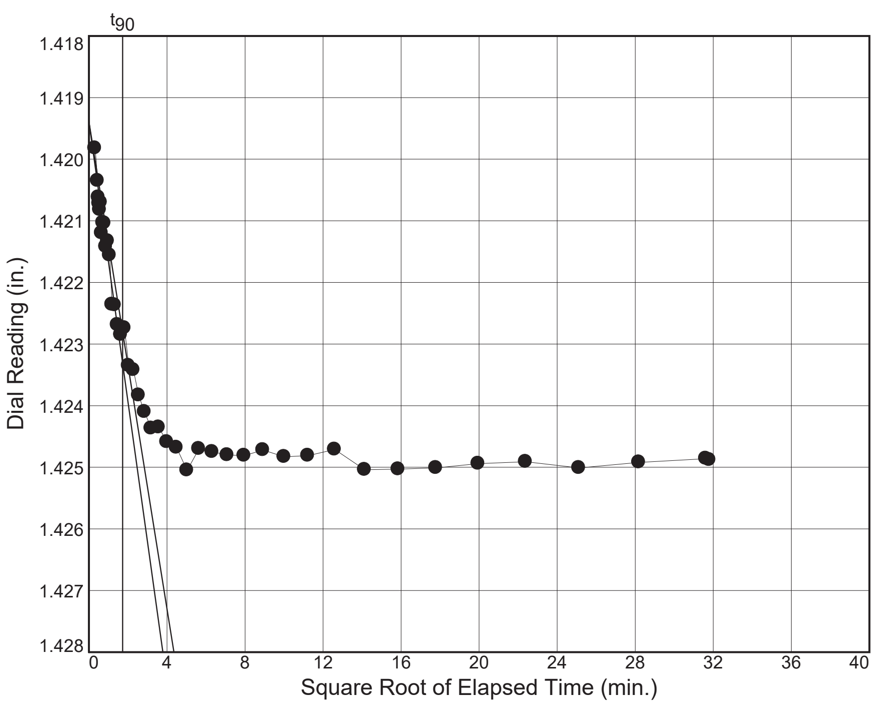
Project No.: N1185278
Project: ERNSTBRIDGE ROAD BRIDGE REPLACEMENT

Source of Sample: B-18-1 Depth: 45-47'



Load No.= 1
Load= 0.25 tsf
D_0 = 1.4188
D_{90} = 1.4189
D_{100} = 1.4190
T_{90} = 7.50 min.

C_v @ T_{90}
0.273 $ft.^2$/day



Load No.= 2
Load= 0.50 tsf
D_0 = 1.4194
D_{90} = 1.4228
D_{100} = 1.4232
T_{90} = 2.97 min.

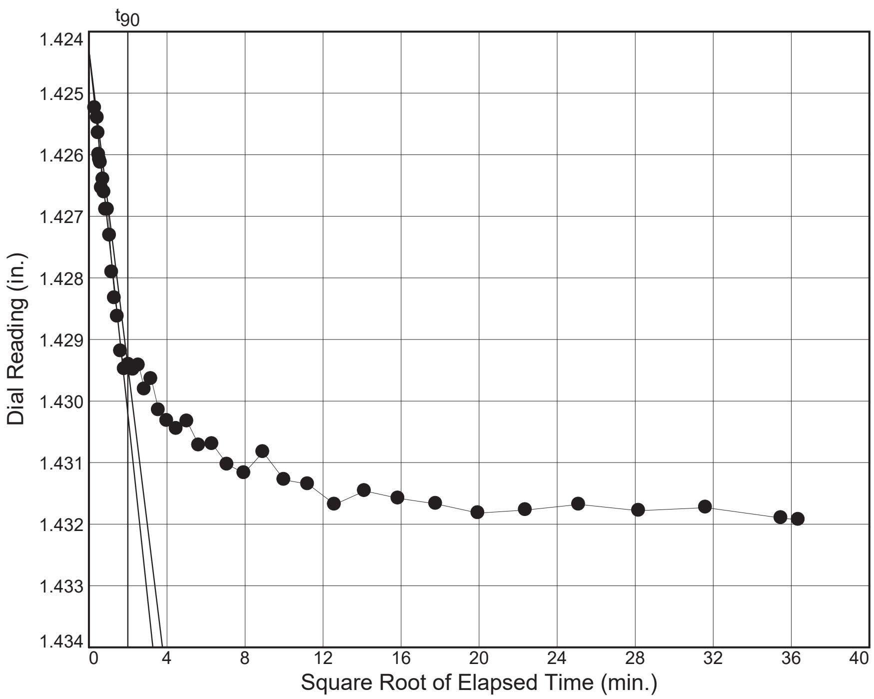
C_v @ T_{90}
0.687 $ft.^2$/day

Exhibit 1514

Dial Reading vs. Time

Project No.: N1185278
Project: ERNSTBRIDGE ROAD BRIDGE REPLACEMENT

Source of Sample: B-18-1 Depth: 45-47'



Load No.= 3

Load= 1.00 tsf

D_0 = 1.4243

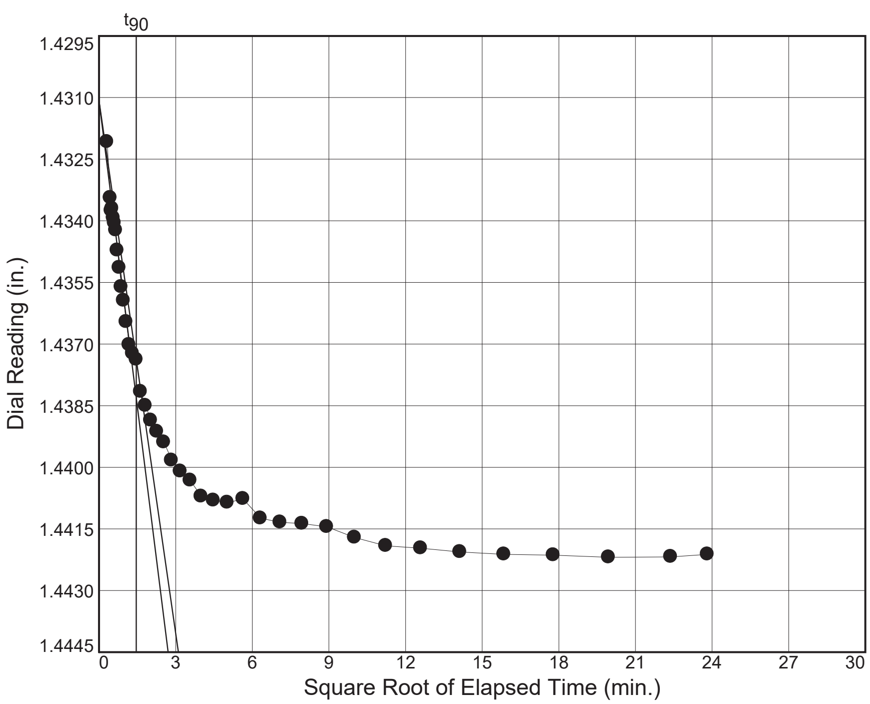
D_{90} = 1.4294

D_{100} = 1.4300

T_{90} = 3.96 min.

C_v @ T_{90}

0.508 ft.^{2}/day



Load No.= 4

Load= 2.00 tsf

D_0 = 1.4311

D_{90} = 1.4374

D_{100} = 1.4381

T_{90} = 2.12 min.

C_v @ T_{90}

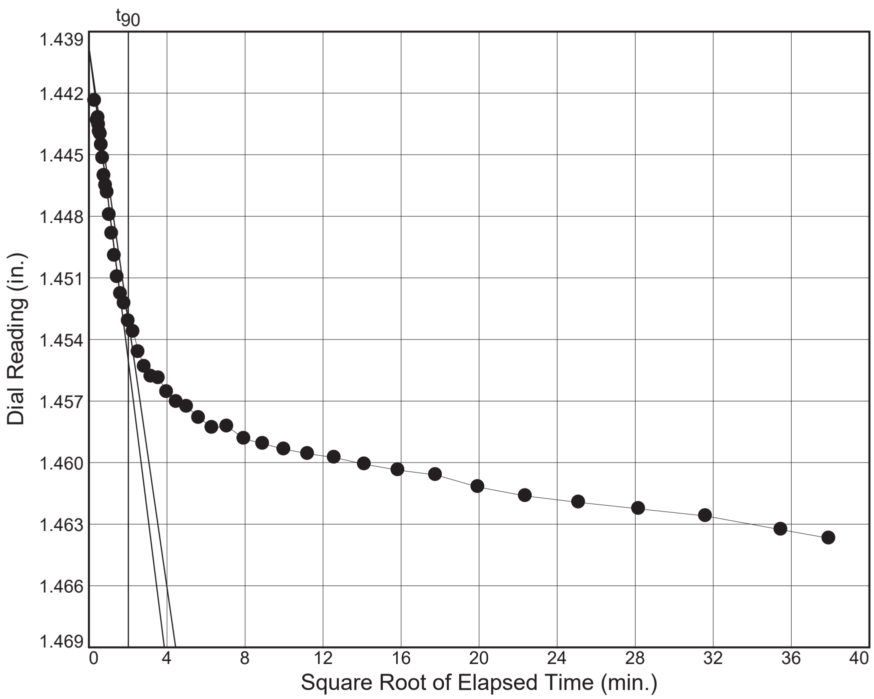
0.933 ft.^{2}/day

Exhibit 1515

Dial Reading vs. Time

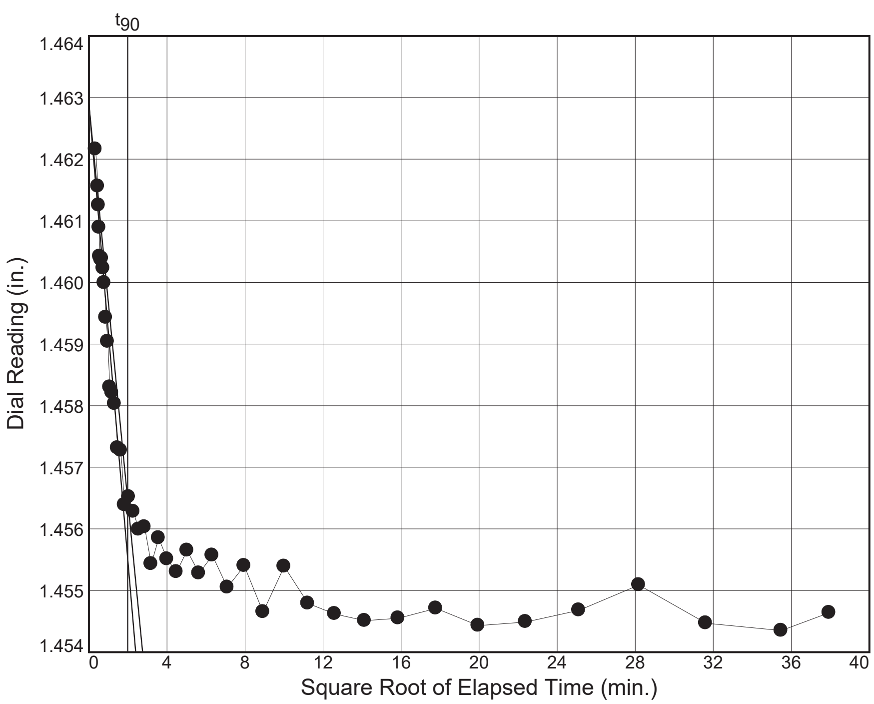
Project No.: N1185278
Project: ERNSTBRIDGE ROAD BRIDGE REPLACEMENT

Source of Sample: B-18-1 Depth: 45-47'



Load No.= 5
Load= 4.00 tsf
D_0 = 1.4398
D_{90} = 1.4530
D_{100} = 1.4545
T_{90} = 4.07 min.

C_v @ T_{90}
0.471 ft.^{2}/day



Load No.= 6
Load= 1.00 tsf
D_0 = 1.4629
D_{90} = 1.4565
D_{100} = 1.4558
T_{90} = 3.93 min.

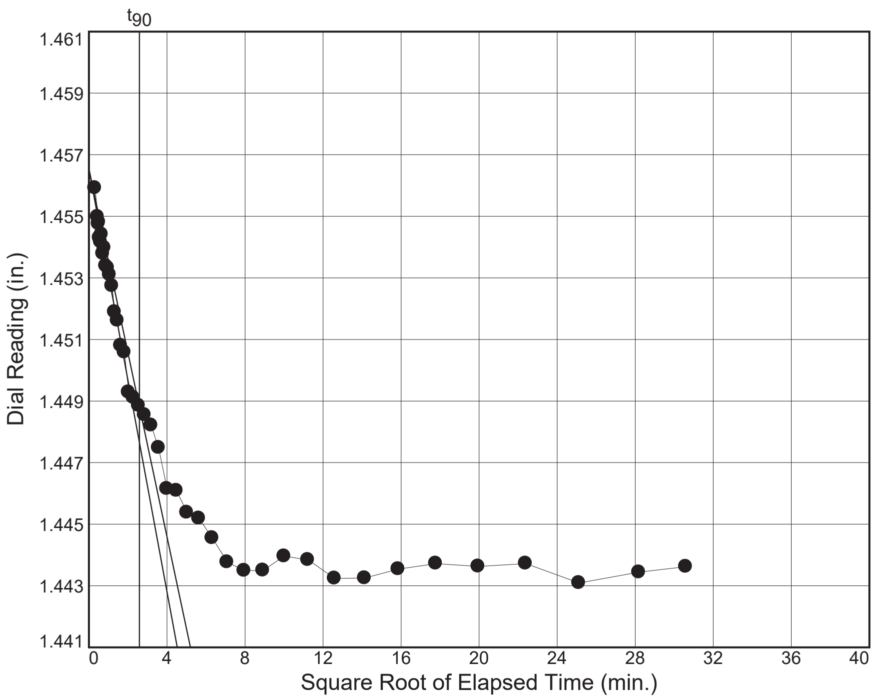
C_v @ T_{90}
0.478 ft.^{2}/day

Exhibit 1516

Dial Reading vs. Time

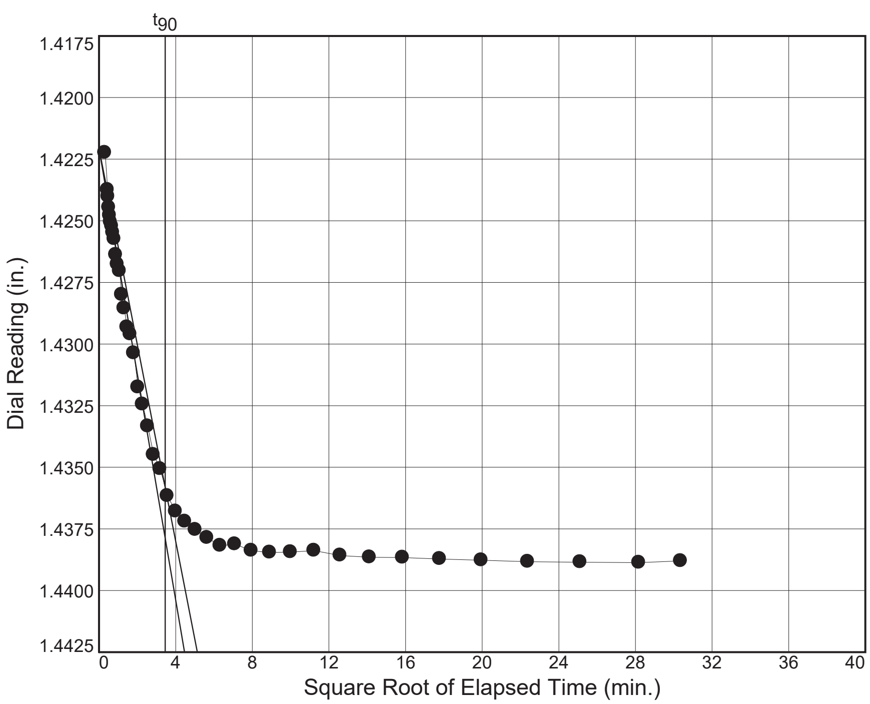
Project No.: N1185278
Project: ERNSTBRIDGE ROAD BRIDGE REPLACEMENT

Source of Sample: B-18-1 Depth: 45-47'



Load No.= 7
Load= 0.25 tsf
D_0 = 1.4565
D_{90} = 1.4488
D_{100} = 1.4479
T_{90} = 6.70 min.

C_v @ T_{90}
0.286 ft.^{2}/day



Load No.= 8
Load= 0.50 tsf
D_0 = 1.4221
D_{90} = 1.4358
D_{100} = 1.4373
T_{90} = 11.90 min.

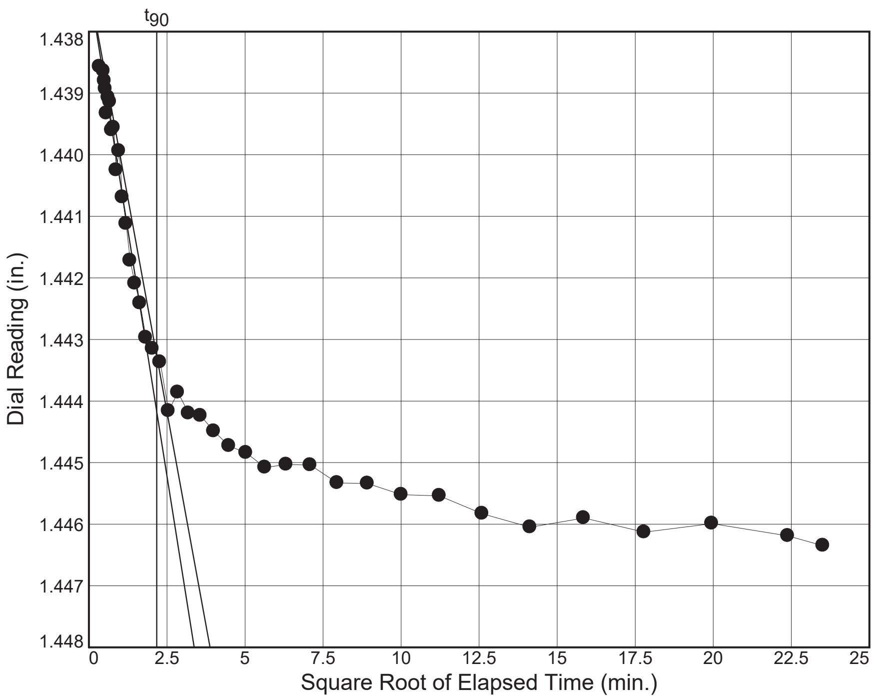
C_v @ T_{90}
0.168 ft.^{2}/day

Exhibit 1517

Dial Reading vs. Time

Project No.: N1185278
Project: ERNSTBRIDGE ROAD BRIDGE REPLACEMENT

Source of Sample: B-18-1 Depth: 45-47'



Load No.= 9

Load= 1.00 tsf

D_0 = 1.4372

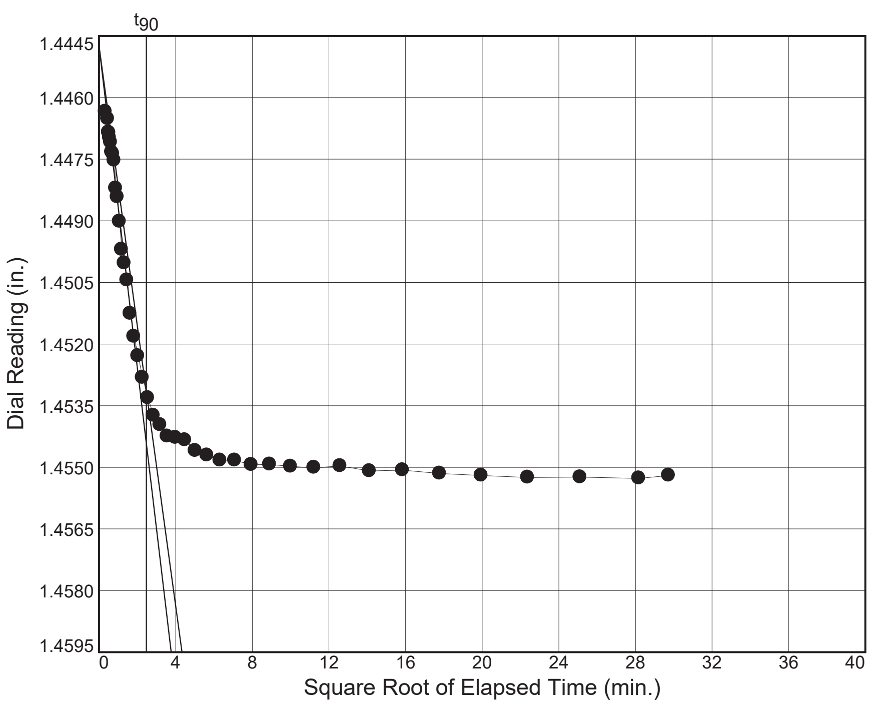
D_{90} = 1.4433

D_{100} = 1.4439

T_{90} = 4.72 min.

C_v @ T_{90}

0.414 ft.^{2}/day



Load No.= 10

Load= 2.00 tsf

D_0 = 1.4447

D_{90} = 1.4532

D_{100} = 1.4541

T_{90} = 6.10 min.

C_v @ T_{90}

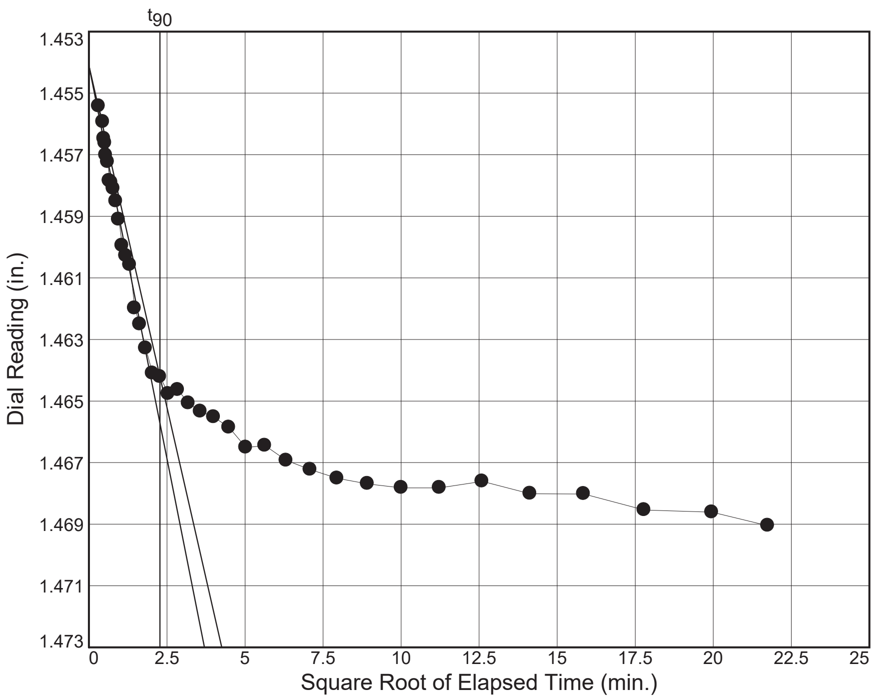
0.315 ft.^{2}/day

Exhibit 1518

Dial Reading vs. Time

Project No.: N1185278
Project: ERNSTBRIDGE ROAD BRIDGE REPLACEMENT

Source of Sample: B-18-1 Depth: 45-47'



Load No.= 11

Load= 4.00 tsf

D_0 = 1.4541

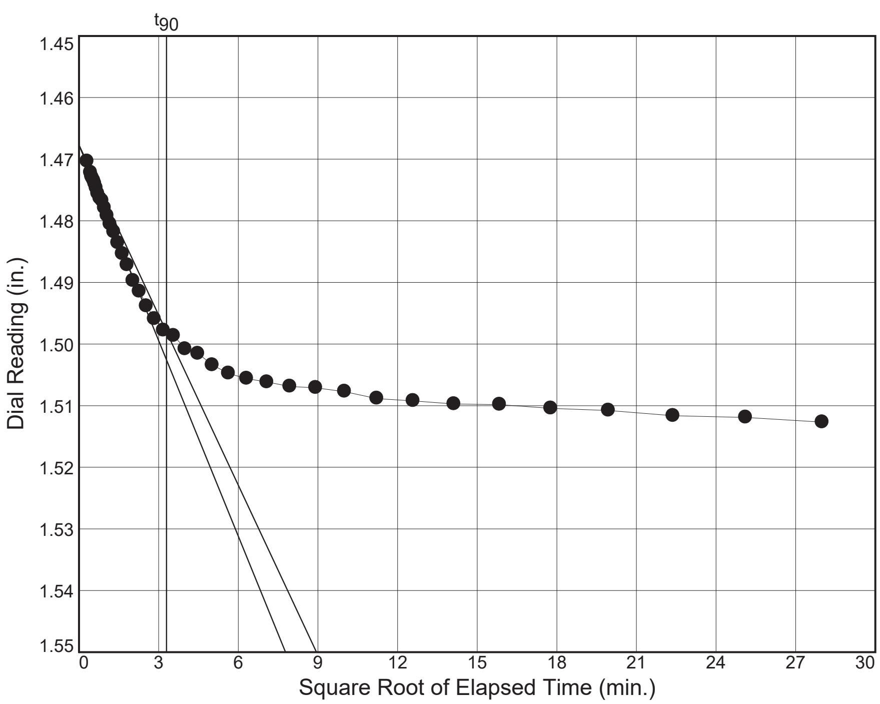
D_{90} = 1.4642

D_{100} = 1.4653

T_{90} = 5.17 min.

C_v @ T_{90}

0.363 ft.^{2}/day



Load No.= 12

Load= 8.00 tsf

D_0 = 1.4677

D_{90} = 1.4980

D_{100} = 1.5014

T_{90} = 10.87 min.

C_v @ T_{90}

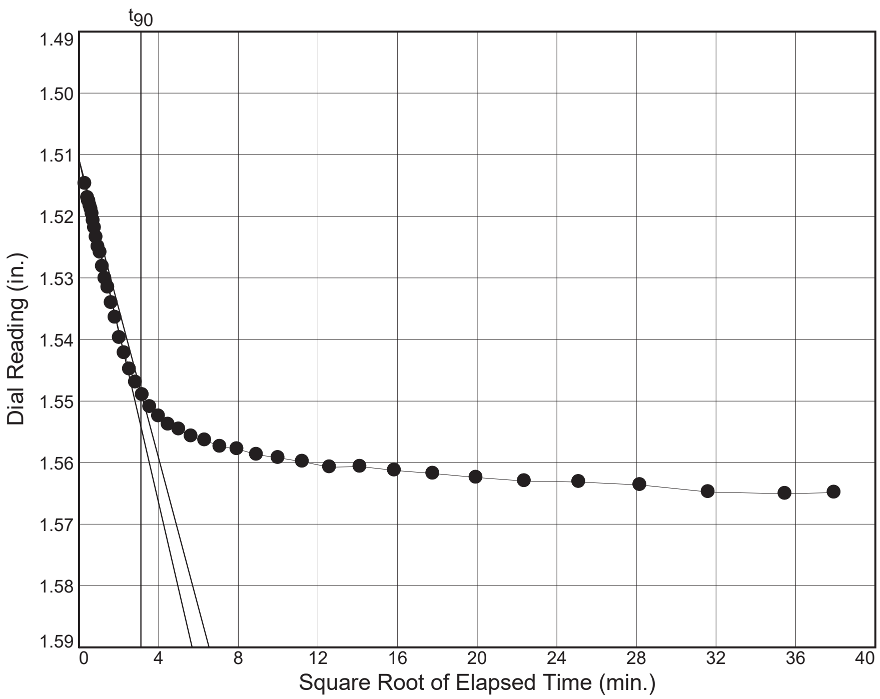
0.163 ft.^{2}/day

Exhibit 1519

Dial Reading vs. Time

Project No.: N1185278
Project: ERNSTBRIDGE ROAD BRIDGE REPLACEMENT

Source of Sample: B-18-1 Depth: 45-47'



Load No.= 13

Load= 16.00 tsf

D_0 = 1.5108

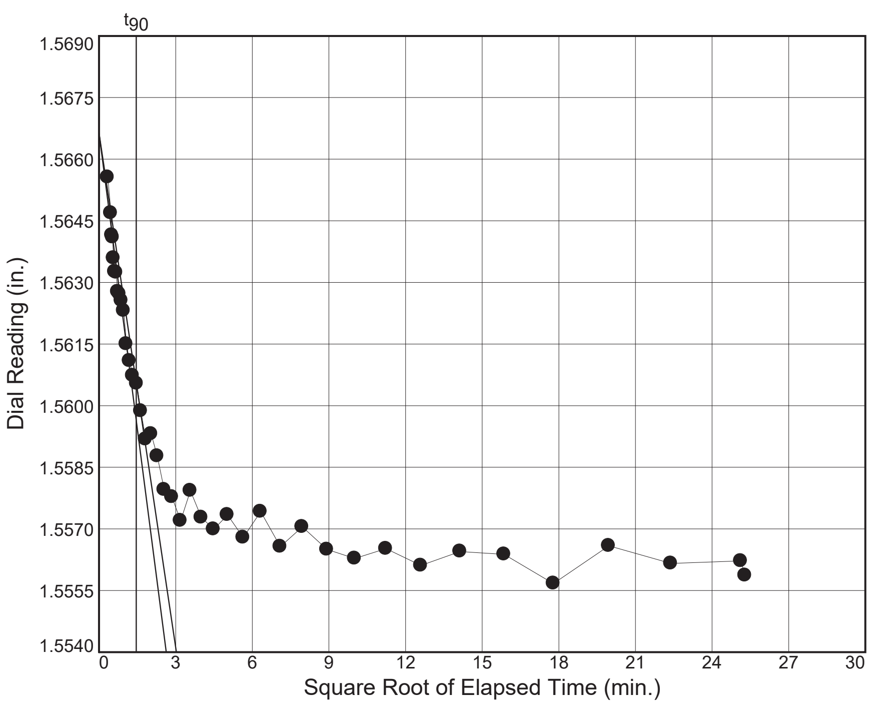
D_{90} = 1.5485

D_{100} = 1.5527

T_{90} = 9.67 min.

C_v @ T_{90}

0.165 ft.^{2}/day



Load No.= 14

Load= 4.00 tsf

D_0 = 1.5666

D_{90} = 1.5606

D_{100} = 1.5599

T_{90} = 2.12 min.

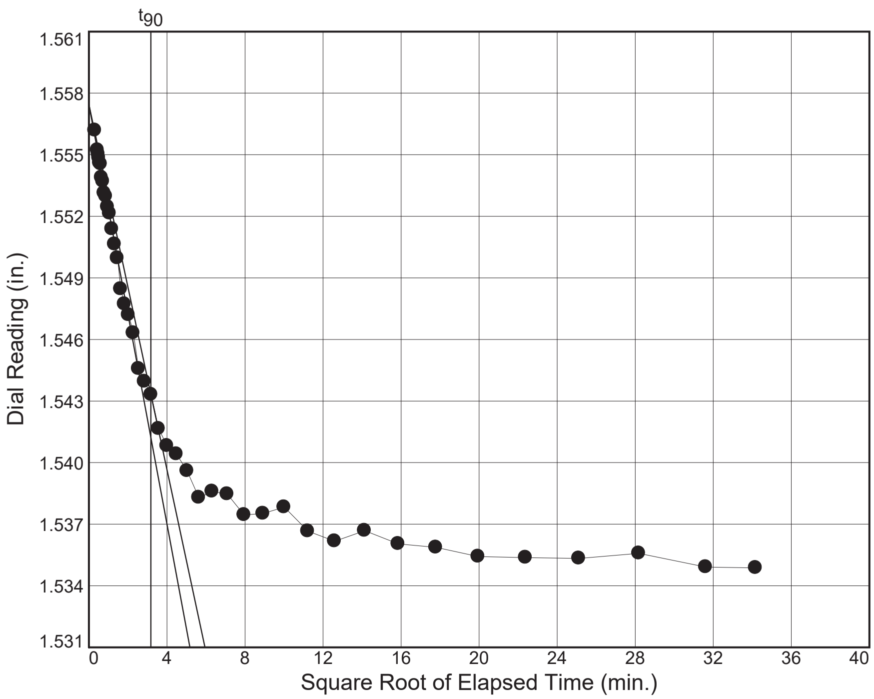
C_v @ T_{90}

0.701 ft.^{2}/day

Dial Reading vs. Time

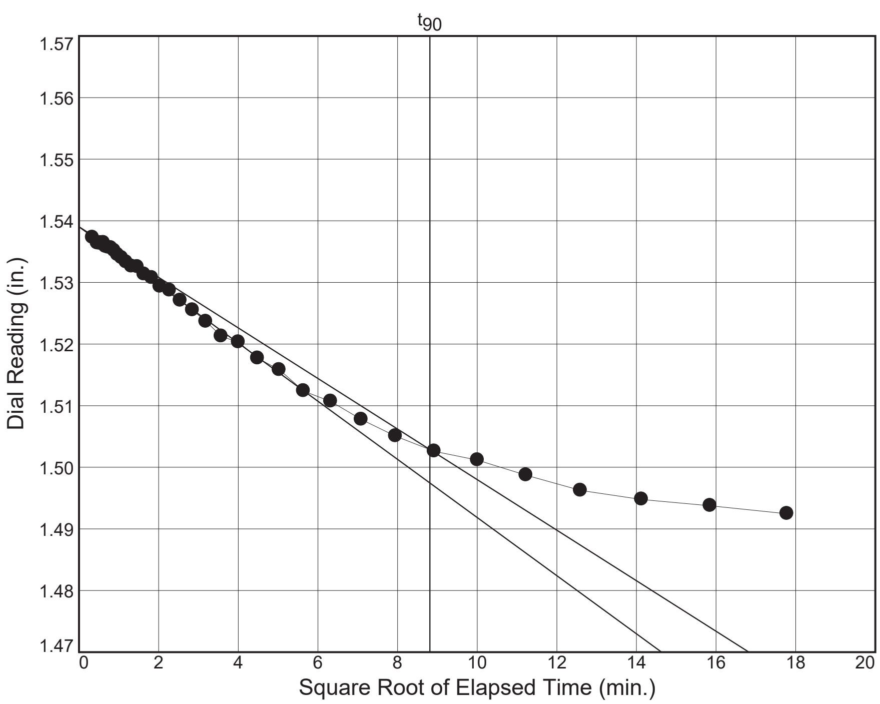
Project No.: N1185278
Project: ERNSTBRIDGE ROAD BRIDGE REPLACEMENT

Source of Sample: B-18-1 Depth: 45-47'



Load No.= 15
Load= 1.00 tsf
D_0 = 1.5574
D_{90} = 1.5433
D_{100} = 1.5418
T_{90} = 10.06 min.

C_v @ T_{90}
0.154 ft.^{2}/day



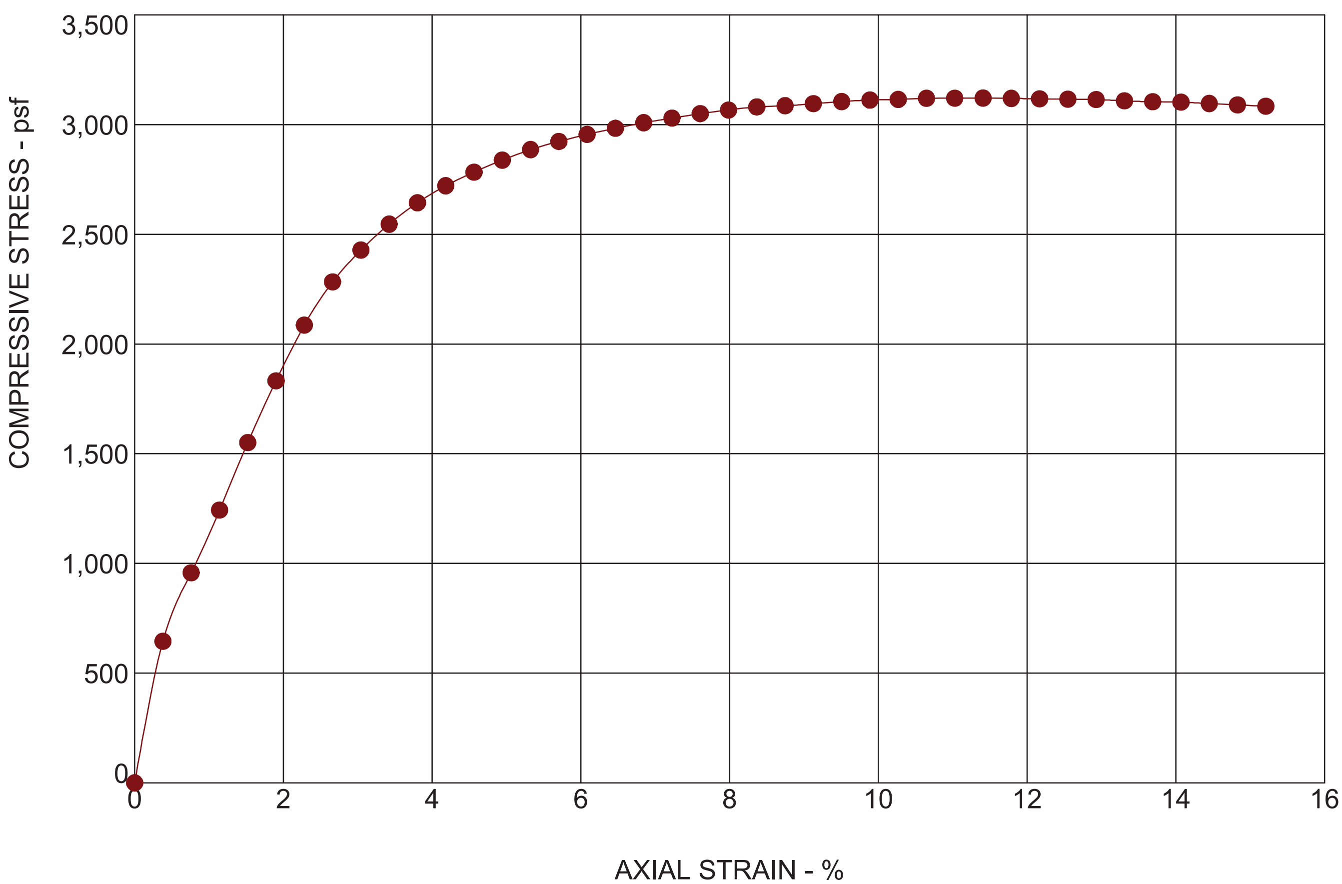
Load No.= 16
Load= 0.25 tsf
D_0 = 1.5390
D_{90} = 1.5029
D_{100} = 1.4989
T_{90} = 77.64 min.

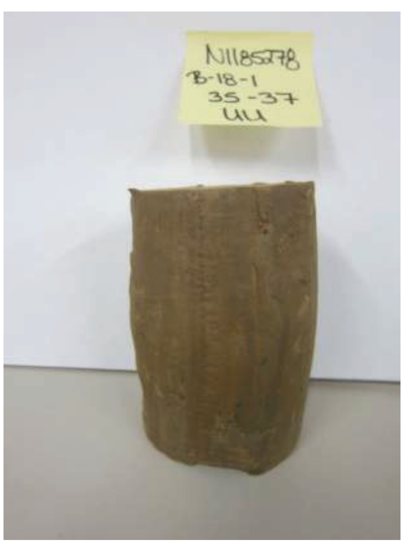
C_v @ T_{90}
0.021 ft.^{2}/day

Exhibit 1521

UNCONSOLIDATED-UNDRAINED TEST

ASTM D2850



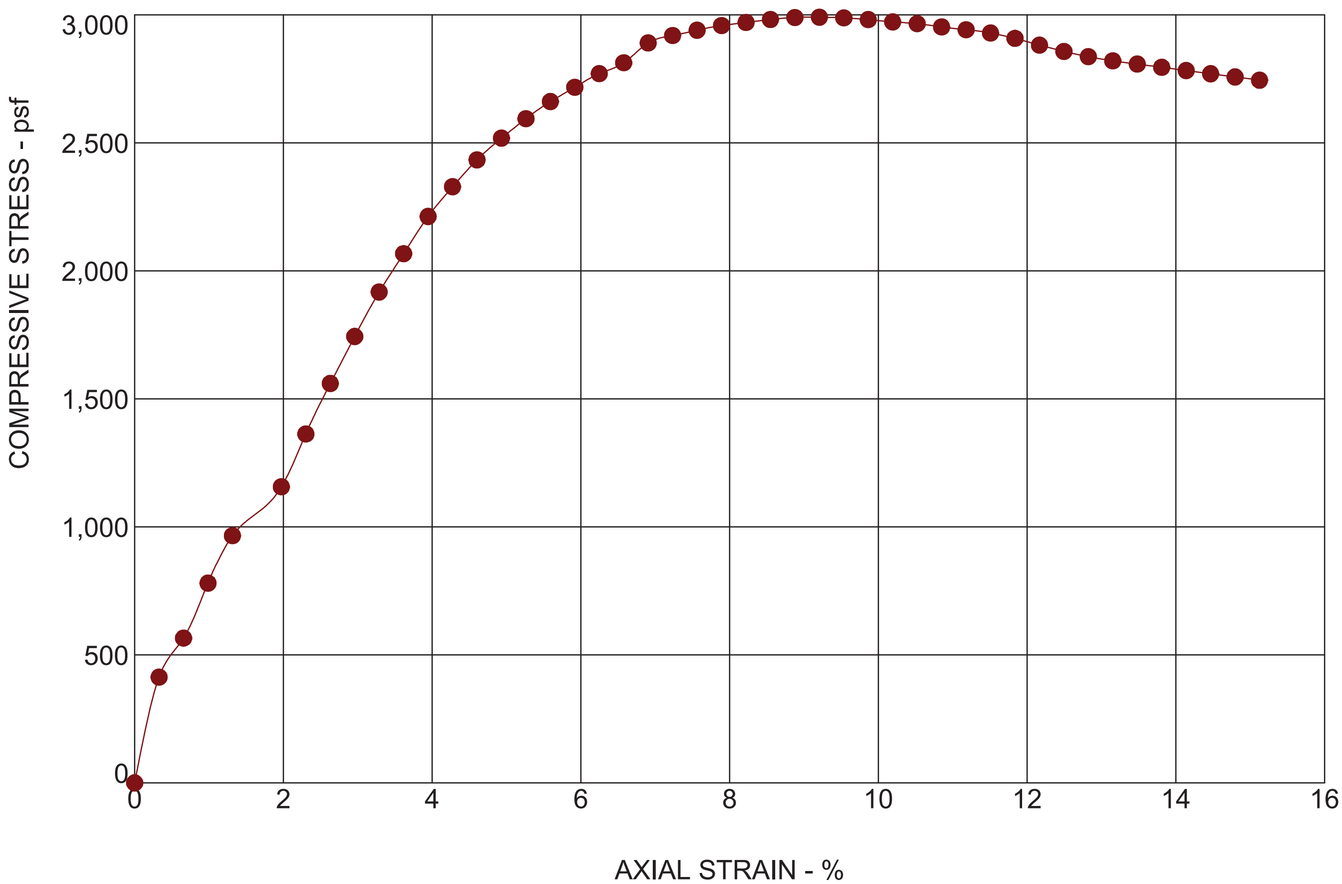
SPECIMEN FAILURE MODE	SPECIMEN TEST DATA		
	Moisture Content:	%	28.5
	Dry Density:	pcf	98.0
	Diameter:	in.	2.86
	Height:	in.	5.26
	Height / Diameter Ratio:		1.84
	Calculated Saturation:	%	100.0
	Calculated Void Ratio:		0.73
	Assumed Specific Gravity:		2.712
	Failure Strain:	%	11.41
	Compressive Strength	psf	3120.86
	Undrained Shear Strength:	psf	1560.43
	Strain Rate:	in/min	0.0526
	Cell Pressure:	psi	35.0
	Remarks: 1510		

SAMPLE TYPE: Shelby Tube	SAMPLE LOCATION: B-18-1 @ 35 - 37 feet			
SAMPLE DESCRIPTION: LEAN CLAY(CL) A-4(6)	LL	PL	PI	Percent < #200 Sieve
	30	22	8	88

PROJECT: Ernstbridge Road Bridge Replacement	Terracon 611 Lunken Park Dr Cincinnati, OH	PROJECT NUMBER: N1185278
SITE: Ernstbridge Road Ryland Heights, KY		CLIENT: WSP USA Inc. Cincinnati, OH

UNCONSOLIDATED-UNDRAINED TEST

ASTM D2850



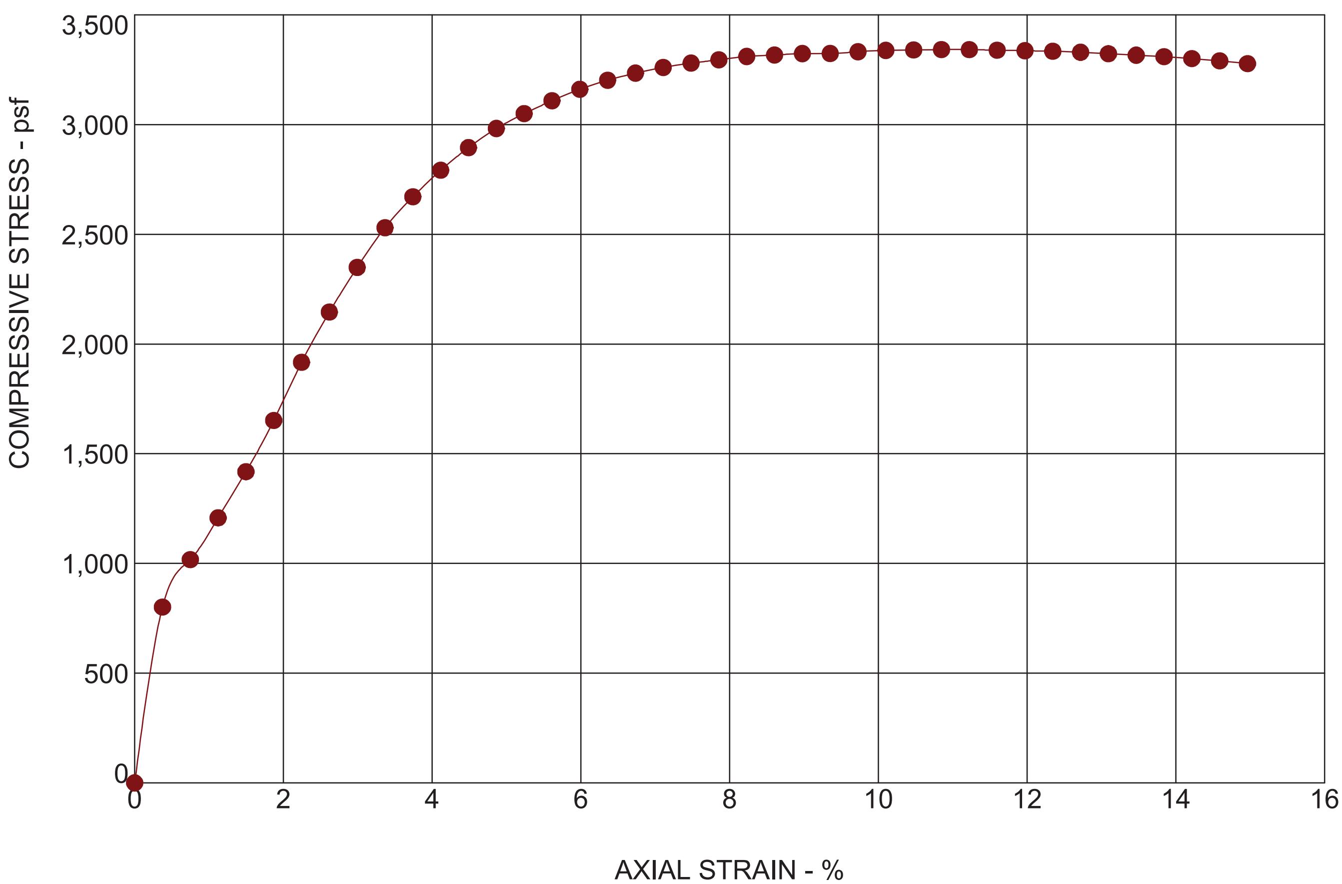
SPECIMEN FAILURE MODE	SPECIMEN TEST DATA		
	Moisture Content:	%	25.8
	Dry Density:	pcf	96.5
	Diameter:	in.	2.87
	Height:	in.	6.08
	Height / Diameter Ratio:		2.12
	Calculated Saturation:	%	91.94
	Calculated Void Ratio:		0.77
	Assumed Specific Gravity:		2.735
	Failure Strain:	%	9.21
	Compressive Strength	psf	2991.02
	Undrained Shear Strength:	psf	1495.51
	Strain Rate:	in/min	0.0608
	Cell Pressure:	psi	45.0
	Remarks: 1513		

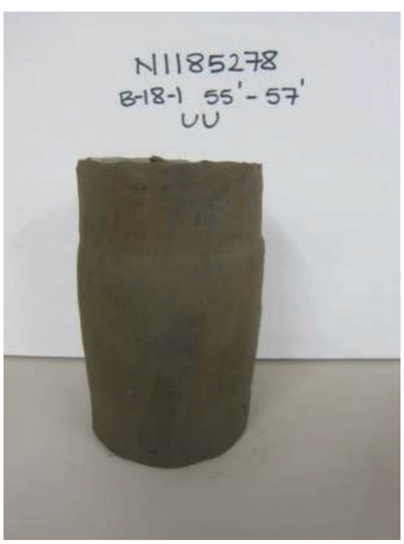
SAMPLE TYPE: Shelby Tube	SAMPLE LOCATION: B-18-1 @ 45 - 47 feet				
SAMPLE DESCRIPTION: LEAN CLAY(CL) A-6(10)	LL	PL	PI	Percent < #200 Sieve	
	32	21	11	94	

PROJECT: Ernstbridge Road Bridge Replacement	Terracon 611 Lunken Park Dr Cincinnati, OH	PROJECT NUMBER: N1185278
SITE: Ernstbridge Road Ryland Heights, KY		CLIENT: WSP USA Inc. Cincinnati, OH

UNCONSOLIDATED-UNDRAINED TEST

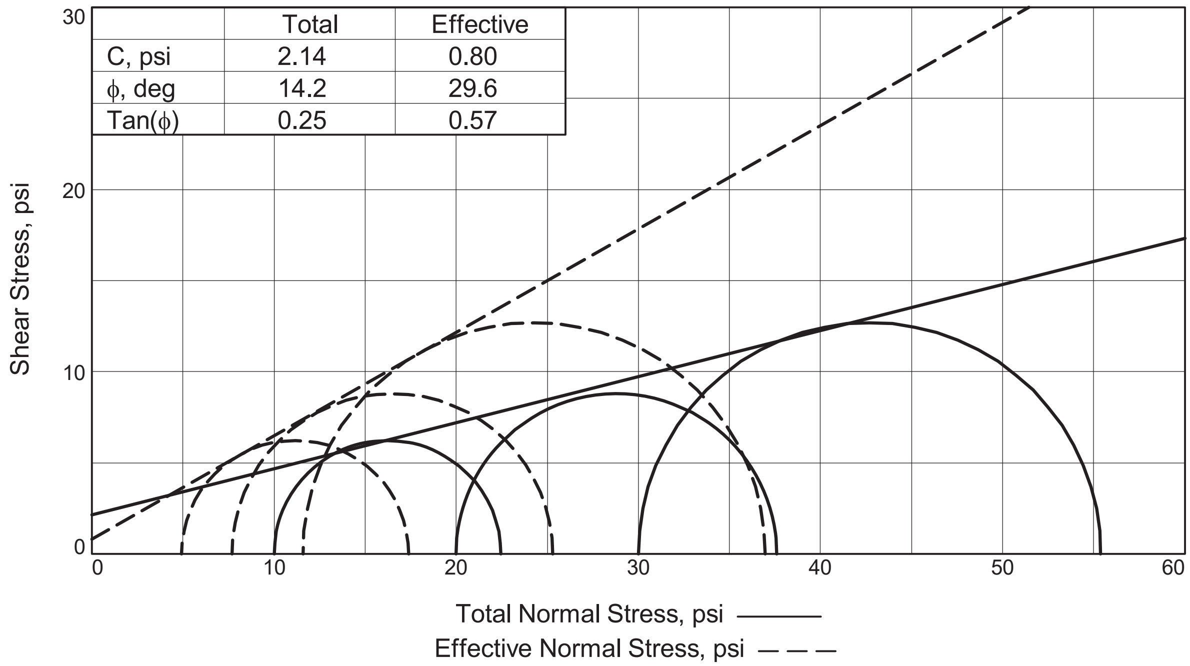
ASTM D2850



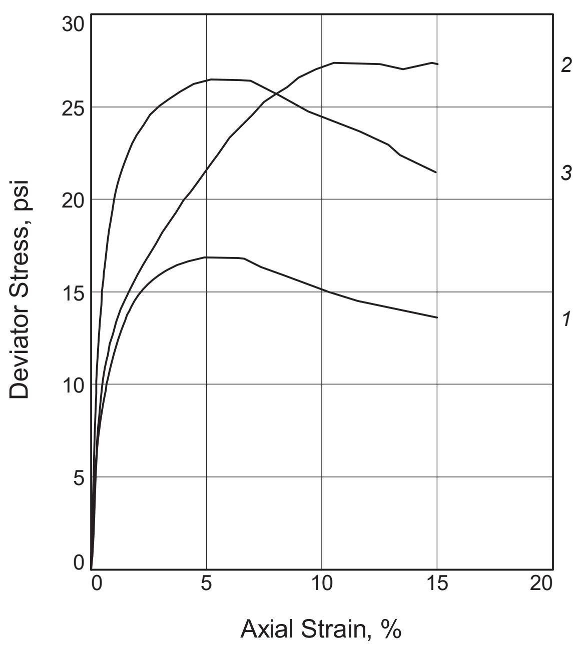
SPECIMEN FAILURE MODE	SPECIMEN TEST DATA		
	Moisture Content:	%	26.9
	Dry Density:	pcf	96.8
	Diameter:	in.	2.88
	Height:	in.	5.35
	Height / Diameter Ratio:		1.86
	Calculated Saturation:	%	98.70
	Calculated Void Ratio:		0.73
	Assumed Specific Gravity:		2.681
	Failure Strain:	%	10.85
	Compressive Strength	psf	3341.84
	Undrained Shear Strength:	psf	1670.92
	Strain Rate:	in/min	0.0535
	Cell Pressure:	psi	55.0
	Remarks: 1516		

SAMPLE TYPE: Shelby Tube				SAMPLE LOCATION: B-18-1 @ 55 - 57 feet
SAMPLE DESCRIPTION: LEAN CLAY A-6(15)	LL	PL	PI	Percent < #200 Sieve
	36	21	15	

PROJECT: Ernstbridge Road Bridge Replacement	611 Lunken Park Dr Cincinnati, OH	PROJECT NUMBER: N1185278
SITE: Ernstbridge Road Ryland Heights, KY		CLIENT: WSP USA Inc. Cincinnati, OH



	Total	Effective
C, psi	2.14	0.80
ϕ, deg	14.2	29.6
Tan(ϕ)	0.25	0.57



Sample No.		1	2	3
Initial	Water Content, %	29.5	29.1	29.9
	Dry Density, pcf	94.0	98.2	95.1
	Saturation, %	96.3	104.6	99.9
	Void Ratio	0.8585	0.7780	0.8369
	Diameter, in.	2.840	2.830	2.850
	Height, in.	5.630	5.550	5.630
At Test	Water Content, %	29.1	26.0	27.8
	Dry Density, pcf	95.9	101.2	98.1
	Saturation, %	99.0	100.0	99.7
	Void Ratio	0.8210	0.7265	0.7798
	Diameter, in.	2.818	2.802	2.824
	Height, in.	5.602	5.497	5.557
Strain rate, in./min.		0.001	0.001	0.001
Back Pressure, psi		50.0	50.0	50.0
Cell Pressure, psi		60.0	70.0	80.0
Fail. Stress, psi		12.5	17.6	25.4
Excess Pore Pr., psi		5.1	12.3	18.4
Ult. Stress, psi				
Excess Pore Pr., psi				
$\bar{\sigma}_1$ Failure, psi		17.4	25.3	37.0
$\bar{\sigma}_3$ Failure, psi		4.9	7.7	11.6

Type of Test:
CU with Pore Pressures

Sample Type: Tube

Description: Lean Clay with Sand A-6(10)

LL= 33 **PL=** 19 **PI=** 14

Specific Gravity= 2.798

Remarks: Three Specimen Series

Client: WSP USA Inc.

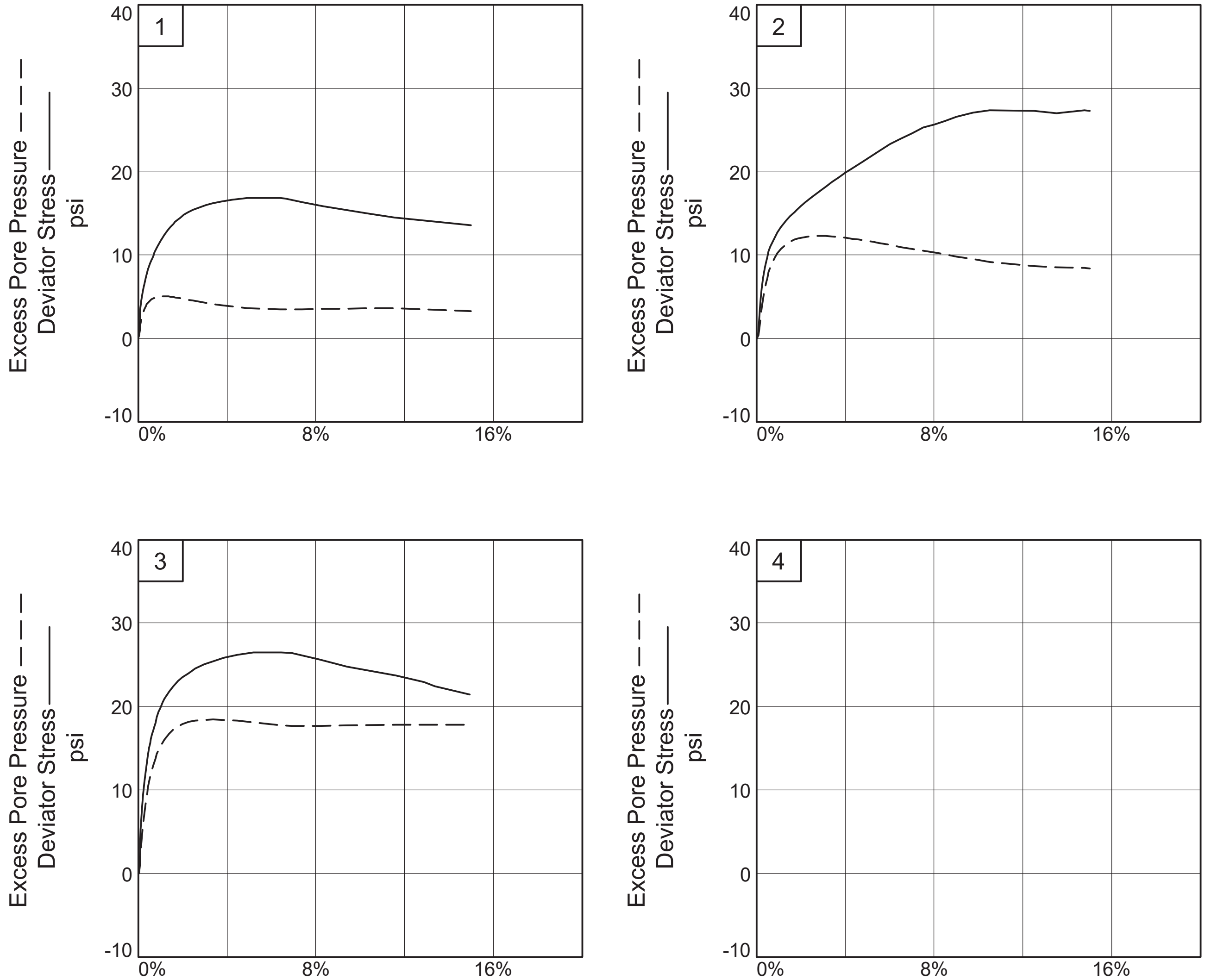
Project: Ernstbridge Road Bridge Replacement

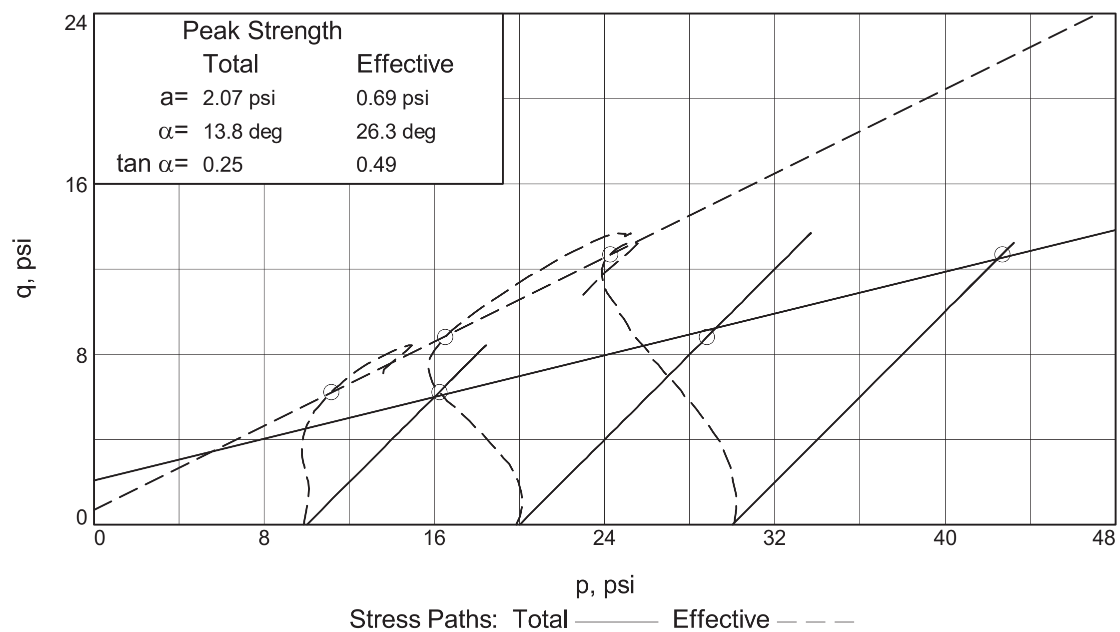
Source of Sample: B-18-1 **Depth:** 25.0-27.0 ft

Sample Number:

Proj. No.: N1185278 **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT
Terracon Consultants, Inc.
Chattanooga, TN





Client: WSP USA Inc.

Project: Ernstbridge Road Bridge

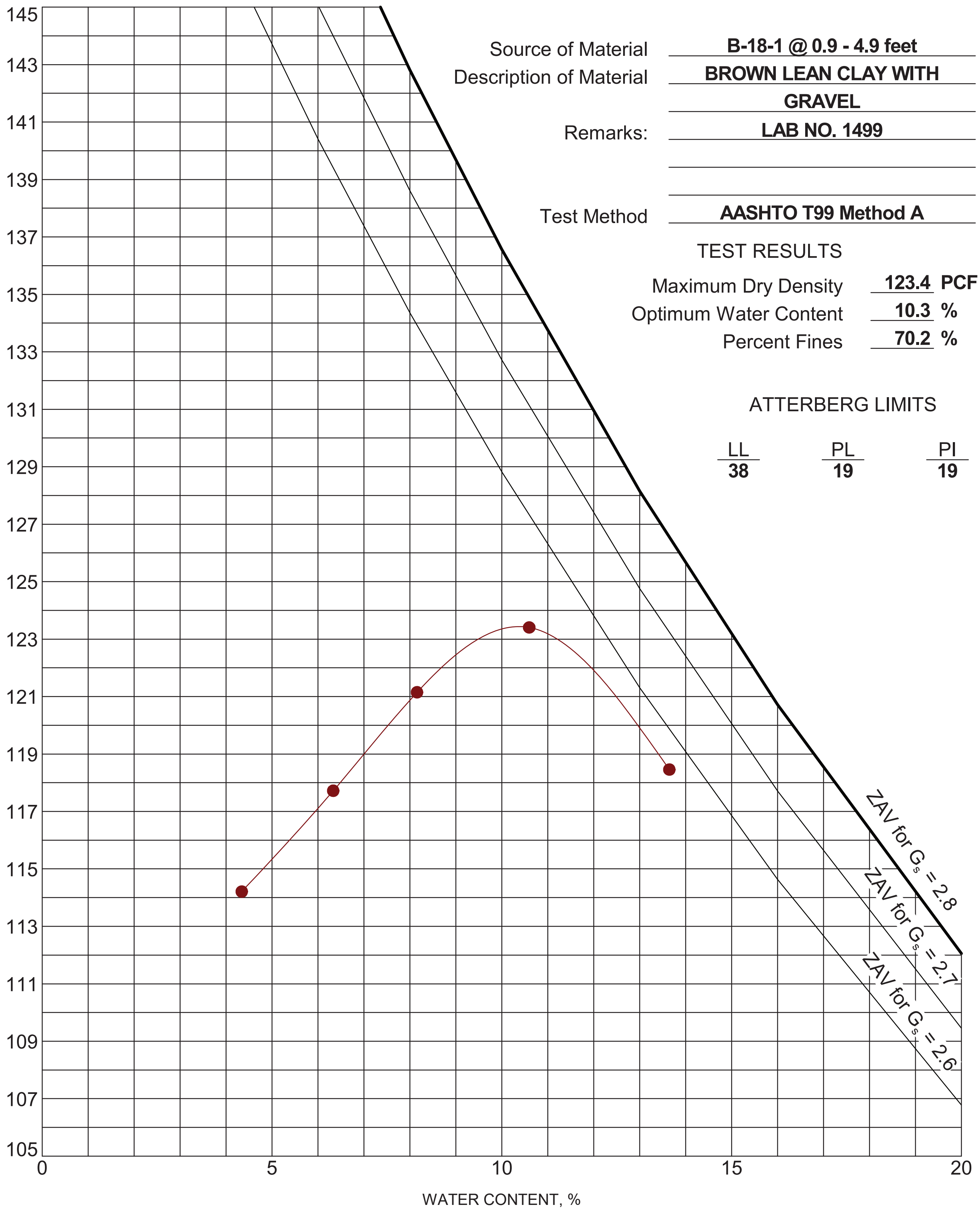
Source of Sample: B-18-1 **Depth:** 25.0-27.0 ft **Sample Number:**

Project No.: N1185278

Terracon Consultants, Inc.

MOISTURE-DENSITY RELATIONSHIP

AASHTO T-99



Source of Material	**B-18-1 @ 0.9 - 4.9 feet**
Description of Material	**BROWN LEAN CLAY WITH GRAVEL**
Remarks:	**LAB NO. 1499**
Test Method	**AASHTO T99 Method A**

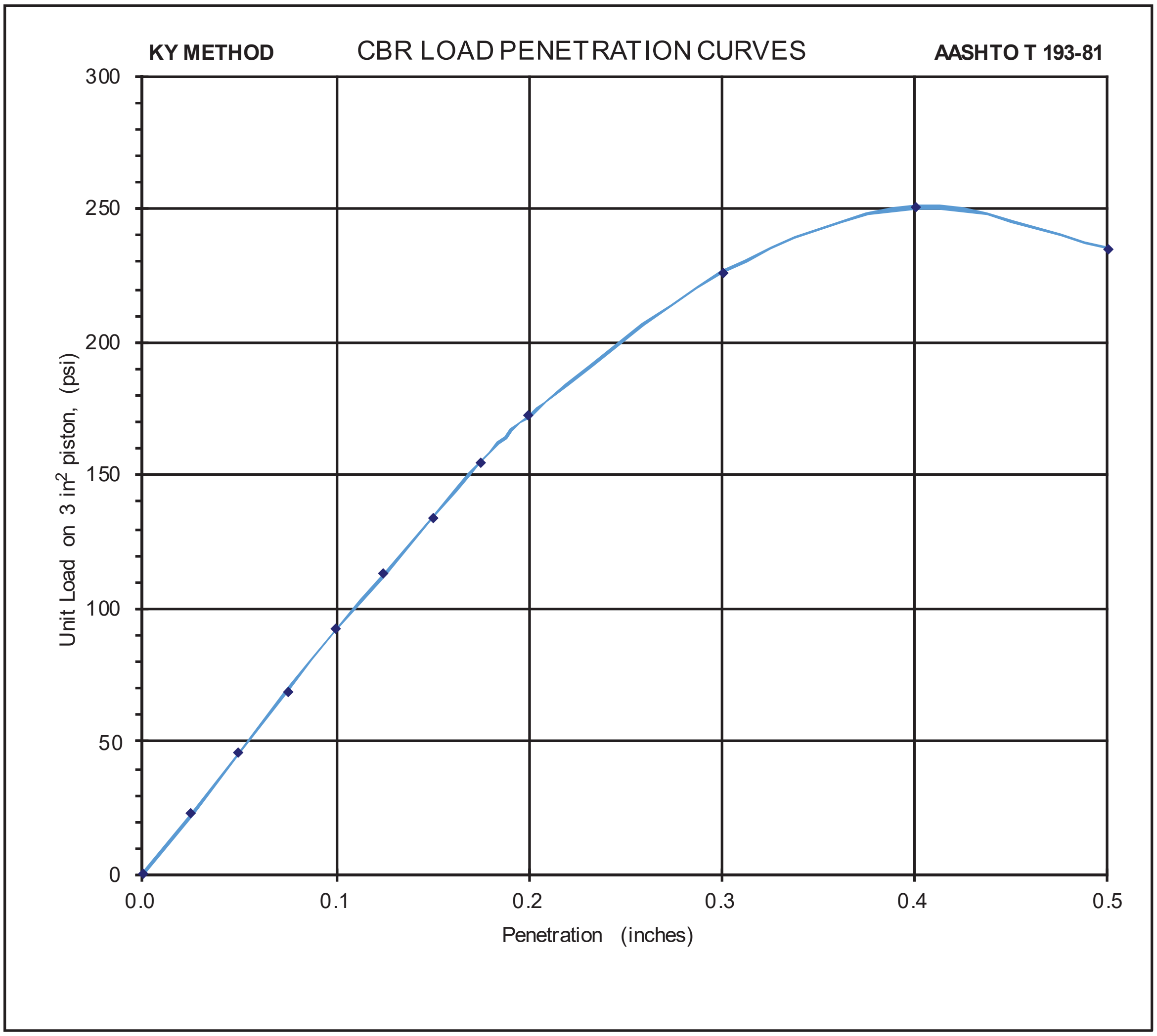
TEST RESULTS

Maximum Dry Density	**123.4**	**PCF**
Optimum Water Content	**10.3**	**%**
Percent Fines	**70.2**	**%**

ATTERBERG LIMITS

LL	PL	PI
38	**19**	**19**

PROJECT: Ernstbridge Road Bridge Replacement	Terracon 611 Lunken Park Dr Cincinnati, OH	PROJECT NUMBER: N1185278
SITE: Ernstbridge Road Ryland Heights, KY		CLIENT: WSP USA Inc. Cincinnati, OH



TERRACON

Client: WSP USA Inc.
W.O.# N1185278
Boring: B-18-1 **Depth:** 0.9-4.9'
Sample Bag - 1
Compaction:
Date: 3-22-19

Project: Ernstbridge Road Bridge Replacement
Description: Brown Lean Clay w/sand A-6(12)
Lab Number: 1499

Sample	Blows	Before Soaking					After Soak	% Swell	CBR @	CBR @
		Wet Unit Weight		% Moisture	Dry Unit Weight		% Moisture		0.1 inches	0.2 inches
		(pcf)	(kN/m^3)	Water Content	(pcf)	(kN/m^3)	Top 1 inch			
Bag - 1		135.1	21.2	10.0%	122.8	19.3	11.6%	0.1	9.2	11.5

Surcharge Weight: 17.5

SUPPORTING INFORMATION

Contents:

Note: All attachments are one page unless noted above.

AASHTO 2007 (LRFD)
Ernstbridge Road Bridge Replacement

PROJECT IDENTIFICATION

Title:	Ernstbridge Road Bridge Replacement
Project Number:	N1185278
Client:	WSP
Designer:	JDD
Station Number:	East Abutment

Description:

H=23 feet. 2H:1V toe Hs=2'. Abutment 5 feet back and 7 feet tall. Cu values based on CPT data. L=0.75H

Company's information:

Name:
Street:

,
Telephone #:
Fax #:
E-Mail:

Original file path and name: N:\Projects\2018\N1185278\Working Files\Calculations-An..... .....Abutmrnt CPT L75.BEN

Original date and time of creating this file: Sun Mar 31 16:0624 2019

PROGRAM MODE: ANALYSIS
of a BRIDGE ABUTMENT
using METAL STRIPS as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	120.0 lb/ft ³
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	125.0 lb/ft ³
Design value of internal angle of friction, ϕ	28.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	124.0 lb/ft ³
Equivalent internal angle of friction, $\phi_{equiv.}$	0.0 °
Equivalent cohesion, $c_{equiv.}$	2000.0 lb/ft ²

Water table is at wall base elevation

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Ka (external stability) = 0.3610 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 4.19 N γ= 0.00

SEISMICITY

Not Applicable

INPUT DATA: Metal strips (Analysis)

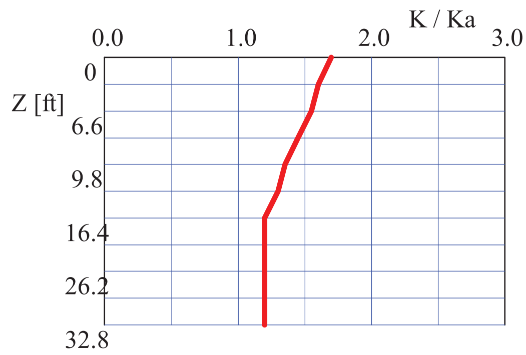
D A T A	Metal strip type #1	Metal strip type #2	Metal strip type #3	Metal strip type #4	Metal strip type #5
Yield strength of steel, Fy [kips/in ²]	65.3	N/A	N/A	N/A	N/A
Gross width of strip, b [in]	2.0	N/A	N/A	N/A	N/A
Vertical spacing, Sv [ft]	Varies	N/A	N/A	N/A	N/A
Design cross section area, Ac [in ²]	0.16	N/A	N/A	N/A	N/A

Ribbed steel strips.
Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0

Friction angle along reinforcement-soil interface,	ρ				
@ the top	60.97	N/A	N/A	N/A	N/A
@ 19.7 ft or below	32.00	N/A	N/A	N/A	N/A
Pullout resistance factor, F*					
@ the top	1.80	N/A	N/A	N/A	N/A
@ 19.7 ft or below	0.62	N/A	N/A	N/A	N/A
Scale-effect correction factor, α	1.00	N/A	N/A	N/A	N/A

Variation of Lateral Earth Pressure Coefficient With Depth

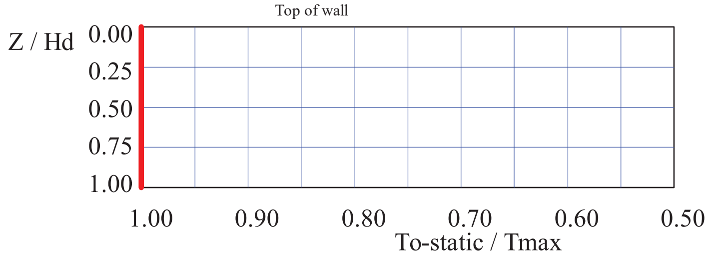
Z	K / Ka
0 ft	1.70
3.3 ft	1.60
6.6 ft	1.55
9.8 ft	1.45
13.1 ft	1.35
16.4 ft	1.30
19.7 ft	1.20



INPUT DATA: Facia and Connection (Analysis)

FACIA type: Segmental precast concrete panels.
Depth of panel is 1.31 ft. Horizontal distance to Center of Gravity of panel is 0.66 ft.
Average unit weight of panel is $\gamma_f = 152.78$ lb/ft ^{3}

Z / Hd	To-static / Tmax
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00



D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	---	N/A	N/A	N/A	N/A
Strength reduction at the connection, CRu = Fyc / Fy	0.90	N/A	N/A	N/A	N/A

INPUT DATA: Geometry and Surcharge loads (of a BRIDGE ABUTMENT)

Design height, Hd	23.00	[ft]	{ Embedded depth is E = 3.00 ft, and height above top of finished bottom grade is H = 20.00 ft }
Batter, ω	0.0	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

U N I F O R M   S U R C H A R G E
Uniformly distributed dead load is 0.0 [lb/ft ²]

ABUTMENT GEOMETRY
Abutment's width, bf = 3.00 at distance from back of wall, cf = 5.00 [ft].
Footing's dimension: height, h' = 7.00, width, b = 3.00, and thickness, t = 1.00 [ft].
Dimensions of bridge bearing plate: height, fh = 0.33, width, fw = 1.64 [ft].

OTHER EXTERNAL LOAD(S)
[S] Vertical Dead Load, Pv-d = 0.0 and Vertical Live Load, Pv-l = 0.0 [lb/ft]. (Total of 0.0 [lb/ft])
The distance from back of the wall is 4.2 [ft].

ANALYZED REINFORCEMENT LAYOUT:

SCALE:

0 2 4 6 8 10[ft]

AASHTO 2007 (LRFD) Input Data

INTERNAL STABILITY

Load factor for vertical earth pressure, EV, from Table 3.4.1-2:	$\gamma_{p\text{-}EV}$	1.35	
Load factor for earthquake loads, EQ, from Table 3.4.1-1:	$\gamma_{p\text{-}EQ}$	1.00	
Load factor for live load surchrge, LS, from Figure C11.5.5-3(b): (Same as in External Stability).	$\gamma_{p\text{-}LS}$	1.75	
Load factor for dead load surchrge, ES: (Same as in External Stability).	$\gamma_{p\text{-}ES}$	1.50	
Resistance factor for reinforcement tension from Table 11.5.6-1:	ϕ	Static	Combined static/seismic
Metal Strips:		0.75	1.00
Resistance factor for reinforcement tension in connectors from Table 11.5.6-1:	ϕ	Static	Combined static/seismic
Metal Strips:		0.75	1.00
Resistance factor for reinforcement pullout from Table 11.5.6-1:	ϕ	0.90	1.20

EXTERNAL STABILITY

		Static		Combined Static/Seismic
Load factor for vertical earth pressure, EV, from Table 3.4.1-2 and Figure C11.5.5-2:				
Sliding and Eccentricity	$\gamma_{p\text{-}EV}$	1.00	$\gamma_{p\text{-}EQ}$	1.00
Bearing Capacity	$\gamma_{p\text{-}EV}$	1.35	$\gamma_{p\text{-}EQ}$	1.35

Load factor of active lateral earth pressure, EH, from Table 3.4.1-2 and Figure C11.5.5-2: $\gamma_{p\text{-}EH}$ 1.50
Load factor of active lateral earth pressure during earthquake (does not multiply P_{AE} and P_{IR}): $(\gamma_{p\text{-}EH})_{EQ}$ 1.50
Load factor for earthquake loads, EQ, from Table 3.4.1-1 (multiplies P_{AE} and P_{IR}): $\gamma_{p\text{-}EQ}$ 1.00

		Static	Combined Static/Seismic
Resistance factor for shear resistance along common interfaces from Table 10.5.5.2.2-1:			
Reinforced Soil and Foundation	ϕ_τ	1.00	1.00
Reinforced Soil and Reinforcement	ϕ_τ	1.00	1.00
Resistance factor for bearing capacity of shallow foundation from Table 10.5.5.2.2-1:	ϕ_b	0.65	0.65

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, CDR = 1.01, Meyerhof stress = 5400 lb/ft².
Foundation Interface: Direct sliding, CDR = 1.677, Eccentricity, e/L = 0.1271, CDR-overturning = 3.39

#	METAL STRIP Elevation [ft]	Length [ft]	Type #	CONNECTION CDR [pullout resistance]	CDR [connection break]	CDR [metal strip strength]	Metal strip strength CDR	Pullout resistance CDR	Direct sliding CDR	Eccentricity e/L	Product name
1	1.15	22.50	1	N/A	0.76	0.85	0.846	1.424	1.933	0.1142	---
2	3.45	22.50	1	N/A	0.81	0.90	0.905	1.337	2.100	0.0893	---
3	5.75	22.50	1	N/A	0.86	0.95	0.950	1.468	2.307	0.0658	---
4	8.05	22.50	1	N/A	0.91	1.01	1.010	1.563	2.541	0.0434	---
5	10.35	22.50	1	N/A	0.98	1.09	1.089	1.633	2.823	0.0217	---
6	12.65	22.50	1	N/A	1.06	1.18	1.176	1.658	3.164	0.0001	---
7	14.95	22.50	1	N/A	1.13	1.25	1.254	1.574	3.577	-0.0223	---
8	17.25	22.50	1	N/A	1.21	1.34	1.339	1.570	4.068	-0.0475	---
9	19.55	22.50	1	N/A	1.32	1.46	1.464	1.539	4.601	-0.0795	---
10	21.85	22.50	1	N/A	1.46	1.62	1.623	1.446	4.989	-0.1291	---

BEARING CAPACITY for GIVEN LAYOUT

	STATIC	SEISMIC	UNITS
(Water table is at wall base elevation)			
Ultimate bearing capacity, q-ult	5447	N/A	[lb/ft ²]
Meyerhof stress, σ_V	5400.1	N/A	[lb/ft ²]
Eccentricity, e	1.97	N/A	[ft]
Eccentricity, e/L	0.087	N/A	
CDR calculated	1.01	N/A	
Base length	22.50	N/A	[ft]

SCALE:

0 2 4 6 8 10[ft]

DIRECT SLIDING for GIVEN LAYOUT
(for METAL STRIPS reinforcements)

Along reinforced and foundation soils interface: CDR-static = 1.677

#	Metal strip Elevation [ft]	Metal strip Length [ft]	CDR Static	CDR Seismic	Metal strip Type #	Product name
1	1.15	22.50	1.933	N/A	1	---
2	3.45	22.50	2.100	N/A	1	---
3	5.75	22.50	2.307	N/A	1	---
4	8.05	22.50	2.541	N/A	1	---
5	10.35	22.50	2.823	N/A	1	---
6	12.65	22.50	3.164	N/A	1	---
7	14.95	22.50	3.577	N/A	1	---
8	17.25	22.50	4.068	N/A	1	---
9	19.55	22.50	4.601	N/A	1	---
10	21.85	22.50	4.989	N/A	1	---

ECCENTRICITY for GIVEN LAYOUT

At interface with foundation: e/L static = 0.1271; Overturning: CDR-static = 3.39

#	Metal strip Elevation [ft]	Metal strip Length [ft]	e / L Static	e / L Seismic	Metal strip Type #	Product name
1	1.15	22.50	0.1142	N/A	1	---
2	3.45	22.50	0.0893	N/A	1	---
3	5.75	22.50	0.0658	N/A	1	---
4	8.05	22.50	0.0434	N/A	1	---
5	10.35	22.50	0.0217	N/A	1	---
6	12.65	22.50	0.0001	N/A	1	---
7	14.95	22.50	-0.0223	N/A	1	---
8	17.25	22.50	-0.0475	N/A	1	---
9	19.55	22.50	-0.0795	N/A	1	---
10	21.85	22.50	-0.1291	N/A	1	---

RESULTS for STRENGTH [Note: Actual CDR = (Yield stress) / (Actual stress)]

#	Metal strip Elevation [ft]	Coverage ratio, Rc=b/Sh	Horizontal spacing, Sh [ft]	Long-term strength Fy·Ac·Rc/b [lb/ft]	Tmax [lb/ft]	Tmd [lb/ft]	Specified minimum CDR static	Actual calculated CDR static	Specified minimum CDR seismic	Actual calculated CDR seismic
1	1.15	0.067	2.460	3084	3646.86	N/A	N/A	0.846	N/A	N/A
2	3.45	0.067	2.460	3084	3408.52	N/A	N/A	0.905	N/A	N/A
3	5.75	0.067	2.460	3084	3245.09	N/A	N/A	0.950	N/A	N/A
4	8.05	0.067	2.460	3084	3054.98	N/A	N/A	1.010	N/A	N/A
5	10.35	0.067	2.460	3084	2831.31	N/A	N/A	1.089	N/A	N/A
6	12.65	0.067	2.460	3084	2623.42	N/A	N/A	1.176	N/A	N/A
7	14.95	0.067	2.460	3084	2459.00	N/A	N/A	1.254	N/A	N/A
8	17.25	0.067	2.460	3084	2302.63	N/A	N/A	1.339	N/A	N/A
9	19.55	0.067	2.460	3084	2106.07	N/A	N/A	1.464	N/A	N/A
10	21.85	0.067	2.460	3084	1899.86	N/A	N/A	1.623	N/A	N/A

AASHTO 2007 (LRFD)
Ernstbridge Road Bridge Replacement

PROJECT IDENTIFICATION

Title:	Ernstbridge Road Bridge Replacement
Project Number:	N1185278
Client:	WSP
Designer:	JDD
Station Number:	East Abutment

Description:

H=23 feet. 2H:1V toe Hs=2'. Abutment 5 feet back and 7 feet tall. Cu values based on CPT data. L=0.8H

Company's information:

Name:
Street:

,
Telephone #:
Fax #:
E-Mail:

Original file path and name: N:\Projects\2018\N1185278\Working Files\Calculations-An.....
..... Abutmrnt CPT L8.BEN

Original date and time of creating this file: Sun Mar 31 16:0624 2019

PROGRAM MODE: ANALYSIS
of a BRIDGE ABUTMENT
using METAL STRIPS as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	120.0 lb/ft ³
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	125.0 lb/ft ³
Design value of internal angle of friction, ϕ	28.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	124.0 lb/ft ³
Equivalent internal angle of friction, $\phi_{equiv.}$	0.0 °
Equivalent cohesion, $c_{equiv.}$	2000.0 lb/ft ²

Water table is at wall base elevation

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Ka (external stability) = 0.3610 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 4.19 N γ= 0.00

SEISMICITY

Not Applicable

INPUT DATA: Metal strips
(Analysis)

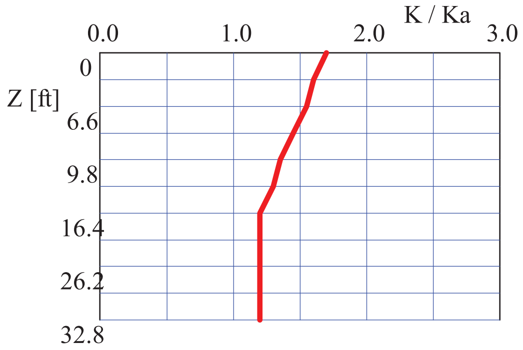
D A T A	Metal strip type #1	Metal strip type #2	Metal strip type #3	Metal strip type #4	Metal strip type #5
Yield strength of steel, Fy [kips/in ²]	65.3	N/A	N/A	N/A	N/A
Gross width of strip, b [in]	2.0	N/A	N/A	N/A	N/A
Vertical spacing, Sv [ft]	Varies	N/A	N/A	N/A	N/A
Design cross section area, Ac [in ²]	0.16	N/A	N/A	N/A	N/A

Ribbed steel strips.
Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0

Friction angle along reinforcement-soil interface,	ρ				
@ the top	60.97	N/A	N/A	N/A	N/A
@ 19.7 ft or below	32.00	N/A	N/A	N/A	N/A
Pullout resistance factor, F*					
@ the top	1.80	N/A	N/A	N/A	N/A
@ 19.7 ft or below	0.62	N/A	N/A	N/A	N/A
Scale-effect correction factor, α	1.00	N/A	N/A	N/A	N/A

Variation of Lateral Earth Pressure Coefficient With Depth

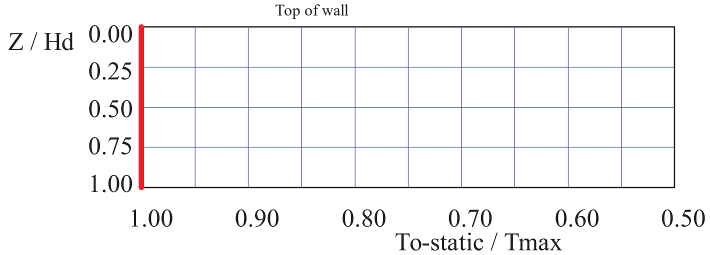
Z	K / Ka
0 ft	1.70
3.3 ft	1.60
6.6 ft	1.55
9.8 ft	1.45
13.1 ft	1.35
16.4 ft	1.30
19.7 ft	1.20



INPUT DATA: Facia and Connection (Analysis)

FACIA type: Segmental precast concrete panels.
Depth of panel is 1.31 ft. Horizontal distance to Center of Gravity of panel is 0.66 ft.
Average unit weight of panel is γ_f = 152.78 lb/ft ³

Z / Hd	To-static / Tmax
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00



D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	---	N/A	N/A	N/A	N/A
Strength reduction at the connection, CRu = Fyc / Fy	0.90	N/A	N/A	N/A	N/A

INPUT DATA: Geometry and Surcharge loads (of a BRIDGE ABUTMENT)

Design height, Hd	23.00	[ft]	{ Embedded depth is E = 3.00 ft, and height above top of finished bottom grade is H = 20.00 ft }
Batter, ω	0.0	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

U N I F O R M   S U R C H A R G E
Uniformly distributed dead load is 0.0 [lb/ft ²]

ABUTMENT GEOMETRY
Abutment's width, bf = 3.00 at distance from back of wall, cf = 5.00 [ft].
Footing's dimension: height, h' = 7.00, width, b = 3.00, and thickness, t = 1.00 [ft].
Dimensions of bridge bearing plate: height, fh = 0.33, width, fw = 1.64 [ft].

OTHER EXTERNAL LOAD(S)
[S] Vertical Dead Load, Pv-d = 0.0 and Vertical Live Load, Pv-l = 0.0 [lb/ft]. (Total of 0.0 [lb/ft])
The distance from back of the wall is 4.2 [ft].

ANALYZED REINFORCEMENT LAYOUT:

SCALE:

0 2 4 6 8 10[ft]

AASHTO 2007 (LRFD) Input Data

INTERNAL STABILITY

Load factor for vertical earth pressure, EV, from Table 3.4.1-2:	$\gamma_{p\text{-}EV}$	1.35	
Load factor for earthquake loads, EQ, from Table 3.4.1-1:	$\gamma_{p\text{-}EQ}$	1.00	
Load factor for live load surchrge, LS, from Figure C11.5.5-3(b): (Same as in External Stability).	$\gamma_{p\text{-}LS}$	1.75	
Load factor for dead load surchrge, ES: (Same as in External Stability).	$\gamma_{p\text{-}ES}$	1.50	
Resistance factor for reinforcement tension from Table 11.5.6-1:	ϕ	Static	Combined static/seismic
Metal Strips:		0.75	1.00
Resistance factor for reinforcement tension in connectors from Table 11.5.6-1:	ϕ	Static	Combined static/seismic
Metal Strips:		0.75	1.00
Resistance factor for reinforcement pullout from Table 11.5.6-1:	ϕ	0.90	1.20

EXTERNAL STABILITY

Load factor for vertical earth pressure, EV, from Table 3.4.1-2 and Figure C11.5.5-2:		Static		Combined Static/Seismic
Sliding and Eccentricity	$\gamma_{p\text{-}EV}$	1.00	$\gamma_{p\text{-}EQ}$	1.00
Bearing Capacity	$\gamma_{p\text{-}EV}$	1.35	$\gamma_{p\text{-}EQ}$	1.35

Load factor of active lateral earth pressure, EH, from Table 3.4.1-2 and Figure C11.5.5-2:	$\gamma_{p\text{-}EH}$	1.50
Load factor of active lateral earth pressure during earthquake (does not multiply P_{AE} and P_{IR}):	$(\gamma_{p\text{-}EH})_{EQ}$	1.50
Load factor for earthquake loads, EQ, from Table 3.4.1-1 (multiplies P_{AE} and P_{IR}):	$\gamma_{p\text{-}EQ}$	1.00

Resistance factor for shear resistance along common interfaces from Table 10.5.5.2.2-1:		Static	Combined Static/Seismic
Reinforced Soil and Foundation	ϕ_{τ}	1.00	1.00
Reinforced Soil and Reinforcement	ϕ_{τ}	1.00	1.00
Resistance factor for bearing capacity of shallow foundation from Table 10.5.5.2.2-1:		Static	Combined Static/Seismic
	ϕ_{b}	0.65	0.65

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, CDR = 1.04, Meyerhof stress = 5257 lb/ft².
Foundation Interface: Direct sliding, CDR = 1.789, Eccentricity, e/L = 0.1085, CDR-overturning = 3.87

METAL STRIP				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	CDR [pullout resistance]	CDR [connection break]	CDR [metal strip strength]	Metal strip strength CDR	Pullout resistance CDR	Direct sliding CDR	Eccentricity e/L	Product name
1	1.15	24.00	1	N/A	0.76	0.85	0.846	1.530	2.074	0.0970	---
2	3.45	24.00	1	N/A	0.81	0.90	0.905	1.443	2.255	0.0750	---
3	5.75	24.00	1	N/A	0.86	0.95	0.950	1.592	2.478	0.0541	---
4	8.05	24.00	1	N/A	0.91	1.01	1.010	1.704	2.732	0.0341	---
5	10.35	24.00	1	N/A	0.98	1.09	1.089	1.790	3.039	0.0147	---
6	12.65	24.00	1	N/A	1.06	1.18	1.176	1.828	3.410	-0.0048	---
7	14.95	24.00	1	N/A	1.13	1.25	1.254	1.749	3.863	-0.0251	---
8	17.25	24.00	1	N/A	1.21	1.34	1.339	1.743	4.405	-0.0481	---
9	19.55	24.00	1	N/A	1.32	1.46	1.464	1.710	5.004	-0.0775	---
10	21.85	24.00	1	N/A	1.46	1.62	1.623	1.606	5.468	-0.1229	---

BEARING CAPACITY for GIVEN LAYOUT

	STATIC	SEISMIC	UNITS
(Water table is at wall base elevation)			
Ultimate bearing capacity, q-ult	5447	N/A	[lb/ft ²]
Meyerhof stress, σ_V	5256.9	N/A	[lb/ft ²]
Eccentricity, e	1.77	N/A	[ft]
Eccentricity, e/L	0.074	N/A	
CDR calculated	1.04	N/A	
Base length	24.00	N/A	[ft]

SCALE:

0 2 4 6 8 10[ft]

DIRECT SLIDING for GIVEN LAYOUT
(for METAL STRIPS reinforcements)

Along reinforced and foundation soils interface: CDR-static = 1.789

#	Metal strip Elevation [ft]	Metal strip Length [ft]	CDR Static	CDR Seismic	Metal strip Type #	Product name
1	1.15	24.00	2.074	N/A	1	---
2	3.45	24.00	2.255	N/A	1	---
3	5.75	24.00	2.478	N/A	1	---
4	8.05	24.00	2.732	N/A	1	---
5	10.35	24.00	3.039	N/A	1	---
6	12.65	24.00	3.410	N/A	1	---
7	14.95	24.00	3.863	N/A	1	---
8	17.25	24.00	4.405	N/A	1	---
9	19.55	24.00	5.004	N/A	1	---
10	21.85	24.00	5.468	N/A	1	---

ECCENTRICITY for GIVEN LAYOUT

At interface with foundation: e/L static = 0.1085; Overturning: CDR-static = 3.87

#	Metal strip Elevation [ft]	Metal strip Length [ft]	e / L Static	e / L Seismic	Metal strip Type #	Product name
1	1.15	24.00	0.0970	N/A	1	---
2	3.45	24.00	0.0750	N/A	1	---
3	5.75	24.00	0.0541	N/A	1	---
4	8.05	24.00	0.0341	N/A	1	---
5	10.35	24.00	0.0147	N/A	1	---
6	12.65	24.00	-0.0048	N/A	1	---
7	14.95	24.00	-0.0251	N/A	1	---
8	17.25	24.00	-0.0481	N/A	1	---
9	19.55	24.00	-0.0775	N/A	1	---
10	21.85	24.00	-0.1229	N/A	1	---

RESULTS for STRENGTH [Note: Actual CDR = (Yield stress) / (Actual stress)]

#	Metal strip Elevation [ft]	Coverage ratio, Rc=b/Sh	Horizontal spacing, Sh [ft]	Long-term strength Fy·Ac·Rc/b [lb/ft]	Tmax [lb/ft]	Tmd [lb/ft]	Specified minimum CDR static	Actual calculated CDR static	Specified minimum CDR seismic	Actual calculated CDR seismic
1	1.15	0.067	2.460	3084	3646.86	N/A	N/A	0.846	N/A	N/A
2	3.45	0.067	2.460	3084	3408.52	N/A	N/A	0.905	N/A	N/A
3	5.75	0.067	2.460	3084	3245.09	N/A	N/A	0.950	N/A	N/A
4	8.05	0.067	2.460	3084	3054.98	N/A	N/A	1.010	N/A	N/A
5	10.35	0.067	2.460	3084	2831.31	N/A	N/A	1.089	N/A	N/A
6	12.65	0.067	2.460	3084	2623.42	N/A	N/A	1.176	N/A	N/A
7	14.95	0.067	2.460	3084	2459.00	N/A	N/A	1.254	N/A	N/A
8	17.25	0.067	2.460	3084	2302.63	N/A	N/A	1.339	N/A	N/A
9	19.55	0.067	2.460	3084	2106.07	N/A	N/A	1.464	N/A	N/A
10	21.85	0.067	2.460	3084	1899.86	N/A	N/A	1.623	N/A	N/A

AASHTO 2007 (LRFD)
Ernstbridge Road Bridge Replacement

PROJECT IDENTIFICATION

Title:	Ernstbridge Road Bridge Replacement
Project Number:	N1185278
Client:	WSP
Designer:	JDD
Station Number:	East Abutment

Description:

H=23 feet. 2H:1V toe Hs=2'. Abutment 5 feet back and 7 feet tall. Cu values based on CPT data. L=0.9H

Company's information:

Name:
Street:
,
Telephone #:
Fax #:
E-Mail:

Original file path and name: N:\Projects\2018\N1185278\Working Files\Calculations-An.....
..... Abutmrnt CPT L9.BEN

Original date and time of creating this file: Wed Apr 03, 2019

PROGRAM MODE: ANALYSIS
of a BRIDGE ABUTMENT
using METAL STRIPS as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	120.0 lb/ft³
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	125.0 lb/ft³
Design value of internal angle of friction, ϕ	28.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	124.0 lb/ft³
Equivalent internal angle of friction, $\phi_{equiv.}$	0.0 °
Equivalent cohesion, $c_{equiv.}$	2000.0 lb/ft²

Water table is at wall base elevation

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Ka (external stability) = 0.3610 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 4.19 N γ= 0.00

SEISMICITY

Not Applicable

INPUT DATA: Metal strips (Analysis)

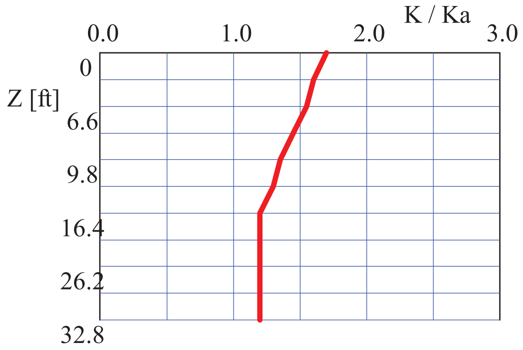
D A T A	Metal strip type #1	Metal strip type #2	Metal strip type #3	Metal strip type #4	Metal strip type #5
Yield strength of steel, Fy [kips/in ²]	65.3	N/A	N/A	N/A	N/A
Gross width of strip, b [in]	2.0	N/A	N/A	N/A	N/A
Vertical spacing, Sv [ft]	Varies	N/A	N/A	N/A	N/A
Design cross section area, Ac [in ²]	0.16	N/A	N/A	N/A	N/A

Ribbed steel strips.
Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0

Friction angle along reinforcement-soil interface,	ρ				
@ the top	60.97	N/A	N/A	N/A	N/A
@ 19.7 ft or below	32.00	N/A	N/A	N/A	N/A
Pullout resistance factor, F*					
@ the top	1.80	N/A	N/A	N/A	N/A
@ 19.7 ft or below	0.62	N/A	N/A	N/A	N/A
Scale-effect correction factor, α	1.00	N/A	N/A	N/A	N/A

Variation of Lateral Earth Pressure Coefficient With Depth

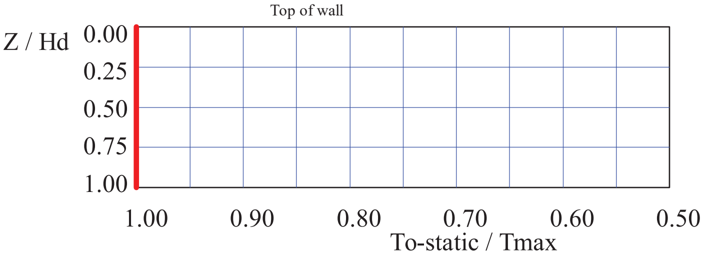
Z	K / Ka
0 ft	1.70
3.3 ft	1.60
6.6 ft	1.55
9.8 ft	1.45
13.1 ft	1.35
16.4 ft	1.30
19.7 ft	1.20



INPUT DATA: Facia and Connection (Analysis)

FACIA type: Segmental precast concrete panels.
Depth of panel is 1.31 ft. Horizontal distance to Center of Gravity of panel is 0.66 ft.
Average unit weight of panel is $\gamma_f = 152.78$ lb/ft³

Z / Hd	To-static / Tmax
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00



D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	---	N/A	N/A	N/A	N/A
Strength reduction at the connection, CRu = Fyc / Fy	0.90	N/A	N/A	N/A	N/A

INPUT DATA: Geometry and Surcharge loads (of a BRIDGE ABUTMENT)

Design height, Hd	23.00	[ft]	{ Embedded depth is E = 3.00 ft, and height above top of finished bottom grade is H = 20.00 ft }
Batter, ω	0.0	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

U N I F O R M   S U R C H A R G E
Uniformly distributed dead load is 0.0 [lb/ft ²]

ABUTMENT GEOMETRY
Abutment's width, bf = 3.00 at distance from back of wall, cf = 5.00 [ft].
Footing's dimension: height, h' = 7.00, width, b = 3.00, and thickness, t = 1.00 [ft].
Dimensions of bridge bearing plate: height, fh = 0.33, width, fw = 1.64 [ft].

OTHER EXTERNAL LOAD(S)
[S] Vertical Dead Load, Pv-d = 0.0 and Vertical Live Load, Pv-l = 0.0 [lb/ft]. (Total of 0.0 [lb/ft])
The distance from back of the wall is 4.2 [ft].

ANALYZED REINFORCEMENT LAYOUT:

SCALE:

0 2 4 6 8 10[ft]

AASHTO 2007 (LRFD) Input Data

INTERNAL STABILITY

Load factor for vertical earth pressure, EV, from Table 3.4.1-2:	$\gamma_{p\text{-}EV}$	1.35	
Load factor for earthquake loads, EQ, from Table 3.4.1-1:	$\gamma_{p\text{-}EQ}$	1.00	
Load factor for live load surchrge, LS, from Figure C11.5.5-3(b): (Same as in External Stability).	$\gamma_{p\text{-}LS}$	1.75	
Load factor for dead load surchrge, ES: (Same as in External Stability).	$\gamma_{p\text{-}ES}$	1.50	
Resistance factor for reinforcement tension from Table 11.5.6-1:	ϕ	Static	Combined static/seismic
Metal Strips:		0.75	1.00
Resistance factor for reinforcement tension in connectors from Table 11.5.6-1:	ϕ	Static	Combined static/seismic
Metal Strips:		0.75	1.00
Resistance factor for reinforcement pullout from Table 11.5.6-1:	ϕ	0.90	1.20

EXTERNAL STABILITY

Load factor for vertical earth pressure, EV, from Table 3.4.1-2 and Figure C11.5.5-2:		Static		Combined Static/Seismic
Sliding and Eccentricity	$\gamma_{p\text{-}EV}$	1.00	$\gamma_{p\text{-}EQ}$	1.00
Bearing Capacity	$\gamma_{p\text{-}EV}$	1.35	$\gamma_{p\text{-}EQ}$	1.35

Load factor of active lateral earth pressure, EH, from Table 3.4.1-2 and Figure C11.5.5-2:	$\gamma_{p\text{-}EH}$	1.50
Load factor of active lateral earth pressure during earthquake (does not multiply P_{AE} and P_{IR}):	$(\gamma_{p\text{-}EH})_{EQ}$	1.50
Load factor for earthquake loads, EQ, from Table 3.4.1-1 (multiplies P_{AE} and P_{IR}):	$\gamma_{p\text{-}EQ}$	1.00

Resistance factor for shear resistance along common interfaces from Table 10.5.5.2.2-1:		Static	Combined Static/Seismic
Reinforced Soil and Foundation	ϕ_τ	1.00	1.00
Reinforced Soil and Reinforcement	ϕ_τ	1.00	1.00
Resistance factor for bearing capacity of shallow foundation from Table 10.5.5.2.2-1:		Static	Combined Static/Seismic
	ϕ_b	0.65	0.65

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, CDR = 1.08, Meyerhof stress = 5067 lb/ft².
Foundation Interface: Direct sliding, CDR = 2.013, Eccentricity, e/L = 0.0808, CDR-overturning = 4.92

METAL STRIP				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	CDR [pullout resistance]	CDR [connection break]	CDR [metal strip strength]	Metal strip strength CDR	Pullout resistance CDR	Direct sliding CDR	Eccentricity e/L	Product name
1	1.15	27.00	1	N/A	0.76	0.85	0.846	1.743	2.356	0.0716	---
2	3.45	27.00	1	N/A	0.81	0.90	0.905	1.656	2.564	0.0539	---
3	5.75	27.00	1	N/A	0.86	0.95	0.950	1.841	2.821	0.0371	---
4	8.05	27.00	1	N/A	0.91	1.01	1.010	1.985	3.115	0.0208	---
5	10.35	27.00	1	N/A	0.98	1.09	1.089	2.103	3.469	0.0049	---
6	12.65	27.00	1	N/A	1.06	1.18	1.176	2.165	3.903	-0.0113	---
7	14.95	27.00	1	N/A	1.13	1.25	1.254	2.098	4.434	-0.0283	---
8	17.25	27.00	1	N/A	1.21	1.34	1.339	2.093	5.079	-0.0479	---
9	19.55	27.00	1	N/A	1.32	1.46	1.464	2.052	5.809	-0.0731	---
10	21.85	27.00	1	N/A	1.46	1.62	1.623	1.927	6.425	-0.1120	---

BEARING CAPACITY for GIVEN LAYOUT

	STATIC	SEISMIC	UNITS
(Water table is at wall base elevation)			
Ultimate bearing capacity, q-ult	5447	N/A	[lb/ft ²]
Meyerhof stress, σ_V	5066.8	N/A	[lb/ft ²]
Eccentricity, e	1.45	N/A	[ft]
Eccentricity, e/L	0.054	N/A	
CDR calculated	1.08	N/A	
Base length	27.00	N/A	[ft]

SCALE:

0 2 4 6 8 10[ft]

DIRECT SLIDING for GIVEN LAYOUT
(for METAL STRIPS reinforcements)

Along reinforced and foundation soils interface: CDR-static = 2.013

#	Metal strip Elevation [ft]	Metal strip Length [ft]	CDR Static	CDR Seismic	Metal strip Type #	Product name
1	1.15	27.00	2.356	N/A	1	---
2	3.45	27.00	2.564	N/A	1	---
3	5.75	27.00	2.821	N/A	1	---
4	8.05	27.00	3.115	N/A	1	---
5	10.35	27.00	3.469	N/A	1	---
6	12.65	27.00	3.903	N/A	1	---
7	14.95	27.00	4.434	N/A	1	---
8	17.25	27.00	5.079	N/A	1	---
9	19.55	27.00	5.809	N/A	1	---
10	21.85	27.00	6.425	N/A	1	---

ECCENTRICITY for GIVEN LAYOUT

At interface with foundation: e/L static = 0.0808; Overturning: CDR-static = 4.92

#	Metal strip Elevation [ft]	Metal strip Length [ft]	e / L Static	e / L Seismic	Metal strip Type #	Product name
1	1.15	27.00	0.0716	N/A	1	---
2	3.45	27.00	0.0539	N/A	1	---
3	5.75	27.00	0.0371	N/A	1	---
4	8.05	27.00	0.0208	N/A	1	---
5	10.35	27.00	0.0049	N/A	1	---
6	12.65	27.00	-0.0113	N/A	1	---
7	14.95	27.00	-0.0283	N/A	1	---
8	17.25	27.00	-0.0479	N/A	1	---
9	19.55	27.00	-0.0731	N/A	1	---
10	21.85	27.00	-0.1120	N/A	1	---

RESULTS for STRENGTH [Note: Actual CDR = (Yield stress) / (Actual stress)]

#	Metal strip Elevation [ft]	Coverage ratio, Rc=b/Sh	Horizontal spacing, Sh [ft]	Long-term strength Fy·Ac·Rc/b [lb/ft]	Tmax [lb/ft]	Tmd [lb/ft]	Specified minimum CDR static	Actual calculated CDR static	Specified minimum CDR seismic	Actual calculated CDR seismic
1	1.15	0.067	2.460	3084	3646.86	N/A	N/A	0.846	N/A	N/A
2	3.45	0.067	2.460	3084	3408.52	N/A	N/A	0.905	N/A	N/A
3	5.75	0.067	2.460	3084	3245.09	N/A	N/A	0.950	N/A	N/A
4	8.05	0.067	2.460	3084	3054.98	N/A	N/A	1.010	N/A	N/A
5	10.35	0.067	2.460	3084	2831.31	N/A	N/A	1.089	N/A	N/A
6	12.65	0.067	2.460	3084	2623.42	N/A	N/A	1.176	N/A	N/A
7	14.95	0.067	2.460	3084	2459.00	N/A	N/A	1.254	N/A	N/A
8	17.25	0.067	2.460	3084	2302.63	N/A	N/A	1.339	N/A	N/A
9	19.55	0.067	2.460	3084	2106.07	N/A	N/A	1.464	N/A	N/A
10	21.85	0.067	2.460	3084	1899.86	N/A	N/A	1.623	N/A	N/A

AASHTO 2007 (LRFD)
Ernstbridge Road Bridge Replacement

PROJECT IDENTIFICATION

Title:	Ernstbridge Road Bridge Replacement
Project Number:	N1185278
Client:	WSP
Designer:	JDD
Station Number:	Wing Wall H=23'

Description:

H=20 feet exposed. 2H:1V toe Hs=2'. L=1H

Company's information:

Name:
Street:
,
Telephone #:
Fax #:
E-Mail:

Original file path and name: N:\Projects\2018\N1185278\Working Files\Calculations-An.....
.....s\MSE\WW H20 L1.BEN

Original date and time of creating this file: Wed Apr 3 18:0624 2019

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using METAL STRIPS as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	120.0 lb/ft ³
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	120.0 lb/ft ³
Design value of internal angle of friction, ϕ	34.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	124.0 lb/ft ³
Equivalent internal angle of friction, $\phi_{equiv.}$	0.0 °
Equivalent cohesion, $c_{equiv.}$	2000.0 lb/ft ²

Water table is at wall base elevation

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Ka (external stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 4.19 N γ= 0.00

SEISMICITY

Not Applicable

INPUT DATA: Metal strips (Analysis)

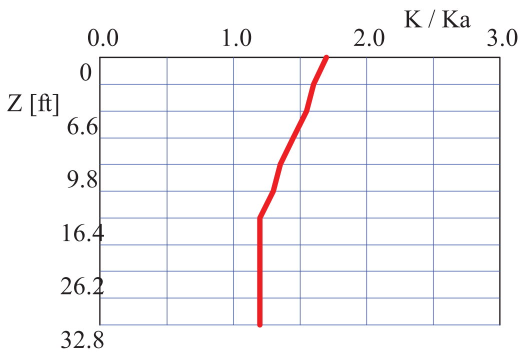
D A T A	Metal strip type #1	Metal strip type #2	Metal strip type #3	Metal strip type #4	Metal strip type #5
Yield strength of steel, Fy [kips/in ²]	65.3	N/A	N/A	N/A	N/A
Gross width of strip, b [in]	2.0	N/A	N/A	N/A	N/A
Vertical spacing, Sv [ft]	Varies	N/A	N/A	N/A	N/A
Design cross section area, Ac [in ²]	0.16	N/A	N/A	N/A	N/A

Ribbed steel strips.
Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0

Friction angle along reinforcement-soil interface,	ρ				
@ the top	60.97	N/A	N/A	N/A	N/A
@ 19.7 ft or below	32.00	N/A	N/A	N/A	N/A
Pullout resistance factor, F*					
@ the top	1.80	N/A	N/A	N/A	N/A
@ 19.7 ft or below	0.62	N/A	N/A	N/A	N/A
Scale-effect correction factor, α	1.00	N/A	N/A	N/A	N/A

Variation of Lateral Earth Pressure Coefficient With Depth

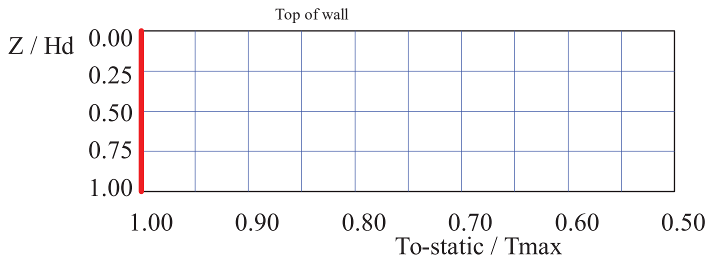
Z	K / Ka
0 ft	1.70
3.3 ft	1.60
6.6 ft	1.55
9.8 ft	1.45
13.1 ft	1.35
16.4 ft	1.30
19.7 ft	1.20



INPUT DATA: Facia and Connection (Analysis)

FACIA type: Segmental precast concrete panels.
Depth of panel is 1.31 ft. Horizontal distance to Center of Gravity of panel is 0.66 ft.
Average unit weight of panel is γ_f = 152.78 lb/ft ^{3}

Z / Hd	To-static / Tmax
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00



D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	---	N/A	N/A	N/A	N/A
Strength reduction at the connection, CRu = Fyc / Fy	0.90	N/A	N/A	N/A	N/A

INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	23.00	[ft]	{ Embedded depth is E = 3.00 ft, and height above top of finished bottom grade is H = 20.00 ft }
Batter, ω	0.0	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	6.6	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

U N I F O R M   S U R C H A R G E

Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 250.0 [lb/ft ²]

ANALYZED REINFORCEMENT LAYOUT:

SCALE:

0 2 4 6 8 10[ft]

AASHTO 2007 (LRFD) Input Data

INTERNAL STABILITY

Load factor for vertical earth pressure, EV, from Table 3.4.1-2:	$\gamma_{p\text{-}EV}$	1.35	
Load factor for earthquake loads, EQ, from Table 3.4.1-1:	$\gamma_{p\text{-}EQ}$	1.00	
Load factor for live load surchrge, LS, from Figure C11.5.5-3(b): (Same as in External Stability).	$\gamma_{p\text{-}LS}$	1.75	
Load factor for dead load surchrge, ES: (Same as in External Stability).	$\gamma_{p\text{-}ES}$	1.50	
Resistance factor for reinforcement tension from Table 11.5.6-1:	ϕ	Static	Combined static/seismic
Metal Strips:		0.75	1.00
Resistance factor for reinforcement tension in connectors from Table 11.5.6-1:	ϕ	Static	Combined static/seismic
Metal Strips:		0.75	1.00
Resistance factor for reinforcement pullout from Table 11.5.6-1:	ϕ	0.90	1.20

EXTERNAL STABILITY

Load factor for vertical earth pressure, EV, from Table 3.4.1-2 and Figure C11.5.5-2:		Static	Combined Static/Seismic	
Sliding and Eccentricity	$\gamma_{p\text{-}EV}$	1.00	$\gamma_{p\text{-}EQ}$	1.00
Bearing Capacity	$\gamma_{p\text{-}EV}$	1.35	$\gamma_{p\text{-}EQ}$	1.35

Load factor of active lateral earth pressure, EH, from Table 3.4.1-2 and Figure C11.5.5-2:	$\gamma_{p\text{-}EH}$	1.50
Load factor of active lateral earth pressure during earthquake (does not multiply Γ_{AE} and Γ_{IR}):	$(\gamma_{p\text{-}EH})_{EQ}$	1.50
Load factor for earthquake loads, EQ, from Table 3.4.1-1 (multiplies Γ_{AE} and Γ_{IR}):	$\gamma_{p\text{-}EQ}$	1.00

Resistance factor for shear resistance along common interfaces from Table 10.5.5.2.2-1:		Static	Combined Static/Seismic
Reinforced Soil and Foundation	ϕ_τ	1.00	1.00
Reinforced Soil and Reinforcement	ϕ_τ	1.00	1.00
Resistance factor for bearing capacity of shallow foundation from Table 10.5.5.2.2-1:		Static	Combined Static/Seismic
	ϕ_b	0.65	0.65

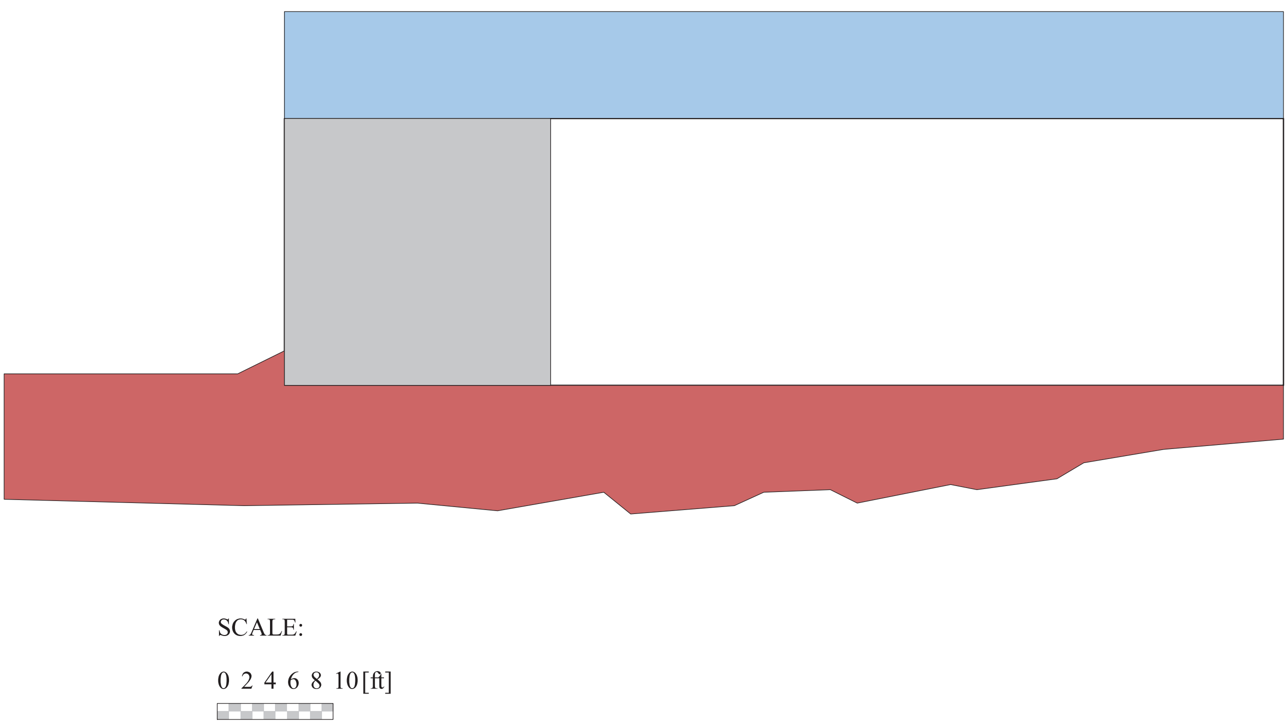
ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, CDR = 1.15, Meyerhof stress = 4750 lb/ft².
Foundation Interface: Direct sliding, CDR = 2.626, Eccentricity, e/L = 0.0931, CDR-overturning = 5.37

METAL STRIP				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	CDR [pullout resistance]	CDR [connection break]	CDR [metal strip strength]	Metal strip strength CDR	Pullout resistance CDR	Direct sliding CDR	Eccentricity e/L	Product name
1	1.15	23.00	1	N/A	0.89	0.99	0.994	1.412	2.726	0.0851	---
2	3.45	23.00	1	N/A	0.97	1.08	1.080	1.304	2.984	0.0701	---
3	5.75	23.00	1	N/A	1.04	1.16	1.156	1.400	3.309	0.0566	---
4	8.05	23.00	1	N/A	1.13	1.26	1.256	1.445	3.693	0.0444	---
5	10.35	23.00	1	N/A	1.26	1.39	1.395	1.445	4.178	0.0337	---
6	12.65	23.00	1	N/A	1.41	1.57	1.567	1.442	4.810	0.0244	---
7	14.95	23.00	1	N/A	1.63	1.81	1.815	1.450	5.667	0.0165	---
8	17.25	23.00	1	N/A	2.00	2.23	2.225	1.403	6.895	0.0100	---
9	19.55	23.00	1	N/A	2.68	2.97	2.975	1.231	8.803	0.0050	---
10	21.85	23.00	1	N/A	4.14	4.60	4.596	0.689	12.172	0.0013	---

BEARING CAPACITY for GIVEN LAYOUT

	STATIC	SEISMIC	UNITS
(Water table is at wall base elevation)			
Ultimate bearing capacity, q-ult	5447	N/A	[lb/ft ²]
Meyerhof stress, σ_V	4749.7	N/A	[lb/ft ²]
Eccentricity, e	1.42	N/A	[ft]
Eccentricity, e/L	0.062	N/A	
CDR calculated	1.15	N/A	
Base length	23.00	N/A	[ft]



DIRECT SLIDING for GIVEN LAYOUT
(for METAL STRIPS reinforcements)

Along reinforced and foundation soils interface: CDR-static = 2.626

#	Metal strip Elevation [ft]	Metal strip Length [ft]	CDR Static	CDR Seismic	Metal strip Type #	Product name
1	1.15	23.00	2.726	N/A	1	---
2	3.45	23.00	2.984	N/A	1	---
3	5.75	23.00	3.309	N/A	1	---
4	8.05	23.00	3.693	N/A	1	---
5	10.35	23.00	4.178	N/A	1	---
6	12.65	23.00	4.810	N/A	1	---
7	14.95	23.00	5.667	N/A	1	---
8	17.25	23.00	6.895	N/A	1	---
9	19.55	23.00	8.803	N/A	1	---
10	21.85	23.00	12.172	N/A	1	---

ECCENTRICITY for GIVEN LAYOUT

At interface with foundation: e/L static = 0.0931; Overturning: CDR-static = 5.37

#	Metal strip Elevation [ft]	Metal strip Length [ft]	e / L Static	e / L Seismic	Metal strip Type #	Product name
1	1.15	23.00	0.0851	N/A	1	---
2	3.45	23.00	0.0701	N/A	1	---
3	5.75	23.00	0.0566	N/A	1	---
4	8.05	23.00	0.0444	N/A	1	---
5	10.35	23.00	0.0337	N/A	1	---
6	12.65	23.00	0.0244	N/A	1	---
7	14.95	23.00	0.0165	N/A	1	---
8	17.25	23.00	0.0100	N/A	1	---
9	19.55	23.00	0.0050	N/A	1	---
10	21.85	23.00	0.0013	N/A	1	---

RESULTS for STRENGTH [Note: Actual CDR = (Yield stress) / (Actual stress)]

#	Metal strip Elevation [ft]	Coverage ratio, Rc=b/Sh	Horizontal spacing, Sh [ft]	Long-term strength Fy·Ac·Rc/b [lb/ft]	Tmax [lb/ft]	Tmd [lb/ft]	Specified minimum CDR static	Actual calculated CDR static	Specified minimum CDR seismic	Actual calculated CDR seismic
1	1.15	0.067	2.460	3084	3103.38	N/A	N/A	0.994	N/A	N/A
2	3.45	0.067	2.460	3084	2856.17	N/A	N/A	1.080	N/A	N/A
3	5.75	0.067	2.460	3084	2669.05	N/A	N/A	1.156	N/A	N/A
4	8.05	0.067	2.460	3084	2456.18	N/A	N/A	1.256	N/A	N/A
5	10.35	0.067	2.460	3084	2211.03	N/A	N/A	1.395	N/A	N/A
6	12.65	0.067	2.460	3084	1967.87	N/A	N/A	1.567	N/A	N/A
7	14.95	0.067	2.460	3084	1699.70	N/A	N/A	1.815	N/A	N/A
8	17.25	0.067	2.460	3084	1386.07	N/A	N/A	2.225	N/A	N/A
9	19.55	0.067	2.460	3084	1036.79	N/A	N/A	2.975	N/A	N/A
10	21.85	0.067	2.460	3084	671.09	N/A	N/A	4.596	N/A	N/A

AASHTO 2007 (LRFD)
Ernstbridge Road Bridge Replacement

PROJECT IDENTIFICATION

Title:	Ernstbridge Road Bridge Replacement
Project Number:	N1185278
Client:	WSP
Designer:	JDD
Station Number:	Wing Wall H=15'

Description:

H=15 feet exposed. 2H:1V toe Hs=7'. L=0.9H

Company's information:

Name:
Street:

,
Telephone #:
Fax #:
E-Mail:

Original file path and name: N:\Projects\2018\N1185278\Working Files\Calculations-An.....
.....s\MSE\WW H15 L8.BEN

Original date and time of creating this file: Sun Mar 31 16:0624 2019

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using METAL STRIPS as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	120.0 lb/ft ³
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	120.0 lb/ft ³
Design value of internal angle of friction, ϕ	34.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	124.0 lb/ft ³
Equivalent internal angle of friction, $\phi_{equiv.}$	0.0 °
Equivalent cohesion, $c_{equiv.}$	2000.0 lb/ft ²

Water table is at wall base elevation

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Ka (external stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 3.99 N γ= 0.00

SEISMICITY

Not Applicable

INPUT DATA: Metal strips (Analysis)

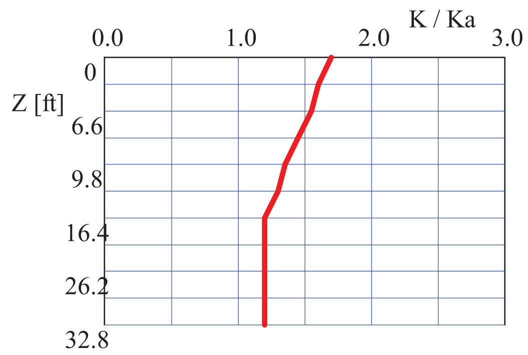
D A T A	Metal strip type #1	Metal strip type #2	Metal strip type #3	Metal strip type #4	Metal strip type #5
Yield strength of steel, Fy [kips/in ²]	65.3	N/A	N/A	N/A	N/A
Gross width of strip, b [in]	2.0	N/A	N/A	N/A	N/A
Vertical spacing, Sv [ft]	Varies	N/A	N/A	N/A	N/A
Design cross section area, Ac [in ²]	0.16	N/A	N/A	N/A	N/A

Ribbed steel strips.
Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0

Friction angle along reinforcement-soil interface,	ρ				
@ the top	60.97	N/A	N/A	N/A	N/A
@ 19.7 ft or below	32.00	N/A	N/A	N/A	N/A
Pullout resistance factor, F*					
@ the top	1.80	N/A	N/A	N/A	N/A
@ 19.7 ft or below	0.62	N/A	N/A	N/A	N/A
Scale-effect correction factor, α	1.00	N/A	N/A	N/A	N/A

Variation of Lateral Earth Pressure Coefficient With Depth

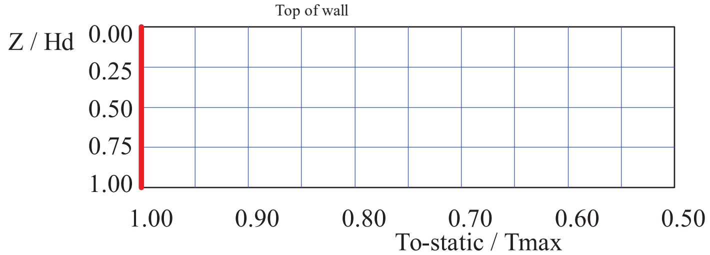
Z	K / Ka
0 ft	1.70
3.3 ft	1.60
6.6 ft	1.55
9.8 ft	1.45
13.1 ft	1.35
16.4 ft	1.30
19.7 ft	1.20



INPUT DATA: Facia and Connection (Analysis)

FACIA type: Segmental precast concrete panels.
Depth of panel is 1.31 ft. Horizontal distance to Center of Gravity of panel is 0.66 ft.
Average unit weight of panel is γ_f = 152.78 lb/ft ^{3}

Z / Hd	To-static / Tmax
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00



D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	---	N/A	N/A	N/A	N/A
Strength reduction at the connection, CRu = Fyc / Fy	0.90	N/A	N/A	N/A	N/A

INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	18.00	[ft]	{ Embedded depth is E = 3.00 ft, and height above top of finished bottom grade is H = 15.00 ft }
Batter, ω	0.0	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	6.6	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

U N I F O R M   S U R C H A R G E

Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 250.0 [lb/ft ²]

ANALYZED REINFORCEMENT LAYOUT:

SCALE:

0 2 4 6 8 10 [ft]

AASHTO 2007 (LRFD) Input Data

INTERNAL STABILITY

Load factor for vertical earth pressure, EV, from Table 3.4.1-2:	γ_{p-EV}	1.35	
Load factor for earthquake loads, EQ, from Table 3.4.1-1:	γ_{p-EQ}	1.00	
Load factor for live load surchrge, LS, from Figure C11.5.5-3(b): (Same as in External Stability).	γ_{p-LS}	1.75	
Load factor for dead load surchrge, ES: (Same as in External Stability).	γ_{p-ES}	1.50	
Resistance factor for reinforcement tension from Table 11.5.6-1:	ϕ	Static	Combined static/seismic
Metal Strips:		0.75	1.00
Resistance factor for reinforcement tension in connectors from Table 11.5.6-1:	ϕ	Static	Combined static/seismic
Metal Strips:		0.75	1.00
Resistance factor for reinforcement pullout from Table 11.5.6-1:	ϕ	0.90	1.20

EXTERNAL STABILITY

Load factor for vertical earth pressure, EV, from Table 3.4.1-2 and Figure C11.5.5-2:		Static	Combined Static/Seismic	
Sliding and Eccentricity	γ_{p-EV}	1.00	γ_{p-EQ}	1.00
Bearing Capacity	γ_{p-EV}	1.35	γ_{p-EQ}	1.35

Load factor of active lateral earth pressure, EH, from Table 3.4.1-2 and Figure C11.5.5-2: γ_{p-EH} 1.50

Load factor of active lateral earth pressure during earthquake (does not multiply P_{AE} and P_{IR}): $(\gamma_{p-EH})_{EQ}$ 1.50

Load factor for earthquake loads, EQ, from Table 3.4.1-1 (multiplies P_{AE} and P_{IR}): γ_{p-EQ} 1.00

Resistance factor for shear resistance along common interfaces from Table 10.5.5.2.2-1:		Static	Combined Static/Seismic
Reinforced Soil and Foundation	ϕ_τ	1.00	1.00
Reinforced Soil and Reinforcement	ϕ_τ	1.00	1.00
Resistance factor for bearing capacity of shallow foundation from Table 10.5.5.2.2-1:		Static	Combined Static/Seismic
	ϕ_b	0.65	0.65

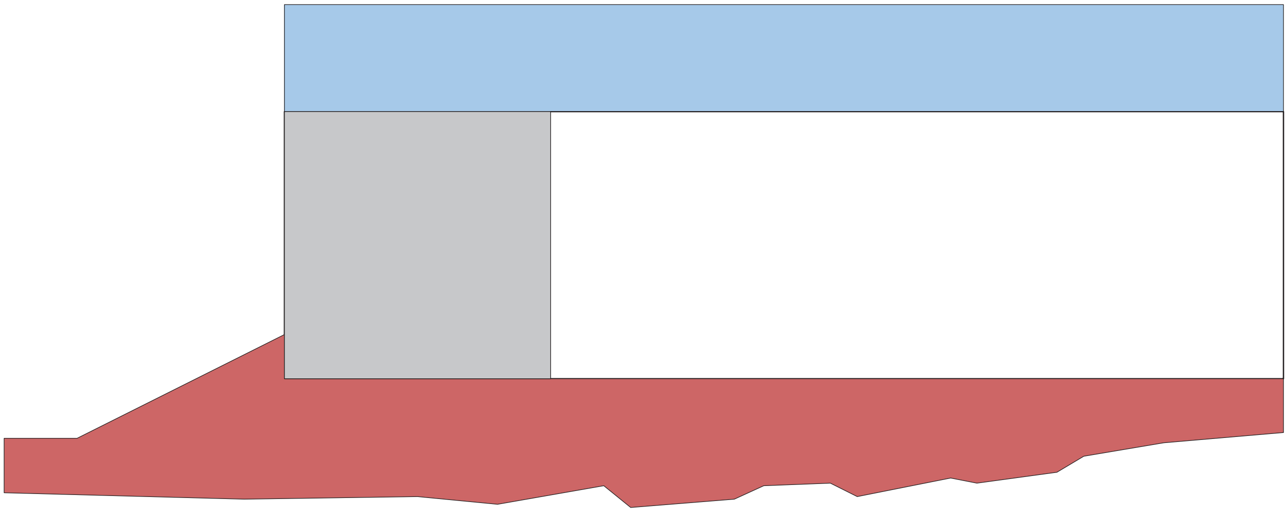
ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, CDR = 1.35, Meyerhof stress = 3845 lb/ft².
Foundation Interface: Direct sliding, CDR = 2.505, Eccentricity, e/L = 0.0993, CDR-overturning = 5.03

METAL STRIP				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	CDR [pullout resistance]	CDR [connection break]	CDR [metal strip strength]	Metal strip strength CDR	Pullout resistance CDR	Direct sliding CDR	Eccentricity e/L	Product name
1	0.90	18.00	1	N/A	1.33	1.48	1.481	1.608	2.607	0.0910	---
2	2.70	18.00	1	N/A	1.42	1.58	1.579	1.638	2.840	0.0754	---
3	4.50	18.00	1	N/A	1.54	1.71	1.714	1.643	3.119	0.0612	---
4	6.30	18.00	1	N/A	1.68	1.87	1.867	1.598	3.457	0.0485	---
5	8.10	18.00	1	N/A	1.85	2.05	2.053	1.508	3.879	0.0371	---
6	9.90	18.00	1	N/A	2.08	2.31	2.308	1.448	4.418	0.0272	---
7	11.70	18.00	1	N/A	2.42	2.69	2.686	1.419	5.130	0.0187	---
8	13.50	18.00	1	N/A	2.96	3.29	3.289	1.334	6.117	0.0116	---
9	15.30	18.00	1	N/A	3.86	4.29	4.287	1.117	7.573	0.0059	---
10	17.10	18.00	1	N/A	5.61	6.24	6.238	0.577	9.939	0.0016	---

BEARING CAPACITY for GIVEN LAYOUT

	STATIC	SEISMIC	UNITS
(Water table is at wall base elevation)			
Ultimate bearing capacity, q-ult	5183	N/A	[lb/ft ²]
Meyerhof stress, σ_V	3845.5	N/A	[lb/ft ²]
Eccentricity, e	1.15	N/A	[ft]
Eccentricity, e/L	0.064	N/A	
CDR calculated	1.35	N/A	
Base length	18.00	N/A	[ft]



SCALE:

0 2 4 6 8 10 [ft]

DIRECT SLIDING for GIVEN LAYOUT
(for METAL STRIPS reinforcements)

Along reinforced and foundation soils interface: CDR-static = 2.505

#	Metal strip Elevation [ft]	Metal strip Length [ft]	CDR Static	CDR Seismic	Metal strip Type #	Product name
1	0.90	18.00	2.607	N/A	1	---
2	2.70	18.00	2.840	N/A	1	---
3	4.50	18.00	3.119	N/A	1	---
4	6.30	18.00	3.457	N/A	1	---
5	8.10	18.00	3.879	N/A	1	---
6	9.90	18.00	4.418	N/A	1	---
7	11.70	18.00	5.130	N/A	1	---
8	13.50	18.00	6.117	N/A	1	---
9	15.30	18.00	7.573	N/A	1	---
10	17.10	18.00	9.939	N/A	1	---

ECCENTRICITY for GIVEN LAYOUT

At interface with foundation: e/L static = 0.0993; Overturning: CDR-static = 5.03

#	Metal strip Elevation [ft]	Metal strip Length [ft]	e / L Static	e / L Seismic	Metal strip Type #	Product name
1	0.90	18.00	0.0910	N/A	1	---
2	2.70	18.00	0.0754	N/A	1	---
3	4.50	18.00	0.0612	N/A	1	---
4	6.30	18.00	0.0485	N/A	1	---
5	8.10	18.00	0.0371	N/A	1	---
6	9.90	18.00	0.0272	N/A	1	---
7	11.70	18.00	0.0187	N/A	1	---
8	13.50	18.00	0.0116	N/A	1	---
9	15.30	18.00	0.0059	N/A	1	---
10	17.10	18.00	0.0016	N/A	1	---

RESULTS for STRENGTH [Note: Actual CDR = (Yield stress) / (Actual stress)]

#	Metal strip Elevation [ft]	Coverage ratio, Rc=b/Sh	Horizontal spacing, Sh [ft]	Long-term strength Fy·Ac·Rc/b [lb/ft]	Tmax [lb/ft]	Tmd [lb/ft]	Specified minimum CDR static	Actual calculated CDR static	Specified minimum CDR seismic	Actual calculated CDR seismic
1	0.90	0.067	2.460	3084	2082.98	N/A	N/A	1.481	N/A	N/A
2	2.70	0.067	2.460	3084	1953.11	N/A	N/A	1.579	N/A	N/A
3	4.50	0.067	2.460	3084	1799.37	N/A	N/A	1.714	N/A	N/A
4	6.30	0.067	2.460	3084	1652.16	N/A	N/A	1.867	N/A	N/A
5	8.10	0.067	2.460	3084	1502.39	N/A	N/A	2.053	N/A	N/A
6	9.90	0.067	2.460	3084	1336.33	N/A	N/A	2.308	N/A	N/A
7	11.70	0.067	2.460	3084	1148.08	N/A	N/A	2.686	N/A	N/A
8	13.50	0.067	2.460	3084	937.74	N/A	N/A	3.289	N/A	N/A
9	15.30	0.067	2.460	3084	719.47	N/A	N/A	4.287	N/A	N/A
10	17.10	0.067	2.460	3084	494.44	N/A	N/A	6.238	N/A	N/A

Ernstbridge Road Bridge Replacement

PROJECT IDENTIFICATION

Title:	Ernstbridge Road Bridge Replacement
Project Number:	N1185278 -
Client:	WSP
Designer:	JDD
Station Number:	East Abutment

Description:

H=22 feet exposed. 2H:1V toe Hs=2'. Abutment 5 feet back and 8 feet tall. L=0.75H

Company's information:

Name:
Street:
,
Telephone #:
Fax #:
E-Mail:

Original file path and name:
Original date and time of creating this file: Wed Apr 03 17:04:33 2019

PROGRAM MODE: Analysis of a General Slope using METALLIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

=========== Soil Layer #: ===========	Unit weight, γ [lb/ft ³]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ²]
....1....... Reinforced Soil.................................	120.0	34.0	0.0
....2....... Retained Soil.....................................	125.0	28.0	50.0
....3....... Foundation Soil.................................	124.0	28.0	100.0
....4.......Silty Sand...	128.0	35.0	0.0

REINFORCEMENT

Reinforcement Type #	Metal Mat Designated Name	Yield Strength of Steel, Fy [kips/in.²]	Design Cross-Section Area per Mat, Ac [inch²]	Gross Width of Mat, b [inch]	Yield Strength Reduction Factor, RFy	Additional Reduction Factor, RFa	Coverage Ratio, Rc Rc = b / Sh
1	---	65.26	0.16	1.97	1.49	1.00	0.07

Interaction Parameters Type #	Metal Mat Designated Name	== Direct Sliding == Cds-phi	Cds-c	====== Pullout ====== F* top	F* @19.7ft.	Alpha	Thickness of Transverse Bars, t [in.]	Distance Between Transverse Bars, St [in.]
1	---	1.18	0.00	1.80	0.62	1.00	0.39	11.81

Relative Orientation of Reinforcement Force, ROR = 0.00. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER
Unit weight of water = 62.45 [lb/ft ³]
Water pressure is defined by phreatic surface in Effective Stress Analysis.

SEISMICITY

Not Applicable

DRAWING OF SPECIFIED GEOMETRY - COMPLEX - Quick Input

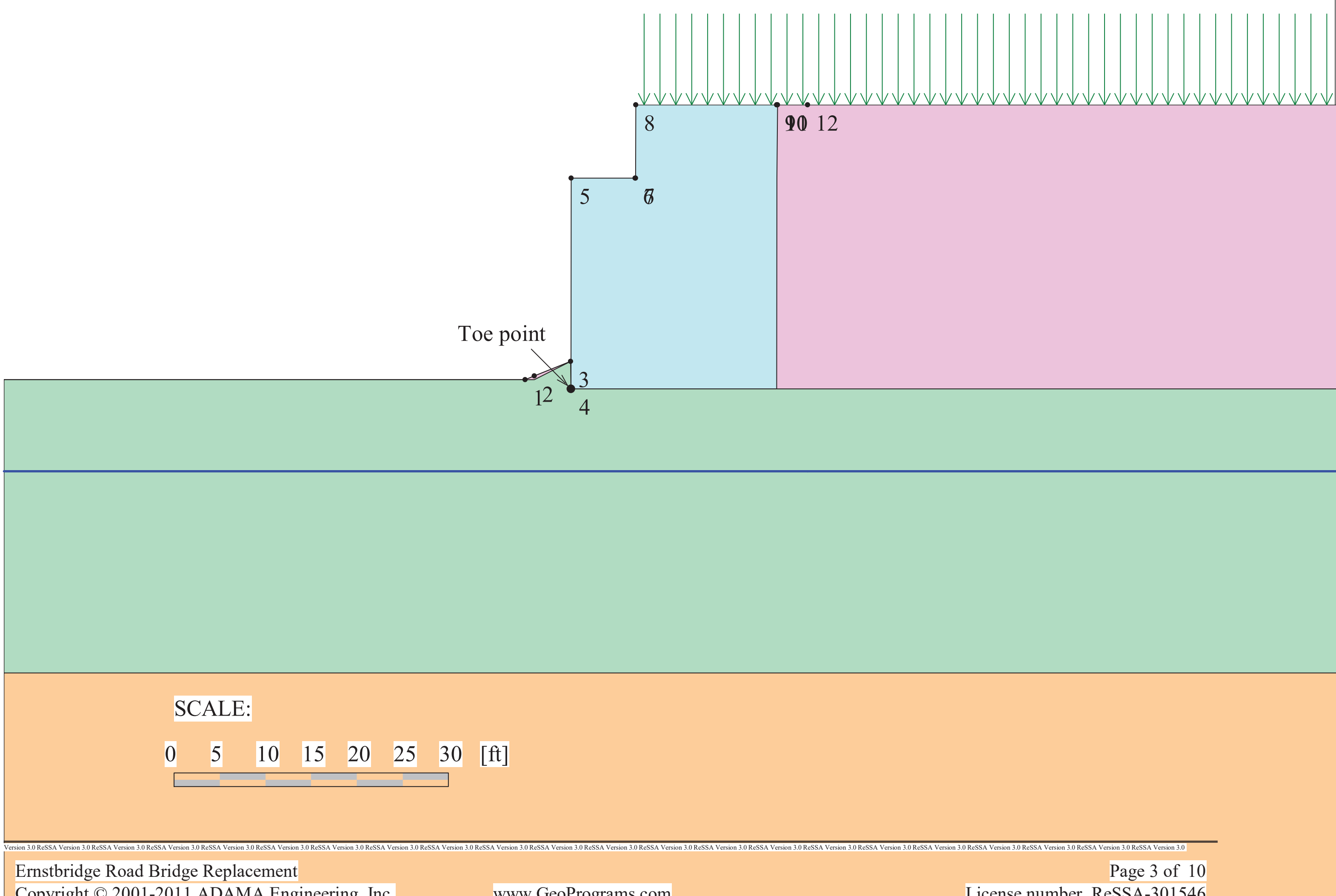
-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.
-- Xw,Yw represents the coordinates of phreatic surface.

GEOMETRY
Soil profile contains 4 layers (see details in next page)

WATER GEOMETRY
Phreatic line was specified.

UNIFORM SURCHARGE
Load Q1 = 250.00 [lb/ft²] inclined from verical at 0.00 degrees, starts at X1s = 108.00 and ends at X1e = 1100.03 [ft].
Surcharge load, Q2....................................None
Surcharge load, Q3....................................None

STRIP LOAD
............................None...................................



TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 4 layers. Coordinates in [ft.]
Water was described by phreatic line.

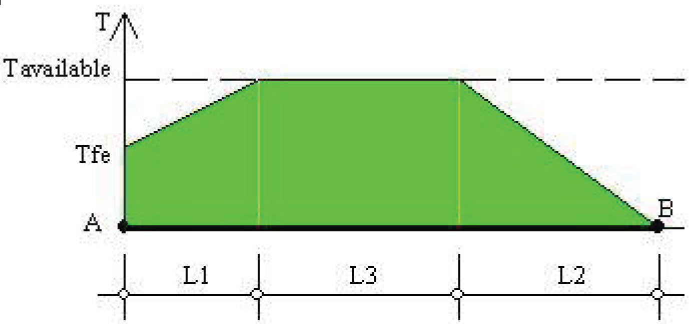
	#	Xi	Yi
Top of Layer 1	1	95.00	526.00
	2	99.97	528.00
	3	100.00	525.00
	4	100.03	548.00
	5	107.03	548.00
	6	107.07	548.00
	7	107.10	556.00
Top of Layer 2	8	95.00	526.00
	9	99.97	528.00
	10	100.00	525.00
	11	122.50	525.00
	12	122.53	548.00
	13	122.60	556.00
	14	125.88	556.00
Top of Layer 3	15	96.00	526.00
	16	99.97	528.00
	17	100.00	525.00
Top of Layer 4	18	328.08	469.00
	19	344.49	469.00
Top of Phreatic Line	21	328.10	516.00
	22	360.90	516.00

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 4 layers. Coordinates in [ft.]
Water was described by phreatic line. Y values are tabulated in the right most column.

#	X	Y1	Y2	Y3	Y4	(phreatic) Yw
1	95.00	526.00	526.00	526.00	469.00	516.00
2	96.00	526.40	526.40	526.00	469.00	516.00
3	99.97	528.00	528.00	528.00	469.00	516.00
4	100.00	525.00	525.00	525.00	469.00	516.00
5	100.03	548.00	525.00	525.00	469.00	516.00
6	107.03	548.00	525.00	525.00	469.00	516.00
7	107.07	548.00	525.00	525.00	469.00	516.00
8	107.10	556.00	525.00	525.00	469.00	516.00
9	122.50	556.00	525.00	525.00	469.00	516.00
10	122.53	556.00	548.00	525.00	469.00	516.00
11	122.60	556.00	556.00	525.00	469.00	516.00
12	125.88	556.00	556.00	525.00	469.00	516.00
13	328.08	556.00	556.00	525.00	469.00	516.00
14	328.10	556.00	556.00	525.00	469.00	516.00
15	344.49	556.00	556.00	525.00	469.00	516.00
16	360.90	556.00	556.00	525.00	469.00	516.00

DISTRIBUTION OF AVAILABLE STRENGTH ALONG EACH REINFORCEMENT LAYER



A = Front-end of reinforcement (at face of slope)
B = Rear-end of reinforcement
AB = L1 + L2 + L3 = Embedded length of reinforcement

Tavailable = Long-term strength of reinforcement
Tfe = Available front-end strength (e.g., connection to facing)

L1 = Front-end 'pullout' length
L2 = Rear-end pullout length
Tavailable prevails along L3

Factor of safety on resistance to pullout on either end of reinforcement, Fs-po = 1.50

Reinforcement Layer #	Designated Name	Height Relative to Toe [ft]	L [ft]	L1 [ft]	L2 [ft]	L3 [ft]	Tfe [lb/ft]	Tavailable [lb/ft]
1	---	1.15	22.50	6.30	14.40	1.80	2028.45	2983.01
2	---	3.45	22.50	6.88	15.62	0.00	2028.45	2972.02 (*)
3	---	5.75	22.50	5.77	16.73	0.00	2028.45	2880.76 (*)
4	---	8.05	22.50	4.94	17.56	0.00	2028.45	2772.53 (*)
5	---	10.35	22.50	4.18	18.32	0.00	2028.45	2643.32 (*)
6	---	12.65	22.50	4.18	18.32	0.00	2028.45	2599.01 (*)
7	---	14.95	22.50	4.43	18.07	0.00	2028.45	2567.46 (*)
8	---	17.25	22.50	3.99	18.51	0.00	2028.45	2450.79 (*)
9	---	19.55	22.50	2.95	19.55	0.00	2028.45	2291.91 (*)
10	---	21.85	22.50	0.65	21.85	0.00	2028.45	2075.29 (*)

(*) This Tavailable is dictated by the pullout resistance capacity, which is smaller than the long-term strength of the reinforcement that is related to its specified yield strength.

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X, Y) [ft]		Exit Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	100.00	525.00	100.00	525.00	100.00	525.00	0.00	N/A	#10 - Overhanging Cliff
2	119.75	556.00	65.92	526.33	84.58	556.15	35.18	1.97	
3	121.50	556.00	71.94	526.18	87.66	556.16	33.85	1.75	
4	123.25	556.01	75.08	526.06	89.85	556.02	33.40	1.62	
5	125.00	556.00	76.52	526.07	91.40	556.20	33.61	1.55	
6	126.75	556.00	76.12	526.30	92.67	556.10	34.09	1.49	
7	128.50	556.00	74.67	526.33	93.33	556.15	35.18	1.45	
8	130.25	556.00	78.02	526.06	95.50	556.10	34.75	1.43	
9	132.00	556.00	77.49	526.36	95.73	557.76	36.32	1.42	
10	133.75	556.00	76.29	526.20	94.99	560.45	39.02	1.42 .	OK
11	135.50	556.00	76.43	526.13	95.59	561.59	40.30	1.42	
12	137.25	556.00	74.65	526.29	94.85	564.54	43.26	1.42	
13	139.00	556.00	74.71	526.24	95.04	566.66	45.24	1.42	
14	140.75	556.00	73.01	526.37	94.72	568.99	47.83	1.43	
15	142.50	556.00	73.05	526.35	95.35	570.28	49.27	1.44	
16	144.25	556.00	73.19	526.26	95.52	572.68	51.51	1.46	
17	146.00	556.00	73.34	526.18	95.67	575.22	53.88	1.47	
18	147.75	556.00	71.57	526.31	94.85	579.15	57.74	1.49	
19	149.50	556.00	70.74	526.04	94.57	581.91	60.74	1.51	
20	151.25	556.00	70.73	526.04	95.21	583.43	62.39	1.53	
21	153.00	556.00	68.96	526.18	94.95	586.26	65.46	1.54	
22	154.75	556.00	69.11	526.11	95.02	589.49	68.47	1.57	
23	156.50	556.00	69.30	526.03	95.68	591.11	70.23	1.59	
24	158.25	556.00	68.53	526.33	95.71	594.62	73.50	1.61	
25	160.00	556.00	67.78	526.05	94.79	599.84	78.57	1.63	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points).

Exit Point #	Exit Point (X , Y) [ft]		Entry Point (X , Y) [ft]		Critical Circle (Xc , Yc , R) [ft]			Fs	STATUS
1	64.84	526.09	139.00	556.00	89.88	570.89	51.33	1.47	
2	66.39	526.04	139.00	556.00	90.78	569.89	50.18	1.46	
3	67.15	526.43	139.00	556.00	91.69	568.89	49.04	1.45	
4	68.83	526.31	137.25	556.00	91.95	566.72	46.56	1.44	
5	70.39	526.25	137.25	556.00	92.86	565.75	45.44	1.43	
6	72.05	526.13	135.50	556.00	93.15	563.64	43.03	1.42	
7	73.05	526.38	135.50	556.00	93.69	563.51	42.48	1.42	
8	74.61	526.32	135.50	556.00	94.64	562.54	41.39	1.42	
. 9	76.29	526.20	133.75	556.00	94.99	560.45	39.02	1.42 .	OK
10	77.58	526.28	133.75	556.00	95.57	560.23	38.42	1.42	
11	79.58	526.00	133.75	556.00	96.54	559.28	37.36	1.42	
12	80.93	526.06	133.75	556.00	97.14	559.02	36.73	1.43	
13	82.20	526.15	132.00	556.00	97.59	556.94	34.43	1.44	
14	83.63	526.16	132.00	556.01	98.60	556.02	33.40	1.45	
15	85.05	526.16	132.00	556.00	98.89	556.25	33.12	1.47	
16	86.59	526.12	133.75	556.00	100.05	557.04	33.72	1.49	
17	88.07	526.11	133.75	556.00	100.71	556.64	33.05	1.52	
18	89.63	526.06	135.50	556.00	101.86	557.44	33.68	1.55	
19	91.16	526.03	135.50	556.00	102.13	557.59	33.41	1.58	
20	92.14	526.19	135.50	556.00	102.81	557.11	32.71	1.63	
21	93.75	526.13	135.50	556.00	103.52	556.59	31.99	1.67	
22	95.43	526.32	139.00	556.00	106.68	556.63	32.33	1.79	
23	96.96	526.92	154.75	556.00	118.11	556.86	36.66	2.99	
24	100.00	525.00	100.00	525.00	100.00	525.00	0.00	N/A	#10 - Overhanging Cliff
25	100.00	525.00	100.00	525.00	100.00	525.00	0.00	N/A	#10 - Overhanging Cliff

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.42

Critical Circle: Xc = 94.99[ft], Yc = 560.45[ft], R = 39.02[ft]. (Number of slices used = 61)

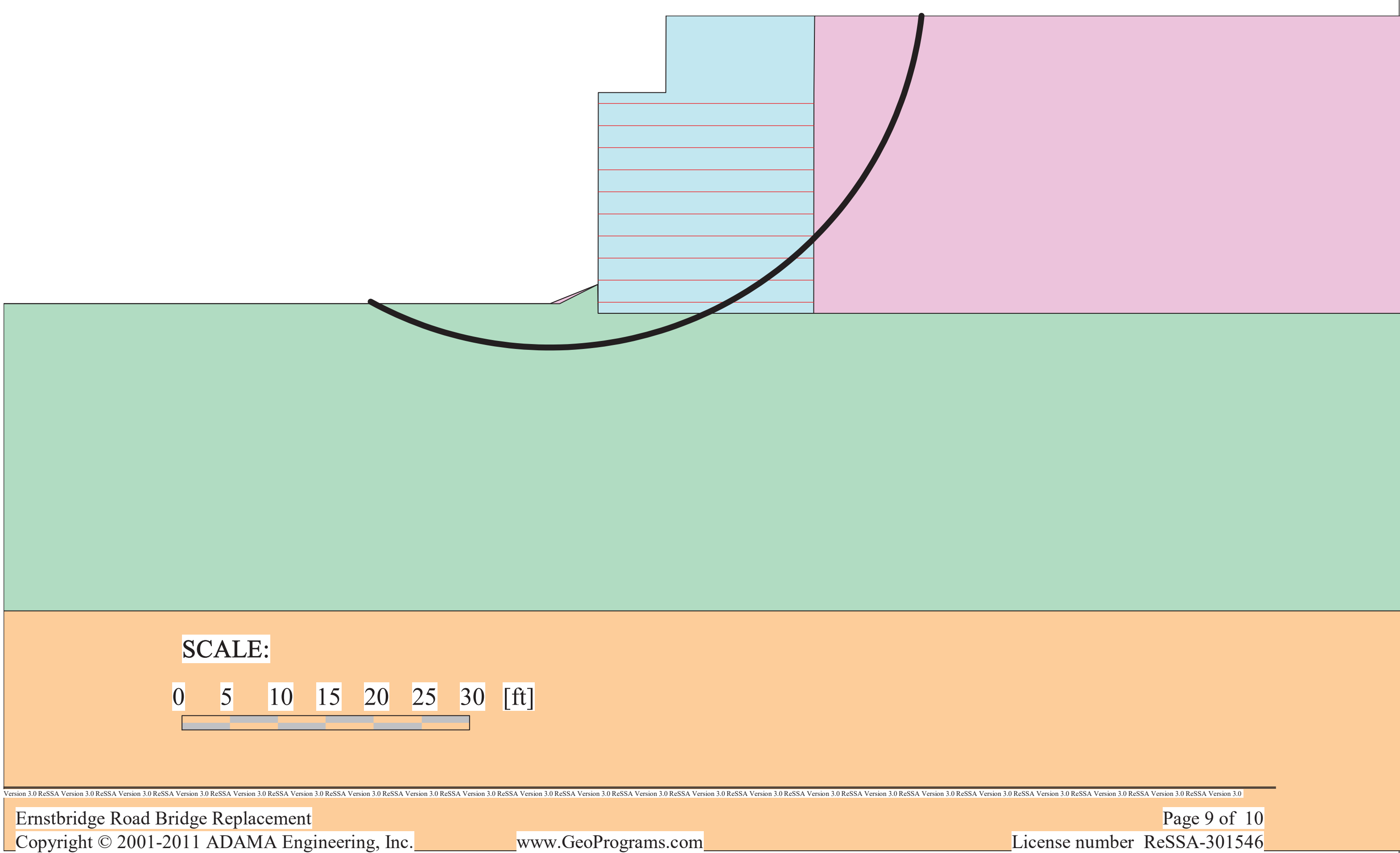
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

N O T C O N D U C T E D

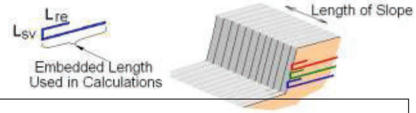
Three-Part Wedge Stability Analysis

N O T C O N D U C T E D

REINFORCEMENT LAYOUT: DRAWING



REINFORCEMENT LAYOUT: TABULATED DATA & QUANTITIES



Layer #	Reinf. Type #	Metallic Mat Designated Name	Height Relative to Toe [ft]	Embedded Length [ft]	Covergae Ratio, Rc	(X, Y) front [ft]		(X, Y) rear [ft]		Lsv * [ft]	Lre [ft]
1	1	---	1.15	22.50	0.07	328.09	1723.59	350.59	1723.59	0.00	0.00
2	1	---	3.45	22.50	0.07	328.09	1725.89	350.59	1725.89	0.00	0.00
3	1	---	5.75	22.50	0.07	328.09	1728.19	350.59	1728.19	0.00	0.00
4	1	---	8.05	22.50	0.07	328.09	1730.49	350.59	1730.49	0.00	0.00
5	1	---	10.35	22.50	0.07	328.10	1732.79	350.60	1732.79	0.00	0.00
6	1	---	12.65	22.50	0.07	328.10	1735.09	350.60	1735.09	0.00	0.00
7	1	---	14.95	22.50	0.07	328.10	1737.39	350.60	1737.39	0.00	0.00
8	1	---	17.25	22.50	0.07	328.11	1739.69	350.61	1739.69	0.00	0.00
9	1	---	19.55	22.50	0.07	328.11	1741.99	350.61	1741.99	0.00	0.00
10	1	---	21.85	22.50	0.07	328.11	1744.29	350.61	1744.29	0.00	0.00

* Vertical distance between layers.

QUANTITIES

Reinf. Type #	Designated Name	Coverage Ratio	Area of reinforcemnt [ft²] / length of slope [ft]
1	---	0.07	15.75

Ernstbridge Road Bridge Replacement

PROJECT IDENTIFICATION

Title:	Ernstbridge Road Bridge Replacement
Project Number:	N1185278 -
Client:	WSP
Designer:	JDD
Station Number:	East Abutment

Description:
H=22 feet exposed. 2H:1V toe Hs=2'. Abutment 5 feet back and 8 feet tall. L=0.75H

Company's information:

Name:
Street:
,
Telephone #:
Fax #:
E-Mail:

Original file path and name: N:\Project ..... lculations-Analyses\MSE\East Abutment L=75H ST.MSE
Original date and time of creating this file: Wed Apr 03 17:04:33 2019

PROGRAM MODE: Analysis of a General Slope using METALLIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

=========== Soil Layer #: ===========	Unit weight, γ [lb/ft ³]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ²]
....1....... Reinforced Soil.................................	120.0	34.0	0.0
....2....... Retained Soil.....................................	125.0	0.0	2000.0
....3....... Foundation Soil.................................	124.0	0.0	1500.0
....4.......Silty Sand..	128.0	35.0	0.0

REINFORCEMENT

Reinforcement Type #	Metal Mat Designated Name	Yield Strength of Steel, Fy [kips/in.²]	Design Cross-Section Area per Mat, Ac [inch²]	Gross Width of Mat, b [inch]	Yield Strength Reduction Factor, RFy	Additional Reduction Factor, RFa	Coverage Ratio, Rc Rc = b / Sh
1	---	65.26	0.16	1.97	1.49	1.00	0.07

Interaction Parameters Type #	Metal Mat Designated Name	== Direct Sliding == Cds-phi	Cds-c	====== Pullout ====== F* top	F* @19.7ft.	Alpha	Thickness of Transverse Bars, t [in.]	Distance Between Transverse Bars, St [in.]
1	---	1.18	0.00	1.80	0.62	1.00	0.39	11.81

Relative Orientation of Reinforcement Force, ROR = 0.00. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER

Unit weight of water = 62.45 [lb/ft ³]
Water pressure is defined by phreatic surface in Effective Stress Analysis.

SEISMICITY

Not Applicable

DRAWING OF SPECIFIED GEOMETRY - COMPLEX - Quick Input

-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.
-- Xw,Yw represents the coordinates of phreatic surface.

GEOMETRY

Soil profile contains 4 layers (see details in next page)

WATER GEOMETRY

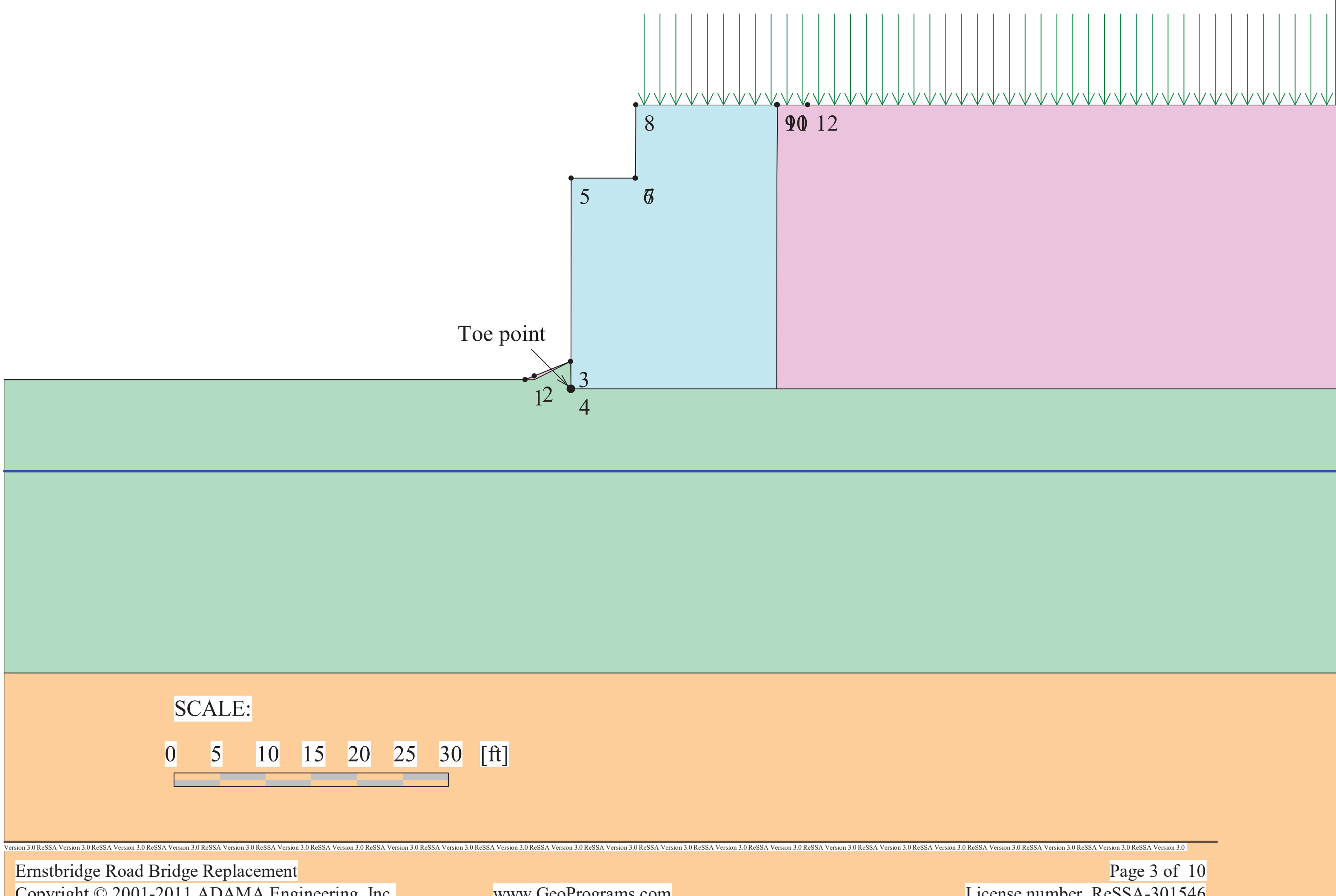
Phreatic line was specified.

UNIFORM SURCHARGE

Load Q1 = 250.00 [lb/ft²] inclined from verical at 0.00 degrees, starts at X1s = 108.00 and ends at X1e = 1100.03 [ft].
Surcharge load, Q2....................................None
Surcharge load, Q3....................................None

STRIP LOAD

............................None..................................



TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 4 layers. Coordinates in [ft.]
Water was described by phreatic line.

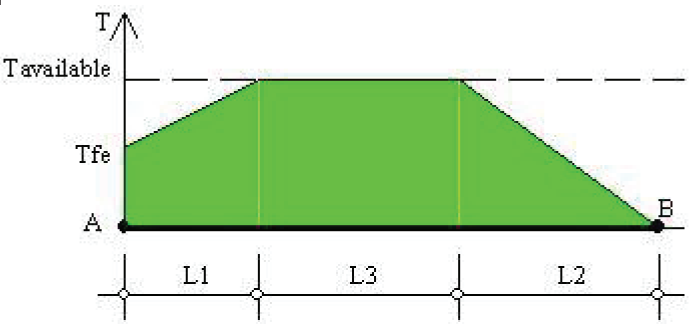
	#	Xi	Yi
Top of Layer 1	1	95.00	526.00
	2	99.97	528.00
	3	100.00	525.00
	4	100.03	548.00
	5	107.03	548.00
	6	107.07	548.00
	7	107.10	556.00
Top of Layer 2	8	95.00	526.00
	9	99.97	528.00
	10	100.00	525.00
	11	122.50	525.00
	12	122.53	548.00
	13	122.60	556.00
	14	125.88	556.00
Top of Layer 3	15	96.00	526.00
	16	99.97	528.00
	17	100.00	525.00
Top of Layer 4	18	328.08	469.00
	19	344.49	469.00
Top of Phreatic Line	21	328.10	516.00
	22	360.90	516.00

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 4 layers. Coordinates in [ft.]
Water was described by phreatic line. Y values are tabulated in the right most column.

#	X	Y1	Y2	Y3	Y4	(phreatic) Yw
1	95.00	526.00	526.00	526.00	469.00	516.00
2	96.00	526.40	526.40	526.00	469.00	516.00
3	99.97	528.00	528.00	528.00	469.00	516.00
4	100.00	525.00	525.00	525.00	469.00	516.00
5	100.03	548.00	525.00	525.00	469.00	516.00
6	107.03	548.00	525.00	525.00	469.00	516.00
7	107.07	548.00	525.00	525.00	469.00	516.00
8	107.10	556.00	525.00	525.00	469.00	516.00
9	122.50	556.00	525.00	525.00	469.00	516.00
10	122.53	556.00	548.00	525.00	469.00	516.00
11	122.60	556.00	556.00	525.00	469.00	516.00
12	125.88	556.00	556.00	525.00	469.00	516.00
13	328.08	556.00	556.00	525.00	469.00	516.00
14	328.10	556.00	556.00	525.00	469.00	516.00
15	344.49	556.00	556.00	525.00	469.00	516.00
16	360.90	556.00	556.00	525.00	469.00	516.00

DISTRIBUTION OF AVAILABLE STRENGTH ALONG EACH REINFORCEMENT LAYER



A = Front-end of reinforcement (at face of slope)
B = Rear-end of reinforcement
AB = L1 + L2 + L3 = Embedded length of reinforcement

Tavailable = Long-term strength of reinforcement
Tfe = Available front-end strength (e.g., connection to facing)

L1 = Front-end 'pullout' length
L2 = Rear-end pullout length
Tavailable prevails along L3

Factor of safety on resistance to pullout on either end of reinforcement, Fs-po = 1.50

Reinforcement Layer #	Designated Name	Height Relative to Toe [ft]	L [ft]	L1 [ft]	L2 [ft]	L3 [ft]	Tfe [lb/ft]	Tavailable [lb/ft]
1	---	1.15	22.50	6.30	14.40	1.80	2028.45	2983.01
2	---	3.45	22.50	6.88	15.62	0.00	2028.45	2972.02 (*)
3	---	5.75	22.50	5.77	16.73	0.00	2028.45	2880.76 (*)
4	---	8.05	22.50	4.94	17.56	0.00	2028.45	2772.53 (*)
5	---	10.35	22.50	4.18	18.32	0.00	2028.45	2643.32 (*)
6	---	12.65	22.50	4.18	18.32	0.00	2028.45	2599.01 (*)
7	---	14.95	22.50	4.43	18.07	0.00	2028.45	2567.46 (*)
8	---	17.25	22.50	3.99	18.51	0.00	2028.45	2450.79 (*)
9	---	19.55	22.50	2.95	19.55	0.00	2028.45	2291.91 (*)
10	---	21.85	22.50	0.65	21.85	0.00	2028.45	2075.29 (*)

(*) This Tavailable is dictated by the pullout resistance capacity, which is smaller than the long-term strength of the reinforcement that is related to its specified yield strength.

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X, Y) [ft]		Exit Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	118.00	556.00	62.21	526.19	82.07	556.14	35.93	4.61	
2	123.09	556.00	74.87	526.06	89.65	556.05	33.43	3.26	
3	128.17	556.00	74.47	526.32	93.07	556.08	35.10	2.94	
4	133.25	556.00	70.20	526.51	94.62	556.45	38.64	2.60	
5	138.34	556.00	70.58	526.23	97.89	556.06	40.44	2.42	
6	143.42	556.00	70.31	526.53	100.90	556.07	42.52	2.36	
7	148.50	556.00	62.49	526.01	99.81	557.31	48.71	2.33	
8	153.59	556.00	57.77	526.66	99.43	561.73	54.46	2.29	
9	158.67	556.00	53.99	526.21	99.74	564.25	59.50	2.27	
10	163.75	556.00	49.36	526.77	99.81	567.80	65.02	2.25	
11	168.84	556.00	45.78	526.06	99.92	571.39	70.61	2.23	
12	173.92	556.00	41.24	526.51	100.08	575.01	76.25	2.22	
13	179.00	556.00	36.71	526.96	100.26	578.67	81.93	2.21	
14	184.08	556.00	32.17	527.42	100.48	582.34	87.66	2.21	
15	189.17	556.00	28.95	526.26	100.72	586.04	93.41	2.20	
16	194.25	556.00	24.49	526.61	100.98	589.76	99.19	2.20	
17	199.33	556.00	20.04	526.96	101.26	593.49	104.99	2.20	
18	204.42	556.00	15.59	527.31	101.56	597.23	110.82	2.19	
19	209.50	556.00	11.13	527.66	101.87	600.99	116.66	2.19	
20	214.58	556.00	8.32	526.01	102.19	604.75	122.52	2.19	
21	219.67	556.00	3.95	526.26	102.52	608.53	128.39	2.19	
22	224.75	556.00	-0.43	526.51	102.86	612.31	134.27	2.19 .	OK
23	229.83	556.00	-1.60	527.95	105.34	614.39	137.51	2.19	
24	234.92	556.00	-0.99	527.20	107.82	616.47	140.75	2.19	
25	240.00	556.00	-0.39	526.47	110.31	618.56	143.99	2.20	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points).

Exit Point #	Exit Point (X, Y) [ft]		Entry Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
. 1	-0.43	526.51	224.75	556.00	102.86	612.31	134.27	2.19	. On extreme X-exit
2	3.95	526.26	219.67	556.00	102.52	608.53	128.39	2.19	
3	8.32	526.01	214.58	556.00	102.19	604.75	122.52	2.19	
4	11.74	526.92	214.58	556.00	104.33	603.05	119.88	2.19	
5	16.19	526.58	209.50	556.00	104.01	599.29	114.03	2.19	
6	20.63	526.24	204.42	556.00	103.71	595.54	108.19	2.19	
7	23.68	527.61	199.33	556.00	103.42	591.81	102.38	2.20	
8	28.21	527.16	194.25	556.00	103.15	588.08	96.58	2.20	
9	33.29	526.06	194.25	556.00	105.33	586.41	93.98	2.20	
10	36.58	527.12	189.17	556.00	105.08	582.71	88.23	2.21	
11	41.18	526.59	184.08	556.00	104.86	579.03	82.50	2.21	
12	45.78	526.07	179.00	556.00	104.67	575.37	76.81	2.22	
13	49.35	526.79	173.92	556.00	104.52	571.75	71.17	2.23	
14	53.50	526.81	173.92	556.00	106.75	570.12	68.64	2.24	
15	58.25	526.09	168.84	556.00	106.33	567.72	63.60	2.26	
16	62.13	526.40	163.75	556.00	105.65	566.24	59.00	2.27	
17	66.14	526.53	158.67	556.00	105.04	564.38	54.28	2.30	
18	70.69	526.13	158.67	556.00	106.39	565.49	53.14	2.32	
19	74.98	526.02	153.59	556.00	105.59	563.78	48.62	2.35	
20	78.69	526.33	148.50	556.00	104.60	562.33	44.35	2.38	
21	82.96	526.22	148.50	556.00	105.30	564.06	43.94	2.42	
22	87.41	526.04	148.50	556.00	106.36	564.67	43.03	2.46	
23	91.38	526.13	148.50	556.00	108.29	563.34	40.88	2.52	
24	95.75	526.37	148.50	556.00	109.89	562.96	39.23	2.66	
25	100.00	525.00	100.00	525.00	100.00	525.00	0.00	N/A	#10 - Overhanging Cliff

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 2.19

Critical Circle: Xc = 102.86[ft], Yc = 612.31[ft], R = 134.27[ft]. (Number of slices used = 60)

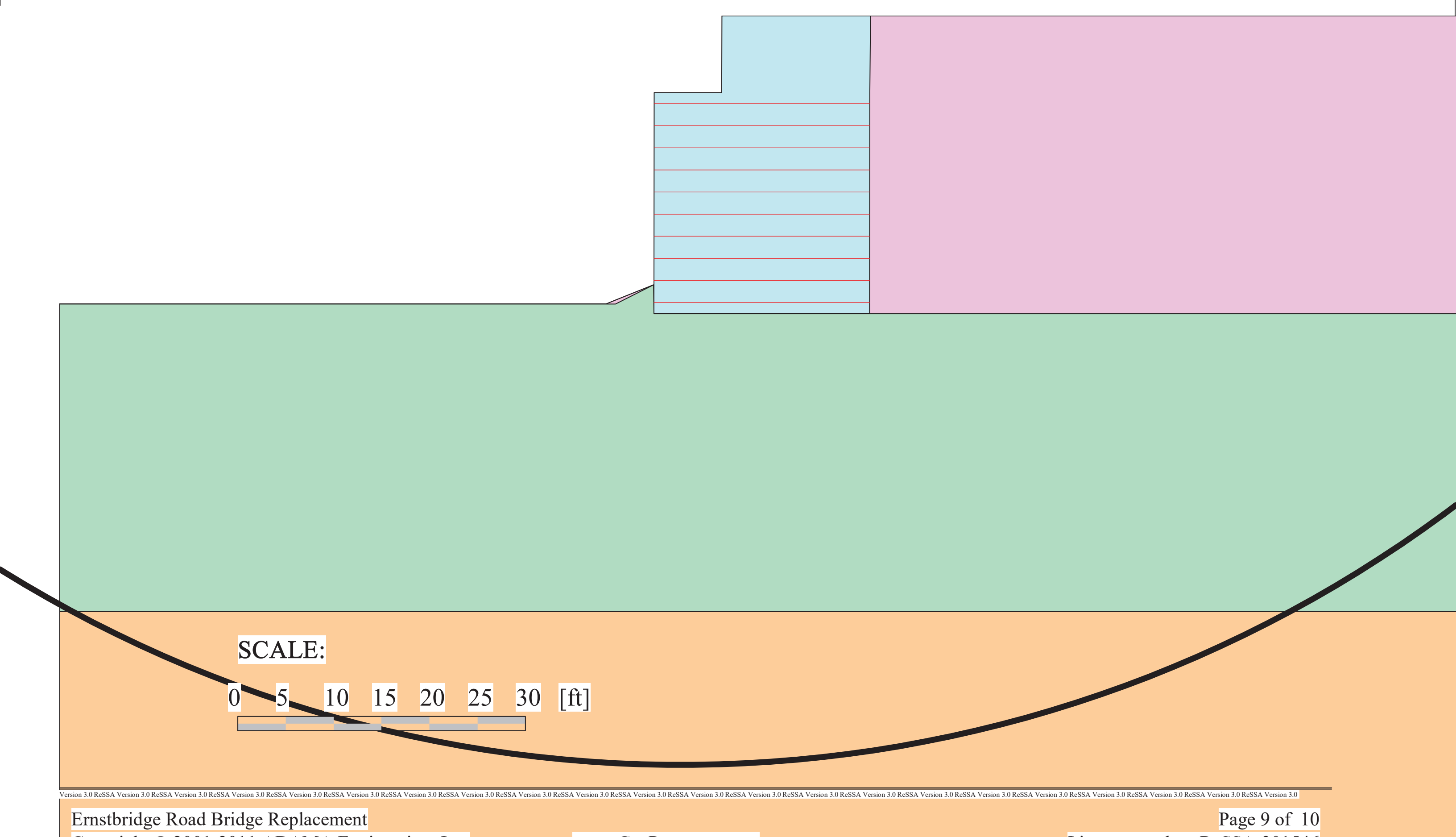
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

N O T C O N D U C T E D

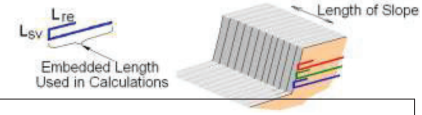
Three-Part Wedge Stability Analysis

N O T C O N D U C T E D

REINFORCEMENT LAYOUT: DRAWING



REINFORCEMENT LAYOUT: TABULATED DATA & QUANTITIES



Layer #	Reinf. Type #	Metallic Mat Designated Name	Height Relative to Toe [ft]	Embedded Length [ft]	Covergae Ratio, Rc	(X, Y) front [ft]		(X, Y) rear [ft]		Lsv * [ft]	Lre [ft]
1	1	---	1.15	22.50	0.07	328.09	1723.59	350.59	1723.59	0.00	0.00
2	1	---	3.45	22.50	0.07	328.09	1725.89	350.59	1725.89	0.00	0.00
3	1	---	5.75	22.50	0.07	328.09	1728.19	350.59	1728.19	0.00	0.00
4	1	---	8.05	22.50	0.07	328.09	1730.49	350.59	1730.49	0.00	0.00
5	1	---	10.35	22.50	0.07	328.10	1732.79	350.60	1732.79	0.00	0.00
6	1	---	12.65	22.50	0.07	328.10	1735.09	350.60	1735.09	0.00	0.00
7	1	---	14.95	22.50	0.07	328.10	1737.39	350.60	1737.39	0.00	0.00
8	1	---	17.25	22.50	0.07	328.11	1739.69	350.61	1739.69	0.00	0.00
9	1	---	19.55	22.50	0.07	328.11	1741.99	350.61	1741.99	0.00	0.00
10	1	---	21.85	22.50	0.07	328.11	1744.29	350.61	1744.29	0.00	0.00

* Vertical distance between layers.

QUANTITIES

Reinf. Type #	Designated Name	Coverage Ratio	Area of reinforcemnt [ft²] / length of slope [ft]
1	---	0.07	15.75

Ernstbridge Road Bridge Replacement

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PROJECT IDENTIFICATION

Title: Ernstbridge Road Bridge Replacement
Project Number: N1185278 -
Client: WSP
Designer:
Station Number: East Abutment

Description:

H=22 feet exposed. 2H:1V toe Hs=2'. Abutment 5 feet back and 8 feet tall. Cu values based on CPT data. L=0.9H

Company's information:

Name:
Street:
,
Telephone #:
Fax #:
E-Mail:

Original file path and name:
Original date and time of creating this file:

PROGRAM MODE: Analysis of a General Slope using METALLIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

=========== Soil Layer #: ===========	Unit weight, γ [lb/ft ³]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ²]
....1....... Reinforced Soil.................................	120.0	34.0	0.0
....2....... Retained Soil.....................................	125.0	28.0	50.0
....3....... Foundation Soil.................................	124.0	28.0	100.0
....4.......Silty Sand...	128.0	35.0	0.0

REINFORCEMENT

Reinforcement Type #	Metal Mat Designated Name	Yield Strength of Steel, Fy [kips/in.²]	Design Cross-Section Area per Mat, Ac [inch²]	Gross Width of Mat, b [inch]	Yield Strength Reduction Factor, RFy	Additional Reduction Factor, RFa	Coverage Ratio, Rc Rc = b / Sh
1	---	65.26	0.16	1.97	1.49	1.00	0.07

Interaction Parameters Type #	Metal Mat Designated Name	== Direct Sliding == Cds-phi	Cds-c	====== Pullout ====== F* top	F* @19.7ft.	Alpha	Thickness of Transverse Bars, t [in.]	Distance Between Transverse Bars, St [in.]
1	---	1.18	0.00	1.80	0.62	1.00	0.39	11.81

Relative Orientation of Reinforcement Force, ROR = 0.00. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER

Unit weight of water = 62.45 [lb/ft ³]
Water pressure is defined by phreatic surface in Effective Stress Analysis.

SEISMICITY

Not Applicable

DRAWING OF SPECIFIED GEOMETRY - COMPLEX - Quick Input

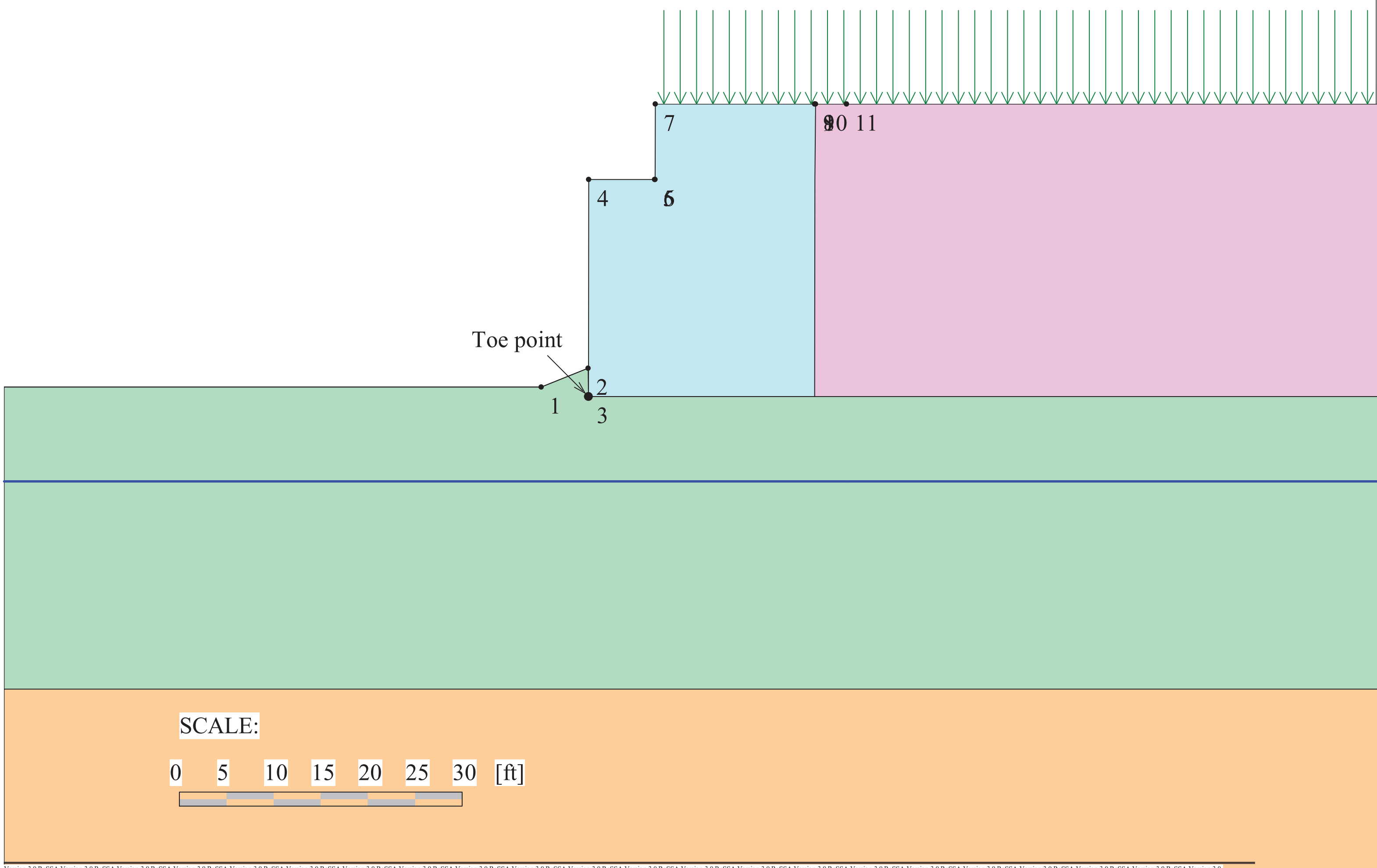
-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.
-- Xw,Yw represents the coordinates of phreatic surface.

GEOMETRY
Soil profile contains 4 layers (see details in next page)

WATER GEOMETRY
Phreatic line was specified.

UNIFORM SURCHARGE
Load Q1 = 250.00 [lb/ft²] inclined from verical at 0.00 degrees, starts at X1s = 108.00 and ends at X1e = 1100.03 [ft].
Surcharge load, Q2....................................None
Surcharge load, Q3....................................None

STRIP LOAD
............................None..................................



TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 4 layers. Coordinates in [ft.]
Water was described by phreatic line.

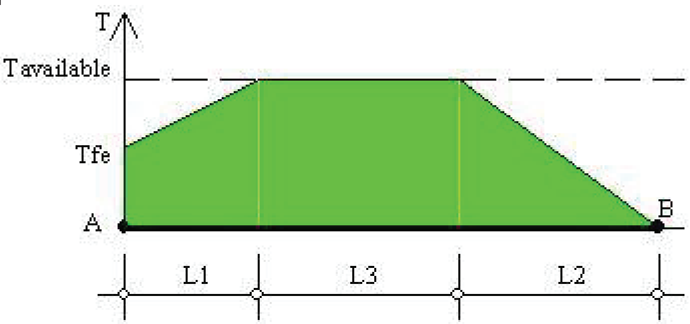
	#	Xi	Yi
Top of Layer 1	1	95.00	526.00
	2	99.97	528.00
	3	100.00	525.00
	4	100.03	548.00
	5	107.03	548.00
	6	107.07	548.00
	7	107.10	556.00
Top of Layer 2	8	95.00	526.00
	9	99.97	528.00
	10	100.00	525.00
	11	124.00	525.00
	12	124.03	548.00
	13	124.10	556.00
	14	127.38	556.00
Top of Layer 3	15	95.00	526.00
	16	99.97	528.00
	17	100.00	525.00
Top of Layer 4	18	328.08	469.00
	19	344.49	469.00
Top of Phreatic Line	21	328.10	516.00
	22	360.90	516.00

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 4 layers. Coordinates in [ft.]
Water was described by phreatic line. Y values are tabulated in the right most column.

#	X	Y1	Y2	Y3	Y4	(phreatic) Yw
1	95.00	526.00	526.00	526.00	469.00	516.00
2	99.97	528.00	528.00	528.00	469.00	516.00
3	100.00	525.00	525.00	525.00	469.00	516.00
4	100.03	548.00	525.00	525.00	469.00	516.00
5	107.03	548.00	525.00	525.00	469.00	516.00
6	107.07	548.00	525.00	525.00	469.00	516.00
7	107.10	556.00	525.00	525.00	469.00	516.00
8	124.00	556.00	525.00	525.00	469.00	516.00
9	124.03	556.00	548.00	525.00	469.00	516.00
10	124.10	556.00	556.00	525.00	469.00	516.00
11	127.38	556.00	556.00	525.00	469.00	516.00
12	328.08	556.00	556.00	525.00	469.00	516.00
13	328.10	556.00	556.00	525.00	469.00	516.00
14	344.49	556.00	556.00	525.00	469.00	516.00
15	360.90	556.00	556.00	525.00	469.00	516.00

DISTRIBUTION OF AVAILABLE STRENGTH ALONG EACH REINFORCEMENT LAYER



A = Front-end of reinforcement (at face of slope)
B = Rear-end of reinforcement
AB = L1 + L2 + L3 = Embedded length of reinforcement

Tavailable = Long-term strength of reinforcement
Tfe = Available front-end strength (e.g., connection to facing)

L1 = Front-end 'pullout' length
L2 = Rear-end pullout length
Tavailable prevails along L3

Factor of safety on resistance to pullout on either end of reinforcement, Fs-po = 1.50

Reinforcement Layer #	Designated Name	Height Relative to Toe [ft]	L [ft]	L1 [ft]	L2 [ft]	L3 [ft]	Tfe [lb/ft]	Tavailable [lb/ft]
1	---	1.15	24.00	6.30	14.40	3.30	2028.45	2983.01
2	---	3.45	24.00	6.96	15.62	1.43	2028.45	2983.01
3	---	5.75	24.00	6.46	17.06	0.48	2028.45	2983.01
4	---	8.05	24.00	5.74	18.26	0.00	2028.45	2893.10 (*)
5	---	10.35	24.00	4.91	19.09	0.00	2028.45	2750.34 (*)
6	---	12.65	24.00	4.99	19.01	0.00	2028.45	2709.61 (*)
7	---	14.95	24.00	5.43	18.57	0.00	2028.45	2689.06 (*)
8	---	17.25	24.00	5.26	18.74	0.00	2028.45	2585.22 (*)
9	---	19.55	24.00	3.97	20.03	0.00	2028.45	2383.01 (*)
10	---	21.85	24.00	1.80	22.20	0.00	2028.45	2157.78 (*)

(*) This Tavailable is dictated by the pullout resistance capacity, which is smaller than the long-term strength of the reinforcement that is related to its specified yield strength.

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X , Y) [ft]		Exit Point (X , Y) [ft]		Critical Circle (Xc , Yc , R) [ft]			Fs	STATUS
1	100.00	525.00	100.00	525.00	100.00	525.00	0.00	N/A	#10 - Overhanging Cliff
2	119.75	556.00	65.92	526.33	84.58	556.15	35.18	2.07	
3	121.50	556.00	71.94	526.18	87.66	556.16	33.85	1.84	
4	123.25	556.01	75.08	526.06	89.85	556.02	33.40	1.71	
5	125.00	556.00	76.52	526.07	91.40	556.20	33.61	1.63	
6	126.75	556.00	76.12	526.30	92.67	556.10	34.09	1.56	
7	128.50	556.00	74.67	526.33	93.33	556.15	35.18	1.51	
8	130.25	556.00	78.02	526.06	95.50	556.10	34.75	1.48	
9	132.00	556.00	76.18	526.29	95.13	557.98	36.93	1.46	
10	133.75	556.00	76.32	526.20	95.73	559.03	38.14	1.46	
11	135.50	556.00	76.43	526.13	95.59	561.59	40.30	1.45 .	OK
12	137.25	556.00	74.60	526.33	95.25	563.70	42.70	1.45	
13	139.00	556.00	74.75	526.24	95.88	564.87	44.03	1.46	
14	140.75	556.00	73.67	526.04	95.16	568.01	47.15	1.46	
15	142.50	556.00	73.05	526.35	95.35	570.28	49.27	1.47	
16	144.25	556.00	73.19	526.26	95.52	572.68	51.51	1.48	
17	146.00	556.00	72.21	526.04	95.71	574.01	53.42	1.50	
18	147.75	556.00	71.54	526.36	95.88	576.58	55.81	1.51	
19	149.50	556.00	70.56	526.12	95.10	580.51	59.67	1.53	
20	151.25	556.00	70.73	526.04	95.21	583.43	62.39	1.54	
21	153.00	556.00	68.96	526.18	94.95	586.26	65.46	1.56	
22	154.75	556.00	69.13	526.10	95.61	587.81	67.15	1.58	
23	156.50	556.00	67.35	526.24	95.38	590.70	70.28	1.60	
24	158.25	556.00	68.55	526.34	96.34	592.75	71.99	1.63	
25	160.00	556.00	64.08	526.38	94.32	598.57	78.27	1.65	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points).

Exit Point #	Exit Point (X, Y) [ft]		Entry Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	64.73	526.15	140.75	556.00	90.56	572.11	52.72	1.50	
2	66.28	526.10	140.75	556.00	91.45	571.09	51.56	1.49	
3	67.15	526.43	139.00	556.00	91.69	568.89	49.04	1.48	
4	68.71	526.37	139.00	556.00	92.60	567.90	47.90	1.47	
5	70.39	526.25	137.25	556.00	92.86	565.75	45.44	1.46	
6	71.94	526.20	137.25	556.00	93.79	564.78	44.34	1.46	
7	73.65	526.05	137.25	556.00	94.31	564.68	43.81	1.46	
8	75.17	526.02	135.50	556.00	95.03	561.76	40.88	1.45	
. 9	76.43	526.13	135.50	556.00	95.59	561.59	40.30	1.45 .	OK
10	77.73	526.21	135.50	556.00	96.54	560.64	39.23	1.46	
11	79.05	526.28	135.50	556.00	97.12	560.42	38.63	1.46	
12	80.95	526.05	133.75	556.00	97.89	557.70	35.90	1.47	
13	82.32	526.09	133.75	556.00	98.51	557.43	35.27	1.48	
14	83.72	526.12	133.75	556.00	99.14	557.13	34.63	1.49	
15	85.14	526.13	133.75	556.00	99.78	556.80	33.98	1.51	
16	86.60	526.12	133.75	556.00	100.43	556.45	33.33	1.54	
17	88.14	526.08	135.50	556.00	101.59	557.24	33.93	1.56	
18	89.63	526.06	135.50	556.00	101.86	557.44	33.68	1.60	
19	91.16	526.03	135.50	556.00	102.53	557.00	32.99	1.63	
20	92.17	526.18	139.00	556.00	103.68	559.78	35.52	1.68	
21	93.75	526.13	137.25	556.00	103.92	557.92	33.38	1.72	
22	95.23	526.40	139.00	556.00	106.68	556.63	32.33	1.84	
23	96.98	526.91	154.75	556.00	118.11	556.86	36.66	2.99	
24	100.00	525.00	100.00	525.00	100.00	525.00	0.00	N/A	#10 - Overhanging Cliff
25	100.00	525.00	100.00	525.00	100.00	525.00	0.00	N/A	#10 - Overhanging Cliff

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.45

Critical Circle: Xc = 95.59[ft], Yc = 561.59[ft], R = 40.30[ft]. (Number of slices used = 61)

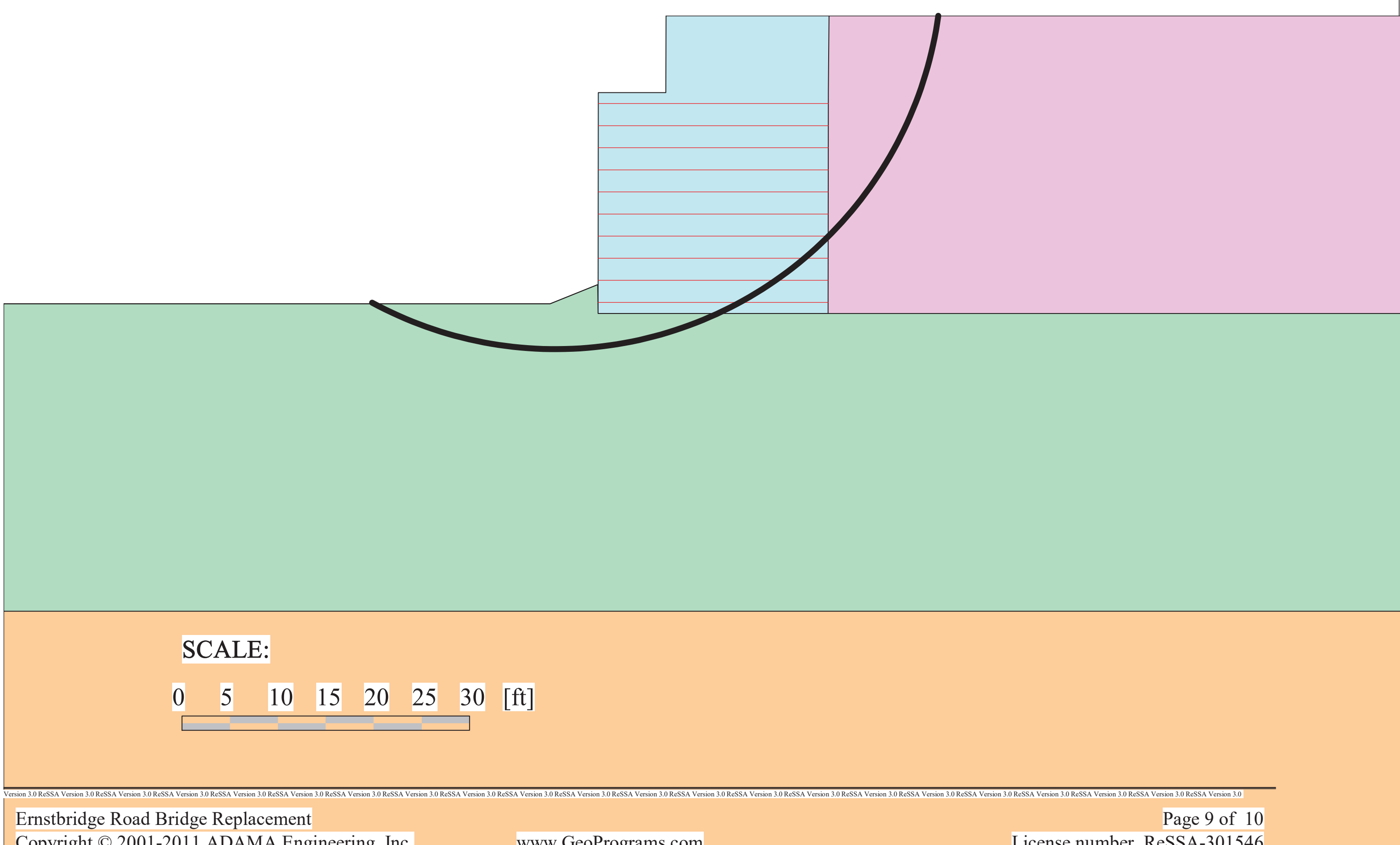
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

N O T C O N D U C T E D

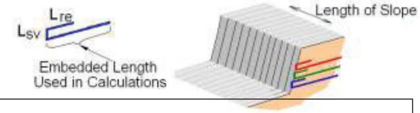
Three-Part Wedge Stability Analysis

N O T C O N D U C T E D

REINFORCEMENT LAYOUT: DRAWING



REINFORCEMENT LAYOUT: TABULATED DATA & QUANTITIES



Layer #	Reinf. Type #	Metallic Mat Designated Name	Height Relative to Toe [ft]	Embedded Length [ft]	Covergae Ratio, Rc	(X, Y) front [ft]		(X, Y) rear [ft]		Lsv * [ft]	Lre [ft]
1	1	---	1.15	24.00	0.07	328.09	1723.59	352.09	1723.59	0.00	0.00
2	1	---	3.45	24.00	0.07	328.09	1725.89	352.09	1725.89	0.00	0.00
3	1	---	5.75	24.00	0.07	328.09	1728.19	352.09	1728.19	0.00	0.00
4	1	---	8.05	24.00	0.07	328.09	1730.49	352.09	1730.49	0.00	0.00
5	1	---	10.35	24.00	0.07	328.10	1732.79	352.10	1732.79	0.00	0.00
6	1	---	12.65	24.00	0.07	328.10	1735.09	352.10	1735.09	0.00	0.00
7	1	---	14.95	24.00	0.07	328.10	1737.39	352.10	1737.39	0.00	0.00
8	1	---	17.25	24.00	0.07	328.11	1739.69	352.11	1739.69	0.00	0.00
9	1	---	19.55	24.00	0.07	328.11	1741.99	352.11	1741.99	0.00	0.00
10	1	---	21.85	24.00	0.07	328.11	1744.29	352.11	1744.29	0.00	0.00

* Vertical distance between layers.

QUANTITIES

Reinf. Type #	Designated Name	Coverage Ratio	Area of reinforcemnt [ft²] / length of slope [ft]
1	---	0.07	16.80

Ernstbridge Road Bridge Replacement

PROJECT IDENTIFICATION

Title:	Ernstbridge Road Bridge Replacement
Project Number:	N1185278 -
Client:	WSP
Designer:	JDD
Station Number:	East Abutment

Description:

H=22 feet exposed. 2H:1V toe Hs=2'. Abutment 5 feet back and 8 feet tall. Cu values based on CPT data. L=0.9H Short Term

Company's information:

Name:
Street:
,
Telephone #:
Fax #:
E-Mail:

Original file path and name: N:\Project ..... alculations-Analyses\MSE\East Abutment L=8H ST.MSE
Original date and time of creating this file: Wed Apr 03 17:01:55 2019

PROGRAM MODE: Analysis of a General Slope using METALLIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

=========== Soil Layer #: ===========	Unit weight, γ [lb/ft ³]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ²]
....1....... Reinforced Soil.................................	120.0	34.0	0.0
....2....... Retained Soil.....................................	125.0	0.0	2000.0
....3....... Foundation Soil.................................	124.0	0.0	1500.0
....4.......Silty Sand...	128.0	35.0	0.0

REINFORCEMENT

Reinforcement Type #	Metal Mat Designated Name	Yield Strength of Steel, Fy [kips/in.²]	Design Cross-Section Area per Mat, Ac [inch²]	Gross Width of Mat, b [inch]	Yield Strength Reduction Factor, RFy	Additional Reduction Factor, RFa	Coverage Ratio, Rc Rc = b / Sh
1	---	65.26	0.16	1.97	1.49	1.00	0.07

Interaction Parameters Type #	Metal Mat Designated Name	== Direct Sliding == Cds-phi	Cds-c	====== Pullout ====== F* top	F* @19.7ft.	Alpha	Thickness of Transverse Bars, t [in.]	Distance Between Transverse Bars, St [in.]
1	---	1.18	0.00	1.80	0.62	1.00	0.39	11.81

Relative Orientation of Reinforcement Force, ROR = 0.00. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER
Unit weight of water = 62.45 [lb/ft ³]
Water pressure is defined by phreatic surface in Effective Stress Analysis.

SEISMICITY

Not Applicable

DRAWING OF SPECIFIED GEOMETRY - COMPLEX - Quick Input

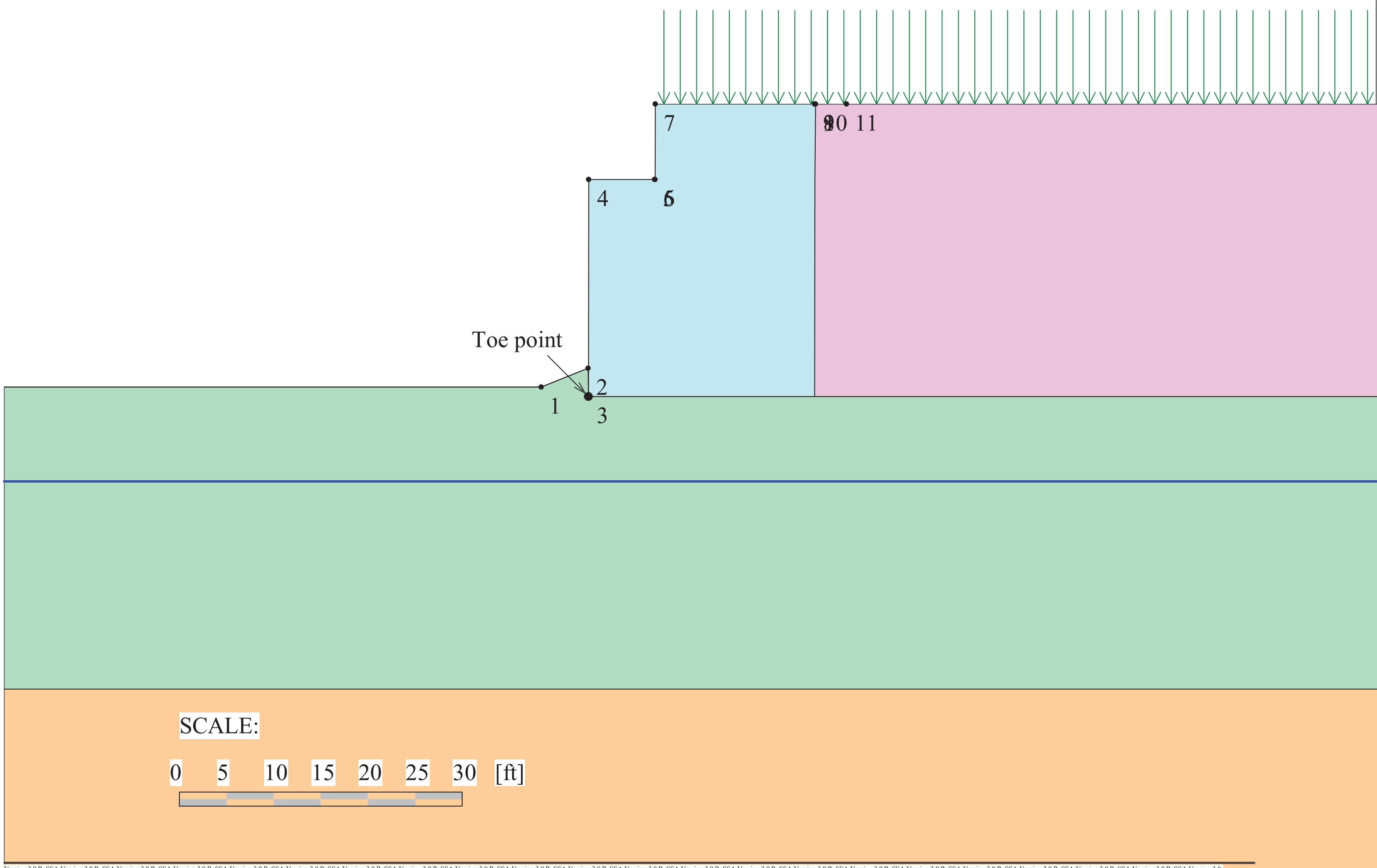
-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.
-- Xw,Yw represents the coordinates of phreatic surface.

GEOMETRY
Soil profile contains 4 layers (see details in next page)

WATER GEOMETRY
Phreatic line was specified.

UNIFORM SURCHARGE
Load Q1 = 250.00 [lb/ft²] inclined from verical at 0.00 degrees, starts at X1s = 108.00 and ends at X1e = 1100.03 [ft].
Surcharge load, Q2....................................None
Surcharge load, Q3....................................None

STRIP LOAD
............................None...................................



TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 4 layers. Coordinates in [ft.]
Water was described by phreatic line.

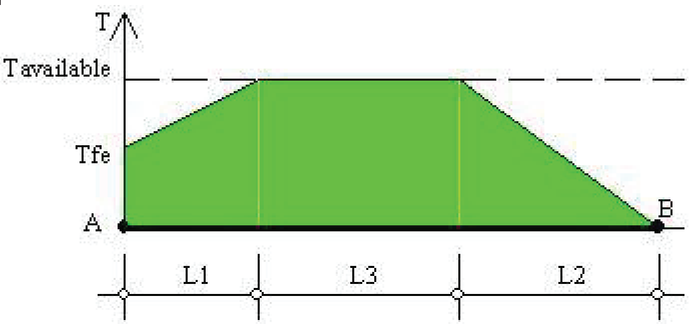
	#	Xi	Yi
Top of Layer 1	1	95.00	526.00
	2	99.97	528.00
	3	100.00	525.00
	4	100.03	548.00
	5	107.03	548.00
	6	107.07	548.00
	7	107.10	556.00
Top of Layer 2	8	95.00	526.00
	9	99.97	528.00
	10	100.00	525.00
	11	124.00	525.00
	12	124.03	548.00
	13	124.10	556.00
	14	127.38	556.00
Top of Layer 3	15	95.00	526.00
	16	99.97	528.00
	17	100.00	525.00
Top of Layer 4	18	328.08	469.00
	19	344.49	469.00
Top of Phreatic Line	21	328.10	516.00
	22	360.90	516.00

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 4 layers. Coordinates in [ft.]
Water was described by phreatic line. Y values are tabulated in the right most column.

#	X	Y1	Y2	Y3	Y4	(phreatic) Yw
1	95.00	526.00	526.00	526.00	469.00	516.00
2	99.97	528.00	528.00	528.00	469.00	516.00
3	100.00	525.00	525.00	525.00	469.00	516.00
4	100.03	548.00	525.00	525.00	469.00	516.00
5	107.03	548.00	525.00	525.00	469.00	516.00
6	107.07	548.00	525.00	525.00	469.00	516.00
7	107.10	556.00	525.00	525.00	469.00	516.00
8	124.00	556.00	525.00	525.00	469.00	516.00
9	124.03	556.00	548.00	525.00	469.00	516.00
10	124.10	556.00	556.00	525.00	469.00	516.00
11	127.38	556.00	556.00	525.00	469.00	516.00
12	328.08	556.00	556.00	525.00	469.00	516.00
13	328.10	556.00	556.00	525.00	469.00	516.00
14	344.49	556.00	556.00	525.00	469.00	516.00
15	360.90	556.00	556.00	525.00	469.00	516.00

DISTRIBUTION OF AVAILABLE STRENGTH ALONG EACH REINFORCEMENT LAYER



A = Front-end of reinforcement (at face of slope)
B = Rear-end of reinforcement
AB = L1 + L2 + L3 = Embedded length of reinforcement

Tavailable = Long-term strength of reinforcement
Tfe = Available front-end strength (e.g., connection to facing)

L1 = Front-end 'pullout' length
L2 = Rear-end pullout length
Tavailable prevails along L3

Factor of safety on resistance to pullout on either end of reinforcement, Fs-po = 1.50

Reinforcement Layer #	Designated Name	Height Relative to Toe [ft]	L [ft]	L1 [ft]	L2 [ft]	L3 [ft]	Tfe [lb/ft]	Tavailable [lb/ft]
1	---	1.15	24.00	6.30	14.40	3.30	2028.45	2983.01
2	---	3.45	24.00	6.96	15.62	1.43	2028.45	2983.01
3	---	5.75	24.00	6.46	17.06	0.48	2028.45	2983.01
4	---	8.05	24.00	5.74	18.26	0.00	2028.45	2893.10 (*)
5	---	10.35	24.00	4.91	19.09	0.00	2028.45	2750.34 (*)
6	---	12.65	24.00	4.99	19.01	0.00	2028.45	2709.61 (*)
7	---	14.95	24.00	5.43	18.57	0.00	2028.45	2689.06 (*)
8	---	17.25	24.00	5.26	18.74	0.00	2028.45	2585.22 (*)
9	---	19.55	24.00	3.97	20.03	0.00	2028.45	2383.01 (*)
10	---	21.85	24.00	1.80	22.20	0.00	2028.45	2157.78 (*)

(*) This Tavailable is dictated by the pullout resistance capacity, which is smaller than the long-term strength of the reinforcement that is related to its specified yield strength.

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X , Y) [ft]		Exit Point (X , Y) [ft]		Critical Circle (Xc , Yc , R) [ft]			Fs	STATUS
1	118.00	556.00	62.21	526.19	82.07	556.14	35.93	4.83	
2	123.09	556.00	74.87	526.06	89.65	556.05	33.43	3.02	
3	128.17	556.00	74.47	526.32	93.07	556.08	35.10	2.97	
4	133.25	556.00	70.20	526.51	94.62	556.45	38.64	2.62	
5	138.34	556.00	70.58	526.23	97.89	556.06	40.44	2.43	
6	143.42	556.00	70.31	526.53	100.90	556.07	42.52	2.37	
7	148.50	556.00	62.46	526.04	98.89	559.94	49.76	2.33	
8	153.59	556.00	57.77	526.66	99.43	561.73	54.46	2.30	
9	158.67	556.00	53.99	526.21	99.74	564.25	59.50	2.27	
10	163.75	556.00	49.36	526.77	99.81	567.80	65.02	2.25	
11	168.84	556.00	45.78	526.06	99.92	571.39	70.61	2.23	
12	173.92	556.00	41.24	526.51	100.08	575.01	76.25	2.22	
13	179.00	556.00	36.71	526.96	100.26	578.67	81.93	2.21	
14	184.08	556.00	32.17	527.42	100.48	582.34	87.66	2.21	
15	189.17	556.00	28.95	526.26	100.72	586.04	93.41	2.20	
16	194.25	556.00	24.49	526.61	100.98	589.76	99.19	2.20	
17	199.33	556.00	20.04	526.96	101.26	593.49	104.99	2.20	
18	204.42	556.00	15.59	527.31	101.56	597.23	110.82	2.19	
19	209.50	556.00	11.13	527.66	101.87	600.99	116.66	2.19	
20	214.58	556.00	8.32	526.01	102.19	604.75	122.52	2.19	
21	219.67	556.00	3.95	526.26	102.52	608.53	128.39	2.19	
22	224.75	556.00	-0.43	526.51	102.86	612.31	134.27	2.19 .	OK
23	229.83	556.00	-1.60	527.95	105.34	614.39	137.51	2.19	
24	234.92	556.00	-0.99	527.20	107.82	616.47	140.75	2.20	
25	240.00	556.00	-0.39	526.47	110.31	618.56	143.99	2.20	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points).

Exit Point #	Exit Point X [ft]	Exit Point Y [ft]	Entry Point X [ft]	Entry Point Y [ft]	Critical Circle Xc [ft]	Critical Circle Yc [ft]	Critical Circle R [ft]	Fs	STATUS
. 1	-0.43	526.51	224.75	556.00	102.86	612.31	134.27	2.19	. On extreme X-exit
2	3.95	526.26	219.67	556.00	102.52	608.53	128.39	2.19	
3	8.32	526.01	214.58	556.00	102.19	604.75	122.52	2.19	
4	11.74	526.92	214.58	556.00	104.33	603.05	119.88	2.19	
5	16.19	526.58	209.50	556.00	104.01	599.29	114.03	2.19	
6	20.63	526.24	204.42	556.00	103.71	595.54	108.19	2.20	
7	23.68	527.61	199.33	556.00	103.42	591.81	102.38	2.20	
8	28.21	527.16	194.25	556.00	103.15	588.08	96.58	2.20	
9	33.29	526.06	194.25	556.00	105.33	586.41	93.98	2.20	
10	36.58	527.12	189.17	556.00	105.08	582.71	88.23	2.21	
11	41.18	526.59	184.08	556.00	104.86	579.03	82.50	2.21	
12	45.77	526.07	179.00	556.00	104.67	575.37	76.81	2.22	
13	49.35	526.79	173.92	556.00	104.52	571.75	71.17	2.23	
14	53.50	526.81	173.92	556.00	106.75	570.12	68.64	2.24	
15	58.25	526.09	168.84	556.00	106.33	567.72	63.60	2.26	
16	62.13	526.40	163.75	556.00	105.65	566.24	59.00	2.28	
17	66.14	526.53	158.67	556.00	105.04	564.38	54.28	2.30	
18	70.69	526.13	158.67	556.00	106.39	565.49	53.14	2.32	
19	74.98	526.02	153.59	556.00	105.59	563.78	48.62	2.35	
20	79.01	526.11	153.59	556.00	106.62	565.21	47.86	2.39	
21	82.96	526.23	148.50	556.00	105.69	563.21	43.41	2.42	
22	87.41	526.04	148.50	556.00	106.77	563.83	42.46	2.46	
23	91.38	526.13	148.50	556.00	108.29	563.34	40.88	2.53	
24	95.16	526.60	153.59	556.00	111.44	567.00	43.56	2.67	
25	100.00	525.00	100.00	525.00	100.00	525.00	0.00	N/A	#10 - Overhanging Cliff

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 2.19

Critical Circle: Xc = 102.86[ft], Yc = 612.31[ft], R = 134.27[ft]. (Number of slices used = 59)

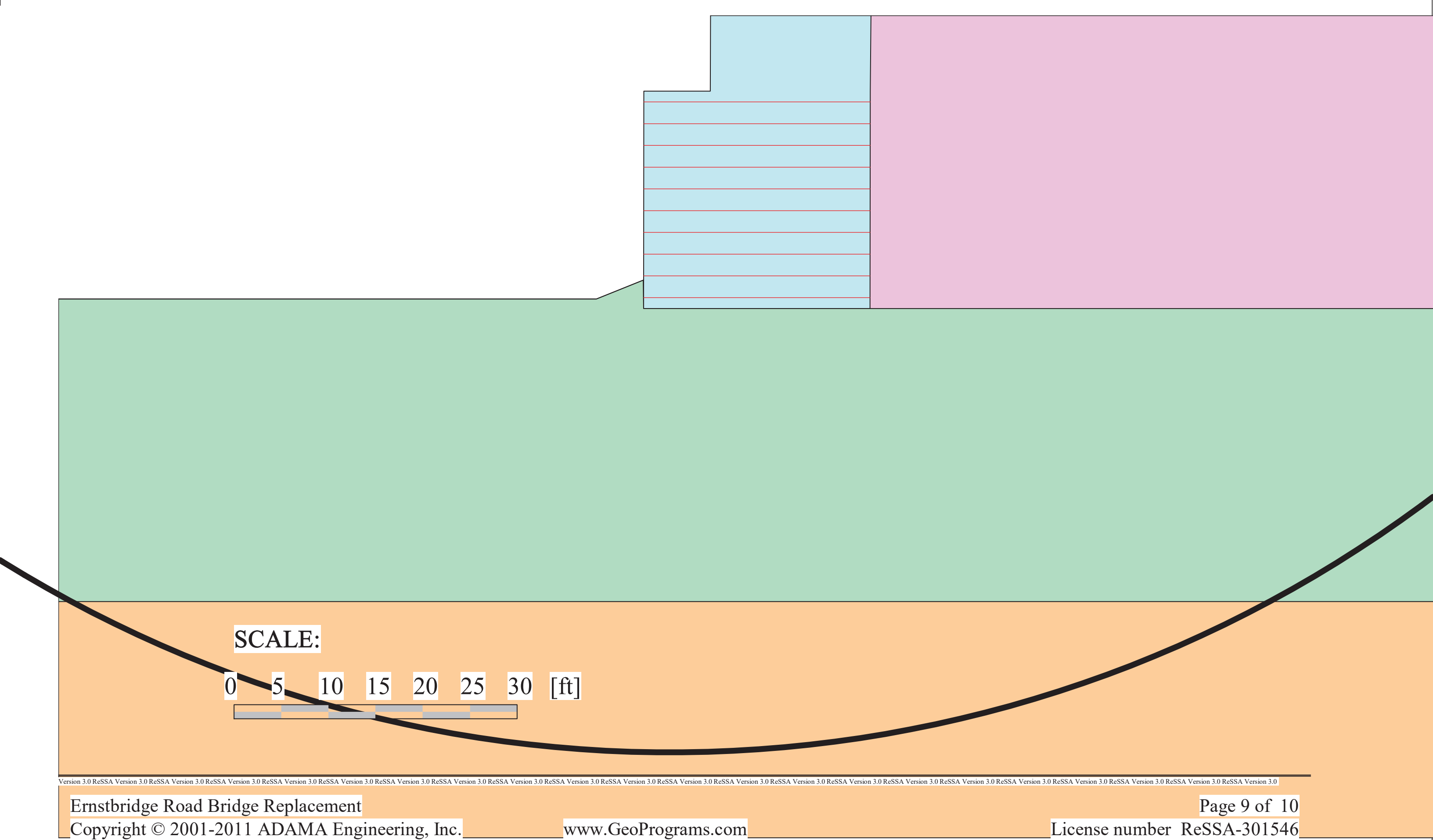
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

N O T C O N D U C T E D

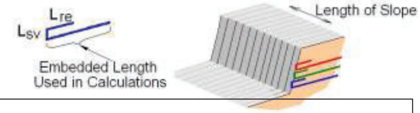
Three-Part Wedge Stability Analysis

N O T C O N D U C T E D

REINFORCEMENT LAYOUT: DRAWING



REINFORCEMENT LAYOUT: TABULATED DATA & QUANTITIES



Layer #	Reinf. Type #	Metallic Mat Designated Name	Height Relative to Toe [ft]	Embedded Length [ft]	Covergae Ratio, Rc	(X, Y) front [ft]		(X, Y) rear [ft]		Lsv * [ft]	Lre [ft]
1	1	---	1.15	24.00	0.07	328.09	1723.59	352.09	1723.59	0.00	0.00
2	1	---	3.45	24.00	0.07	328.09	1725.89	352.09	1725.89	0.00	0.00
3	1	---	5.75	24.00	0.07	328.09	1728.19	352.09	1728.19	0.00	0.00
4	1	---	8.05	24.00	0.07	328.09	1730.49	352.09	1730.49	0.00	0.00
5	1	---	10.35	24.00	0.07	328.10	1732.79	352.10	1732.79	0.00	0.00
6	1	---	12.65	24.00	0.07	328.10	1735.09	352.10	1735.09	0.00	0.00
7	1	---	14.95	24.00	0.07	328.10	1737.39	352.10	1737.39	0.00	0.00
8	1	---	17.25	24.00	0.07	328.11	1739.69	352.11	1739.69	0.00	0.00
9	1	---	19.55	24.00	0.07	328.11	1741.99	352.11	1741.99	0.00	0.00
10	1	---	21.85	24.00	0.07	328.11	1744.29	352.11	1744.29	0.00	0.00

* Vertical distance between layers.

QUANTITIES

Reinf. Type #	Designated Name	Coverage Ratio	Area of reinforcemnt [ft²] / length of slope [ft]
1	---	0.07	16.80

Ernstbridge Road Bridge Replacement

PROJECT IDENTIFICATION

Title:	Ernstbridge Road Bridge Replacement
Project Number:	N1185278 -
Client:	WSP
Designer:	JDD
Station Number:	East Abutment

Description:

H=22 feet exposed. 2H:1V toe Hs=2'. Abutment 5 feet back and 8 feet tall. L=0.9H

Company's information:

Name:
Street:
,
Telephone #:
Fax #:
E-Mail:

Original file path and name:
Original date and time of creating this file: Wed Apr 03 16:46:19 2019

PROGRAM MODE: Analysis of a General Slope using METALLIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

=========== Soil Layer #: ===========	Unit weight, γ [lb/ft ³]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ²]
....1....... Reinforced Soil.................................	120.0	34.0	0.0
....2....... Retained Soil.....................................	125.0	28.0	50.0
....3....... Foundation Soil.................................	124.0	28.0	100.0
....4.......SIlty Sand...	128.0	35.0	0.0

REINFORCEMENT

Reinforcement Type #	Metal Mat Designated Name	Yield Strength of Steel, Fy [kips/in.²]	Design Cross-Section Area per Mat, Ac [inch²]	Gross Width of Mat, b [inch]	Yield Strength Reduction Factor, RFy	Additional Reduction Factor, RFa	Coverage Ratio, Rc Rc = b / Sh
1	---	65.26	0.16	1.97	1.49	1.00	0.07

Interaction Parameters Type #	Metal Mat Designated Name	== Direct Sliding == Cds-phi	Cds-c	====== Pullout ====== F* top	F* @19.7ft.	Alpha	Thickness of Transverse Bars, t [in.]	Distance Between Transverse Bars, St [in.]
1	---	1.18	0.00	1.80	0.62	1.00	0.39	11.81

Relative Orientation of Reinforcement Force, ROR = 0.00. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER

Unit weight of water = 62.45 [lb/ft ³]
Water pressure is defined by phreatic surface in Effective Stress Analysis.

SEISMICITY

Not Applicable

DRAWING OF SPECIFIED GEOMETRY - COMPLEX - Quick Input

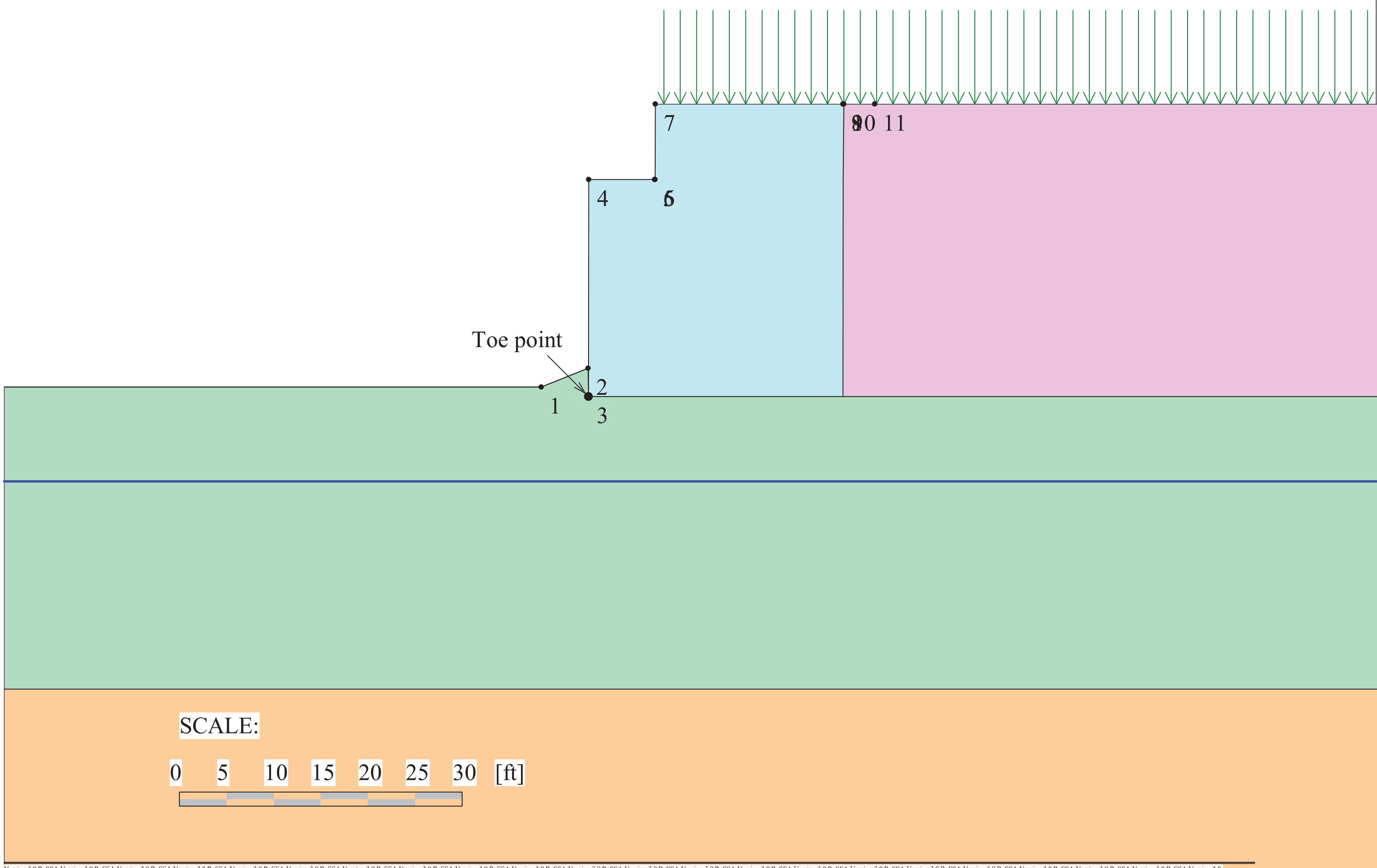
-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.
-- Xw,Yw represents the coordinates of phreatic surface.

GEOMETRY
Soil profile contains 4 layers (see details in next page)

WATER GEOMETRY
Phreatic line was specified.

UNIFORM SURCHARGE
Load Q1 = 250.00 [lb/ft²] inclined from verical at 0.00 degrees, starts at X1s = 108.00 and ends at X1e = 1100.03 [ft].
Surcharge load, Q2....................................None
Surcharge load, Q3....................................None

STRIP LOAD
............................None...................................



TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 4 layers. Coordinates in [ft.]
Water was described by phreatic line.

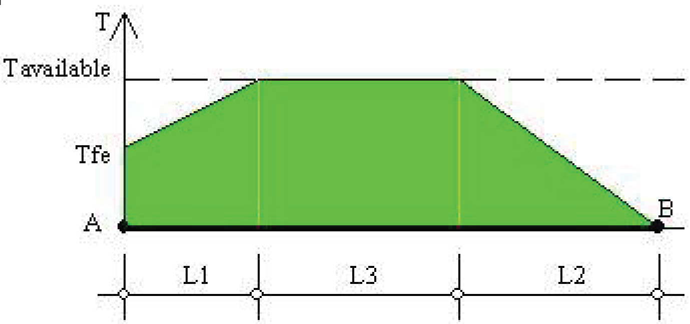
	#	Xi	Yi
Top of Layer 1	1	95.00	526.00
	2	99.97	528.00
	3	100.00	525.00
	4	100.03	548.00
	5	107.03	548.00
	6	107.07	548.00
	7	107.10	556.00
Top of Layer 2	8	95.00	526.00
	9	99.97	528.00
	10	100.00	525.00
	11	127.00	525.00
	12	127.03	548.00
	13	127.10	556.00
	14	130.38	556.00
Top of Layer 3	15	95.00	526.00
	16	99.97	528.00
	17	100.00	525.00
Top of Layer 4	18	328.08	469.00
	19	344.49	469.00
Top of Phreatic Line	21	328.10	516.00
	22	360.90	516.00

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 4 layers. Coordinates in [ft.]
Water was described by phreatic line. Y values are tabulated in the right most column.

#	X	Y1	Y2	Y3	Y4	(phreatic) Yw
1	95.00	526.00	526.00	526.00	469.00	516.00
2	99.97	528.00	528.00	528.00	469.00	516.00
3	100.00	525.00	525.00	525.00	469.00	516.00
4	100.03	548.00	525.00	525.00	469.00	516.00
5	107.03	548.00	525.00	525.00	469.00	516.00
6	107.07	548.00	525.00	525.00	469.00	516.00
7	107.10	556.00	525.00	525.00	469.00	516.00
8	127.00	556.00	525.00	525.00	469.00	516.00
9	127.03	556.00	548.00	525.00	469.00	516.00
10	127.10	556.00	556.00	525.00	469.00	516.00
11	130.38	556.00	556.00	525.00	469.00	516.00
12	328.08	556.00	556.00	525.00	469.00	516.00
13	328.10	556.00	556.00	525.00	469.00	516.00
14	344.49	556.00	556.00	525.00	469.00	516.00
15	360.90	556.00	556.00	525.00	469.00	516.00

DISTRIBUTION OF AVAILABLE STRENGTH ALONG EACH REINFORCEMENT LAYER



A = Front-end of reinforcement (at face of slope)
B = Rear-end of reinforcement
AB = L1 + L2 + L3 = Embedded length of reinforcement

Tavailable = Long-term strength of reinforcement
Tfe = Available front-end strength (e.g., connection to facing)

L1 = Front-end 'pullout' length
L2 = Rear-end pullout length
Tavailable prevails along L3

Factor of safety on resistance to pullout on either end of reinforcement, Fs-po = 1.50

Reinforcement Layer #	Designated Name	Height Relative to Toe [ft]	L [ft]	L1 [ft]	L2 [ft]	L3 [ft]	Tfe [lb/ft]	Tavailable [lb/ft]
1	---	1.15	27.00	6.30	14.40	6.30	2028.45	2983.01
2	---	3.45	27.00	6.96	15.62	4.43	2028.45	2983.01
3	---	5.75	27.00	6.46	17.03	3.51	2028.45	2983.01
4	---	8.05	27.00	6.33	18.73	1.93	2028.45	2983.01
5	---	10.35	27.00	6.35	20.65	0.00	2028.45	2961.48 (*)
6	---	12.65	27.00	6.58	20.42	0.00	2028.45	2927.83 (*)
7	---	14.95	27.00	7.48	19.52	0.00	2028.45	2938.76 (*)
8	---	17.25	27.00	7.91	19.09	0.00	2028.45	2864.86 (*)
9	---	19.55	27.00	7.16	19.84	0.00	2028.45	2667.87 (*)
10	---	21.85	27.00	3.97	23.03	0.00	2028.45	2313.92 (*)

(*) This Tavailable is dictated by the pullout resistance capacity, which is smaller than the long-term strength of the reinforcement that is related to its specified yield strength.

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X, Y) [ft]		Exit Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	100.00	525.00	100.00	525.00	100.00	525.00	0.00	N/A	#10 - Overhanging Cliff
2	119.75	556.00	65.92	526.33	84.58	556.15	35.18	2.28	
3	121.50	556.00	71.94	526.18	87.66	556.16	33.85	2.03	
4	123.25	556.01	75.08	526.06	89.85	556.02	33.40	1.87	
5	125.00	556.00	73.61	526.08	90.48	556.19	34.52	1.77	
6	126.75	556.00	76.12	526.30	92.67	556.10	34.09	1.69	
7	128.50	556.00	74.67	526.33	93.33	556.15	35.18	1.63	
8	130.25	556.00	75.08	526.08	94.27	556.51	35.98	1.59	
9	132.00	556.00	76.24	526.28	96.18	556.05	35.82	1.56	
10	133.75	556.00	76.34	526.20	96.10	558.34	37.73	1.55	
11	135.50	556.00	75.21	526.00	95.78	560.24	39.95	1.54	
12	137.25	556.00	74.64	526.33	96.04	562.08	41.66	1.53	
13	139.00	556.00	73.56	526.11	95.75	564.11	44.00	1.53 .	OK
14	140.75	556.00	73.68	526.03	95.58	567.06	46.50	1.53	
15	142.50	556.00	73.09	526.36	96.23	568.27	47.88	1.53	
16	144.25	556.00	73.23	526.27	96.44	570.53	49.97	1.54	
17	146.00	556.00	72.21	526.04	95.71	574.01	53.42	1.55	
18	147.75	556.00	71.54	526.36	95.88	576.58	55.81	1.56	
19	149.50	556.00	70.60	526.11	96.12	577.84	57.68	1.57	
20	151.25	556.00	70.76	526.03	96.28	580.57	60.21	1.59	
21	153.00	556.00	70.04	526.36	96.41	583.46	62.90	1.60	
22	154.75	556.00	67.18	526.33	95.28	587.46	67.28	1.62	
23	156.50	556.00	67.35	526.24	95.38	590.70	70.28	1.64	
24	158.25	556.00	65.59	526.40	95.75	591.80	72.02	1.66	
25	160.00	556.00	67.73	526.08	96.73	593.88	73.74	1.68	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points).

Exit Point #	Exit Point (X , Y) [ft]		Entry Point (X , Y) [ft]		Critical Circle (Xc , Yc , R) [ft]			Fs	STATUS
1	64.45	526.32	142.50	556.00	91.67	572.20	53.35	1.56	
2	66.17	526.16	142.50	556.00	92.13	572.31	52.95	1.55	
3	67.67	526.15	140.75	556.00	92.78	569.06	49.72	1.54	
4	68.59	526.44	140.75	556.00	93.26	569.09	49.26	1.54	
5	70.15	526.39	140.75	556.00	94.17	568.10	48.13	1.53	
6	71.87	526.23	140.75	556.00	94.66	568.07	47.65	1.53	
. 7	73.56	526.11	139.00	556.00	95.75	564.11	44.00	1.53 .	OK
8	74.77	526.24	139.00	556.00	96.28	564.02	43.47	1.53	
9	76.03	526.36	139.00	556.00	97.21	563.07	42.38	1.53	
10	78.01	526.06	139.00	556.00	97.76	562.93	41.82	1.54	
11	79.30	526.15	139.00	556.00	98.31	562.75	41.25	1.54	
12	80.61	526.21	139.00	556.00	98.87	562.55	40.67	1.55	
13	81.97	526.26	139.00	556.00	99.84	561.55	39.55	1.57	
14	83.92	526.02	137.25	556.00	100.63	558.72	36.72	1.58	
15	85.39	526.01	139.00	556.00	101.00	561.01	38.33	1.60	
16	86.81	526.03	139.00	556.00	101.60	560.70	37.69	1.63	
17	88.26	526.03	139.00	556.00	102.21	560.35	37.05	1.66	
18	89.73	526.02	139.00	556.00	102.83	559.97	36.39	1.69	
19	90.65	526.21	140.75	556.00	103.51	561.61	37.66	1.73	
20	92.17	526.18	139.00	556.00	103.68	559.78	35.52	1.77	
21	93.75	526.13	142.50	556.00	105.21	562.14	37.79	1.82	
22	95.26	526.38	140.75	556.00	107.14	557.89	33.67	1.93	
23	96.98	526.91	154.75	556.00	118.11	556.86	36.66	2.99	
24	100.00	525.00	100.00	525.00	100.00	525.00	0.00	N/A	#10 - Overhanging Cliff
25	100.00	525.00	100.00	525.00	100.00	525.00	0.00	N/A	#10 - Overhanging Cliff

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.53

Critical Circle: Xc = 95.75[ft], Yc = 564.11[ft], R = 44.00[ft]. (Number of slices used = 61)

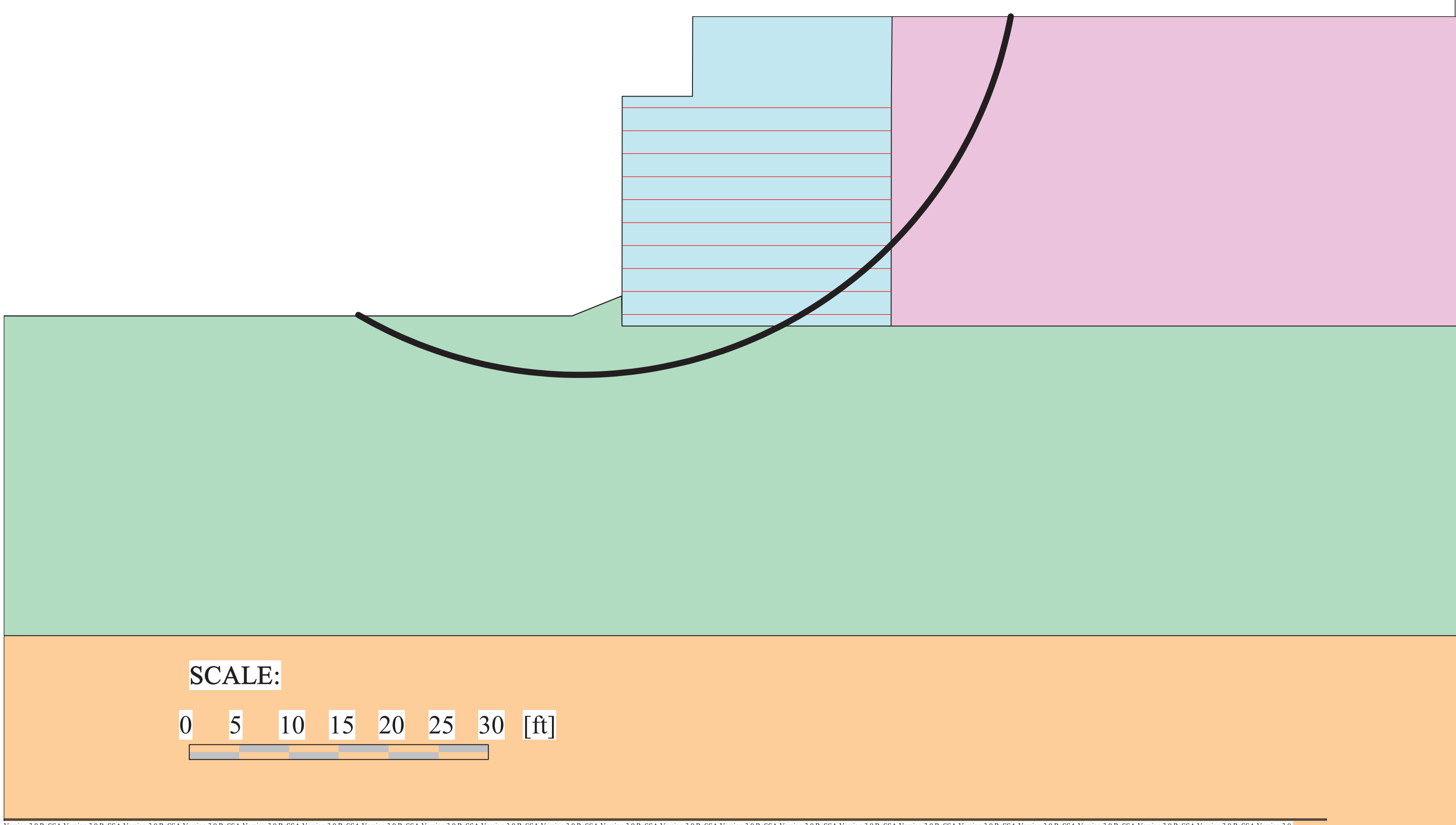
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

N O T C O N D U C T E D

Three-Part Wedge Stability Analysis

N O T C O N D U C T E D

REINFORCEMENT LAYOUT: DRAWING



Ernstbridge Road Bridge Replacement

PROJECT IDENTIFICATION

Title:	Ernstbridge Road Bridge Replacement
Project Number:	N1185278 -
Client:	WSP
Designer:	JDD
Station Number:	East Abutment

Description:
H=22 feet exposed. 2H:1V toe Hs=2'. Abutment 5 feet back and 8 feet tall. L=0.9H. Short-term

Company's information:

Name:
Street:
,
Telephone #:
Fax #:
E-Mail:

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PROGRAM MODE: Analysis of a General Slope using METALLIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

=========== Soil Layer #: ===========	Unit weight, γ [lb/ft ³]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ²]
....1....... Reinforced Soil................................	120.0	34.0	0.0
....2....... Retained Soil....................................	125.0	0.0	2000.0
....3....... Foundation Soil................................	124.0	0.0	1500.0
....4.......SIlty Sand...	128.0	35.0	0.0

REINFORCEMENT

Reinforcement Type #	Metal Mat Designated Name	Yield Strength of Steel, Fy [kips/in.²]	Design Cross-Section Area per Mat, Ac [inch²]	Gross Width of Mat, b [inch]	Yield Strength Reduction Factor, RFy	Additional Reduction Factor, RFa	Coverage Ratio, Rc Rc = b / Sh
1	---	65.26	0.16	1.97	1.49	1.00	0.07

Interaction Parameters Type #	Metal Mat Designated Name	== Direct Sliding == Cds-phi	Cds-c	====== Pullout ====== F* top	F* @19.7ft.	Alpha	Thickness of Transverse Bars, t [in.]	Distance Between Transverse Bars, St [in.]
1	---	1.18	0.00	1.80	0.62	1.00	0.39	11.81

Relative Orientation of Reinforcement Force, ROR = 0.00. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER

Unit weight of water = 62.45 [lb/ft ³]
Water pressure is defined by phreatic surface in Effective Stress Analysis.

SEISMICITY

Not Applicable

DRAWING OF SPECIFIED GEOMETRY - COMPLEX - Quick Input

-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.
-- Xw,Yw represents the coordinates of phreatic surface.

GEOMETRY

Soil profile contains 4 layers (see details in next page)

WATER GEOMETRY

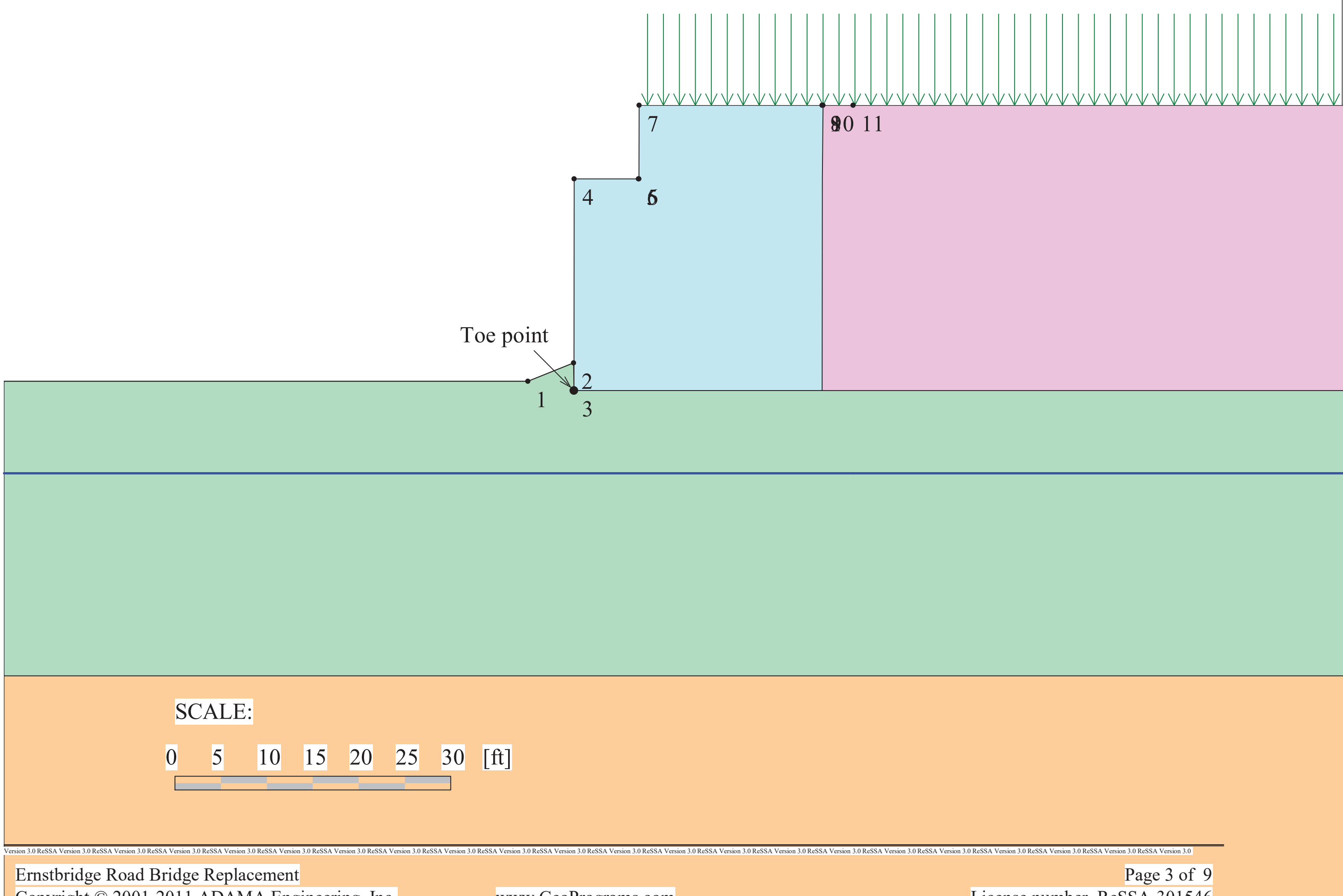
Phreatic line was specified.

UNIFORM SURCHARGE

Load Q1 = 250.00 [lb/ft²] inclined from verical at 0.00 degrees, starts at X1s = 108.00 and ends at X1e = 1100.03 [ft].
Surcharge load, Q2....................................None
Surcharge load, Q3....................................None

STRIP LOAD

............................None..................................



TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 4 layers. Coordinates in [ft.]
Water was described by phreatic line.

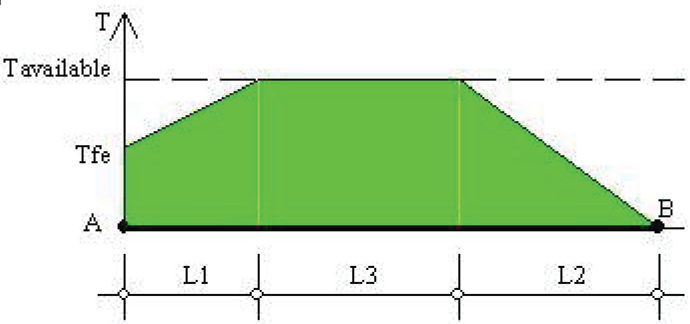
	#	Xi	Yi
Top of Layer 1	1	95.00	526.00
	2	99.97	528.00
	3	100.00	525.00
	4	100.03	548.00
	5	107.03	548.00
	6	107.07	548.00
	7	107.10	556.00
Top of Layer 2	8	95.00	526.00
	9	99.97	528.00
	10	100.00	525.00
	11	127.00	525.00
	12	127.03	548.00
	13	127.10	556.00
	14	130.38	556.00
Top of Layer 3	15	95.00	526.00
	16	99.97	528.00
	17	100.00	525.00
Top of Layer 4	18	328.08	469.00
	19	344.49	469.00
Top of Phreatic Line	21	328.10	516.00
	22	360.90	516.00

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 4 layers. Coordinates in [ft.]
Water was described by phreatic line. Y values are tabulated in the right most column.

#	X	Y1	Y2	Y3	Y4	(phreatic) Yw
1	95.00	526.00	526.00	526.00	469.00	516.00
2	99.97	528.00	528.00	528.00	469.00	516.00
3	100.00	525.00	525.00	525.00	469.00	516.00
4	100.03	548.00	525.00	525.00	469.00	516.00
5	107.03	548.00	525.00	525.00	469.00	516.00
6	107.07	548.00	525.00	525.00	469.00	516.00
7	107.10	556.00	525.00	525.00	469.00	516.00
8	127.00	556.00	525.00	525.00	469.00	516.00
9	127.03	556.00	548.00	525.00	469.00	516.00
10	127.10	556.00	556.00	525.00	469.00	516.00
11	130.38	556.00	556.00	525.00	469.00	516.00
12	328.08	556.00	556.00	525.00	469.00	516.00
13	328.10	556.00	556.00	525.00	469.00	516.00
14	344.49	556.00	556.00	525.00	469.00	516.00
15	360.90	556.00	556.00	525.00	469.00	516.00

DISTRIBUTION OF AVAILABLE STRENGTH ALONG EACH REINFORCEMENT LAYER



A = Front-end of reinforcement (at face of slope)
B = Rear-end of reinforcement
AB = L1 + L2 + L3 = Embedded length of reinforcement

Tavailable = Long-term strength of reinforcement
Tfe = Available front-end strength (e.g., connection to facing)

L1 = Front-end 'pullout' length
L2 = Rear-end pullout length
Tavailable prevails along L3

Factor of safety on resistance to pullout on either end of reinforcement, Fs-po = 1.50

Reinforcement Layer #	Designated Name	Height Relative to Toe [ft]	L [ft]	L1 [ft]	L2 [ft]	L3 [ft]	Tfe [lb/ft]	Tavailable [lb/ft]
1	---	1.15	27.00	6.30	14.40	6.30	2028.45	2983.01
2	---	3.45	27.00	6.96	15.62	4.43	2028.45	2983.01
3	---	5.75	27.00	6.46	17.03	3.51	2028.45	2983.01
4	---	8.05	27.00	6.33	18.73	1.93	2028.45	2983.01
5	---	10.35	27.00	6.35	20.65	0.00	2028.45	2961.48 (*)
6	---	12.65	27.00	6.58	20.42	0.00	2028.45	2927.83 (*)
7	---	14.95	27.00	7.48	19.52	0.00	2028.45	2938.76 (*)
8	---	17.25	27.00	7.91	19.09	0.00	2028.45	2864.86 (*)
9	---	19.55	27.00	7.16	19.84	0.00	2028.45	2667.87 (*)
10	---	21.85	27.00	3.97	23.03	0.00	2028.45	2313.92 (*)

(*) This Tavailable is dictated by the pullout resistance capacity, which is smaller than the long-term strength of the reinforcement that is related to its specified yield strength.

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X, Y) [ft]		Exit Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	118.00	556.00	62.21	526.19	82.07	556.14	35.93	5.25	
2	123.09	556.00	74.87	526.06	89.65	556.05	33.43	3.26	
3	128.17	556.00	74.47	526.32	93.07	556.08	35.10	2.88	
4	133.25	556.00	62.20	526.29	91.50	556.03	41.75	2.65	
5	138.34	556.00	70.58	526.23	97.89	556.06	40.44	2.45	
6	143.42	556.00	70.31	526.53	100.90	556.07	42.52	2.38	
7	148.50	556.00	62.49	526.01	99.81	557.31	48.71	2.34	
8	153.59	556.00	57.78	526.67	99.74	560.75	54.05	2.30	
9	158.67	556.00	53.97	526.22	99.43	565.35	59.97	2.27	
10	163.75	556.00	49.36	526.77	99.81	567.80	65.02	2.25	
11	168.84	556.00	49.87	526.15	102.16	569.76	68.09	2.24	
12	173.92	556.00	41.24	526.51	100.08	575.01	76.25	2.23	
13	179.00	556.00	36.71	526.96	100.26	578.67	81.93	2.22	
14	184.08	556.00	37.26	526.29	102.67	580.69	85.07	2.21	
15	189.17	556.00	28.95	526.26	100.72	586.04	93.41	2.20	
16	194.25	556.00	24.49	526.61	100.98	589.76	99.19	2.20	
17	199.33	556.00	20.04	526.96	101.26	593.49	104.99	2.20	
18	204.42	556.00	15.59	527.31	101.56	597.23	110.82	2.20	
19	209.50	556.00	11.13	527.66	101.87	600.99	116.66	2.19	
20	214.58	556.00	8.32	526.01	102.19	604.75	122.52	2.19	
21	219.67	556.00	3.95	526.26	102.52	608.53	128.39	2.19	
22	224.75	556.00	-0.43	526.51	102.86	612.31	134.27	2.19 .	OK
23	229.83	556.00	-1.60	527.95	105.34	614.39	137.51	2.19	
24	234.92	556.00	-0.99	527.20	107.82	616.47	140.75	2.20	
25	240.00	556.00	-0.39	526.47	110.31	618.56	143.99	2.20	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points).

Exit Point #	Exit Point (X , Y) [ft]		Entry Point (X , Y) [ft]		Critical Circle (Xc , Yc , R) [ft]			Fs	STATUS
. 1	-0.43	526.51	224.75	556.00	102.86	612.31	134.27	2.19	. On extreme X-exit
2	3.95	526.26	219.67	556.00	102.52	608.53	128.39	2.19	
3	7.29	527.26	219.67	556.00	104.66	606.82	125.74	2.19	
4	11.74	526.92	214.58	556.00	104.33	603.05	119.88	2.19	
5	16.19	526.58	209.50	556.00	104.01	599.29	114.03	2.20	
6	20.63	526.24	204.42	556.00	103.71	595.54	108.19	2.20	
7	23.68	527.61	199.33	556.00	103.42	591.81	102.38	2.20	
8	28.21	527.16	194.25	556.00	103.15	588.08	96.58	2.20	
9	33.29	526.06	194.25	556.00	105.33	586.41	93.98	2.20	
10	36.58	527.12	189.17	556.00	105.08	582.71	88.23	2.21	
11	41.18	526.59	184.08	556.00	104.86	579.03	82.50	2.21	
12	45.77	526.07	179.00	556.00	104.67	575.37	76.81	2.22	
13	49.35	526.79	173.92	556.00	104.52	571.75	71.17	2.23	
14	53.50	526.81	173.92	556.00	106.75	570.12	68.64	2.24	
15	58.25	526.09	168.84	556.00	106.33	567.72	63.60	2.26	
16	62.13	526.40	163.75	556.00	105.65	566.24	59.00	2.28	
17	66.55	526.11	163.75	556.00	107.29	566.63	57.45	2.30	
18	70.69	526.13	158.67	556.00	106.39	565.49	53.14	2.33	
19	74.98	526.02	153.59	556.00	105.59	563.78	48.62	2.36	
20	79.01	526.11	153.59	556.00	106.62	565.21	47.86	2.39	
21	82.96	526.24	148.50	556.00	106.07	562.39	42.91	2.43	
22	87.41	526.05	148.50	556.00	107.57	562.21	41.40	2.49	
23	91.45	526.09	153.59	556.00	109.97	567.12	45.01	2.56	
24	95.16	526.60	153.59	556.00	111.44	567.00	43.56	2.71	
25	100.00	525.00	100.00	525.00	100.00	525.00	0.00	N/A	#10 - Overhanging Cliff

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 2.19
Critical Circle: Xc = 102.86[ft], Yc = 612.31[ft], R = 134.27[ft]. (Number of slices used = 59)

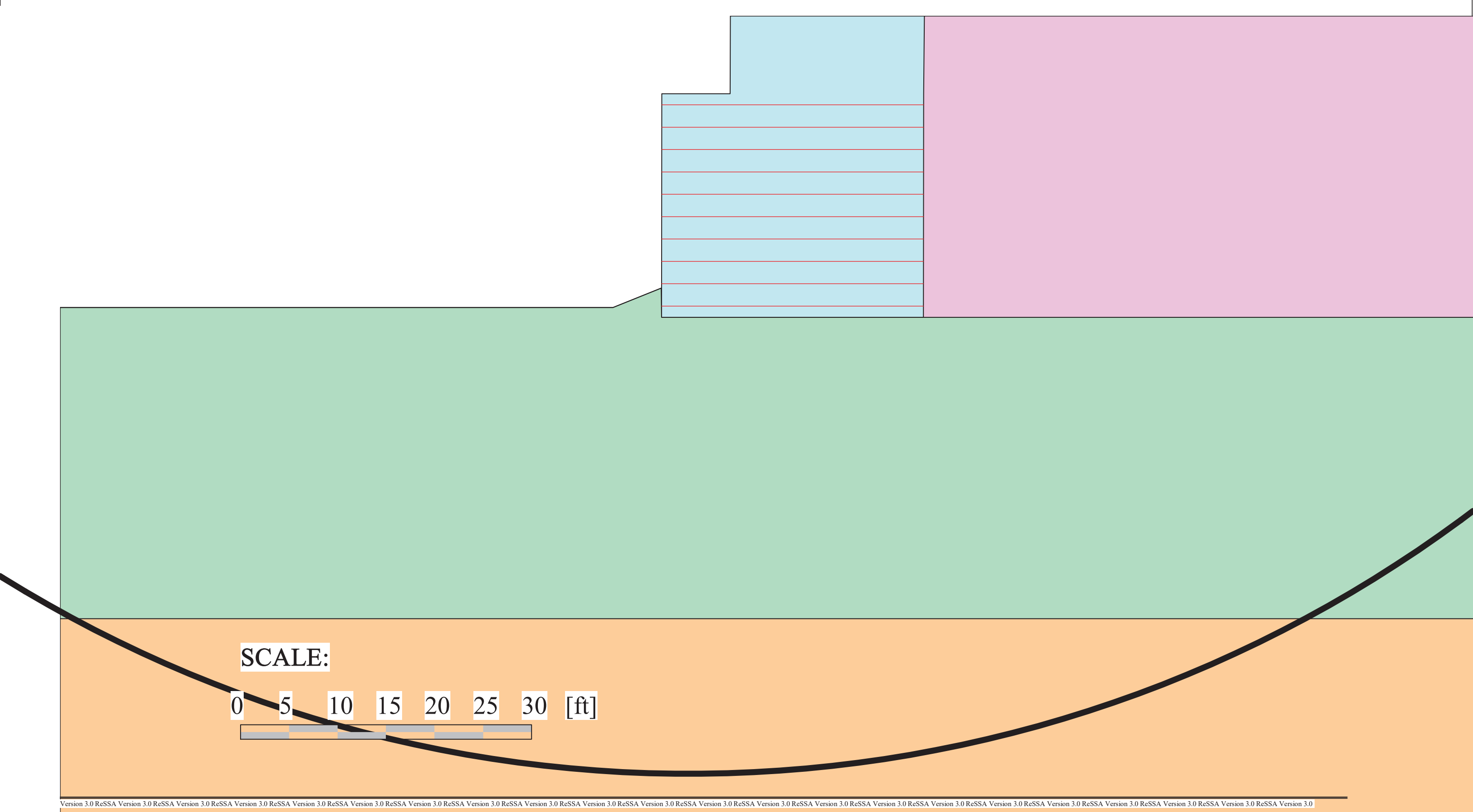
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

N O T C O N D U C T E D

Three-Part Wedge Stability Analysis

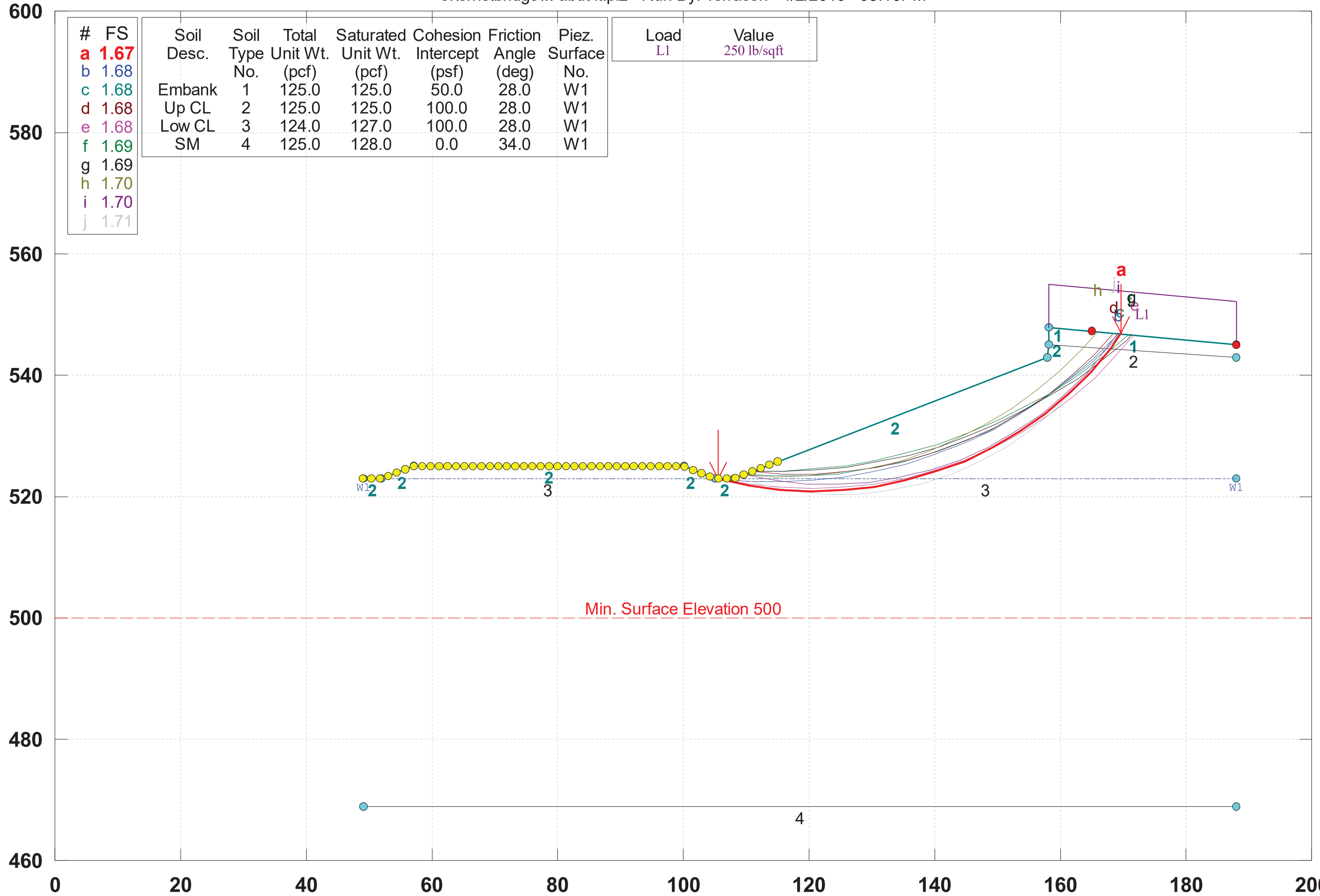
N O T C O N D U C T E D

REINFORCEMENT LAYOUT: DRAWING



Ernstbridge Rd - West Abutment Long-Term2.5H:1V Slope

o:\ernstbridge\w abut lt.pl2   Run By: Terracon   4/2/2019   08:10PM



STABL6H FSmin=1.67

Safety Factors Are Calculated By The Modified Bishop Method

Terracon

Ernstbridge Rd - West Abutment Short-Term - 2.5H:1V Slope

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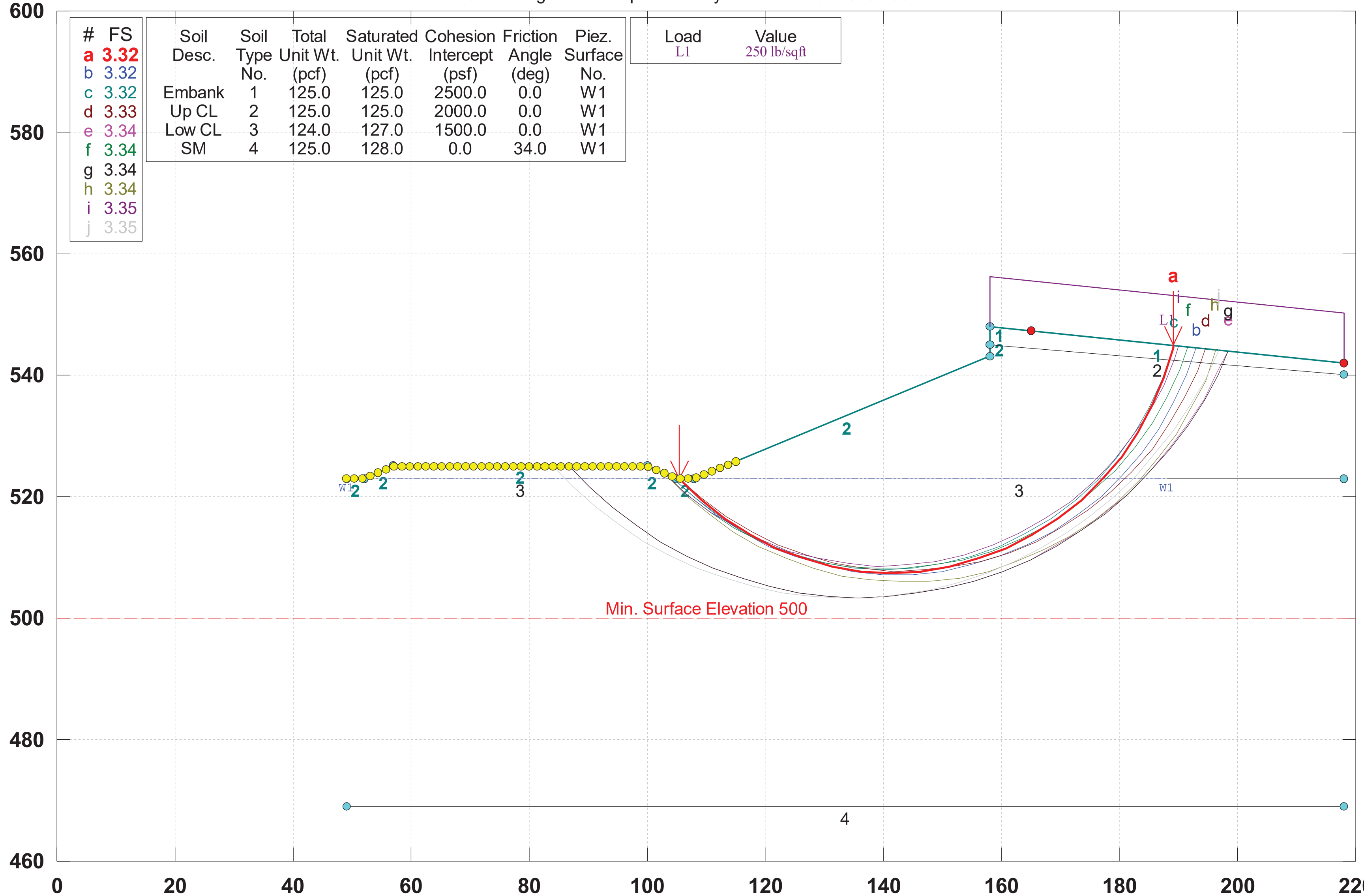


STABL6H FSmin=3.32

Safety Factors Are Calculated By The Modified Bishop Method

Terracon

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

Ernstbridge Road Bridge Replacement ■ Ryland Heights, KY
April 8, 2019 ■ Terracon Project No. N1185278

Terracon GeoReport

SAMPLING	WATER LEVEL	FIELD TESTS
Grab Sample; Shelby Tube; Standard Penetration Test	**Water Initially Encountered**; **Water Level After a Specified Period of Time**; **Water Level After a Specified Period of Time**	**N** – **Standard Penetration Test Resistance (Blows/Ft.)**
	Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.	**(HP)** – **Hand Penetrometer**
		(T) – **Torvane**
		(DCP) – **Dynamic Cone Penetrometer**
		UC – **Unconfined Compressive Strength**
		(PID) – **Photo-Ionization Detector**
		(OVA) – **Organic Vapor Analyzer**

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on the Unified Soil Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

LOCATION AND ELEVATION NOTES

Unless otherwise noted, Latitude and Longitude are approximately determined using a hand-held GPS device. The accuracy of such devices is variable. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

STRENGTH TERMS

RELATIVE DENSITY OF COARSE-GRAINED SOILS (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance		CONSISTENCY OF FINE-GRAINED SOILS (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance		
Descriptive Term (Density)	**Standard Penetration or N-Value Blows/Ft.**	**Descriptive Term (Consistency)**	**Unconfined Compressive Strength Qu, (tsf)**	**Standard Penetration or N-Value Blows/Ft.**
Very Loose	0 - 3	Very Soft	less than 0.25	0 - 1
Loose	4 - 9	Soft	0.25 to 0.50	2 - 4
Medium Dense	10 - 29	Medium Stiff	0.50 to 1.00	4 - 8
Dense	30 - 50	Stiff	1.00 to 2.00	8 - 15
Very Dense	> 50	Very Stiff	2.00 to 4.00	15 - 30
		Hard	> 4.00	> 30

RELATIVE PROPORTIONS OF SAND AND GRAVEL		RELATIVE PROPORTIONS OF FINES	
Descriptive Term(s) of other constituents	Percent of Dry Weight	**Descriptive Term(s) of other constituents**	**Percent of Dry Weight**
Trace	<15	Trace	<5
With	15-29	With	5-12
Modifier	>30	Modifier	>12

GRAIN SIZE TERMINOLOGY		PLASTICITY DESCRIPTION	
Major Component of Sample	**Particle Size**	**Term**	**Plasticity Index**
Boulders	Over 12 in. (300 mm)	Non-plastic	0
Cobbles	12 in. to 3 in. (300mm to 75mm)	Low	1 - 10
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)	Medium	11 - 30
Sand	#4 to #200 sieve (4.75mm to 0.075mm	High	> 30
Silt or Clay	Passing #200 sieve (0.075mm)		

CPT GENERAL NOTES

DESCRIPTION OF MEASUREMENTS AND CALIBRATIONS

To be reported per ASTM D5778:

Uncorrected Tip Resistance, q_c
Measured force acting on the cone divided by the cone's projected area

Corrected Tip Resistance, q_t
Cone resistance corrected for porewater and net area ratio effects
$q_t = q_c + u_2(1 - a)$
Where a is the net area ratio, a lab calibration of the cone typically between 0.70 and 0.85

Pore Pressure, u
Pore pressure measured during penetration
u_1 - sensor on the face of the cone
u_2 - sensor on the shoulder (more common)

Sleeve Friction, f_s
Frictional force acting on the sleeve divided by its surface area

Normalized Friction Ratio, F_r
The ratio as a percentage of f_s to q_t, accounting for overburden pressure

To be reported per ASTM D7400, if collected:

Shear Wave Velocity, V_s
Measured in a Seismic CPT and provides direct measure of soil stiffness

DESCRIPTION OF GEOTECHNICAL CORRELATIONS

Normalized Tip Resistance, Q_{tn}
$Q_{tn} = ((q_t - \sigma_{V0})/P_a)(P_a/\sigma'_{V0})^n$
$n = 0.381(I_c) + 0.05(\sigma'_{V0}/P_a) - 0.15$

Over Consolidation Ratio, OCR
$OCR\ (1) = 0.25(Q_{tn})^{1.25}$
$OCR\ (2) = 0.33(Q_{tn})$

Undrained Shear Strength, S_u
$S_u = Q_{tn} \times \sigma'_{V0}/N_{kt}$
N_{kt} is a soil-specific factor (shown on S_u plot)

Sensitivity, S_t
$S_t = (q_t - \sigma_{V0}/N_{kt}) \times (1/f_s)$

Effective Friction Angle, ϕ'
$\phi'\ (1) = \tan^{-1}(0.373[\log(q_t/\sigma'_{V0}) + 0.29])$
$\phi'\ (2) = 17.6 + 11[\log(Q_{tn})]$

Unit Weight, γ
$\gamma = (0.27[\log(F_r)]+0.36[\log(q_t/atm)]+1.236) \times \gamma_{water}$
σ_{V0} is taken as the incremental sum of the unit weights

Small Strain Shear Modulus, G_0
$G_0\ (1) = \rho V_s^2$
$G_0\ (2) = 0.015 \times 10^{(0.55Ic + 1.68)}(q_t - \sigma_{V0})$

Soil Behavior Type Index, I_c
$I_c = [(3.47 - \log(Q_{tn}))^2 + (\log(F_r) + 1.22)^2]^{0.5}$

SPT N_{60}
$N_{60} = (q_t/atm) / 10^{(1.1268 - 0.2817Ic)}$

Elastic Modulus, E_s (assumes $q/q_{ultimate} \sim 0.3$, i.e. FS = 3)
$E_s\ (1) = 2.6\Psi G_0$ where $\Psi = 0.56 - 0.33\log Q_{tn,clean\ sand}$
$E_s\ (2) = G_0$
$E_s\ (3) = 0.015 \times 10^{(0.55Ic + 1.68)}(q_t - \sigma_{V0})$
$E_s\ (4) = 2.5q_t$

Constrained Modulus, M
$M = \alpha_M(q_t - \sigma_{V0})$
For $I_c > 2.2$ (fine-grained soils)
$\alpha_M = Q_{tn}$ with maximum of 14
For $I_c < 2.2$ (coarse-grained soils)
$\alpha_M = 0.0188 \times 10^{(0.55Ic + 1.68)}$

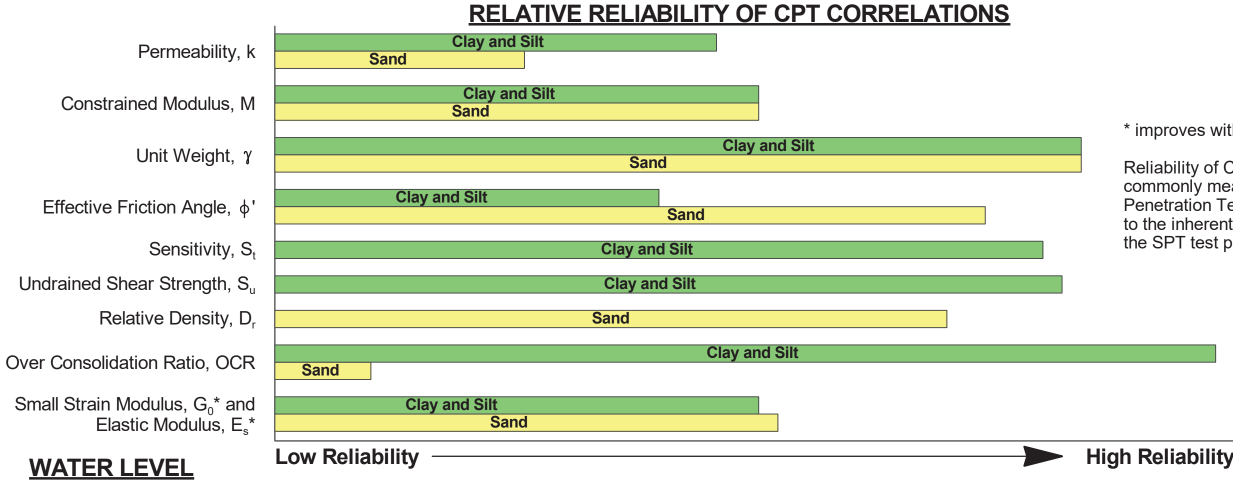
Hydraulic Conductivity, k
For $1.0 < I_c < 3.27$ $k = 10^{(0.952 - 3.04Ic)}$
For $3.27 < I_c < 4.0$ $k = 10^{(-4.52 - 1.37Ic)}$

Relative Density, D_r
$D_r = (Q_{tn} / 350)^{0.5} \times 100$

REPORTED PARAMETERS

CPT logs as provided, at a minimum, report the data as required by ASTM D5778 and ASTM D7400 (if applicable). This minimum data include q_t, f_s, and u. Other correlated parameters may also be provided. These other correlated parameters are interpretations of the measured data based upon published and reliable references, but they do not necessarily represent the actual values that would be derived from direct testing to determine the various parameters. To this end, more than one correlation to a given parameter may be provided. The following chart illustrates estimates of reliability associated with correlated parameters based upon the literature referenced below.

RELATIVE RELIABILITY OF CPT CORRELATIONS



* improves with seismic V_s measurements

Reliability of CPT-predicted N_{60} values as commonly measured by the Standard Penetration Test (SPT) is not provided due to the inherent inaccuracy associated with the SPT test procedure.

WATER LEVEL

The groundwater level at the CPT location is used to normalize the measurements for vertical overburden pressures and as a result influences the normalized soil behavior type classification and correlated soil parameters. The water level may either be "measured" or "estimated:"

Measured - Depth to water directly measured in the field

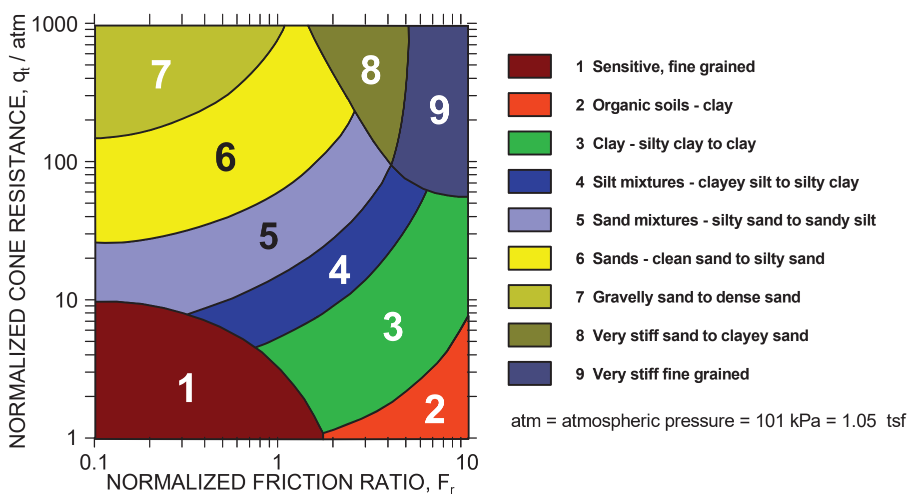
Estimated - Depth to water interpolated by the practitioner using pore pressure measurements in coarse grained soils and known site conditions

While groundwater levels displayed as "measured" more accurately represent site conditions at the time of testing than those "estimated," in either case the groundwater should be further defined prior to construction as groundwater level variations will occur over time.

CONE PENETRATION SOIL BEHAVIOR TYPE

The estimated stratigraphic profiles included in the CPT logs are based on relationships between corrected tip resistance (q_t), friction resistance (f_s), and porewater pressure (u_2). The normalized friction ratio (F_r) is used to classify the soil behavior type.

Typically, silts and clays have high F_r values and generate large excess penetration porewater pressures; sands have lower F_r's and do not generate excess penetration porewater pressures. The adjacent graph (Robertson *et al.*) presents the soil behavior type correlation used for the logs. This normalized SBT chart, generally considered the most reliable, does not use pore pressure to determine SBT due to its lack of repeatability in onshore CPTs.



atm = atmospheric pressure = 101 kPa = 1.05 tsf

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Mayne, P.W., (2013). "Geotechnical Site Exploration in the Year 2013," Georgia Institue of Technology, Atlanta, GA.
Robertson, P.K., Cabal, K.L. (2012). "Guide to Cone Penetration Testing for Geotechnical Engineering," Signal Hill, CA.
Schmertmann, J.H., (1970). "Static Cone to Compute Static Settlement over Sand," *Journal of the Soil Mechanics and Foundations Division*, 96(SM3), 1011-1043.

Terracon

Exhibit:

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]					Soil Classification: Group Symbol	Soil Classification: Group Name [B]
Coarse-Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$ [E]		GW	Well-graded gravel [F]
			$Cu < 4$ and/or [$Cc<1$ or $Cc>3.0$] [E]		GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH		GM	Silty gravel [F, G, H]
			Fines classify as CL or CH		GC	Clayey gravel [F, G, H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	$Cu \geq 6$ and $1 \leq Cc \leq 3$ [E]		SW	Well-graded sand [I]
			$Cu < 6$ and/or [$Cc<1$ or $Cc>3.0$] [E]		SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH		SM	Silty sand [G, H, I]
			Fines classify as CL or CH		SC	Clayey sand [G, H, I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	$PI > 7$ and plots on or above "A"		CL	Lean clay [K, L, M]
			$PI < 4$ or plots below "A" line [J]		ML	Silt [K, L, M]
		Organic:	Liquid limit - oven dried	< 0.75	OL	Organic clay [K, L, M, N]
			Liquid limit - not dried			Organic silt [K, L, M, O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line		CH	Fat clay [K, L, M]
			PI plots below "A" line		MH	Elastic Silt [K, L, M]
		Organic:	Liquid limit - oven dried	< 0.75	OH	Organic clay [K, L, M, P]
			Liquid limit - not dried			Organic silt [K, L, M, Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor				PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve.

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.

[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains $\geq$ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains $\geq$ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains $\geq$ 30% plus No. 200 predominantly sand, add "sandy" to group name.

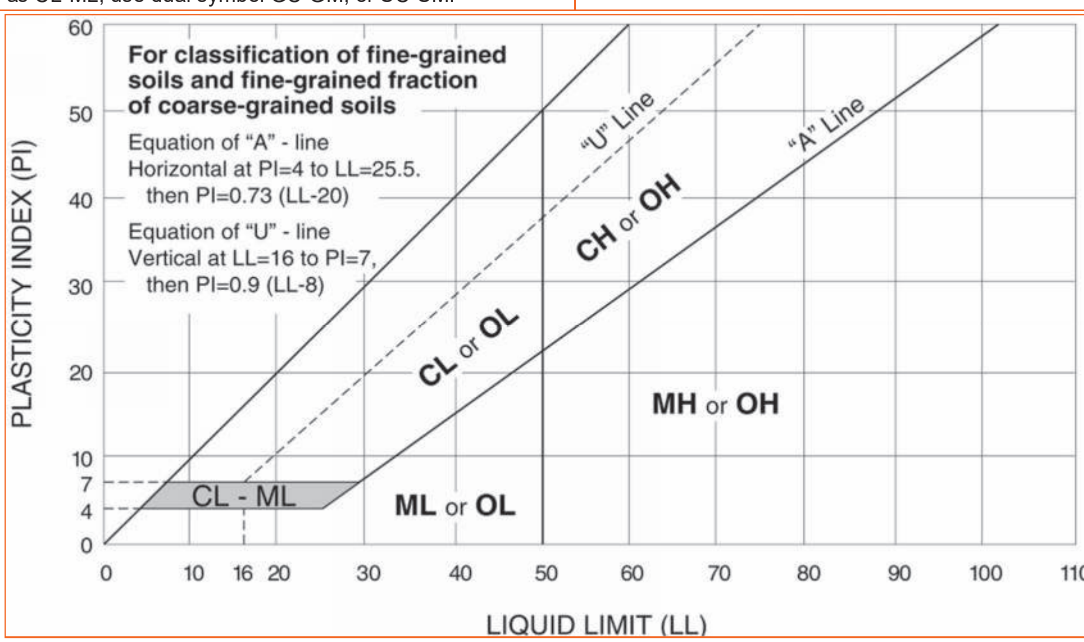
[M] If soil contains $\geq$ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] $PI \geq 4$ and plots on or above "A" line.

[O] $PI < 4$ or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.



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		Organic:	$\frac{\text{Liquid limit - oven dried}}{\text{Liquid limit - not dried}} < 0.75$	OL	Organic clay [K, L, M, N] Organic silt [K, L, M, O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line	CH	Fat clay [K, L, M]
			PI plots below "A" line	MH	Elastic Silt [K, L, M]
		Organic:	$\frac{\text{Liquid limit - oven dried}}{\text{Liquid limit - not dried}} < 0.75$	OH	Organic clay [K, L, M, P] Organic silt [K, L, M, Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

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