Final Roadway Exploration Report

MOT-4-19.30

State Route 4 Settlement Improvement Project

Dayton, Ohio
April 10, 2024

Prepared for:
Chagrin Valley Engineering, Ltd
22999 Forbes Road, Suite B
Oakwood, Ohio 44146

By:
HDR Engineering, Inc.
9999 Carver Road, Suite 210
Blue Ash, Ohio 45242

Table of Contents

EXECUTIVE SUMMARY 1
1. INTRODUCTION 2
2. GEOLOGY AND OBSERVATIONS 2
2.1 Project Setting 2
2.2 Soil and Geologic Setting 3
2.2.1 Project Soils 3
2.2.2 Bedrock Geology 4
3. EXPLORATION 4
3.1 Previous Explorations 4
3.2 Site Reconnaissance 7
3.3 Subsurface Exploration 8
3.4 Laboratory Testing 9
4. FINDINGS 9
4.1 Encountered Subsurface Conditions 9
5. ANALYSIS AND RECOMMENDATIONS 11
5.1 Discussion 11
5.1.1 Undercut and Replacement with Geogrid-Reinforced Soil Mat 13
5.1.2 Potential Voids 14
5.1.3 Subgrade Strength 15
5.1.4 Constructability 15
5.2 Recommendations 15
5.2.1 Geogrid-Reinforced Soil Mat 15
5.2.2 Pavement Design 16
6. LIMITATIONS 16
7. REFERENCES 17

Appendices

Appendix A. Exhibits A-1
Appendix B. Historic Information B-1
Appendix C. Site Photos C-1
Appendix D. Boring Logs and Pavement Core Photos D-1
Appendix E. Geotechnical Plan and Profile Sheets E-1
Appendix F. Analyses F-1
Appendix G. Plan Notes G-7

EXECUTIVE SUMMARY

This report summarizes the results of the geotechnical study performed by HDR Engineering, Inc. (HDR) in support of the MOT-4-19.30 settlement improvement project along State Route 4 (SR 4) at the Stanley Avenue interchange in Dayton, Ohio. This section of roadway was constructed over an existing landfill in the late 1950's and is experiencing settlement that is impacting the mainline and Ramps J, K, and L. The Ohio Department of Transportation began exploring possible alternatives for a more permanent solution before teaming with Chagrin Valley Engineering, Ltd to continue this work.

Ten soil borings were performed as part of the geotechnical exploration program to assess the subsurface conditions along SR 4 and associated ramps within the project limits. The explorations extended through the embankment fill to approximately 20 feet below the underlying refuse material to provide a more complete representation of the soil profile at the project site. This report includes the geotechnical information obtained from these borings, as well as the laboratory testing performed under this study. The exploration findings, along with the laboratory test results, are presented in more detail in Section 3 as well as in Appendix D of this report. The generalized soil profile as encountered in these borings consists of an overlying layer of granular embankment fill, underlain by refuse materials and construction debris, over granular glacial outwash soils containing occasional layers of cohesive glacial till. As the borings were located within the pavement limits of either SR 4 or Ramps J, K and L, the surficial materials consist of asphaltic concrete with some areas underlain with reinforced Portland-cement concrete. Further discussion on the encountered subsurface conditions is found in Section 4.

Given the encountered subsurface conditions at the site, including the encountered thicknesses of the existing embankment material and underlying refuse material, it was decided to construct a geogrid-reinforced soil mat to create a more uniform and interconnected subgrade and aid in reducing the noticeable effects of the differential settlement within the pavement system. To construct the geogrid mat, the existing subgrade material is to be undercut to a depth of 36 inches below the bottom of the proposed pavement system and three layers of Tensar InterAx™ geogrid installed. The first or bottom layer is to consist of a layer of NX850-FG, a composite geosynthetic consisting of a NX850 geogrid bonded to a nonwoven geotextile. The second or middle layer will consist of NX850 geogrid to be placed at 24 inches below the bottom of the pavement system, and the third or upper layer of NX850 geogrid is to be placed at 12 inches below the proposed bottom of the pavement system. The backfill materials for the 36-inch undercut are to consist of the on-site embankment materials, which classify as a gravel (A-1-a) or gravel with sand (A-1-b). If needed, borrow material meeting the requirements of ODOT Item 703.16 Type B Granular material with a fines content of less than 15% passing the No. 200 sieve may be utilized as well. Additional discussions and recommendations with regards to the geogrid-reinforced soil mat are provided in Section 5.

1. INTRODUCTION

This report summarizes the results of the geotechnical study performed by HDR Engineering, Inc. (HDR) in support of the MOT-4-19.30 settlement improvement project along State Route 4 (SR 4) at the Stanley Avenue interchange in Dayton, Ohio. This section of roadway was constructed over an existing landfill in the late 1950's and is experiencing settlement that is impacting the mainline and Ramps J, K, and L. The City of Dayton is responsible for maintaining the roadway within the project area. HDR understands that the City has periodically performed asphalt patching and overlays (mill and fill), and resetting of the guardrail, in response to the differential settlement that is affecting rideability along SR 4 and the ramps. However, these repairs have proven to be stopgap measures as settlement of the roadway has continued. As such, the Ohio Department of Transportation (ODOT) began exploring possible alternatives for a more permanent solution before teaming with Chagrin Valley Engineering, Ltd (CVE) to continue this work.

Four feasible alternatives to reduce the effects of the continued settlement and consolidation of the refuse material at the project site were considered by the CVE team as part of this study. An evaluation of these alternatives (1. Undercut and Replacement, 2. Rigid Inclusions, 3. Injection Grouting, and 4. a No-Build alternative) as well as recommendations with regards to a preferred alternative are presented in the geotechnical design memo dated October 13, 2023. Based on a meeting with ODOT on September 27, 2023 and a follow up email from the ODOT Project Manager, Jonathon Koester, dated November 15, 2023, it is understood that the Undercut and Replacement alternative utilizing the on-site materials as the granular backfill for the geogrid-reinforced soil mat was selected by ODOT as the preferred alternative.

With the selection of the preferred alternative, the remaining scope of work relative to the geotechnical study for the MOT-4-19.30 project included:

- updating and finalizing the evaluations related to the use of a partial undercut and replacement with a geogrid-reinforced soil mat to reduce the effects of continued settlement and consolidation of the refuse material along SR 4 and Ramps J, K, and L, and
- development of this Roadway Exploration Report, which presents the analyses and recommendations for implementation of the selected alternative.

2. GEOLOGY AND OBSERVATIONS

2.1 Project Setting

Located within the City of Dayton, SR 4 is a major arterial highway that parallels the Mad River within the project area as shown on the Site Vicinity and Topographic Map (Exhibit No. 1) in Appendix A. The project site is located within a highly developed urban area, with residential housing and the occasional business located to the north of the highway, and a recreational sports complex to the south. Dayton Children's Hospital is also located within the project area, immediately to the north of the SR 4/Stanley Avenue interchange at the corner of Stanley Avenue and Valley Street.

The surrounding area is relatively level, with any considerable elevation changes occurring along the excavation and/or embankment slopes for the mainline alignment, various ramps, and near the right-of-way boundaries. This also includes the spill-through abutments for the left and right bridge structures carrying SR 4 over Stanley Avenue at the western project limits. As shown on Exhibit No. 1, the existing topography at the project site ranges from approximately El. 775 to El. 760 along SR 4, from about El. 775 to El 750 along Ramp J, and from roughly El. 760 to El. 750 along Ramps K and L.

2.2 Soil and Geologic Setting

Montgomery County is located in the Southern Ohio Loamy Till Plains region within the Wisconsin-age, glaciated Central Lowland Till Plains section of southwestern Ohio (Exhibit No. 2). Montgomery County is a broad, nearly level to gently-rolling till plain, where Wisconsin-age glaciers wore away and filled in the former rolling to moderately steep limestone topography. This glacial action and the stream development that followed resulted in the formation of a series of creeks and rivers across the county, including the Mad River, Stillwater River, Twin Creek, Wolf Creek, and Great Miami River. The Great Miami River flows north-to-south through the middle of Montgomery County, and together with its tributaries, drains most of the county.

The project site is located near the confluence of the Mad River and the Great Miami River, within the confines of an ancient, buried river valley that is part of the Great Miami/Mad River Buried Aquifer. The soils in this valley generally consist of unconsolidated Pleistocene glacial deposits, predominately Wisconsinan and Illinoian in age, when the glaciers retreated and melt waters were discharged through the valleys, partially filling them with sand and gravel outwash. The soils and materials overlying the site are much more recent, and the result of human activity. The underlying bedrock in the vicinity of the project site generally consists of Ordovician-age limestone and shale.

2.2.1 Project Soils

According to the Surficial Geology of the Ohio portion of the Dayton region map from the Ohio Department of Natural Resources (ODNR) dated 2011 (Exhibit No. 3), the surficial materials in the project area generally consist of Wisconsinan Sand and Gravel (SG) with interbedded layers of unsorted silt, clay, sand, gravel, and boulders (T), underlain by Ordovician-age Limestone and Shale bedrock (L-S). The soils along the south side of the project site consist of similar materials, with the exception of an overlying layer of more recently deposited alluvium (a) from the adjacent Mad River. Sand and gravel mining is prevalent throughout the project area. A review of ODNR mine maps (Exhibit No. 4), historic topographic maps from the United States Geological Survey, and the MOT-4-19.73 plans (ODOT, 1958) indicate aggregate mining occurred within the project limits prior to 1955, with these water-filled gravel pits extending under portions of Ramp J and K. Gravel mining continues to be conducted in the surrounding areas; however, these aggregate mines are generally outside of an approximately 4 to 6 mile radius of the project site.

The USDA Soil Survey of Montgomery County indicate the most prevalent surficial soil types within the project limits consist of urban fills and man-made deposits including Udorthents (Ud), gravel pits (Gp) and Made Land (Mb) as shown in Exhibit No. 5a. As shown on Exhibit Nos. 5b through 5d in Appendix A, the soil survey does not provide steel and concrete corrosion risk nor pH levels for the soils within the project site. As these soils are urban fills and have been subjected to development and disturbances throughout the years, site specific testing will need to be performed to determine the potential risk to construction materials.

2.2.2 Bedrock Geology

As shown on Exhibit No. 6 (Bedrock Geology Map), the bedrock geology mapped within the majority of the project area is the Ordovician-age Grant Lake Formation, with the underlying Miamitown Shale-Fairview Formation, undivided mapped along Ramp L and the northern project boundary. However, some publications and maps combine these three formations, designating this unit as the Grant Lake and Fairview Formations, Miamitown Shale Formation, undivided. As a combined formation, the underlying bedrock units are comprised of interbedded layers of fossiliferous shale and limestone. Evenly proportioned shale and limestone is thin- to thick-bedded in the upper half, and thin- to medium-bedded in the lower half. An interval of predominantly shale (90%) with interbedded limestone (approximately 10%) may be found near the middle of the bedrock unit. The term Grant Lake Formation is used where the unit becomes predominantly shale (50% to 60%) based on USGS publications. The top of bedrock in the project area, as shown on Exhibit No. 7 (Bedrock Topography Map), is at approximately El. 500, or 250 to 275 feet below the existing ground surface.

3. EXPLORATION

3.1 Previous Explorations

Multiple subsurface explorations have been performed within the MOT-4-19.30 project limits, from the initial soil explorations (auger and test borings) for this section of SR 4 in 1958, to test borings and dynamic cone penetrometer (DCP) testing at the site in 2016, 2017, and 2021. These later explorations were performed as part of an ODOT study to pursue more long-term solutions to the on-going settlement at the project site, including in-situ grouting to fill voids and stabilize the ground beneath the embankment, constructing a reinforced concrete slab beneath the pavement section to bridge over the settlement areas, and the use of a geogrid-reinforced mat.

The interchange was constructed over an existing landfill in the late 1950’s, with the footprint of both the active and abandoned portions of the landfill at the time of construction extending from about Mainline Station 85+00 to Station 96+00. The historic borings performed in 1958 within the project area indicate the thickness of the landfill or refuse material ranging from 0 to 15 feet, with an average depth of approximately 7 feet. The refuse material was described as random fill: gravel, ashes, cans, glass, rubber, rags, tires, bottles, and tin. A later test boring performed in 2017 (B-001-0-16) that penetrated 8 feet into the landfill described the material as refuse containing glass with sand, silt, clay, and stone fragments. The soils underlying the refuse material as encountered in the 1958 borings consisted of gravel (A-1-a), gravel with sand (A-1-b), gravel with sand, silt, and clay (A-2-6) with occasional gravel with sand and silt (A-2-4), coarse and fine sand (A-3a), and silt and clay (A-6a).

The refuse material is overlain by embankment fill placed during construction, consisting of medium dense to very dense gravel and stone fragments (A-1-b, A-2-4) with varying amounts of sand, silt and concrete fragments based on two borings drilled in 2017. The relative density of the granular embankment was corroborated by the DCP testing performed in 2016 and 2021 at the site to depths of roughly 2 to 10 feet below the existing ground surface.

Per the historic plans, the mainline pavement is comprised of a composite section consisting of approximately 10 inches of asphalt over 9 inches of reinforced concrete. Pavement cores obtained during the previous geotechnical explorations indicate pavement thicknesses similar the design section (19 inches) up to 36 inches. The difference in thickness is in additional asphalt, which has been placed to raise and level paved surfaces in areas that have settled due to degradation and consolidation of the landfill materials. The pavement cores that indicate additional asphalt are mainly located between Mainline Station 87+50 and Station 93+00, which roughly corresponds with the settlement limits as indicated by the District in their 2016 study between Station 86+00 and Station 93+00.

A list of the historic borings and DCP tests located within the project limits are provided in Table 3-1 and Table 3-2, respectively, with the approximate locations of these previous explorations shown on Exhibit No. 8 (Boring Location Plan) in Appendix A. Copies of the boring and DCP logs, the graphical logs for the 1958 borings, as well as other information as presented on the historic construction plans and soil profile sheets, are located in Appendix B.

Table 3-1: Historic Boring Designations

Project (Year)	Original Designation	Original Station – Offset[1] (Alignment)	Latitude	Longitude	Updated Designation
MOT-4-19.73 (1958)	--	85+50, CL (SR 4)	39.779262	-85.161106	B-001-2-58
MOT-4-19.73 (1958)	--	2+00, CL (Ramp J)	39.779312	-84.160591	B-001-3-58
MOT-4-19.73 (1958)	--	88+00, 50' LT (SR 4)	39.779809	-84.160511	B-003-2-58
MOT-4-19.73 (1958)	--	88+00, 50' RT (SR 4)	39.779587	-84.160307	B-003-3-58
MOT-4-19.73 (1958)	--	5+00, CL (Ramp L)	39.779974	-84.160640	B-003-4-58
MOT-4-19.73 (1958)	--	88+00, 135 RT (SR 4)	39.764060	-84.160130	B-003-5-58
MOT-4-19.73 (1958)	--	90+00, CL (SR 4)	39.780022	-84.159832	B-005-1-58
MOT-4-19.73 (1958)	--	90+00, 50' LT (SR 4)	39.780129	-84.159929	B-005-2-58
MOT-4-19.73 (1958)	--	90+00, 115' LT (SR 4)	39.780283	-84.160070	B-005-3-58
MOT-4-19.73 (1958)	--	90+00, 50' RT (SR 4)	39.779907	-84.159729	B-005-4-58
MOT-4-19.73 (1958)	--	90+00, 100' RT (SR 4)	39.779793	-84.159626	B-005-5-58
MOT-4-19.73 (1958)	--	90+00, 192' RT (SR 4)	39.779587	-84.159442	B-005-6-58
MOT-4-19.73 (1958)	--	9+50, CL (Ramp K)	39.779517	-84.158997	B-007-1-58
MOT-4-19.73 (1958)	--	1+00, 30' LT (Ramp L)	39.780408	-84.159311	B-008-1-58
MOT-4-19.73 (1958)	--	93+00, 5' RT (SR 4)	39.780333	-84.158827	B-009-1-58
MOT-4-19.73 (1958)	--	93+00, 50' RT (SR 4)	39.780439	-84.158906	B-009-2-58
MOT-4-19.73 (1958)	--	6+00, CL (Ramp K)	39.780341	-84.158327	B-010-1-58
MOT-4-19.30 (2016)	B-001-0-16	90+69, 53' RT (SR 4)	39.780297	-84.158906	B-001-0-16
MOT-4-19.30 (2016)	B-002-0-16	92+86, 51' RT (SR 4)	39.779984	-84.159550	B-002-0-16

Notes:
1. Stationing listed as presented in respective project plans

Table 3-2: Historic DCP Locations

Project (Year)	Original Designation	Original Station – Offset[1] (Alignment)	Latitude	Longitude	Updated Designation
D7 MOT SR4 DCP (2016)	DCP-1	80+12, 16 LT (Ramp K)	39.779864	-84.158706	D-001-0-16
D7 MOT SR4 DCP (2016)	DCP-2	90+67, 68 RT (SR 4)	39.779945	-84.159527	D-002-0-16
D7 MOT SR4 DCP (2016)	DCP-3	91+21, 68 RT (SR 4)	39.780025	-84.159369	D-003-0-16
D7 MOT SR4 DCP (2016)	DCP-4	92+62, 68 RT (SR 4)	39.780224	-84.158949	D-004-0-16
D7 MOT SR4 DCP (2016)	DCP-5	93+44, 68 RT (SR 4)	39.780334	-84.158701	D-005-0-16
D7 MOT SR4 DCP (2016)	DCP-6	94+07, 68 RT (SR 4)	39.780416	-84.158508	D-006-0-16
D7 MOT SR4 DCP (2016)	DCP-7	90+47, 16 LT (SR 4)	39.780107	-84.159751	D-007-0-16
D7 MOT SR4 DCP (2016)	DCP-8	91+15, 15 LT (SR 4)	39.780208	-84.159546	D-008-0-16
D7 MOT SR4 DCP (2016)	DCP-9	91+85, 14 LT (SR 4)	39.780308	-84.159333	D-009-0-16
MOT-4-19.93(2021)	D-001-0-21	85+57, 31 RT (SR 4)	39.779190	-84.161033	D-001-0-21
MOT-4-19.93 (2021)	D-002-0-21	87+36, 33 LT (SR 4)	39.779638	-84.160688	D-002-0-21
MOT-4-19.93 (2021)	D-003-0-21	87+79, 48 RT (SR 4)	39.779537	-84.160388	D-003-0-21
MOT-4-19.93 (2021)	D-004-0-21	89+17, 46 LT (SR 4)	39.779970	-84.160197	D-004-0-21
MOT-4-19.93 (2021)	D-005-0-21	89+88, 34 RT (SR 4)	39.779903	-84.159825	D-005-0-21
MOT-4-19.93 (2021)	D-006-0-21	91+04, 47 RT (SR 4)	39.780050	-84.159459	D-006-0-21
MOT-4-19.93 (2021)	D-007-0-21	91+11, 33 LT (SR 4)	39.780244	-84.159594	D-007-0-21

Notes:
1. Stationing for 2016 DCP locations approximated from provided information and location maps using SR 4 centerline alignment.

3.2 Site Reconnaissance

A geotechnical site reconnaissance of the project area was performed on May 10, 2023, as well as during the subsurface exploration program from May 15 to 22, 2023. The reconnaissance consisted of observations made while walking the mainline and ramp alignments, as well as the infields and median, and noting the general terrain, overall condition of the exiting roadways and pavement, and any additional pertinent observations. Representative photos of these observations have been included in Appendix C.

As previously mentioned, the project site is located within a highly developed urban area, with residential housing and the occasional business located to the north of the highway, and a recreational sports complex to the south. Mature trees and dense overgrown vegetation were noted along the south embankment slope for Ramp K and along the north embankment slope for Ramp L. The median between north and southbound SR 4, as well as the infields between the ramps and SR 4, are level to gently sloping, maintained grassed areas. No signs of settlement or distress were noted in these areas, but undulations in the vertical profile were noted along all travel lanes of SR 4 and Ramps J and K. Vehicles were observed to be bouncing, tilting, and wobbling in these areas, with gouges noted in the pavement surface where vehicles had bottomed out. Those areas where settlement was readily distinguishable were along the mainline from approximately Sta. 87+00 to Sta. 93+00, from about Sta. 5+00 to Sta. 9+00 along Ramp K, and on Ramp L from roughly Sta. 2+50 to Sta. 5+00. Longitudinal cracking was also noted between lanes of traffic with regular intervals of transverse cracking. Areas of alligator cracking and map cracking were also noted along Ramp K from approximately Sta. 6+50 to Sta. 8+00 as well as at the merge of northbound SR 4 and Ramp J (approximately Ramp J Sta. 0+50 to Sta.1+50). Similar conditions were noted occasionally at the interface of the travel lanes and shoulders along SR 4.

3.3 Subsurface Exploration

Ten soil borings were performed as part of the geotechnical exploration program to assess the subsurface conditions along SR 4 and associated ramps within the project limits. The explorations extended through the embankment fill to approximately 20 feet below the underlying refuse material to provide a more complete representation of the soil profile at the project site. The locations of the explorations are shown on the Boring Location Plan (Exhibit No. 8) in Appendix A. The test locations were located and marked in the field during the initial site reconnaissance on May 10, 2023, with the as-drilled locations surveyed after completion of the borings. These locations are reflected on the boring location plan, boring logs and Table 3-3.

Table 3-3. Summary of Explorations

Exploration Number	Exploration Type	Latitude	Longitude	Ground Surface (El., ft)	Top of Refuse Material (El. ft)	Bottom of Refuse Material (El. ft)	Bottom of Exploration Depth (ft)	Bottom of Exploration (El. ft)
B-001-1-23	C3	39.779141	-84.160889	772.6	756.1	750.1	42.0	730.6
B-002-1-23	C3	39.779490	-84.160409	770.9	755.4	743.4	47.5	723.4
B-002-2-23	C3	39.779310	-84.160214	768.2	764.2	757.2	33.5	734.7
B-003-1-23	C3	39.779984	-84.160505	770.3	761.3	743.1	46.0	724.3
B-004-1-23	C3	39.779853	-84.159766	768.1	756.1	738.6	50.0	718.1
B-004-2-23	C3	39.780079	-84.160106	771.5	761.0	746.0	46.5	725.0
B-008-0-23	C3	39.780288	-84.159458	768.1	759.1	746.6	41.5	726.6
B-009-0-23	C3	39.780282	-84.158882	764.2	756.2	743.2	43.0	721.2
B-010-0-23	C3	39.779882	-84.158816	763.0	755.5	738.0	45.3	717.7
B-011-0-23	C3	39.780667	-84.157898	761.5	751.5	748.0	34.0	727.5

Notes:
1. C3 = Uncontrolled Fill Borings

The borings were drilled with a Diedrich D-50 track rig by Central Star Drilling under the supervision of an HDR geotechnical engineer from May 15 to 22, 2023. The rig was calibrated on March 7, 2022 and had an energy ratio of 86.8%. All borings were drilled utilizing 3.25-inch internal diameter hollow stem augers to advance the borings to the explored depths. The sampling of the soils was accomplished in accordance with the *Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils* (ASTM D 1586) at continuous 1.5-foot intervals to the determined bottom of the refuse material, and at 2.5-foot intervals thereafter until reaching the boring termination depth approximately 20 feet below the refuse material. In the split-barrel sampling procedure, a standard 2-inch outside diameter split-barrel sampling spoon is driven into the ground with a 140-pound hammer falling 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of a typical 18-inch penetration is recorded as the standard penetration test (SPT) resistance or N_{SPT}-value. The N_{SPT}-value is then corrected to an energy ratio of 60%, termed N_{60}, which is used for design. Retrieval of undisturbed soil samples within an occasional cohesive layer were also attempted in accordance with the *Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes* (ASTM D 1587). However, these attempts were unsuccessful due to the generally high granular content of the subsurface soils or limited thickness of the cohesive layers. Pavement cores were also obtained using a 10-inch diameter core barrel at select boring locations along the mainline. Photos of the pavement cores and the typed boring logs are provided in Appendix D.

3.4 Laboratory Testing

Representative soil samples were selected for laboratory testing to confirm the field classification and to assess the various engineering properties of the soils. Soil index testing was performed at HDR's materials laboratory and included 237 natural moisture content tests (per ASTM D 2216), 80 Atterberg limit determinations (per ASTM D 4318), and 80 grain size analyses (per ASTM D 422). Due to the composition of obtained samples, 2-hour hydrometer tests (per ASTM D 422) could not be performed on nine of the selected test samples due to a lack of appropriate testing material. Results of these tests are presented on the boring logs provided in Appendix D.

4. FINDINGS

4.1 Encountered Subsurface Conditions

The generalized soil profile as encountered in the ten recent soil borings consists of an overlying layer of granular embankment fill, underlain by refuse materials and construction debris, over granular glacial outwash soils containing occasional layers of cohesive glacial till. As the borings were located within the pavement limits of either SR 4 or Ramps J, K and L, the surficial materials consisted of asphalt concrete with some areas underlain with reinforced Portland-cement concrete. A summary of the encountered pavement thicknesses is provided in Table 4-1. Photographs of the pavement core as obtained from select boring locations as indicated in the table below are provided in Appendix D.

Table 4-1: Encountered Pavement Types and Thicknesses

Exploration Number	Asphalt Thickness (in.)	Reinforced Concrete Thickness (in.)	Total Pavement Thickness (in.)	Pavement Core Photo Provided
B-001-1-23	12	--	12	Yes
B-002-1-23	12	9	21	--
B-002-2-23	9	--	9	--
B-003-1-23	14	--	14	Yes
B-004-1-23	27	--	27	Yes
B-004-2-23	10	8	18	Yes
B-008-0-23	13	--	13	Yes
B-009-0-23	17	--	17	Yes
B-010-0-23	30	12	42	--
B-011-0-23	12	--	12	Yes

Embankment fill as encountered in each of the borings consisted of generally medium dense to very dense Gravel (A-1-a) with Gravel with Sand (A-1-b), Gravel with Sand and Silt (A-2-4) and Coarse and Fine Sand (A-3a) encountered to a lesser extent. This overlying granular fill is consistent with the fill material as specified in the 1958 construction plans. The thickness of the fill material ranges from 3.3 feet to 15.5 feet as presented on the graphical depictions of the encountered subsurface conditions for both SR 4 and its associated ramps in Appendix E.

The underlying refuse material generally consisted of medium dense to very dense Gravel (A-1-a), Gravel with Sand (A-1-b), Gravel with Sand and Silt (A-2-4) and Sandy Silt (A-4a) with Coarse and Fine Sand (A-3a) and Fine Sand (A-3) encountered to a lesser extent. The material tended to have a dark brown to black coloring, with noted amounts construction debris including brick, glass, asphalt, wood, and metal fragments. Very loose to loose materials were also encountered in Borings B-008-0-23, B-009-0-23 and B-010-0-23 beginning at depths of approximately 11 feet to 13 feet below the existing ground surface, and extending to a depth of 18.5 feet (El. 749.6) in B-008-0-23 along southbound SR 4 to an increased depth of 23 feet (El. 740) in B-010-0-23 to the south on Ramp K. The thicknesses of the refuse material range from approximately 3 feet to 18.5 feet. Layer elevations are presented in Table 3-3.

Native granular soils were encountered beneath the refuse material. The medium dense to very dense soils consisted of predominantly Gravel (A-1-a), Gravel with Sand (A-1-b), Gravel with Sand and Silt (A-2-4), and Sandy Silt (A-4a) with Coarse and Fine Sand (A-3a) encountered to a lesser extent. The native soils were encountered at depths ranging from 33.5 feet to 50 feet below existing grade (El. 757.2 to El. 738.0) and extended to the boring termination depth in each of the borings except at B-008-0-23 and B-009-0-23. These borings encountered a layer of glacial till consisting of stiff to hard cohesive Sandy Silt (A-4a) at depths of 40 feet (El. 728.1) and 36 feet (El. 728.2), respectively, until reaching their termination depths.

Free water was encountered in several of the borings during drilling within the native granular layer. A summary of the locations and elevations where water was encountered is provided in

Table 4-2 below. In addition, field observations and laboratory testing indicated elevated moisture contents within the refuse material in Borings B-003-1-23, B-004-2-23, B-008-0-32, and B-009-0-23 from approximately El. 755 to El. 744, with laboratory test moisture contents ranging from approximately 23 percent to 47 percent. Water was also added in some bore holes to assist in drilling through the denser soils (B-001-1-23, B-002-1-23, B-002-2-23) or to mitigate/prevent sand heave into the augers due to negative water gradients, which occurred in Borings B-004-1-23, B-004-2-23, and B-009-0-23. As the borings were sealed immediately upon completion due to their locations within the roadway pavement, delayed water readings were not obtained. Groundwater levels and possible perched water conditions can vary throughout the year depending on precipitation and other seasonal variations.

Table 4-2: Encountered Groundwater Elevations

Exploration Number	Existing Ground Surface Elevation (ft)	Groundwater Depth (ft)	Groundwater Elevation (ft)
B-001-1-23	772.6	--	--
B-002-1-23	770.9	--	--
B-002-2-23	768.2	--	--
B-003-1-23	770.3	33.0	737.3
B-004-1-23	768.1	31.0	737.1
B-004-2-23 [1]	771.5	42.5	729.0
B-008-0-23	768.1	30.0	738.1
B-009-0-23 [2]	764.2	--	--
B-010-0-23	763.0	27.0	736.0
B-011-0-23	761.5	22.5	739.0

Notes:
1. B-004-2-23 Groundwater depth estimated based on 1 foot of sand heave at 42.5 feet. Water added to borehole at 32.5 feet to assist in drilling activities and obscuring true groundwater levels
2. B-009-0-23: Water and bentonite slurry added to borehole to assist in drilling and prevent sand heave.

5. ANALYSIS AND RECOMMENDATIONS

5.1 Discussion

SR 4 and portions of Ramps J, K, and L within the SR 4/Stanley Avenue interchange were constructed in the late 1950's over what was noted at the time to be both active and abandoned dumps. These sections of SR 4 and the ramps have experienced continued settlement and pavement deformation throughout the duration of its service life, which is impacting rideability and creating unsafe conditions for drivers that encounter these depressions and dips in the pavement. Undulations and dips in the vertical profile were readily distinguishable during our site reconnaissance from approximately Sta. 87+00 to Sta. 93+00 along the mainline, from about Sta. 5+00 to Sta. 9+00 along Ramp K, and from roughly Sta. 2+50 to Sta. 5+00 on Ramp L. Survey information as presented in Appendix F indicates these limits extend slightly farther, from approximately Sta. 85+00 to Sta. 95+00 along SR 4, from about Sta. 3+50 to Sta. 10+00 on Ramp

K, and from roughly Sta. 0+50 to Sta. 5+50 on Ramp L. Settlement, which was not visually evident, is also occurring along Ramp J from approximately Sta. 2+25 to Sta. 3+75.

Review of the MOT-4-19.73 construction plans (ODOT, 1958) indicates that the limits of active and abandoned dumps that were compacted and buried roughly correspond to these areas. However, they appear to extend beyond these areas to both the south and west. Along the mainline, what is designated as an abandoned dump extends to roughly Sta. 96+10. Along Ramp J and Ramp K, the limits of an active dump extended to about Sta. 4+50 and to Sta. 10+25, respectively. The dump or refuse material as encountered within these limits in the 2023 borings generally consisted of intermittently-observed construction debris (brick, glass, asphalt, wood, metal items) dispersed within a gravel with sand and silt soil matrix. No voids or observed discrete pockets of construction debris or other landfill material were encountered.

The historic borings drilled in 1958 note that the refuse consisted of random materials (gravel, ashes, cans, glass, rubber, rags, tires, bottles, and tin.) These materials were to be compacted with a 50-ton pneumatic tired roller according to the MOT-4-19.73 construction plans, with granular material used to aid in leveling the area during the compaction process. This description helps to explain the condition of the refuse material as encountered in the 2023 borings, along with that in historic Boring B-001-0-16.

The type, quantity, and thickness of the refuse material varied across the site, along with the depth of the refuse material from the existing road surface, which makes the differential settlement that has occurred as a result of degradation and compression of the refuse material difficult to predict. To more fully mitigate the effects of this differential settlement occurring along SR 4 and Ramps J, K, and L would require:

- the use of rigid inclusions to transfer the overlying embankment and traffic loading below the refuse material to the underlying native soils,
- the improvement of the refuse material using a deep cement grout injection program to compact and strengthen the refuse, or
- the removal of the underlying refuse material.

However, these alternatives were determined to be impractical given the depth, thickness, and limits of the refuse material, together with the environmental impacts of exposing/interacting with the refuse, and the cost effectiveness of these options. As such, it was decided to construct a geogrid-reinforced soil mat to create a more uniform and interconnected subgrade, help bridge the underlying layer of refuse material, and aid in reducing the noticeable effects of the differential settlement.

The advantages of a geogrid-reinforced soil mat are it allows for part-width construction, which could minimize traffic impacts within the interchange, limits the potential to encounter and expose the refuse material and construction debris to those areas near the northeastern edges of the abandoned dump where refuse material was encountered at a relatively shallow depth, and allows for reuse of the existing on-site granular embankment material. However, as this improvement does not augment or densify the layer of refuse and construction debris, there is a potential the embankment will continue to settle due to degradation and compression of the refuse material.

5.1.1 Undercut and Replacement with Geogrid-Reinforced Soil Mat

Construction of the geogrid-reinforced soil mat involves the removal of the existing pavement system, undercutting the existing granular subgrade materials, and replacing the material with granular embankment material with interbedded layers of geogrid. From discussions with representatives of Tensar Corporation (Tensar), a global ground stabilization and soil reinforcement provider, the effects of the differential settlement occurring along the mainline and associated ramps may be greatly diminished by undercutting the existing subgrade to a depth of 36 inches below the bottom of the proposed pavement system and installing a minimum of three layers of Tensar InterAx™ geogrid. The first or lowest layer is to consist of a layer of NX850-FG (FG stands for FilterGrid™, a composite geosynthetic consisting of a NX850 geogrid bonded to a nonwoven geotextile) placed along the bottom of the 36-inch undercut. This composite geosynthetic is recommended to provide not only the needed stabilization and confinement to restrict movement, but also to provide separation between the geogrid-reinforced soil mat and the underlying soils and prevent the migration of fines into the backfill. Prior to placement of this initial geogrid layer, the bottom of the 36-inch undercut should be compacted according to CMS 204.03 and pass a proof roll per CMS 204.06 to create a solid platform to construct the geogrid-reinforced soil mat upon. Barring the discovery of the existing pavement thickness exceeding 36 inches or encountering refuse material at the bottom of the undercut in localized areas, the proposed bottom elevation of the geogrid-reinforced soil mat will be the maximum depth of the undercut in most cases. Only additional reworking and recompaction of the encountered soils at the bottom of the excavation is anticipated should they fail to pass the proof-roll. A second, middle layer of geogrid consisting of NX850 geogrid is to be placed at 24 inches below the bottom of the pavement system, and a third, upper layer of NX850 is to be placed at 12 inches below the proposed bottom of the pavement system.

The backfill material for the 36-inch undercut is to consist of the on-site embankment materials which classify as a gravel (A-1-a) or gravel with sand (A-1-b). These gravels are generally well-graded with fine contents of less than 15%, as shown on the boring logs in Appendix D and highlighted on the lab sheet provided in Appendix F. However, additional suitable off-site materials may be necessary to achieve the proposed subgrade elevation. These off-site borrow materials are to meet the requirement of ODOT Item 703.16 Type B Granular material, with a fines content of less than 15% passing the No. 200 sieve. A minimum loose lift thickness of 6 inches of aggregate or backfill materials is to be placed prior to the operation of tracked vehicles over the geogrid, and each lift compacted per the compaction requirements as presented in CMS Item 204.03.

The recommended limits for the undercut and replacement (geotechnical improvements) are summarized in Table 5-1 below and on the Paving and Geotechnical Limits plan (Exhibit No. 9).

Table 5-1: Estimated Geotechnical Improvement Limits[1]

Alignment	Beginning Station	Ending Station	Pavement Section Width (ft) [2]	Anticipated Improvement Section Width (ft) [3,4]	Estimated Improvement Area (sq. ft.) [4]
Northbound SR 4	85+25	96+75	40	50	57,500
Southbound SR 4 [5]	85+25	91+00	40	50	29,000
Southbound SR 4 [6]	91+00	96+75	50	60	34,500
Ramp J	0+50	4+50	27	37	15,000
Ramp K	3+75	10+25	27	37	24,000
Ramp L	2+00	6+00	27	37	15,000
				TOTAL	**175,000**

Notes:
1. The limits of the geotechnical improvements do not include MOT or new pavement/overlays that may extend beyond this work as part of the overall roadway improvement project.
2. Includes paved travel lanes and shoulders.
3. Assumes an additional 5 feet to the left and right of the paved areas.
4. Approximated and rounded to the nearest 500 sq ft.
5. 2-lane alignment east of Ramp L Exit
6. 3-lane alignment approaching Ramp L Exit

These improvements are to extend at least 5 feet beyond the outside edge of the paved shoulders given the depth of the undercut and the assumed thickness of the pavement section.

5.1.2 Potential Voids

As previously discussed, the construction of a geogrid-reinforced soil mat will provide a more uniform and interconnected support system for the overlying pavement system while helping to reduce or alleviate the noticeable effects of the differential settlement. This additional support is achieved based on the principle of the InterAx™ geogrid serving to interlock the aggregates between the geogrid layers such that the aggregate is considered to be fully confined. However, this improvement does not mitigate the source of the settlement that is occurring at the site, and the underlying embankment material may continue to settle. Should this occur, the geogrid-reinforced soil mat will aid in supporting the pavement by redistributing the loading and straddling future areas of settlement. However, there is the potential for a void to develop between the underlying embankment fill and the geogrid mat. As such, supplemental analyses were performed to estimate the tolerable size of a potential void due to possible continued embankment settlement. References regarding the use of geogrid mats to span voids and sinkholes are listed in this report with excerpts from these references provided in Appendix F.

The InterAx™ geogrids, along with its predecessor the TriAx® geogrids, work under the principal of increasing the modulus of the aggregate layer through confinement of the aggregate. This concept of an increased modulus of the aggregate layer was not considered in the available technical resources and references regarding the use of geogrid mats to span over voids. Rather, these resources incorporate the tensile strength of the geogrid into their formulas. This tensile strength is not readily available for the InterAx™ geogrids based on Tensar's more modulus-based approach on their latest geogrid products. However, the tensile strength is still noted on the data sheet for the TriAx® geogrids. As such, the analyses were performed using the parameters provided for the Triax® geogrid to gain general insight into the overall performance of the proposed geogrid-reinforced soil mat should a void occur under SR 4 or one of its associated ramps at the Stanley Avenue interchange due to continued settlement. Based on the analyses included in Appendix F, the geogrid mat may withstand an infinitely long void with a width of approximately 5 feet or a circular void that is approximately 10 feet in diameter.

5.1.3 Subgrade Strength

An estimate of the subgrade strength was developed considering the expected backfill material for the geogrid-reinforced soil mat utilizing available grain sizes collected within or near the upper 6 feet of the recent test borings, the historic DCP logs and test data, and published correlations. Based on this available information, a design CBR of 20 is recommended. This is equivalent to a subgrade resilient modulus (M_R) of 24,000 psi.

5.1.4 Constructability

Maintenance of traffic is a concern, particularly with regards to ramp access, as excavations of approximately 5 feet below the existing pavement grade are required. With a predominantly granular profile, this will require the use of non-vertical slopes for sidewall stability and to maintain the limits of the excavation, or the installation of temporary shoring should clear space from an obstruction or running traffic be an issue. To eliminate this concern, we understand the MOT scheme will temporarily close Ramps J, K, and L.

5.2 Recommendations

SR 4 and portions of Ramps J, K, and L within the SR 4/Stanley Avenue interchange were constructed in the late 1950s over what was noted at the time to be both active and abandoned dumps. Today, both the mainline and ramps within the project area have experienced continued settlement and pavement deformation as a result of compression and degradation of the underlying refuse material, which is impacting rideability and creating unsafe conditions for drivers that encounter these depressions and dips in the pavement. As such, it was decided that a geogrid-reinforced soil mat be constructed to create a more uniform and interconnected subgrade, help bridge the refuse material, and aid in reducing the noticeable effects of the differential settlement. However, as the geogrid mat does not improve or densify the layer of refuse and construction debris, there is a potential the embankment continues to settle.

5.2.1 Geogrid-Reinforced Soil Mat

- Install the geogrid reinforced soil mat within the geotechnical limits as shown on Exhibit 9 and in the Geotechnical Plan and Profile sheets in Appendix E. These limits generally extend 5 feet beyond the edge of the proposed pavement.

- Undercut the existing subgrade to a depth of 36 inches below the bottom of the proposed pavement. Compact the bottom of the bottom of the undercut in accordance with CMS

204.03 and perform a proof roll per CMS 204.06 to create a solid platform to construct the geogrid-reinforced soil mat upon.

- Install three layers of Tensar InterAxTM geogrid with the undercut: A layer of Tensar NX850-FG along the bottom of the 36-inch undercut and two layers of NX850 geogrid, one at 24 inches below the bottom of the pavement system, and the second at 12 inches below the proposed bottom of the pavement system.

- Drain the geogrid reinforced soil mat to an underdrain, catch basin, or pipe.

- Include the plan note "ITEM 204 – GEOGRID (AS PER PLAN)" provided in Appendix G in the general notes.

5.2.1.1 GRANULAR BACKFILL MATERIALS

- The granular backfill materials to be utilized within the geogrid-reinforced soil mat are to consist of Item 204 granular embankment material as per plan. (See the plan note in Appendix G.) Include this note in the general notes.

- The granular backfill materials to be utilized within the geogrid-reinforced soil mat are to consist of the existing on-site embankment materials meeting the Department Group Classifications A-1-a and A-1-b, with the maximum grain size being less than 3 inches and the fines contents (passing the No. 200 sieve) no more than 15%.

- If additional granular backfill material is necessary to meet the planned subgrade elevation, materials conforming to ODOT Item 703.16 Type B Granular material is acceptable provided the fines contents does not exceed 15%.

5.2.2 Pavement Design

- A CBR of 20 and a subgrade resilient modulus (Mr) of 24,000 psi is recommended for pavement design.

6. LIMITATIONS

This geotechnical report documents the findings of HDR Engineering, Inc., for the geotechnical aspects related to the MOT-4-19.30 roadway improvement project in Dayton, Ohio. The report has been prepared for use by Chagrin Valley Engineering and the Ohio Department of Transportation for specific application to the project, in accordance with generally accepted engineering practice. No warranty, expressed or implied, is made. Any analyses or recommendations submitted are based on the field explorations performed at the locations indicated, on specific laboratory tests on individual samples taken during the exploration, and information obtained from outside sources. The geotechnical report and findings do not reflect variations that could occur between exploration locations or at other points in time. Variations in conditions, if any, may become evident during the construction period, at which time, a re-evaluation of any recommendations may become necessary. In the event of such changes, the recommendations and changes should be reviewed by HDR's geotechnical staff.

7. REFERENCES

State of Ohio Department of Transportation, (Updated January 2023); "*Specifications for Geotechnical Explorations".*

U.S. Department of the Interior, U.S. Geological Survey, (2023); *"The National Map, US Topo: Dayton North Quadrangle, Ohio, Montgomery County, 7.5-minute series".*

Ohio Department of Natural Resources, Division of Geologic Survey, (1998); *"Physiographic Regions of Ohio".*

Ohio Department of Natural Resources, Division of Geologic Survey, Brockman, C.X., Pavey, R.R., Schumacher, G.A., Shrake, D.L., Swinford, E.M., and Vorbau, K.E., with GIS Production and Cartography by Powers, D.M., Wells, J.G., and Martin, D.R., (2011); *"Surficial geology of the Ohio portion of the Dayton 30 x 60-minute quadrangles".*

United States Department of Agriculture: Natural Resources Conservation Service, (2022); *"Web Soil Survey"*. http://websoilsurvey.nrcs.usda.gov/app/

Ohio Department of Natural Resources, Division of Geologic Survey, Swinford, E.M., (1994); *"Reconnaissance Bedrock Geology of the Dayton North, Ohio quadrangle".*

Ohio Department of Natural Resources, Division of Geologic Survey, Leow, J.A. and Brockman, C.S., (1994); *"Bedrock topography of the Dayton North, Ohio, quadrangle".*

Ohio Department of Transportation, (2020); *"Geotechnical Bulletin 7: Drilled Shaft Landslide Stabilization Design".*

American Association of State Highway and Transportation Officials, (2020); *"AASHTO LRFD Bridge Design Specifications".*

Sloan, Joel A., Filz, George M., Collin, Fames G., Kumar, Pawan. *"Column-Supported Embankments: Full-Scale Tests and Design Recommendations."* 2nd Edition. Virginia Tech Center for Geotechnical Practice Research (CGRP) Report 77. (2014).

Federal Highway Administration, Elias, Victor et al. *"Mechanically Stabilized Earth Walls and Reinforced Soil Slopes: Design and Construction Guidelines (Updated Version)"*, FHWA-NHI-00-043, 2001

Giroud, Jeanpierre & Bonaparte, Rudolph & Beech, J. & Gross, Beth. (1988). Load-carrying capacity of a soil layer supported by a geosynthetic overlying a void. Proceedings of the International Geotechnical Symposium on Theory and Practice of Earth Reinforcement. 185-190.

Giroud, Jeanpierre & Bonaparte, Rudolph & Beech, J.F. & Gross, Beth. (1990). Design of soil layer-geosynthetic systems overlying voids. Geotextiles and Geomembranes. 9. 11-50. 10.1016/0266-1144(90)90004-V.

Koerner, Robert M.. "Designing with Geosynthetics." Fifth Edition. (2005).

Lee, Hyung. (2017). *Recommended Practice for Incorporating Geogrids in ME Pavement Design*.

Agaiby, Sherif & Jones, Colin. (2011). Design of reinforced fill systems over voids. Canadian Geotechnical Journal. 32. 939-945. 10.1139/t95-093.

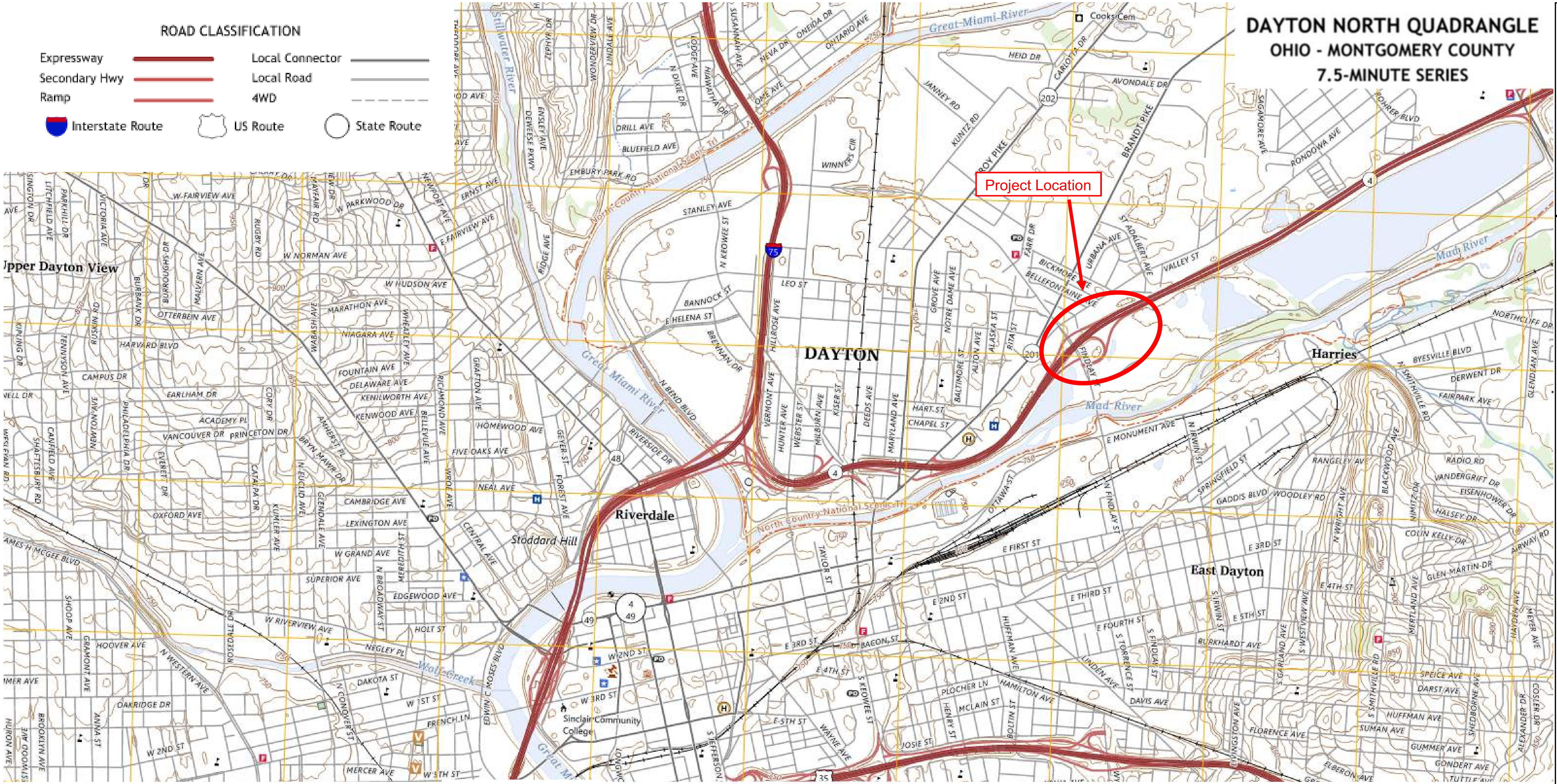
Nancey, A & Philippe, Delmas. (2022). *Basal reinforcement of embankment over voids and soil subsidence area – new standard XP G38065*. IOP Conference Series: Materials Science and Engineering. 1260. 012041. 10.1088/1757-899X/1260/1/012041.

Blivet, J.C. & Khay, Murm & Villard, Pascal & Gourc, Jean-Pierre. (2000). *Design method for geosynthetic as reinforcement for embankment subjected to localized sinkholes.* GeoEng2000, international conference on geotechnical and geological engineering. 1-6.

Brusa, N & Naughton, P. & Scotto, M. (2018). *Design of geosynthetics over areas prone to subsidence*. (from Proceedings of the 11th International Conference on Geosynthetics, Seoul, Korea).

Unified Facilities Criteria, UFC-3-260-02: Pavement Design For Airfields, 30-Jun-2001

Appendix A. Exhibits



DAYTON NORTH, OH

2023

MN

GN

6°20´
113 MILS

1°48´
32 MILS

UTM GRID AND 2023 MAGNETIC NORTH DECLINATION AT CENTER OF SHEET

U.S. National Grid
100,000 - m Square ID
GK
Grid Zone Designation
16S

SCALE 1:24 000

KILOMETERS 1 0.5 0 1 2

METERS 1000 500 0 1000 2000

MILES 1 0.5 0 1

FEET 1000 0 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000

CONTOUR INTERVAL 10 FEET
NORTH AMERICAN VERTICAL DATUM OF 1988

This map was produced to conform with the National Geospatial Program US Topo Product Standard.

OHIO

QUADRANGLE LOCATION

1	2	3
4		5
6	7	8

1 West Milton
2 Tipp City
3 New Carlisle
4 Trotwood
5 Fairborn
6 Miamisburg
7 Dayton South
8 Bellbrook

ADJOINING QUADRANGLES

Produced by the United States Geological Survey

North American Datum of 1983 (NAD83)
World Geodetic System of 1984 (WGS84). Projection and 1 000-meter grid: Universal Transverse Mercator, Zone 16S

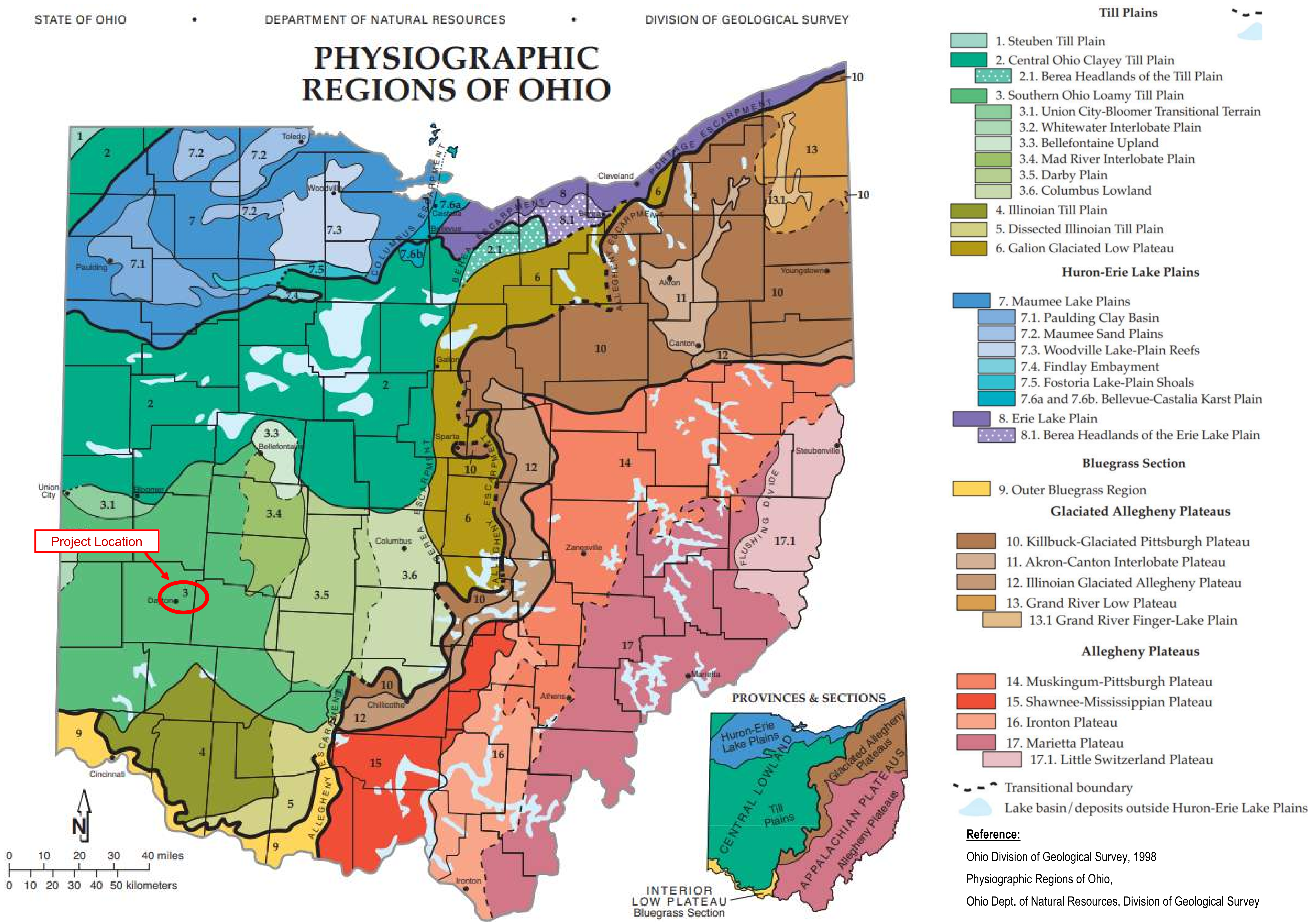
This map is not a legal document. Boundaries may be generalized for this map scale. Private lands within government reservations may not be shown. Obtain permission before entering private lands.

Imagery....................NAIP, June 2017 - December 2017
Roads....................U.S. Census Bureau, 2016 - 2016
Names....................GNIS, 1979 - 2023
Hydrography....................National Hydrography Dataset, 2006 - 2022
Contours....................National Elevation Dataset, 2010
Boundaries....................Multiple sources; see metadata file 2020 - 2022
Public Land Survey System....................BLM, 2017
Wetlands....................FWS National Wetlands Inventory 2004 - 2007

Calculated: DCM
Checked: DMV

Exhibit No. 1 : Site Vicinity and Topographic Map

Project: MOT-4-19.30
PID : 117239



Reference:

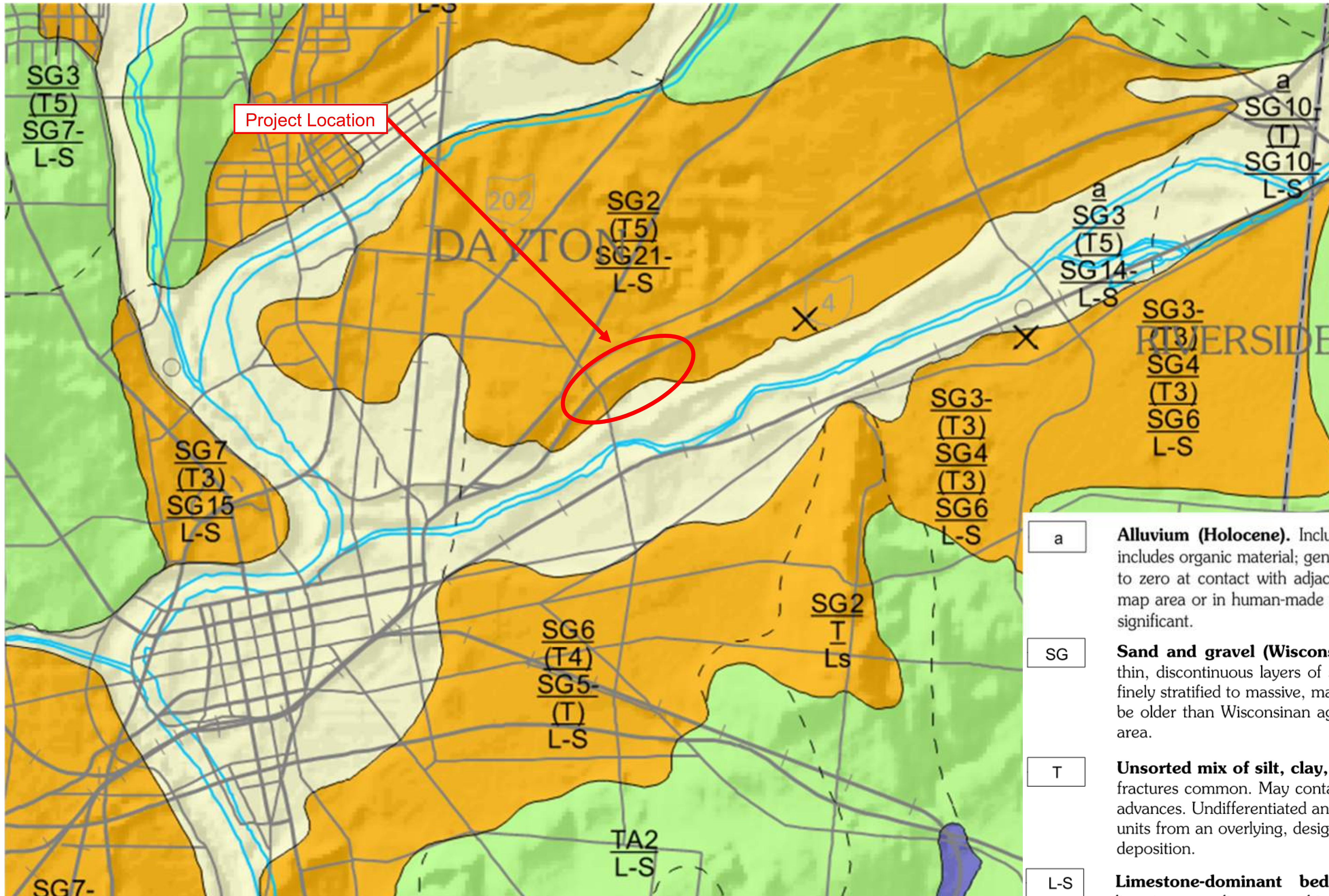
Ohio Division of Geological Survey, 1998
Physiographic Regions of Ohio,
Ohio Dept. of Natural Resources, Division of Geological Survey

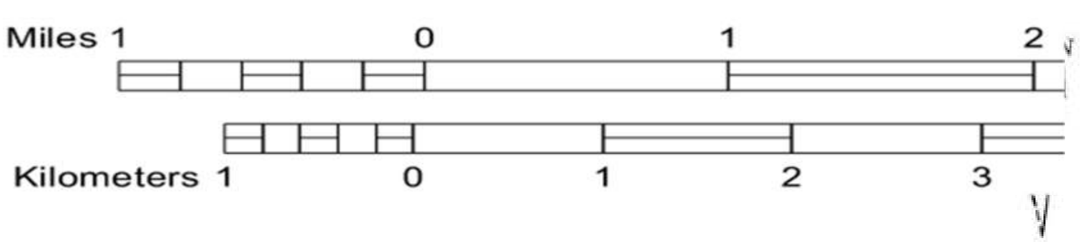
Calculated: DCM
Checked: DMV

Exhibit No. 2 : Physiographic Regions of Ohio

Project: MOT-4-19.30
PID : 117239

Exhibit No. 3 : Surficial Geology





KEY TO LITHOLOGIC COLORS*

- w - Water
- m - Man made
- a - Alluvium
- o - Organics
- C - Clay (Wisconsinan)
- L - Silt (Wisconsinan)
- S - Sand (Wisconsinan)
- SG - Sand and Gravel (Wisconsinan)
- IC - Ice-contact (Wisconsinan)
- T - Unsorted mix (Wisconsinan)
- TA - Loam till (Wisconsinan)
- G - Gravel (Wisconsinan)
- L-S Limestone-dominant bedrock (Ordivician)
- Ls - Limestone
- S-L - Interbedded shale, limestone, and dolomite (Upper Ordivician)
- D - Dolomite

a — **Alluvium (Holocene).** Includes a wide variety of textures from silt and clay to boulders; commonly includes organic material; generally not compacted. Rarely greater than 20-ft thick; unit considered to thin to zero at contact with adjacent polygons. Present in floodplains of modern streams throughout entire map area or in human-made water retention features. Mapped only where areal extent and thickness are significant.

SG — **Sand and gravel (Wisconsinan).** Intermixed and interbedded sand and gravel commonly containing thin, discontinuous layers of silt and clay; grains well to moderately sorted, moderately to well rounded; finely stratified to massive, may be cross bedded; locally may contain organics. In deep buried valleys, may be older than Wisconsinan age. Present as valley wall terraces and in buried valleys throughout the map area.

T — **Unsorted mix of silt, clay, sand, gravel, and boulders (Wisconsinan**). Variable carbonate content, fractures common. May contain silt, sand, and gravel lenses. Deposited directly from several separate ice advances. Undifferentiated and nonspecified age in buried valleys or where separated by intervening nontill units from an overlying, designated till. Surface may be wave planed or modified by lacustrine erosion and deposition.

L-S — **Limestone-dominant bedrock and bedrock-derived colluvium (Ordivician).** Interbedded limestone, medium gray, thin to medium bedded, fossiliferous, and shale, gray, thin to medium bedded. Limestone ranges from 50%–80% of the unit, although shale-rich beds are present. Includes Point Pleasant, Fairview, Grant Lake, Arnheim, Liberty, and Whitewater Formations. On side-slopes and toe-slopes, unit is colluvium, predominantly clay with downslope-oriented limestone slabs and organic matter. Colluvium has relatively low shear strength and is the source of numerous landslides, especially on steep slopes.

* Small area of organic deposits.

× Quarry, mine, or strip mine; floored in bedrock; may contain reclaimed areas.

× Sand-and-gravel pit. Pit bottom generally underlain by unconsolidated lithologic units of surrounding polygon(s). May contain reclaimed areas.

—— Boundary between map-unit areas having different uppermost, continuous lithologies or significant bedrock lithology change; underlying lithologies may or may not differ.

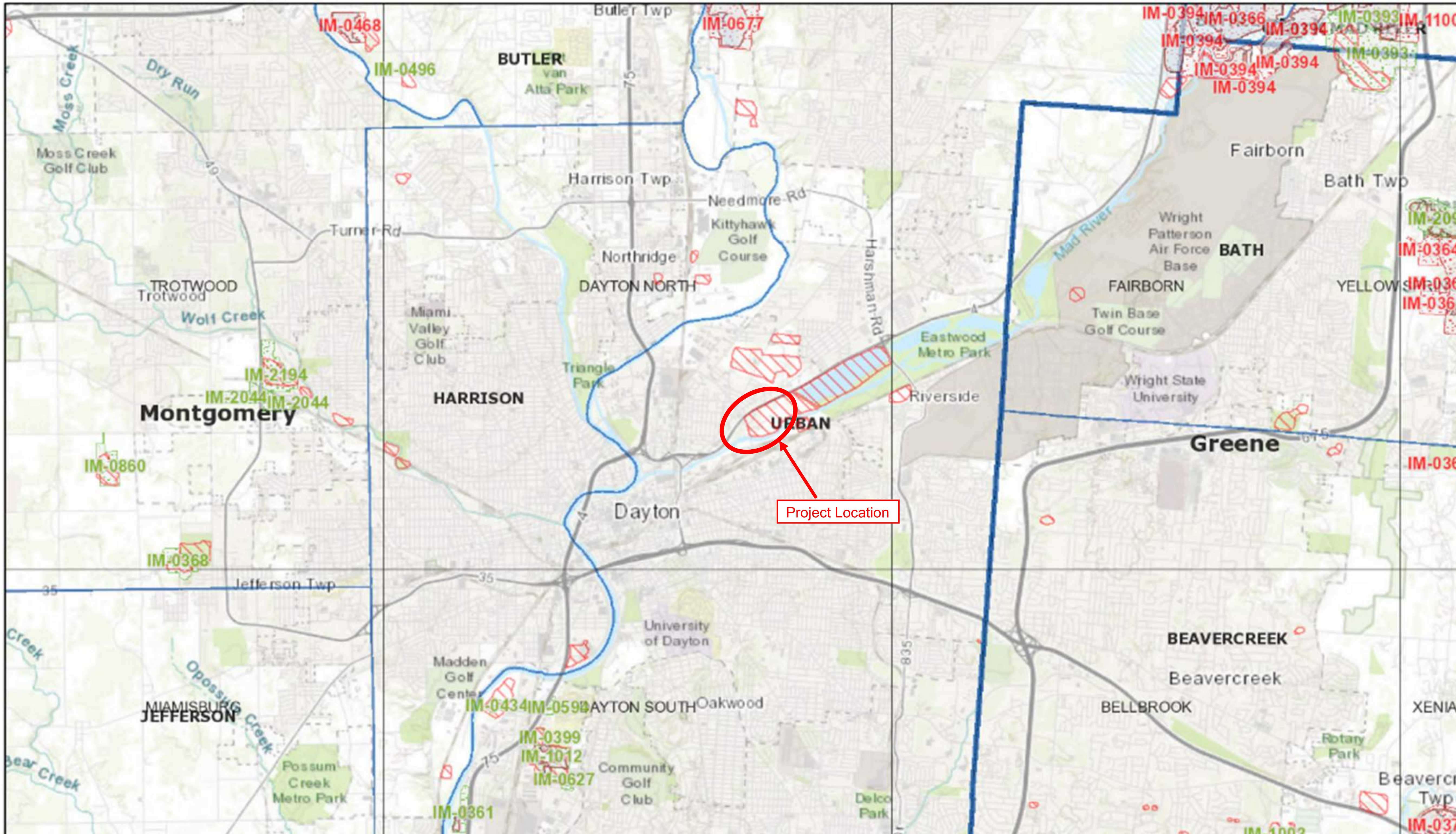
------ Boundary between map-unit areas having the same uppermost, continuous lithology but different thicknesses or different underlying lithologies.

Reference:

Brockman, C.S., Pavey, R.R., Schumacher, G.A., Shrake, D.L., Swinford, E.M., and Vorbau, K.E., with GIS production and cartography by Powers, D.M., Wells, J.G., and Martin, D.R., 2011, Surficial geology of the Ohio portion of the Dayton 30 x 60-minute quadrangle: Columbus, Ohio Department of Natural Resources, Division of Geological Survey Map SG-2-DAY, scale 1:100,000.

Calculated: DCM
Checked: DMV

Project: MOT-4-19.30
PID: 117239



August 2, 2023

Quadrangle 24K (7.5 min)	
Abandoned before 1977	
Past	
Current	
Proposed	
Original Application	
Adjacent Area Application	
Historic - From Geology Maps	
Past	
Current	
Proposed	
Original Application	
Adjacent Area Application	
Known	
Partially Known	
Past	
Current	
Permitted	

1:144,448

0 0.75 1.5 3 mi

0 1.5 3 6 km

County of Montgomery, OH, Esri, HERE, Garmin, USGS, NGA, EPA, USDA, NPS

Ohio Dept. of Natural Resources

Calculated: DCM

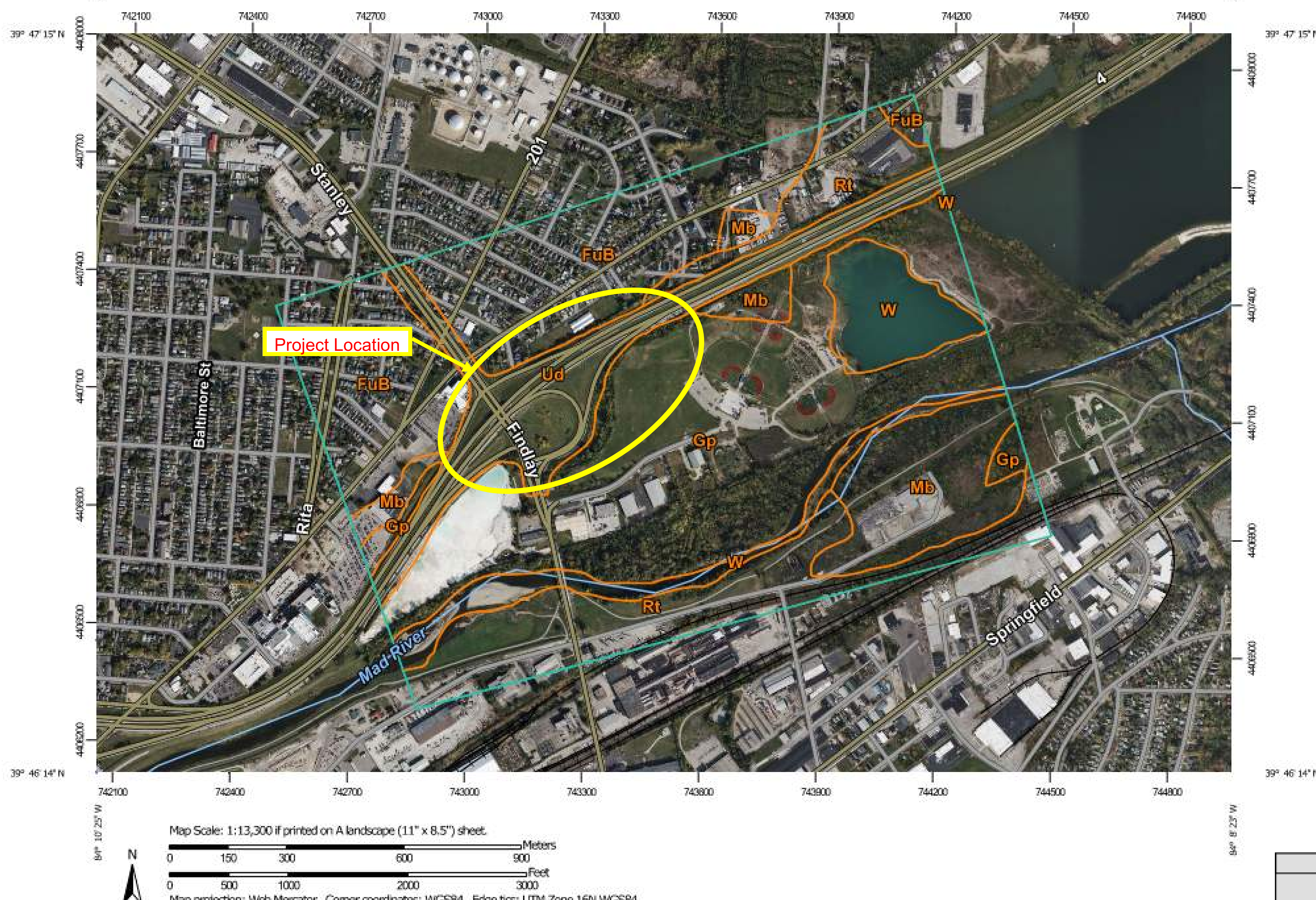
Checked: DMV

Exhibit No. 4 : Mine Map

Project: MOT-4-19.30

PID : 117239

Soil Map—Montgomery County, Ohio
(MOT-4-19.30)



MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:15,800.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Montgomery County, Ohio
Survey Area Data: Version 21, Sep 9, 2022

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Oct 9, 2020—Nov 5, 2020

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Water Features

Streams and Canals

Transportation

Rails

Interstate Highways

US Routes

Major Roads

Local Roads

Background

Aerial Photography

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Map Unit Polygons

Soil Map Unit Lines

Soil Map Unit Points

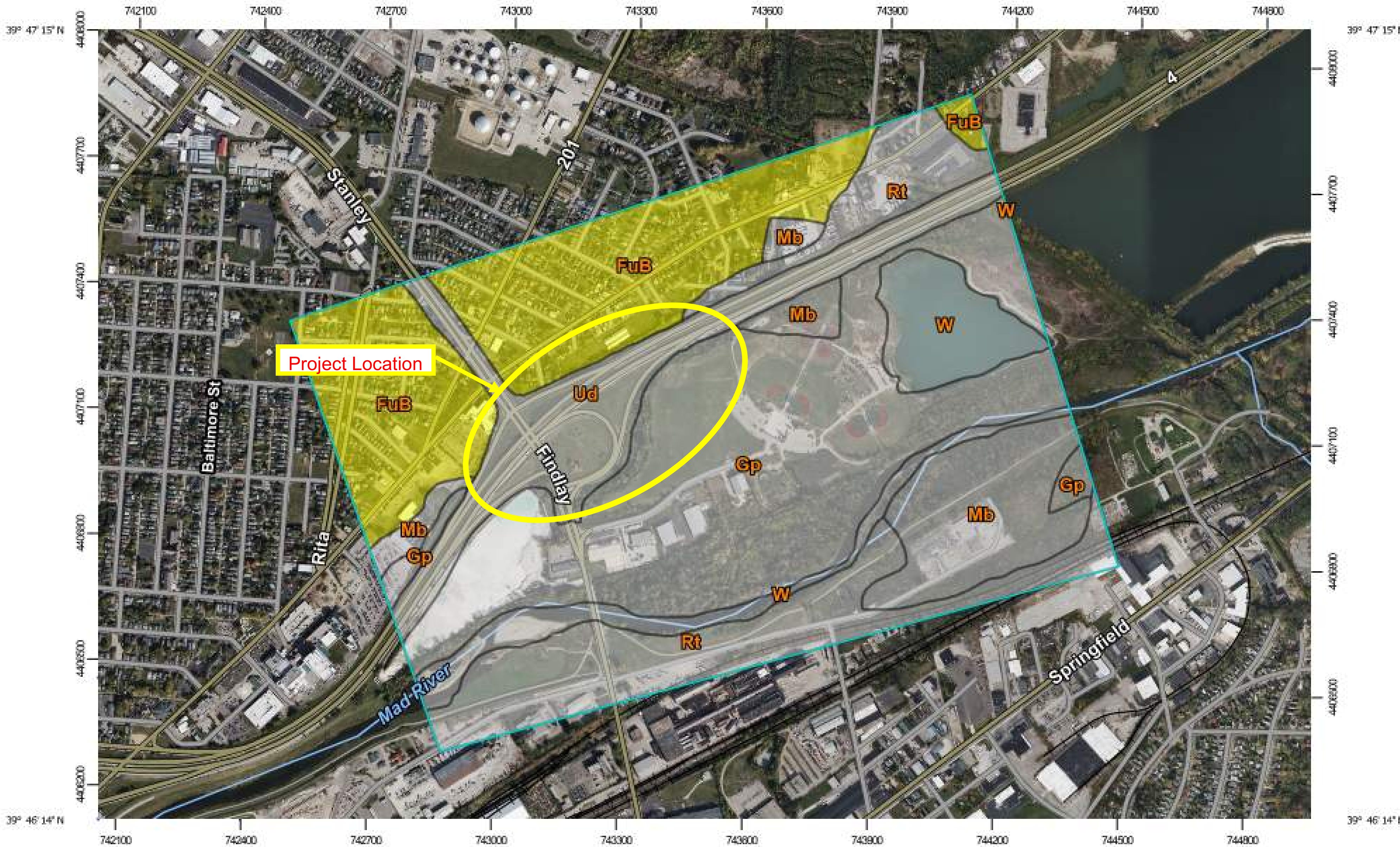
Map Unit Symbol	Map Unit Name
FuB	Fox-Urban land complex, 2 to 6 percent slopes
Gp	Gravel pits
Mb	Made land
Rt	Ross-Urban land complex
Ud	Udorthents
W	Water
Totals for Area of Interest	

Calculated: DCM
Checked: DMV

Exhibit No. 5a : Soil Survey Map
Soil Types

Project: MOT-4-19.30
PID : 117239

Corrosion of Concrete—Montgomery County, Ohio
(MOT-4-19.30)



Map Scale: 1:13,300 if printed on A landscape (11" x 8.5") sheet.

Meters: 0, 150, 300, 600, 900

Feet: 0, 500, 1000, 2000, 3000

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 16N WGS84

Water Features

Streams and Canals

Transportation

Rails

Interstate Highways

US Routes

Major Roads

Local Roads

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Rating Polygons

High

Moderate

Low

Not rated or not available

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:15,800.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Montgomery County, Ohio
Survey Area Data: Version 21, Sep 9, 2022

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Oct 9, 2020—Nov 5, 2020

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

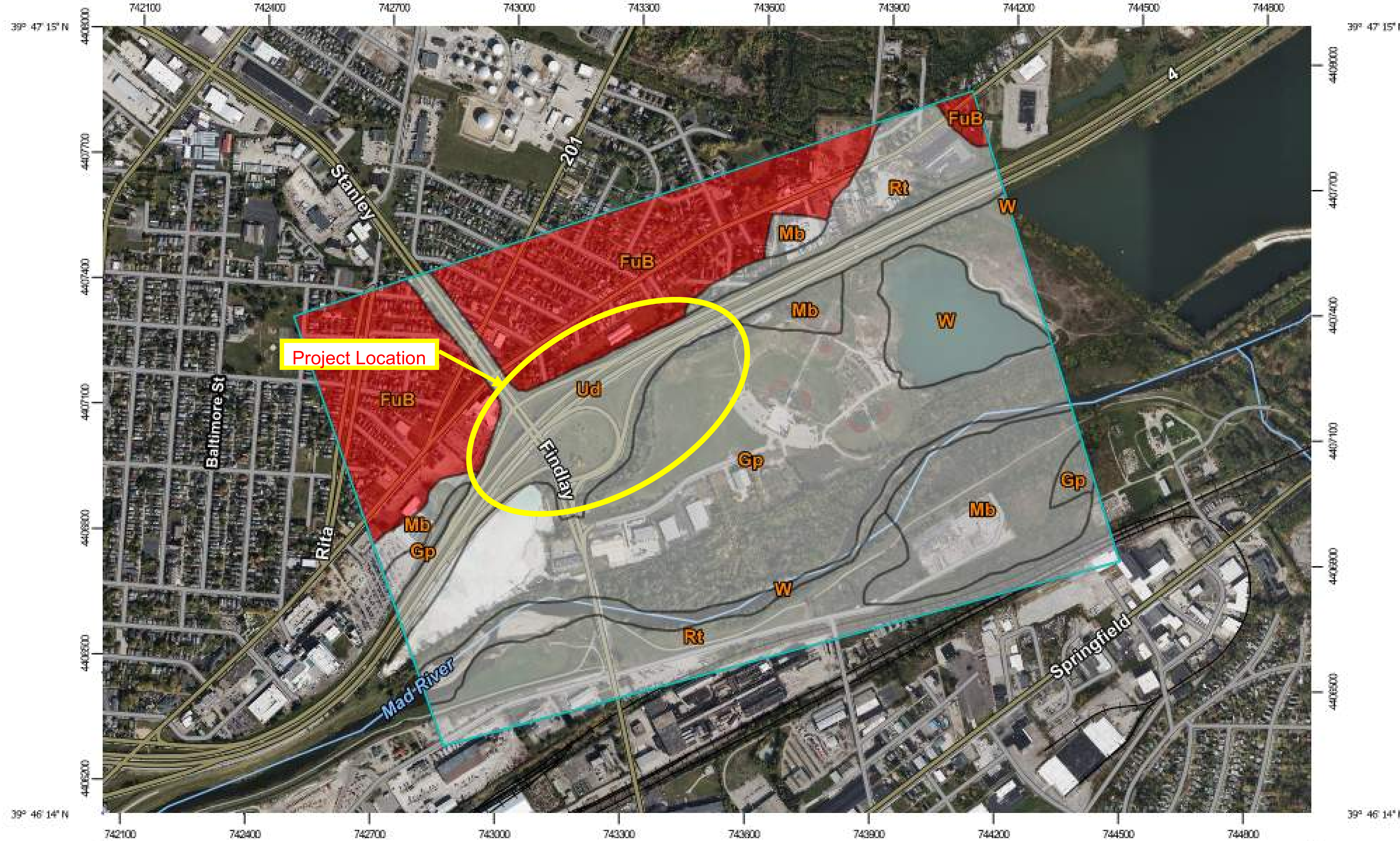
Map unit symbol	Map unit name	Rating
FuB	Fox-Urban land complex, 2 to 6 percent slopes	Moderate
Gp	Gravel pits	
Mb	Made land	
Rt	Ross-Urban land complex	
Ud	Udorthents	
W	Water	
Totals for Area of Interest		

Calculated: DCM
Checked: DMV

Exhibit No. 5b : Soil Survey Map
Corrosion of Concrete

Project: MOT-4-19.30
PID : 117239

Corrosion of Steel—Montgomery County, Ohio
(MOT-4-19.30)



Map Scale: 1:13,300 if printed on A landscape (11" x 8.5") sheet.

N

0 150 300 600 900 Meters

0 500 1000 2000 3000 Feet

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 16N WGS84

Water Features

Streams and Canals

Transportation

Rails

Interstate Highways

US Routes

Major Roads

Local Roads

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Rating Polygons

High

Moderate

Low

Not rated or not available

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:15,800.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Montgomery County, Ohio
Survey Area Data: Version 21, Sep 9, 2022

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Oct 9, 2020—Nov 5, 2020

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

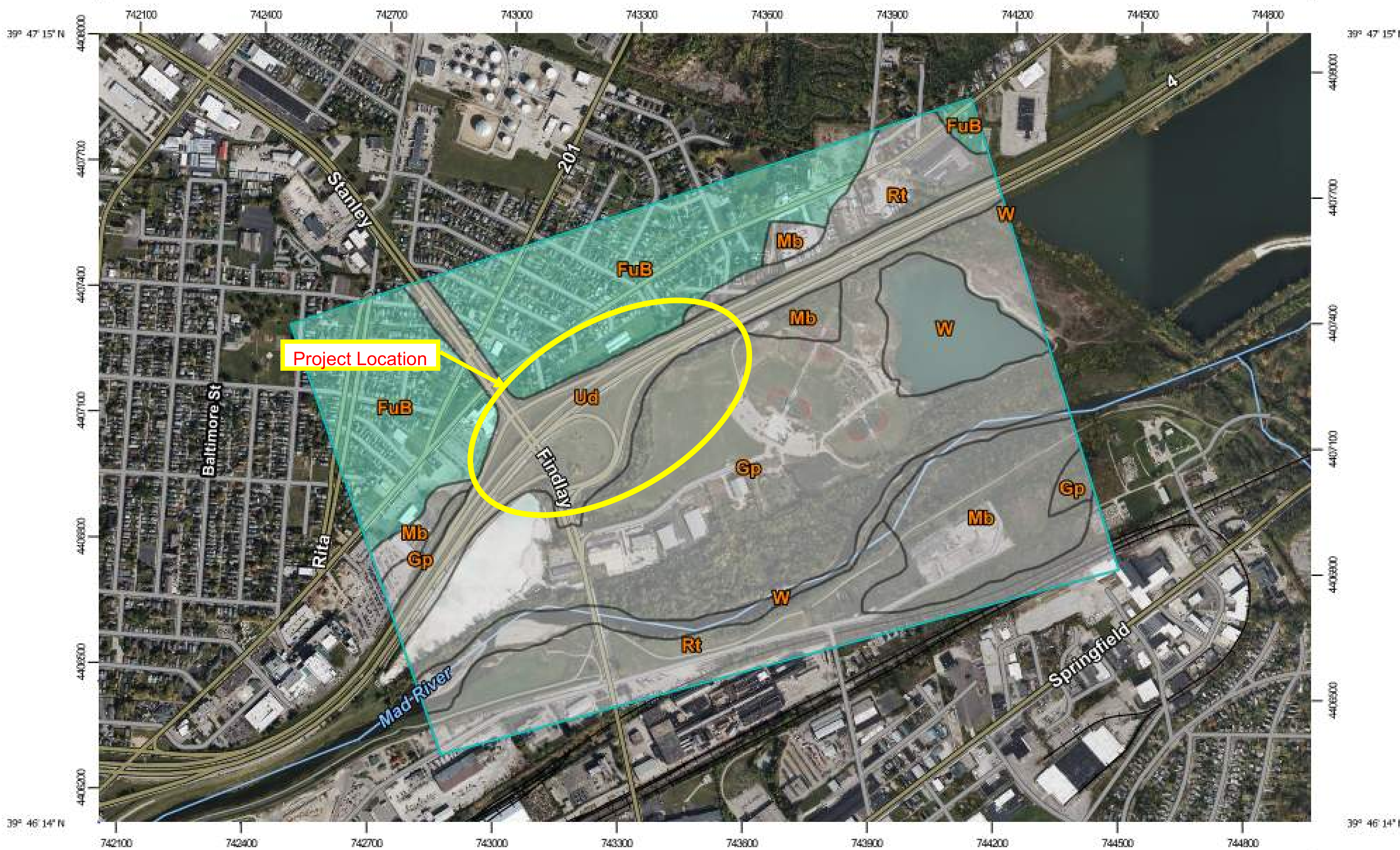
Map unit symbol	Map unit name	Rating
FuB	Fox-Urban land complex, 2 to 6 percent slopes	High
Gp	Gravel pits	
Mb	Made land	
Rt	Ross-Urban land complex	
Ud	Udorthents	
W	Water	
Totals for Area of Interest		

Calculated: DCM
Checked: DMV

Exhibit No. 5c : Soil Survey Map
Corrosion of Steel

Project: MOT-4-19.30
PID : 117239

pH (1 to 1 Water)—Montgomery County, Ohio
(MOT-4-19.30)



Map Scale: 1:13,300 if printed on A landscape (11" x 8.5") sheet.

Meters: 0 150 300 600 900

Feet: 0 500 1000 2000 3000

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 16N WGS84

Area of Interest (AOI)

- Area of Interest (AOI)

Soils

Soil Rating Polygons

- Ultra acid (ph < 3.5)
- Extremely acid (pH 3.5 - 4.4)
- Very strongly acid (pH 4.5 - 5.0)
- Strongly acid (pH 5.1 - 5.5)
- Moderately acid (pH 5.6 - 6.0)
- Slightly acid (pH 6.1 - 6.5)
- Neutral (pH 6.6 - 7.3)
- Slightly alkaline (pH 7.4 - 7.8)
- Moderately alkaline (pH 7.9 - 8.4)
- Strongly alkaline (pH 8.5 - 9.0)
- Very strongly alkaline (pH > 9.0)
- Not rated or not available

Water Features

- Streams and Canals

Transportation

- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:15,800.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Montgomery County, Ohio
Survey Area Data: Version 21, Sep 9, 2022

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Oct 9, 2020—Nov 5, 2020

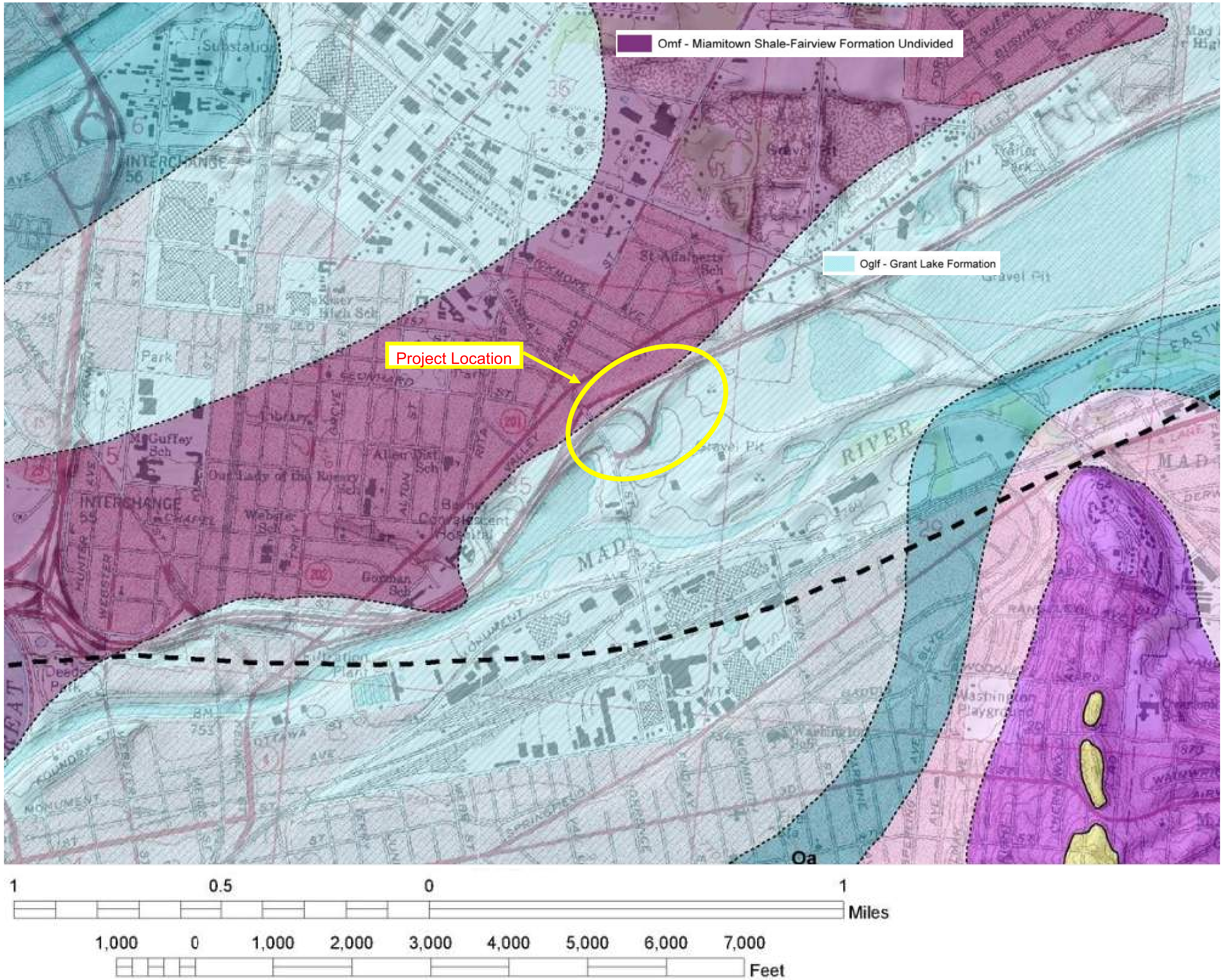
The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map unit symbol	Map unit name	Rating
FuB	Fox-Urban land complex, 2 to 6 percent slopes	7.2
Gp	Gravel pits	
Mb	Made land	
Rt	Ross-Urban land complex	
Ud	Udorthents	
W	Water	
Totals for Area of Interest		

Calculated: DCM
Checked: DMV

Exhibit No. 5d : Soil Survey Map
pH Levels

Project: MOT-4-19.30
PID: 117239



Explanation

Ow - Waynesville Formation
Oa - Arnheim formation
Oglf - Grant Lake Formation
Omf - Miamitown Shale-Fairview Formation Undivided

Contacts

Exposed
Concealed

Facies Changes and Mappable Limits

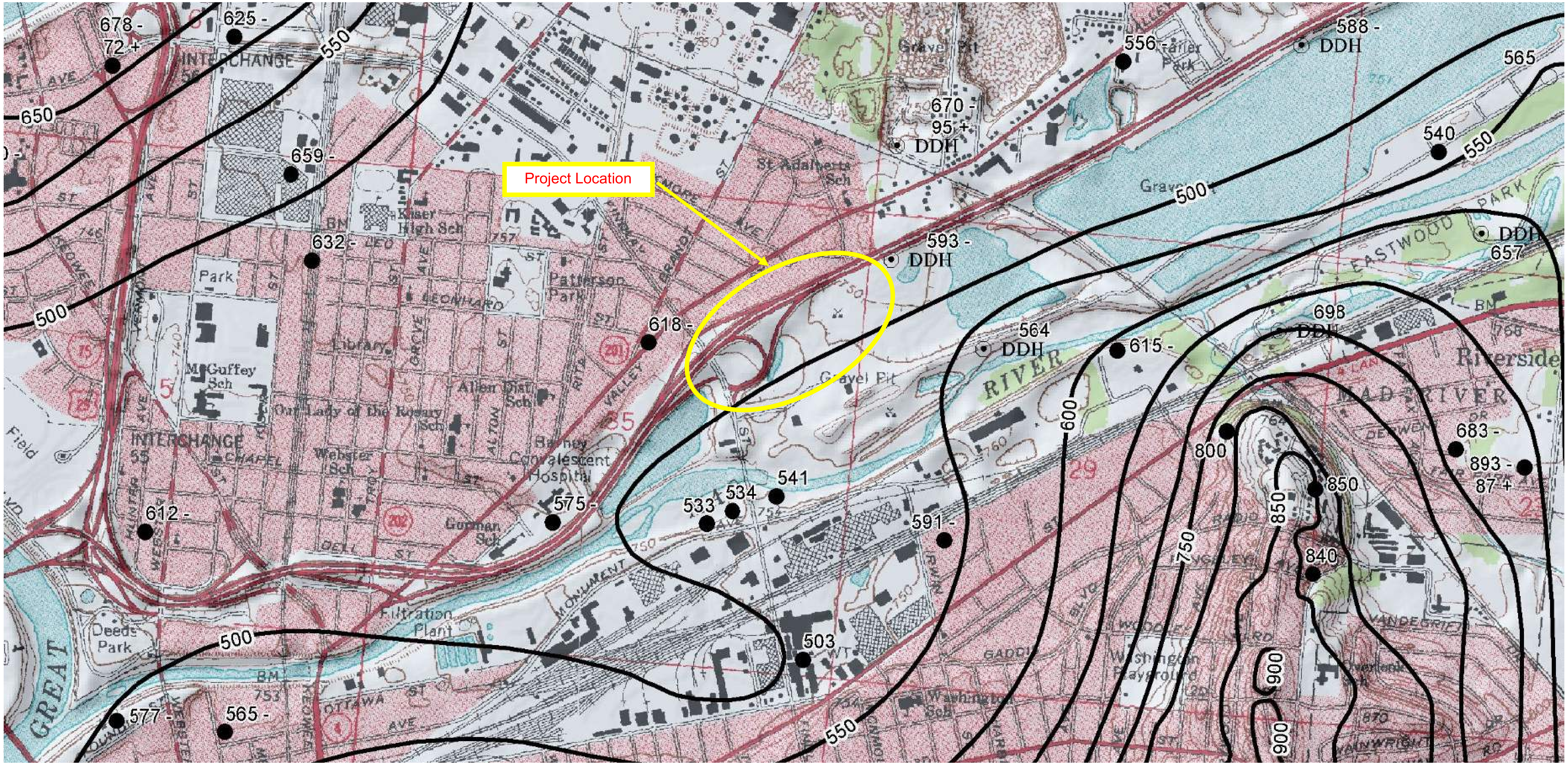
Northern mappable limit of Odwl, Ow, Oa, and Ogl
Western limit of Scse and Sm-b

- **Title:** Reconnaissance bedrock geology of the Dayton North, Ohio, quadrangle
- **Author(s):** Swinford, E.M.
- **Publishing Organization:** Ohio Division of Geological Survey
- **Series and Number:** Digital Map Series BG-2 Dayton North (supersedes Open-File Map version)
- **Publication Date:** 1994

Calculated: DCM
Checked: DMV

Exhibit No. 6 : Bedrock Geology Map

Project: MOT-4-19.30
PID : 117239



1 0.5 0 1 Miles

1,000 0 1,000 2,000 3,000 4,000 5,000 6,000 7,000 Feet

1 0.5 0 1 Kilometers

N

LOCATION MAP

- **Title:** Bedrock topography of the Dayton North, Ohio, quadrangle
- **Author(s):** Leow, J.A., and Brockman, C.S.
- **Publishing Organization:** Ohio Division of Geological Survey
- **Series and Number:** Digital Map Series BT-3B Dayton North (supersedes Open-File Map version)
- **Publication Date:** 1994

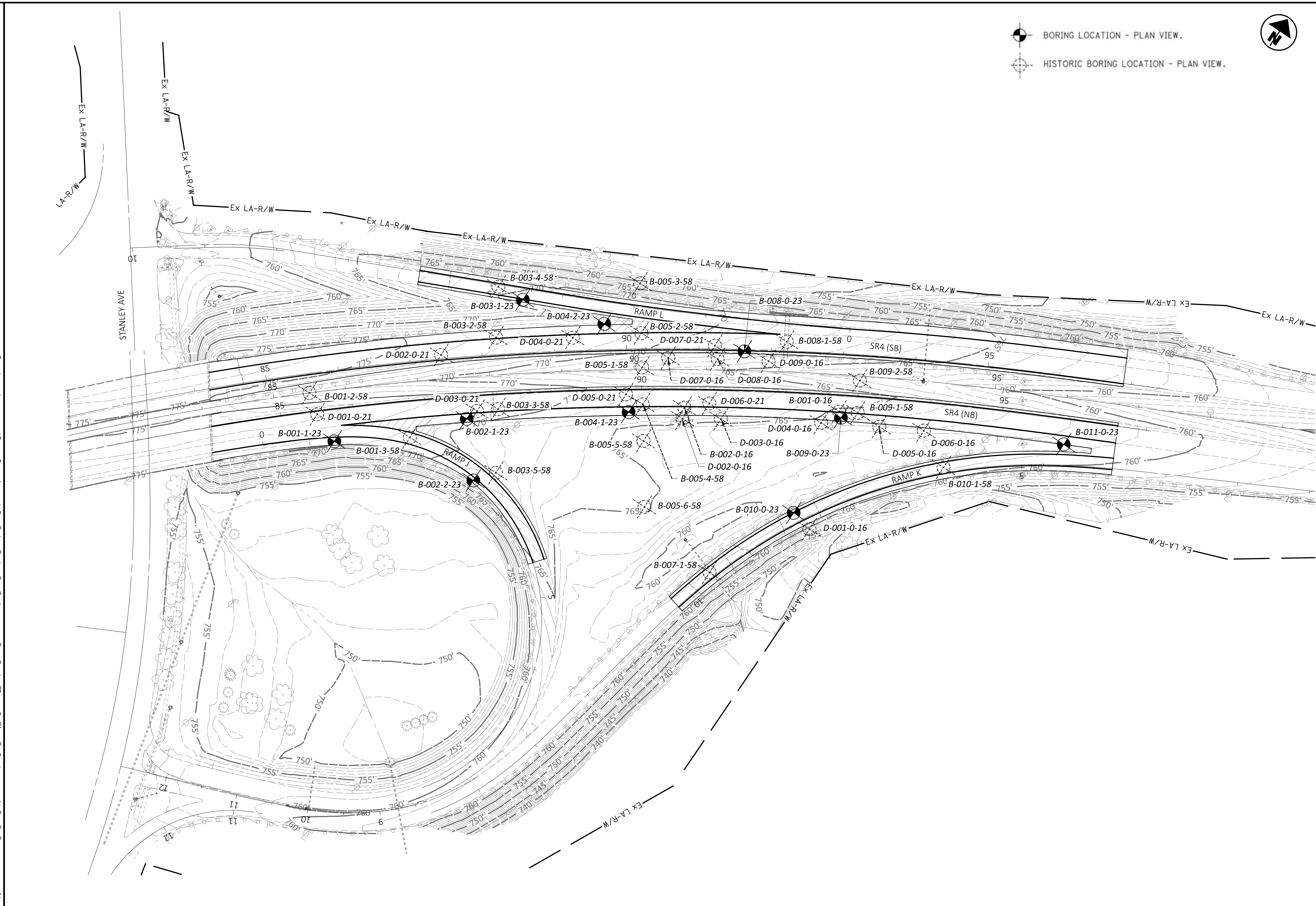
Calculated: DCM

Checked: DMV

Exhibit No. 7 : Bedrock Topography Map

Project: MOT-4-19.30

PID : 117239



HORIZONTAL SCALE IN FEET

0 30 60 120

EXHIBIT NO. 8 : BORING LOCATION PLAN

DESIGN AGENCY

HDR

DESIGNER
DCM

REVIEWER
DMV 08/17/23

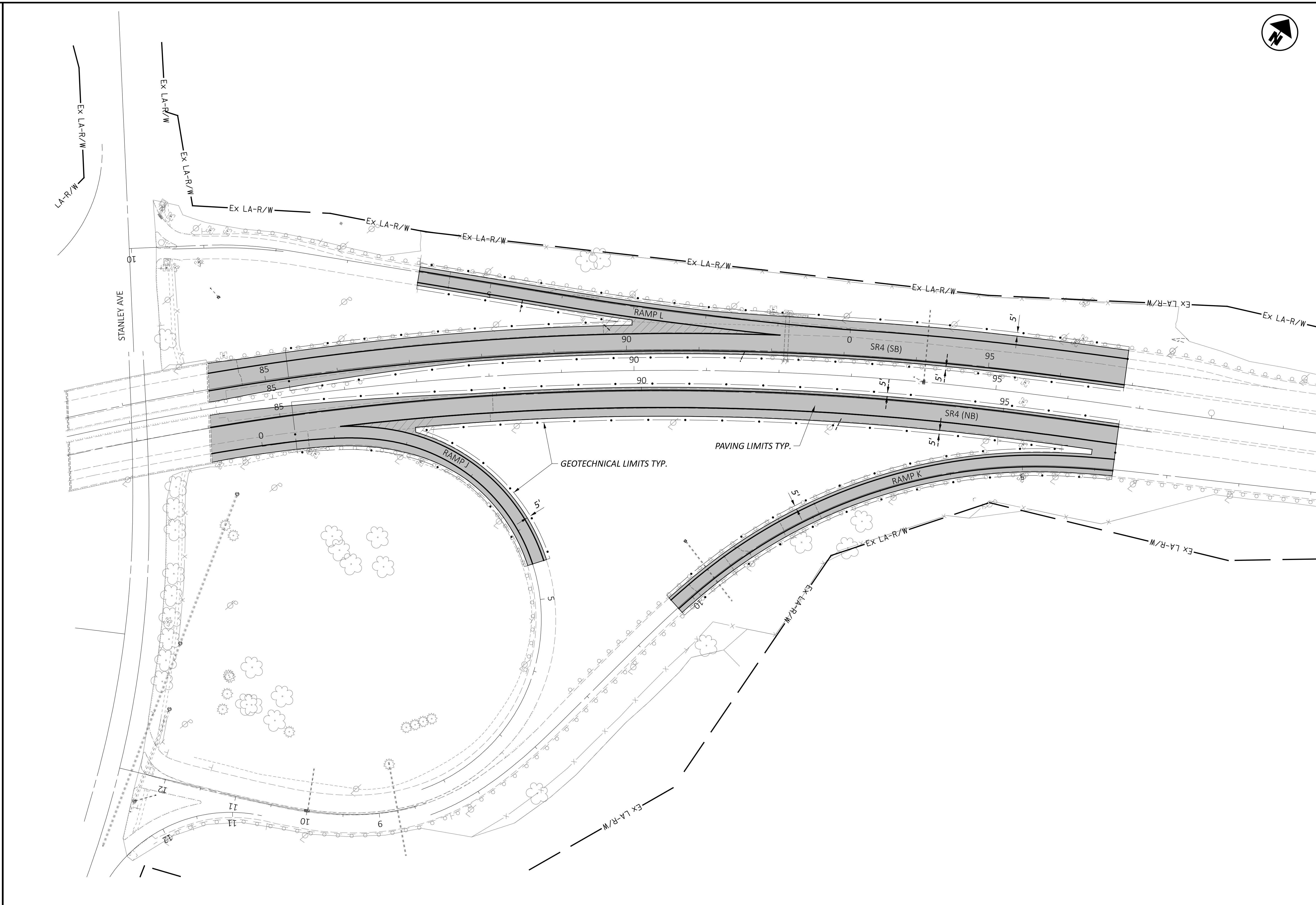
PROJECT ID
117239

SHEET	TOTAL
1	0

MOT-4-19.30

MODEL: 117239_IP900 [Sheet] PAPERSIZE: 17x11 (in.) DATE: 10/12/2023 TIME: 8:36:50 AM USER: MOMOHAMMED

pw:\\pwhdruseas01:HDR_US_East_01\Documents\Chagrin_Valley_Engineering_Ltd\CVE_ODT_MOT-4-19.30\6.0_CAD_BIM\6.2_WIP\01_Design\117239\400 Engineering\Geotechnical\Sheets\Boring Location Plan



HORIZONTAL SCALE IN FEET
0 30 60 120

EXHIBIT NO. 9 : PAVING AND GEOTECHNICAL LIMITS PLAN

DESIGN AGENCY
HDR

DESIGNER
DCM
REVIEWER
DMV 08/17/23
PROJECT ID
117239
SHEET 1
TOTAL 0

MOT-4-19.30

MODEL: 117239_IP900 [Sheet] PAPERSIZE: 17x11 (in.) DATE: 10/12/2023 TIME: 8:37:00 AM USER: MOMOHAMMED
pw:\\pwhdruseas01:HDR_US_East_01\Documents\Chagrin_Valley_Engineering_Ltd\CVE_ODT_MOT-4-19.30\6.0_CAD_BIM\6.2_WIP\01_Design\117239\400 Engineering\Geotechnical\Sheets\Geotechnical Limits

Appendix B. Historic Information

MOT-4-19.73 (ODOT, 1958) – Historic Soil Profile Sheets

MOT-4-19.30 (Advanced Materials, 2016) – DCP

MOT-4-19.30 (ODOT 2016) – Exploratory Borings

MOT-4-19.30 (ODOT, 2021) - DCP

MOT-4-19.73 (ODOT, 1958) – Construction Drawings

MOT-4-19.14 (ODOT, 2003) – Construction Drawings

MOT-4-19.73 (ODOT, 1958) – Historic Soil Profile Sheets

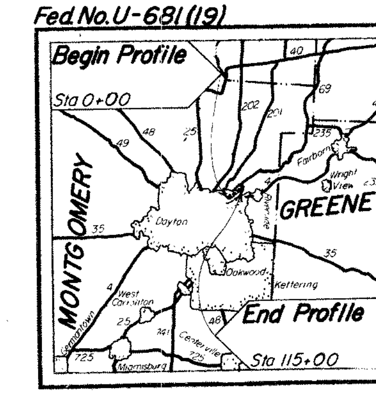
SOIL PROFILE
MONTGOMERY COUNTY
MOT-4-19.73

1/14

STATE HIGHWAY TESTING AND RESEARCH LABORATORY
O S U CAMPUS COLUMBUS, OHIO

NOTE: The information shown by this subgrade profile was secured for the use of the State of Ohio and is not to be construed as a part of the plans governing the construction of the project

Fed. No. U-681 (19)



LOCATION MAP

Recon - PAH - 5-23-58
Drilling Auger - DJH, CAS, BEB, TRS, CH, PAH - 5-23-58
Core - EFM - 5-20-58
Drafting - ZK, RVS, PAH, LNL - 7-18-58

Auger boring - plan view.
Drive sample boring - plan view.
Auger boring plotted to vertical scale only.
Water content nearly equal to or greater than liquid limit.
Number of blows for "Standard Penetration" Test. x=Number of blows for the first six inches; y=Number of blows for the second six inches
w Free water.
Sod & Topsoil = X' = Approximate depth.
Berm material.

Samples Taken
Lab Nos. So.
89493-89521 Incl, 90168-90176 Incl,
90422-90424 Incl, 90467-90521 Incl,
90742-90754 Incl, 90843-90906 Incl,
90974-90975 Incl, 91244-91258 Incl,
91511, 91521-91535 Incl, 91545-91548 Incl.

NOTE: Figures beside borings indicate water content in percent.

LEGEND FOR PROJECT-AVERAGE RESULTS OF TESTS - 136 SAMPLES TESTED

DESCRIPTION	H R B CLASS	OHIO CLASS	% AGG	% C SAND	% F SAND	% SILT	% CLAY	LIQUID LIMIT	PLASTICITY INDEX	WATER CONTENT	SAMPLES TESTED
Gravel	A-1-a (0)	A-1-a	67	19	8	5	1	NP	NP	11	47
Gravel with sand	A-1-b (0)	A-1-b	46	26	13	13	2	NP	NP	17	9
F.S Fine sand	A-3 (0)	A-3	11	29	51	7	2	NP	NP	29	1
Coarse and fine sand	—	A-3a	13	36	39	10	2	NP	NP	15	6
Foundry sand	—	—	20	10	50	16	4	NP	NP	18	9
Gravel with sand and silt	A-2-4 (0)	A-2-4	52	11	11	17	9	28	5	20	11
Gravel with sand, silt, and clay	A-2-6 (1)	A-2-6	52	12	9	14	13	33	14	15	8
Sandy silt	A-4 (4)	A-4a	18	9	20	34	19	18	3	33	8
Silt	A-4 (8)	A-4b	1	2	6	66	25	30	3	65	8
Silt and clay	A-6 (9)	A-6a	9	7	11	42	31	33	13	36	10
Silty clay	A-6 (9)	A-6b	23	7	10	32	28	37	19	33	8
Elastic clay	A-7-5 (12)	A-7-5	0	1	1	52	46	47	17	127	2
Clay	A-7-6 (12)	A-7-6	12	8	10	33	37	46	20	40	9
Lime	Visual classification										
Random fill	Visual classification										

SUMMARY OF SOIL TEST DATA

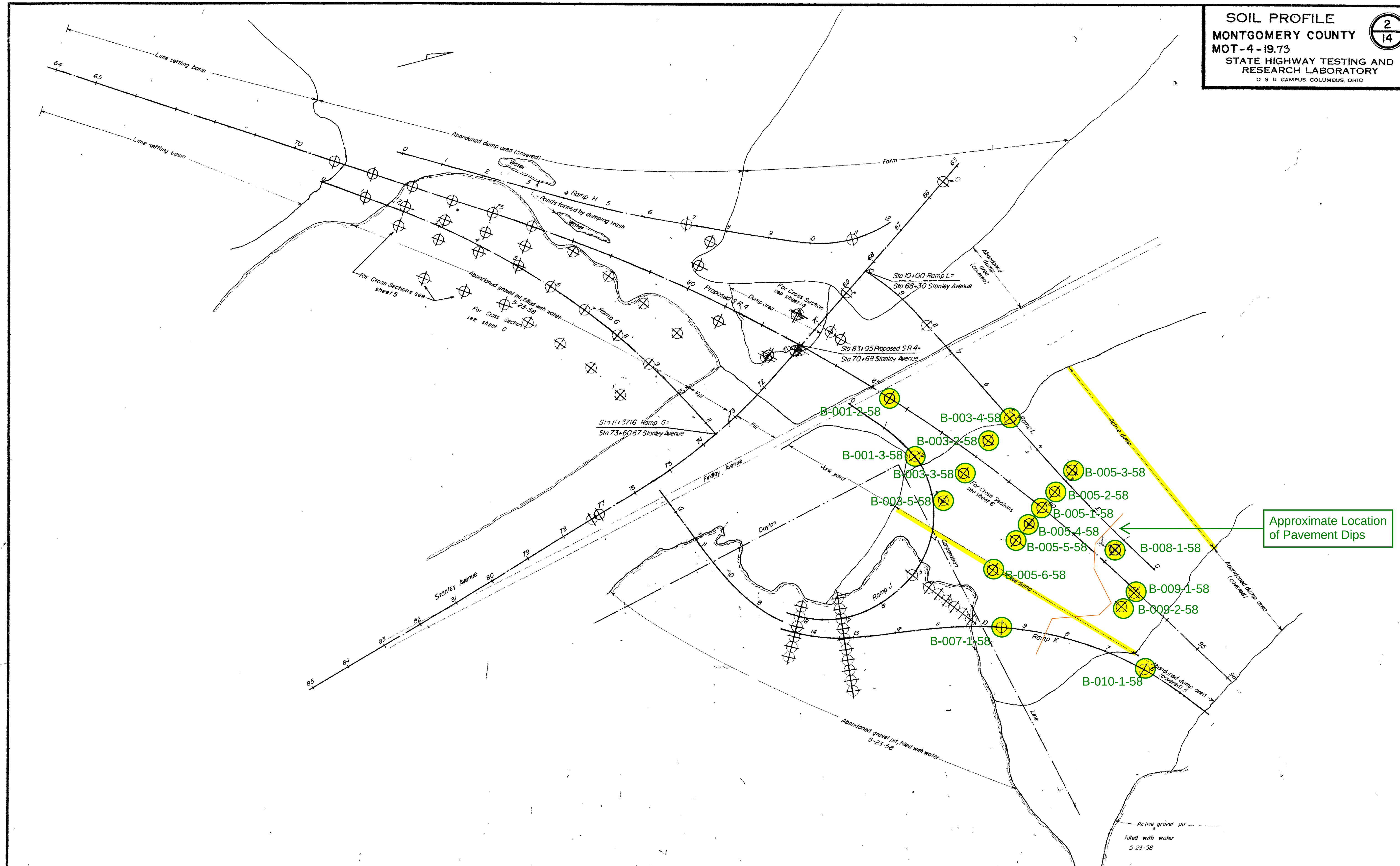
NOTE: NP shown in Liquid Limit and Plasticity Index columns indicates that the material is non-plastic.

Station & Offset		Depth From-To	% Agg	% CS	% FS	% Silt	% Clay	L.L.	P.I.	% WC.	SHTL Class
71+00	CL	0.0-1.0 1.0-6.0 6.0-6.3	77	WATER Visual classification 12	2	9	0	NP	NP	10	LIME A-1-a
72+00	CL	0.2-8.0	79	13	3	4	1	NP	NP	4	A-1-a
72+00	60'RT	0.0-3.0 3.0-8.0 8.0-12.0 12.0-15.0	65 26 49 66	16 36 22 16	10 26 17 9	8 12 10 7	1 0 2 2	NP NP NP NP	NP NP NP NP	8 15 13 11	A-1-a A-1-b A-1-b A-1-a
74+00	CL	0.0-26.0 26.0-27.0	0	WATER 0	8	67	25	28	6	41	A-4b
74+00	50'RT	0.0-18.0 18.0-22.0	0	WATER 1	1	66	32	35	5	62	A-4b
74+00	100'RT	0.0-21.0 21.0-24.0	0	WATER 1	1	59	39	40	17	57	A-6b
74+00	200'RT	0.0-21.5 21.5-24.0	0	WATER 0	1	66	33	34	11	59	A-6a
75+00	CL	0.0-25.0 25.0-26.0	0	WATER 1	7	74	18	NP	NP	47	A-4b
75+00	50'RT	0.0-25.0 25.0-27.0	0	WATER 0	7	73	20	27	3	42	A-4b
75+00	100'RT	0.0-25.0 25.0-29.0	0	WATER 1	1	70	28	37	11	52	A-6a
75+00	200'RT	0.0-21.0 21.0-24.0	0	WATER 0	0	56	44	41	13	66	A-7-6
76+00	CL	0.0-1.5 1.5-3.0	28	WATER 13	18	25	26	17	5	14	A-4a
76+00	50'RT	0.0-12.0 12.0-13.0	55	WATER 26	8	11	0	NP	NP	9	A-1-a
76+00	100'RT	0.0-19.0 19.0-23.0	SANDY SILT	WATER						75	A-4a
76+00	200'RT	0.0-23.0 23.0-32.0	SANDY SILT	WATER						52	A-4a
80+00	60'LT	0.2-2.0 2.0-6.0 6.0-9.0	21 77 80	13 5 11	20 4 6	22 10 2	24 4 1	38 NP NP	21 NP NP	17 11 5	A-6b A-1-a A-1-a
80+00	125'LT	0.2-4.0	3	4	65	22	6	NP	NP	14	FOUNDRY SAND
		4.0-7.0	0	6	74	17	3	NP	NP	5	FOUNDRY SAND
		7.0-11.0	23	11	48	16	2	NP	NP	10	FOUNDRY SAND
		11.0-13.0	42	18	18	15	7	NP	NP	48	FOUNDRY SAND
		13.0-15.0	63	10	6	13	8	20	11	21	A-2-6
81+00	40'RT	0.6-3.0 3.0-5.0 5.0-8.0	54 46 62	14 10 9	9 14 8	13 14 13	10 16 8	31 32 31	9 10 10	14 21 17	A-2-4 A-2-4 A-2-4

Station & Offset		Depth From-To	% Agg	% CS	% FS	% Silt	% Clay	LL	PI	% WC	SHTL Class
82+50	55'RT	0.6-3.0 3.0-8.0 8.0-12.0 12.0-14.0 14.0-16.0 16.0-19.0 19.0-22.0	51 44 69 56 7 32 75	14 13 12 8 3 6 6	9 10 8 5 2 5 5	13 19 7 23 50 33 9	13 14 4 8 38 24 5	32 31 21 20 28 27 24	14 13 4 5 10 11 10	13 13 5 20 23 17 12	A-2-6 A-2-6 A-1-a A-2-4 A-4b A-6a A-2-4
82+60	60'LT	0.0-4.0	RANDOM FILL-GRAVEL AND ASHES								
		4.0-7.0	RANDOM FILL-GRAVEL AND ASHES								
		7.0-10.0	RANDOM FILL-SAND, GRAVEL AND ASHES								
		10.0-12.0	RANDOM FILL-GROUND UP BRICK AND ASHES								
		12.0-15.0	RANDOM FILL-ASHES, SLAG, SAND, AND ETC.								
		15.0-18.0	RANDOM FILL-ASHES, ETC.								
		18.0-22.0	RANDOM FILL-WET ASHES								
		22.0-25.0	RANDOM FILL-WET ASHES AND GRAVEL								
		25.0-27.0	RANDOM FILL-WET ASHES								
		27.0-30.0	RANDOM FILL-WET ASHES								
		30.0-31.0	60	11	23	5	1	NP	NP	17	A-1-a
83+00	15'RT	0.6-3.0 3.0-10.0 10.0-12.0 12.0-18.0	33 0 52 81	16 5 10 7	31 80 9 8	14 10 18 3	6 5 11 1	NP NP 33 NP	NP NP 15 NP	12 12 18 6	A-2-4 A-3a A-2-6 A-1-a
83+80	60'LT	0.8-3.0 3.0-9.0 9.0-11.0 11.0-15.0	32 47 54 79	12 7 36 15	11 9 4 3	23 21 5 2	22 16 1 1	32 39 NP NP	17 20 NP NP	18 17 5 6	A-6b A-6b A-1-a A-1-a
85+50 B-001-2-58	CL	0.8-3.0	RANDOM FILL-BROKEN BRICK								
		3.0-6.0 6.0-9.0 9.0-14.0	64 24 83	10 15 6	11 14 6	10 27 4	5 20 1	NP 32 NP	NP 13 NP	11 19 7	A-1-a A-6a A-1-a
88+00 B-003-2-58	50'LT	0.0-3.0	RANDOM FILL-GRAVEL								
		3.0-8.0	RANDOM FILL-ASHES, GLASS, RUBBER CANS								
88+00 B-003-3-58	50'RT	0.0-4.0	RANDOM FILL-ASHES, CANS, GLASS, ETC.								
		4.0-9.0	RANDOM FILL-ASHES, CANS, GLASS, ETC.								
		9.0-11.0	RANDOM FILL-ASHES, GRAVEL, ETC.								
		11.0-17.0	69	15	7	7	2	NP	NP	11	A-1-a
90+00 B-005-1-58	CL	0.0-4.0	RANDOM FILL-GRAVEL, GLASS, CANS, ETC.								
		4.0-8.0	RANDOM FILL-CANS, GLASS, ETC.								
		8.0-12.0	32	10	41	13	4	NP	NP	13	A-3a
90+00 B-005-2-58	50'LT	0.0-3.0	RANDOM FILL-GRAVEL								
		3.0-10.0	RANDOM FILL-ASHES, CANS, ETC.								
		10.0-15.0	RANDOM FILL-GRAVEL AND ASHES								
		15.0-19.0	74	11	6	6	3	NP	NP	11	A-1-a
90+00 B-005-3-58	115'LT	0.0-6.0	RANDOM FILL-ASHES, GRAVEL, GLASS, AND CANS								
		6.0-10.0	61	9	11	15	4	NP	NP	6	A-1-b
90+00 B-005-4-58	50'RT	0.0-13.0	RANDOM FILL-ASHES, GRAVEL, CANS, RAGS, TIRES, AND BOTTLES								
		13.0-16.0	76	6	6	7	5	29	11	13	A-2-6
90+00 B-005-5-58	100'RT	0.0-2.0	RANDOM FILL-GRAVEL								
		2.0-7.5	RANDOM FILL-ASHES, CANS, RAGS, AND GLASS								
90+00 B-005-6-58	192'RT	0.0-5.0	RANDOM FILL-ASHES, CANS, GLASS, ETC.								
		5.0-7.0	73	15	4	5	3	26	6	15	A-1-a

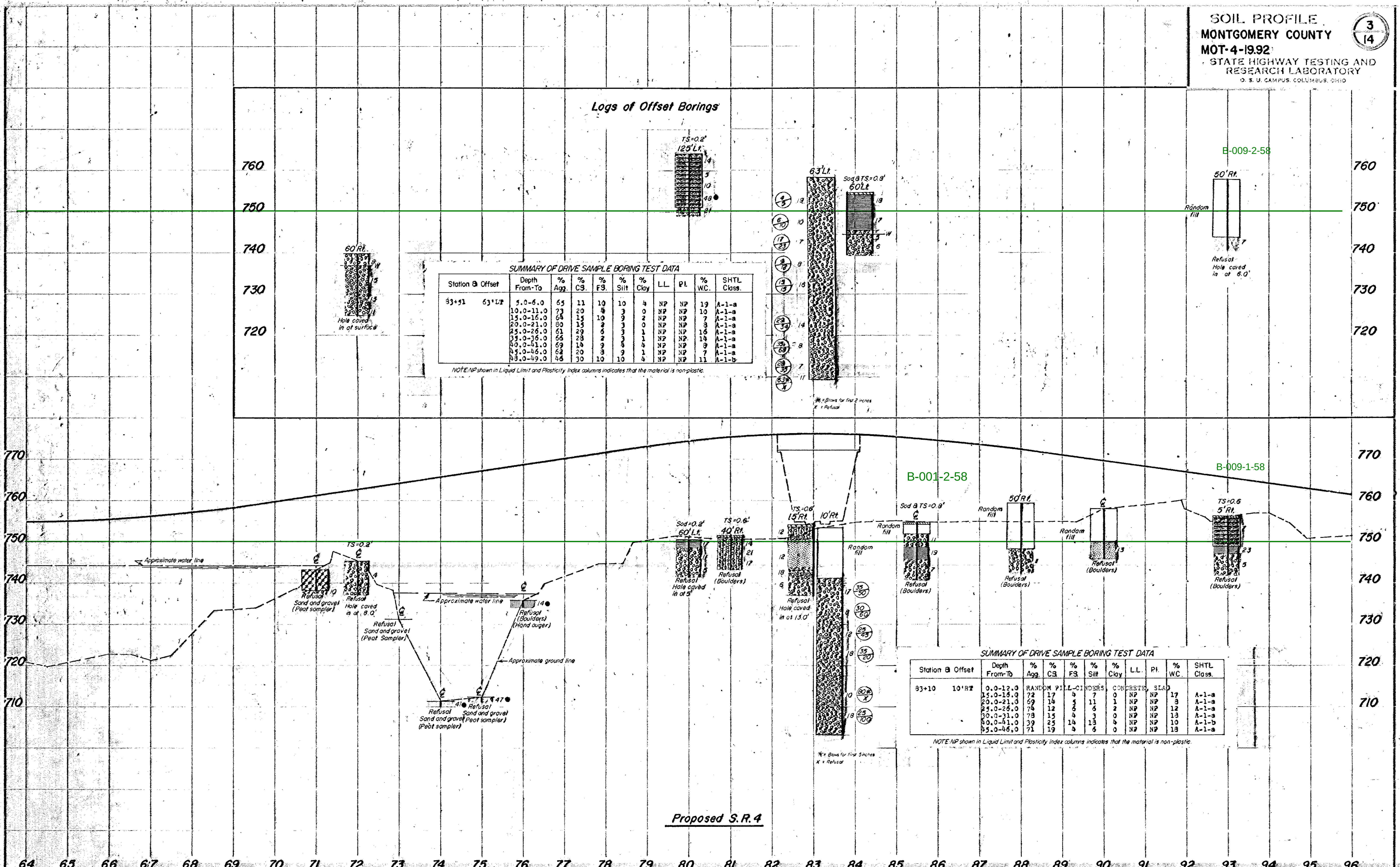
Station & Offset		Depth From-To	% Agg	% CS.	% FS	% Silt	% Clay	L.L.	P.I.	% W.C.	SHTL Class.
93+00 B-009-1-58	5'RT	0.6-7.0	17	9	64	8	2	NP	NP	7	FOUNDRY SAND
		7.0-9.0 9.0-14.0	0 82	12 11	24 4	34 3	30 0	35 NP	15 NP	23 5	A-6a A-1-a
93+00 B-009-2-58	50'RT	0.0-14.0	RANDOM FILL-ASHES, GRAVEL, TIN, CANS, GLASS, RAGS, ETC.								
		14.0-17.0	82	9	4	3	2	NP	NP	7	A-1-a
97+00	CL	0.0-2.0 2.0-9.0	47 87	9 8	12 3	17 1	15 1	33 NP	11 NP	20 5	A-2-6 A-1-a
97+00	50'RT	0.5-2.0 2.0-12.0	87 90	6 5	2 3	4 1	1 1	31 NP	5 NP	5 4	A-1-a A-1-a
101+50	CL	0.0-2.0 2.0-9.0 9.0-12.0	0 55 19	12 17 13	18 5 20	30 21 30	40 2 18	41 NP 18	19 NP 5	25 8 9	A-7-6 A-1-b A-4a
105+50	CL	0.8-2.0 2.0-4.0 4.0-11.0	18 0 75	8 7 18	12 12 4	32 40 2	30 41 1	51 46 NP	26 21 NP	26 26 4	A-7-6 A-7-6 A-1-a
109+00	CL	0.6-4.0 4.0-6.0 6.0-12.0 12.0-17.0	24 7 58 90	12 10 30 5	12 13 7 2	27 34 3 2	25 36 2 1	39 43 NP NP	13 15 NP NP	18 25 5 5	A-6a A-7-6 A-1-a A-1-a
112+00	CL	0.6-4.0 4.0-8.0 8.0-11.0	31 67 89	18 13 9	11 6 1	9 10 1	31 4 0	53 27 NP	29 8 NP	25 8 3	A-7-6 A-2-4 A-1-a
RAMP-G											
6+00	100'RT	0.0-20.0 20.0-21.0	20	WATER 11	5	30	34	39	22	61	A-6b
7+00	100'LT	0.0-7.0 7.0-8.0	60	WATER 5	5	21	9	NP	NP	41	A-2-4
9+00	100'LT	0.0-13.0 13.0-14.0	0	WATER 2	4	56	38	NP	NP	67	A-4b
RAMP-H											
7+00	CL	0.0-5.0	RANDOM FILL-STONES AND TIN CANS								
		5.0-11.0	RANDOM FILL-BROKEN CONCRETE BLOCK								
		11.0-13.0 13.0-20.0	13 63	7 25	10 6	28 3	42 3	32 16	14 3	16 11	A-6a A-1-a
11+00	CL	0.6-5.0 5.0-7.0 7.0-14.0	19 39 82	7 23 13	12 9 2	25 13 3	37 16 0	50 32 NP	26 15 NP	25 14 2	A-7-6 A-2-6 A-1-a
RAMP-J											
2+00 B-001-3-58	CL	0.0-1.0 2.0-9.0	32 66	15 20	18 6	21 5	14 3	28 NP	6 NP	20 5	A-2-4 A-1-a
RAMP-K											
2+00	CL	0.0-1.0	0	5	10	78	7	NP	NP	—	A-4b
2+00	50'LT	0.0-6.0 6.0-13.0	-	WATER -	-	-	-	65	18	—	A-7-5

Station & Offset		Depth From-To	% Agg	% CS	% F.S	% Silt	% Clay	L.L.	PI	% W.C.	SHTL Class.
2+00	75'LT	0.0-5.5 5.5-13.0	0	WATER 3	22	44	31	NP	NP	84	A-4a
2+00	100'LT	0.0-2.0 2.0-7.0	-	WATER -	-	-	—	NP	NP	-	A-4b
2+00	125'LT	0.0-6.0 6.0-15.0	11	WATER 29	51	7	2	NP	NP	29	A-3
2+00	150'LT	0.0-3.0 3.0-7.0	25	WATER 32	18	25	0	NP	NP	47	A-1-b
2+00	175'LT	0.0-6.0 6.0-16.0	0	WATER 1	4	47	48	48	21	112	A-7-6
2+00	200'LT	0.0-6.5 6.5-27.0	32	WATER 21	15	31	1	NP	NP	27	A-2-4
2+00	225'LT	0.0-7.0 7.0-12.0	0	WATER 1	1	52	46	47	17	127	A-7-5
2+00	250'LT	0.0-3.0 3.0-7.0	0	WATER 1	16	49	34	31	12	52	A-6a
3+00	50'LT	0.0-3.0 3.0-3.5	0	WATER 3	35	49	13	NP	NP	64	A-4a
3+00	75'LT	0.0-6.0 6.0-13.0	54	WATER 9	10	14	13	NP	NP	26	A-2-4
3+00	100'LT	0.0-5.0 5.0-12.0	0	WATER 2	4	57	37	37	14	59	A-6a
3+00	125'LT	0.0-6.0 6.0-12.0	0	WATER 2	3	54	41	38	17	59	A-6b
3+00	150'LT	0.0-6.0 6.0-12.0	27	WATER 11	17	25	20	NP	NP	41	A-4a
6+00 B-010-1-58	CL	0.6-3.0	5	12	64	11	3	NP	NP	9	FOUNDRY SAND
		3.0-9.0	75	12	8	2	3	NP	NP	8	A-1-a
9+50 B-007-1-58	CL	0.0-1.0	RANDOM FILL-ASHES								
		1.0-10.0	89	4	4	3	0	NP	NP	4	A-1-a
RAMP-L											
1+00 B-008-1-58	30'LT	0.0-10.0	RANDOM FILL-CANS, GLASS, ETC.								
		10.0-16.0	81	12	3	3	1	NP	NP	5	A-1-a
5+00 B-003-4-58	CL	0.6-5.0 5.0-12.0	44 82	13 8	16 5	18 3	9 2	NP NP	NP NP	19 7	A-1-b A-1-a
8+00	CL	0.6-1.5 1.5-8.0	44 88	13 7	13 3	13 2	17 0	40 NP	21 NP	11 3	A-2-6 A-1-a
STANLEY AVENUE											
65+50	CL	0.6-1.5 1.5-9.0	32 86	10 6	17 4	21 3	20 1	33 NP	16 NP	12 4	A-6b A-1-a
69+00	CL	0.6-2.0 2.0-7.0 7.0-14.0	32 56 93	5 30 3	11 8 3	25 5 1	27 1 0	39 NP NP	20 NP NP	20 5 4	A-6b A-1-a A-1-a
77+00	CL	0.2-5.0 5.0-11.0 11.0-13.0 13.0-16.0 16.0-18.0	37 82 81 38 14	7 11 9 11 10	8 3 4 13 18	25 3 5 25 41	23 1 1 13 17	42 NP NP 19 19	18 NP NP 5 6	29 3 6 10 11	A-7-6 A-1-a A-1-a A-4a A-4a
77+20	CL	0.0-3.0	RANDOM FILL-ASHES AND TIN CANS								
		3.0-6.0	RANDOM FILL-WRECKING RESULTS AND TIN CANS								
		6.0-9.0	RANDOM FILL-ASHES, TIN CANS, ETC.								
		9.0-12.0 12.0-17.0 17.0-20.0	0 70 68	10 21 15	25 8 13	31 0 3	34 1 1	38 NP NP	11 NP NP	44 4 12	A-6a A-1-a A-1-a


SOIL PROFILE
MONTGOMERY COUNTY
MOT-4-19.73
STATE HIGHWAY TESTING AND RESEARCH LABORATORY
O S U CAMPUS, COLUMBUS, OHIO
2
14
Lime settling basin
Abandoned dump area (covered)
Water
Ponds formed by dumping trash
Ramp H
Form
Ramp G
Abandoned gravel pit, filled with water 5-23-58
For Cross Sections see sheet 5
For Cross Section see sheet 6
Proposed S.R. 4
Dump area
For Cross Section see sheet 4
Sta 10+00 Ramp L= Sta 68+30 Stanley Avenue
Sta 83+05 Proposed S.R. 4= Sta 70+68 Stanley Avenue
Sta 11+37.16 Ramp G= Sta 73+60.67 Stanley Avenue
Fill
Stanley Avenue
Findlay Avenue
Junk yard
Dayton
Corporation
Line
Ramp J
Ramp K
Ramp L
Active dump
Abandoned dump area (covered)
Abandoned gravel pit, filled with water 5-23-58
Active gravel pit filled with water 5-23-58
B-001-2-58
B-001-3-58
B-003-4-58
B-003-2-58
B-003-3-58
B-003-5-58
B-005-3-58
B-005-2-58
B-005-1-58
B-005-4-58
B-005-5-58
B-005-6-58
B-008-1-58
B-009-1-58
B-009-2-58
B-007-1-58
B-010-1-58
Approximate Location of Pavement Dips

SOIL PROFILE
MONTGOMERY COUNTY
MOT-4-19.92
STATE HIGHWAY TESTING AND RESEARCH LABORATORY
O. S. U. CAMPUS, COLUMBUS, OHIO

3/14



SUMMARY OF DRIVE SAMPLE BORING TEST DATA

Station & Offset		Depth From-To	% Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	% W.C.	SHTL Class.
83+51	63'LT	5.0-6.0	65	11	10	10	4	NP	NP	19	A-1-a
		10.0-11.0	73	20	4	3	0	NP	NP	10	A-1-a
		15.0-16.0	64	15	10	9	2	NP	NP	7	A-1-a
		20.0-21.0	80	15	2	3	0	NP	NP	8	A-1-a
		25.0-26.0	61	29	6	3	1	NP	NP	16	A-1-a
		35.0-36.0	66	28	2	3	1	NP	NP	14	A-1-a
		40.0-41.0	69	14	9	4	4	NP	NP	9	A-1-a
		45.0-46.0	62	20	8	9	1	NP	NP	7	A-1-a
		48.0-49.0	46	30	10	10	4	NP	NP	11	A-1-b

NOTE: NP shown in Liquid Limit and Plasticity Index columns indicates that the material is non-plastic.

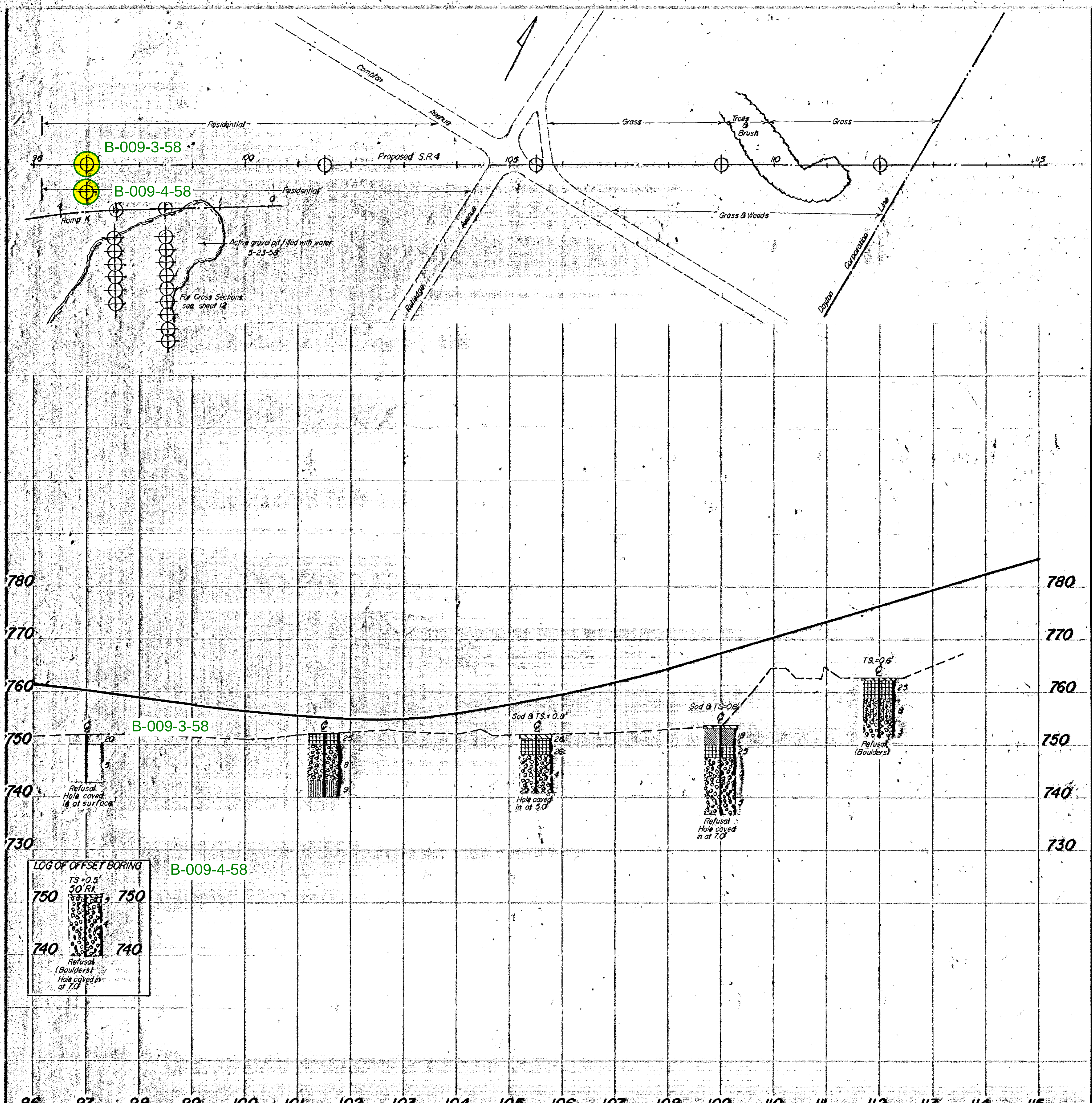
SUMMARY OF DRIVE SAMPLE BORING TEST DATA

Station & Offset		Depth From-To	% Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	% W.C.	SHTL Class.
83+10	10'RT	0.0-12.0	RANDOM FILL-CINDERS, CONCRETE, SLAG								
		15.0-16.0	72	17	4	7	0	NP	NP	17	A-1-a
		20.0-21.0	69	14	5	11	1	NP	NP	8	A-1-a
		25.0-26.0	74	12	6	6	2	NP	NP	12	A-1-a
		30.0-31.0	78	15	4	3	0	NP	NP	13	A-1-a
		40.0-41.0	39	25	14	13	4	NP	NP	10	A-1-b
		45.0-46.0	71	19	4	6	0	NP	NP	18	A-1-a

NOTE: NP shown in Liquid Limit and Plasticity Index columns indicates that the material is non-plastic.

SOIL PROFILE
MONTGOMERY COUNTY
MOT-4-19.92
STATE HIGHWAY TESTING AND RESEARCH LABORATORY
O. S. U. CAMPUS, COLUMBUS, OHIO

4
14



SOIL PROFILE

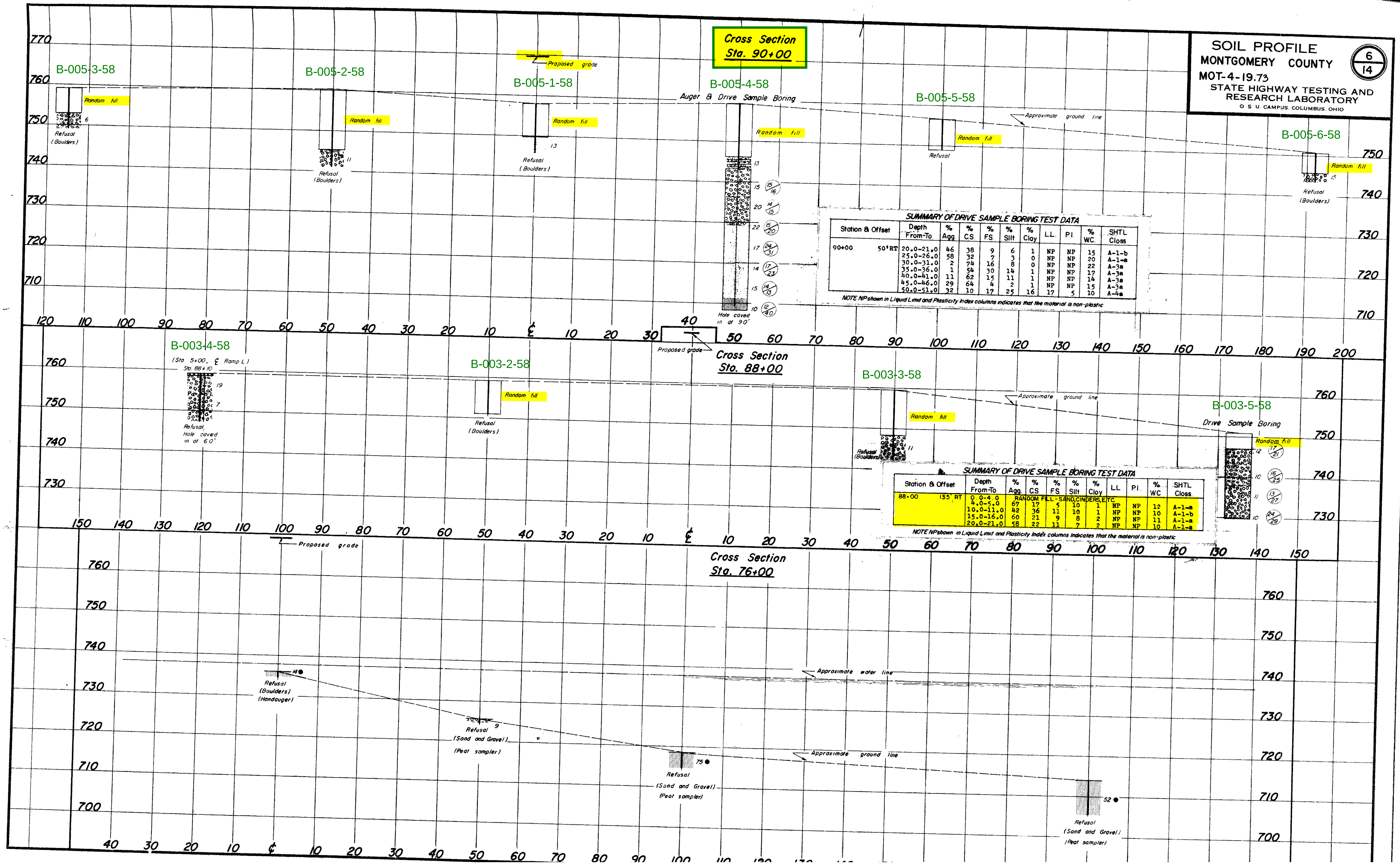
MONTGOMERY COUNTY

MOT-4-19.73

STATE HIGHWAY TESTING AND RESEARCH LABORATORY

O. S. U. CAMPUS, COLUMBUS, OHIO

6 / 14



SUMMARY OF DRIVE SAMPLE BORING TEST DATA

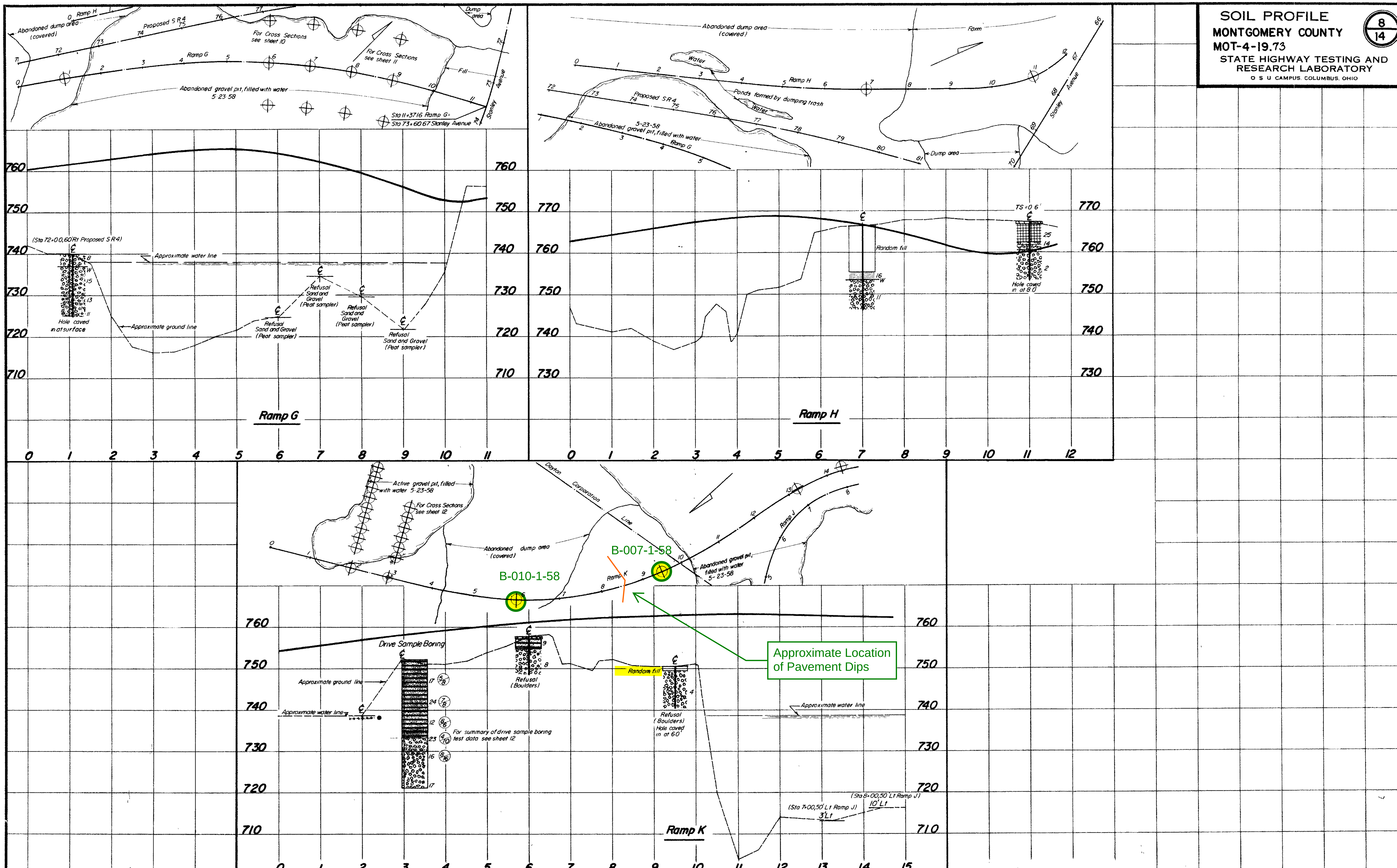
Station & Offset		Depth From-To	% Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	% W.C.	SHTL Class
90+00	50' RT	20.0-21.0	46	38	9	6	1	NP	NP	15	A-1-b
		25.0-26.0	58	32	7	3	0	NP	NP	20	A-1-a
		30.0-31.0	2	74	16	8	0	NP	NP	22	A-3a
		35.0-36.0	1	54	30	14	1	NP	NP	17	A-3a
		40.0-41.0	11	62	15	11	1	NP	NP	14	A-3a
		45.0-46.0	29	64	4	2	1	NP	NP	15	A-3a
		50.0-51.0	32	10	17	25	16	17	5	10	A-4a

NOTE: NP shown in Liquid Limit and Plasticity Index columns indicates that the material is non-plastic

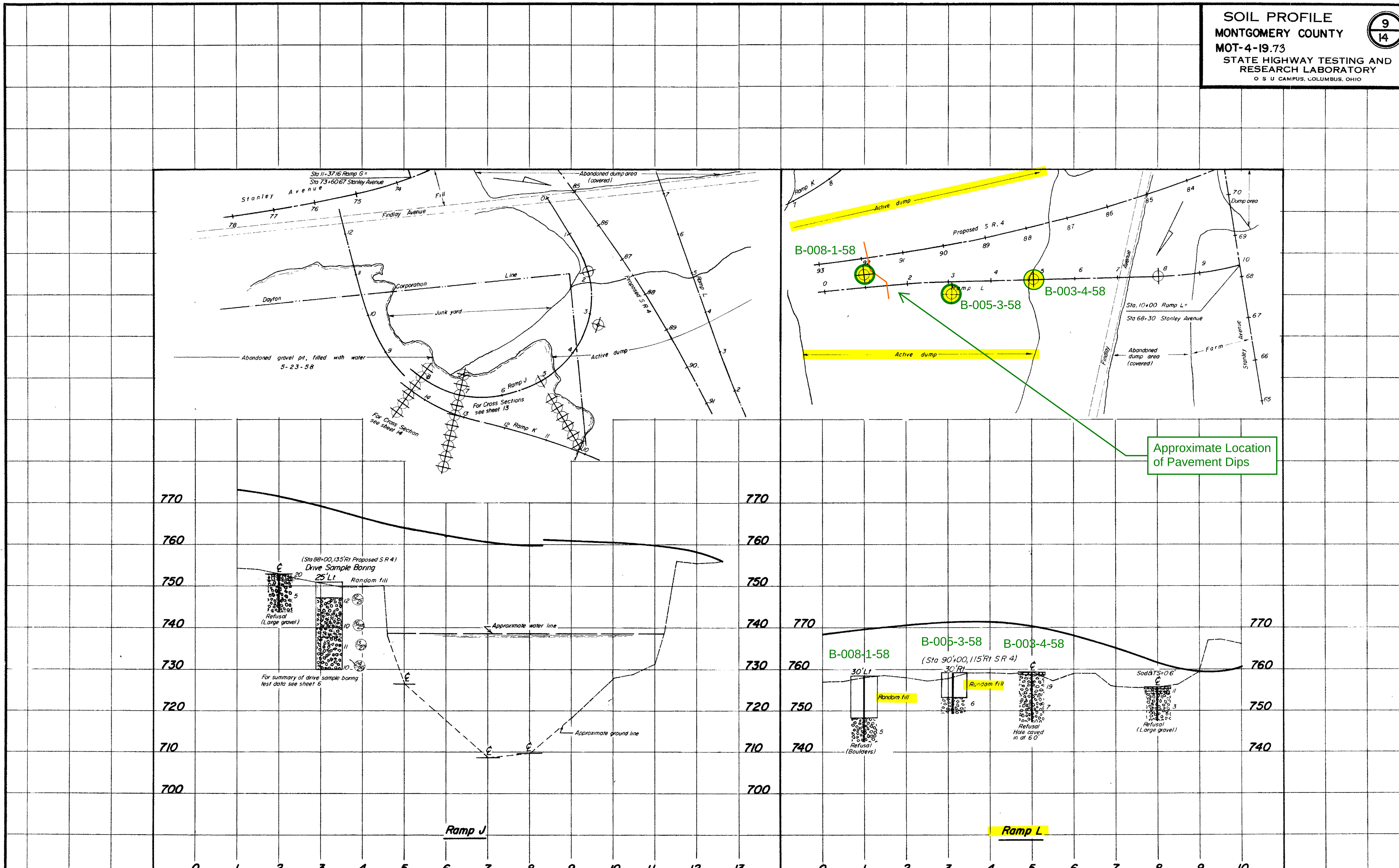
SUMMARY OF DRIVE SAMPLE BORING TEST DATA

Station & Offset		Depth From-To	% Agg.	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	% W.C.	SHTL Class
88+00	135' RT	0.0-4.0	RANDOM FILL - SAND, CINDERS, ETC.								
		4.0-5.0	67	17	5	10	1	NP	NP	12	A-1-a
		10.0-11.0	42	36	11	10	1	NP	NP	10	A-1-b
		15.0-16.0	60	21	9	8	2	NP	NP	11	A-1-a
		20.0-21.0	58	22	11	7	2	NP	NP	10	A-1-a

NOTE: NP shown in Liquid Limit and Plasticity Index columns indicates that the material is non-plastic



SOIL PROFILE
MONTGOMERY COUNTY
MOT-4-19.73
8
14
STATE HIGHWAY TESTING AND RESEARCH LABORATORY
O S U CAMPUS, COLUMBUS, OHIO
Ramp G
Ramp H
Ramp K
Abandoned dump area (covered)
Proposed S R 4
For Cross Sections see sheet 10
For Cross Sections see sheet 11
Abandoned gravel pit, filled with water 5-23-58
Sta 11+37.16 Ramp G
Sta 73+60.67 Stanley Avenue
Stanley Avenue
Dump area
Fill
(Sta 72+00, 60'Rt Proposed S R 4)
Approximate water line
Approximate ground line
Hole caved in at surface
Refusal Sand and Gravel (Peat sampler)
Farm
Water
Ponds formed by dumping trash
Random fill
Hole caved in at 8.0'
TS=0.6'
Dayton Corporation Line
Active gravel pit, filled with water 5-23-58
For Cross Sections see sheet 12
Ramp J
B-007-1-58
B-010-1-58
Approximate Location of Pavement Dips
Drive Sample Boring
Refusal (Boulders)
Refusal (Boulders) Hole caved in at 6.0'
For summary of drive sample boring test data see sheet 12
(Sta 7+00, 50' Lt Ramp J)
(Sta 8+00, 50' Lt Ramp J)



SOIL PROFILE
MONTGOMERY COUNTY
MOT-4-19.73
STATE HIGHWAY TESTING AND RESEARCH LABORATORY
O S U CAMPUS, COLUMBUS, OHIO
9
14
Sta 11+37.16 Ramp G =
Sta 73+60.67 Stanley Avenue
Stanley Avenue
Findlay Avenue
Abandoned dump area (covered)
Fill
Corporation Line
Dayton
Junk yard
Proposed S R 4
Ramp L
Abandoned gravel pit, filled with water
5-23-58
Active dump
Ramp J
Ramp K
For Cross Sections see sheet 13
For Cross Section see sheet 14
Active dump
Proposed S R. 4
B-008-1-58
B-005-3-58
B-003-4-58
Sta. 10+00 Ramp L =
Sta 68+30 Stanley Avenue
Abandoned dump area (covered)
Farm
Dump area
Approximate Location of Pavement Dips
(Sta 88+00, 135' Rt Proposed S R 4)
Drive Sample Boring
25' Lt
Random fill
Refusal (Large gravel)
Approximate water line
For summary of drive sample boring test data see sheet 6
Approximate ground line
Ramp J
30' Lt
(Sta 90+00, 115' Rt S R 4)
30' Rt
Random fill
Refusal (Boulders)
Refusal Hole caved in at 6.0'
Sod & TS 0.6'
Refusal (Large gravel)
Ramp L

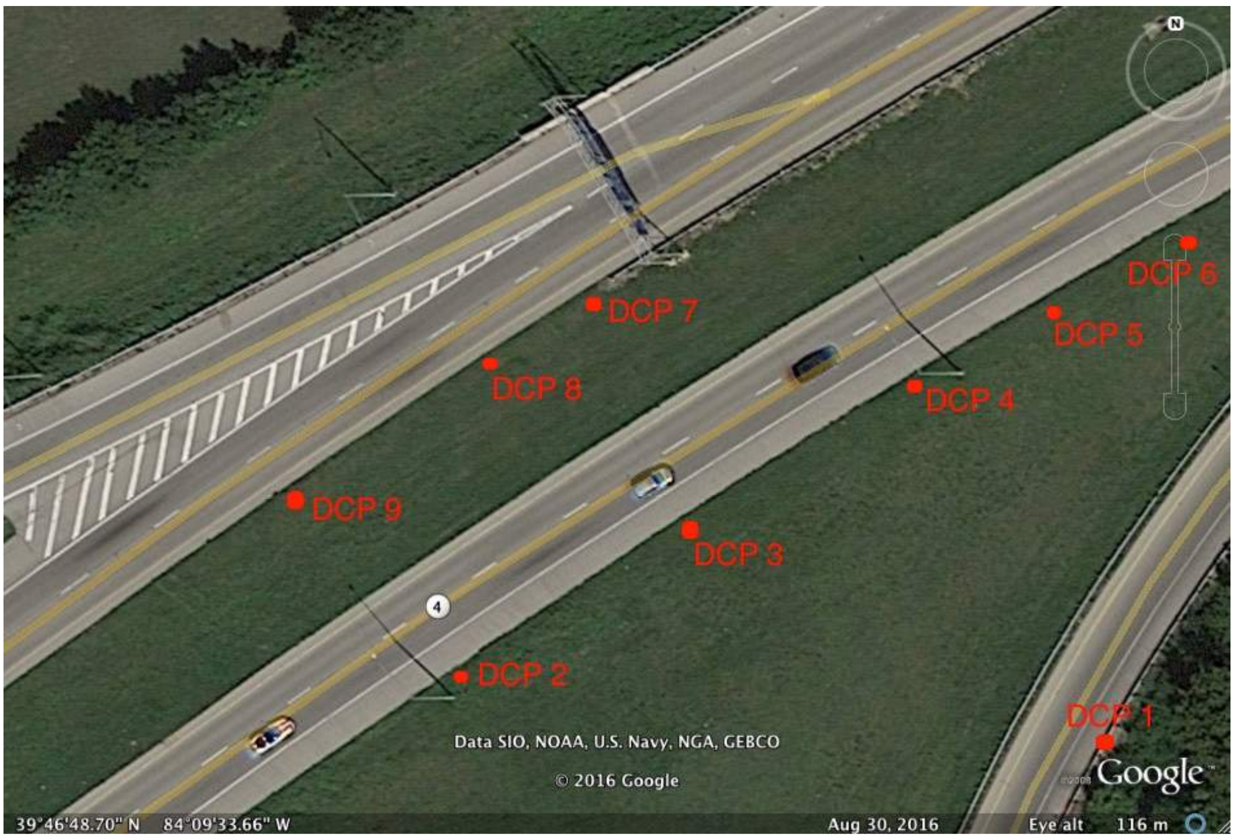
MOT-4-19.30 (Advanced Materials, 2016) – DCP

Grilliot, Daniel

Subject: FW: D7 MOT SR4 DCP Data
Attachments: D7 MOT SR4 DCP1.pdf; D7 MOT SR4 DCP2.pdf; D7 MOT SR4 DCP3.pdf; D7 MOT SR4 DCP4.pdf; D7 MOT SR4 DCP5.pdf; D7 MOT SR4 DCP6.pdf; D7 MOT SR4 DCP7.pdf; D7 MOT SR4 DCP8.pdf; D7 MOT SR4 DCP9.pdf

From: Eilerman, Lee
Sent: Monday, December 12, 2016 8:57 AM
To: Grilliot, Daniel <Daniel.Grilliot@dot.ohio.gov>
Subject: FW: D7 MOT SR4 DCP Data

From: Joe Kindler [mailto:joe@amllc.org]
Sent: Friday, December 09, 2016 4:38 PM
To: Eilerman, Lee <Lee.Eilerman@dot.ohio.gov>
Subject: D7 MOT SR4 DCP Data



Respectfully,

Joe Kindler
Advanced Materials, LLC
P.O. Box 3414 | Dublin, OH 43016

614-561-6440
800-423-8451 FAX
joe@amllc.org | www.amllc.org

DCP 1
Redesignated D-001-0-16

Dynamic Cone Penetrometer Log Sheet

Location Information

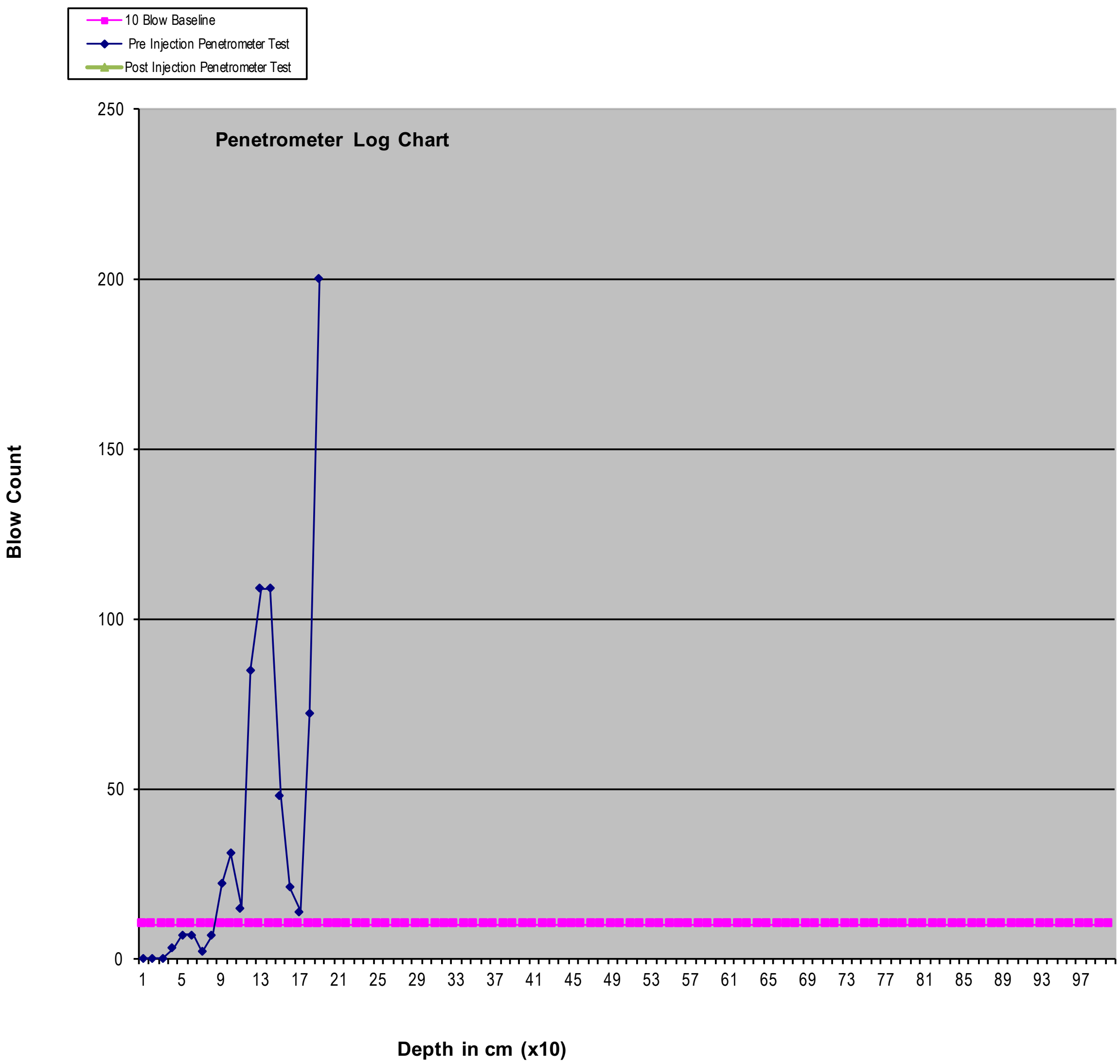
County		Pavement Material	
State		Pavement Thickness	
Roadway		Base Material	
Penetrometer Operators		Base Thickness	
Start Time		SubBase material	
Finish Time		SubBase Thickness	
Penetrometer test #	1 Date 11/17/16	Data Recorder	
Other Information	ODOT 7 SR4 North bound Ramp Right Berm Outside Gaurdrail		

in	ft	cm	Pre		Post	in	ft	cm	Pre		Post
depth	depth	depth	Blows		Blows	depth	depth	depth	Blows		Blows
3.94	0.33	10	0.2			200.79	16.73	510			
7.87	0.66	20	0.2			204.72	17.06	520			
11.81	0.98	30	0.2			208.66	17.38	530			
15.75	1.31	40	3.2			212.60	17.71	540			
19.69	1.64	50	7			216.54	18.04	550			
23.62	1.97	60	7			220.47	18.37	560			
27.56	2.30	70	2			224.41	18.70	570			
31.50	2.62	80	7			228.35	19.02	580			
35.43	2.95	90	22			232.28	19.35	590			
39.37	3.28	100	31			236.22	19.68	600			
43.31	3.61	110	15			240.16	20.01	610			
47.24	3.94	120	85			244.09	20.34	620			
51.18	4.26	130	109			248.03	20.66	630			
55.12	4.59	140	109			251.97	20.99	640			
59.06	4.92	150	48			255.91	21.32	650			
62.99	5.25	160	21			259.84	21.65	660			
66.93	5.58	170	14			263.78	21.98	670			
70.87	5.90	180	72			267.72	22.30	680			
74.80	6.23	190	200			271.65	22.63	690			
78.74	6.56	200				275.59	22.96	700			
82.68	6.89	210				279.53	23.29	710			
86.61	7.22	220				283.46	23.62	720			
90.55	7.54	230				287.40	23.94	730			
94.49	7.87	240				291.34	24.27	740			
98.43	8.20	250				295.28	24.60	750			
102.36	8.53	260				299.21	24.93	760			
106.30	8.86	270				303.15	25.26	770			
110.24	9.18	280				307.09	25.58	780			
114.17	9.51	290				311.02	25.91	790			
118.11	9.84	300				314.96	26.24	800			
122.05	10.17	310				318.90	26.57	810			
125.98	10.50	320				322.83	26.90	820			
129.92	10.82	330				326.77	27.22	830			
133.86	11.15	340				330.71	27.55	840			
137.80	11.48	350				334.65	27.88	850			
141.73	11.81	360				338.58	28.21	860			
145.67	12.14	370				342.52	28.54	870			
149.61	12.46	380				346.46	28.86	880			
153.54	12.79	390				350.39	29.19	890			
157.48	13.12	400				354.33	29.52	900			
161.42	13.45	410				358.27	29.85	910			
165.35	13.78	420				362.20	30.18	920			
169.29	14.10	430				366.14	30.50	930			
173.23	14.43	440				370.08	30.83	940			
177.17	14.76	450				374.02	31.16	950			
181.10	15.09	460				377.95	31.49	960			
185.04	15.42	470				381.89	31.82	970			
188.98	15.74	480				385.83	32.14	980			
192.91	16.07	490				389.76	32.47	990			
196.85	16.40	500				393.70	32.80	1000			

Dynamic Cone Penetrometer Log Sheet

Location Information

County		Pavement Material	
State		Pavement Thickness	
Roadway		Base Material	
Penetrometer Operators		Base Thickness	
Start Time		SubBase material	
Finish Time		SubBase Thickness	
Penetrometer test #	ODOT 7 SR4 North bound Ramp Right Ber Date 11/17/16	Data Recorder	
Other Information			



DCP 2
Redesignated D-002-0-16

Dynamic Cone Penetrometer Log Sheet

Location Information

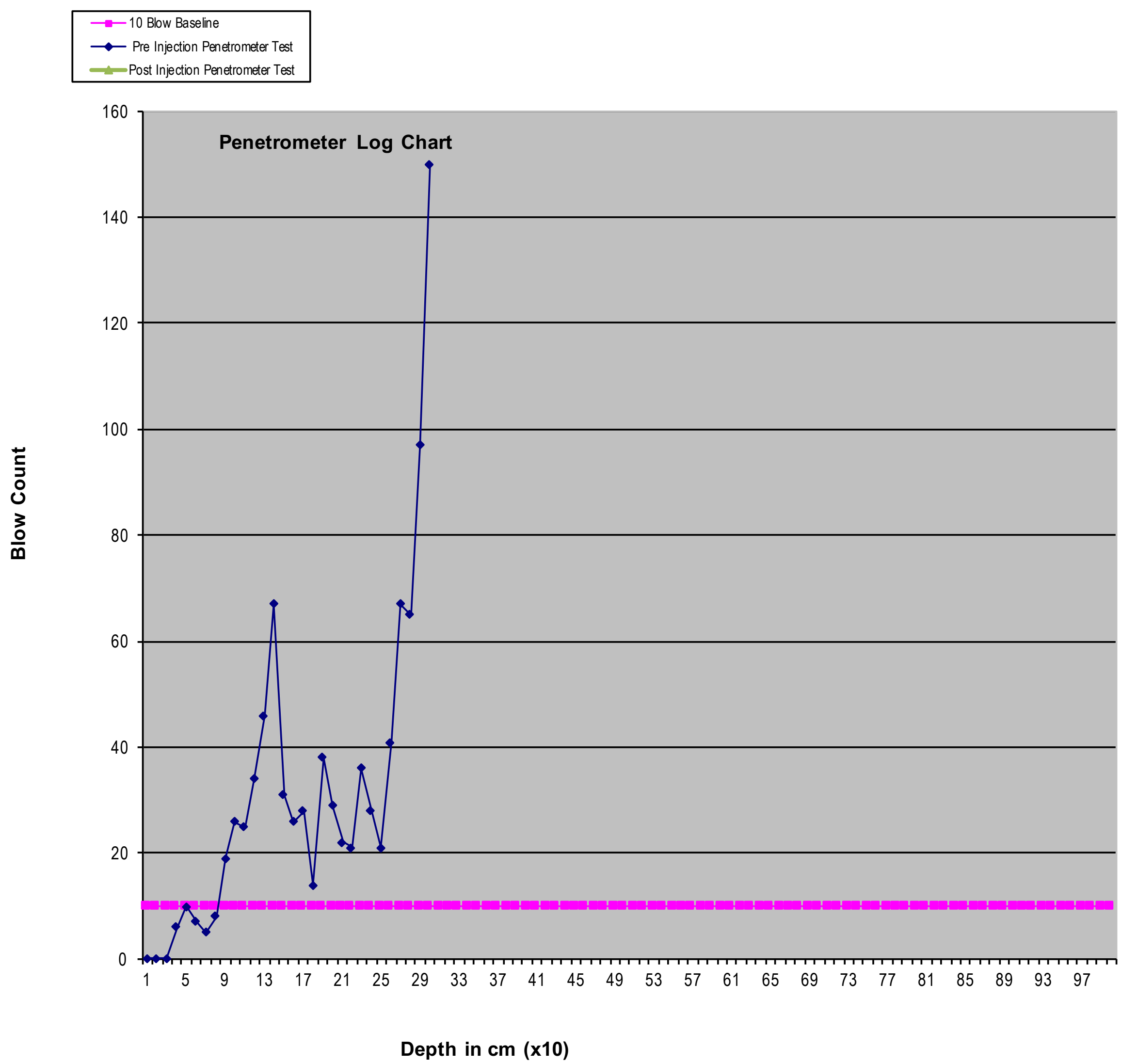
County		Pavement Material	
State		Pavement Thickness	
Roadway		Base Material	
Penetrometer Operators		Base Thickness	
Start Time		SubBase material	
Finish Time		SubBase Thickness	
Penetrometer test #	2 Date 11/18/16	Data Recorder	
Other Information	ODOT 7 SR4 North Bound Right Berm 5' off Pavement Edge Light Post 3F5		

in depth	ft depth	cm depth	Pre Blows		Post Blows	in depth	ft depth	cm depth	Pre Blows		Post Blows
3.94	**0.33**	10	0.2			200.79	16.73	510			
7.87	**0.66**	20	0.2			204.72	17.06	520			
11.81	**0.98**	30	0.2			208.66	17.38	530			
15.75	**1.31**	40	6			212.60	17.71	540			
19.69	**1.64**	50	10			216.54	18.04	550			
23.62	**1.97**	60	7			220.47	18.37	560			
27.56	**2.30**	70	5			224.41	18.70	570			
31.50	**2.62**	80	8			228.35	19.02	580			
35.43	**2.95**	90	19			232.28	19.35	590			
39.37	**3.28**	100	26			236.22	19.68	600			
43.31	**3.61**	110	25			240.16	20.01	610			
47.24	**3.94**	120	34			244.09	20.34	620			
51.18	**4.26**	130	46			248.03	20.66	630			
55.12	**4.59**	140	67			251.97	20.99	640			
59.06	**4.92**	150	31			255.91	21.32	650			
62.99	**5.25**	160	26			259.84	21.65	660			
66.93	**5.58**	170	28			263.78	21.98	670			
70.87	**5.90**	180	14			267.72	22.30	680			
74.80	**6.23**	190	38			271.65	22.63	690			
78.74	**6.56**	200	29			275.59	22.96	700			
82.68	**6.89**	210	22			279.53	23.29	710			
86.61	**7.22**	220	21			283.46	23.62	720			
90.55	**7.54**	230	36			287.40	23.94	730			
94.49	**7.87**	240	28			291.34	24.27	740			
98.43	**8.20**	250	21			295.28	24.60	750			
102.36	**8.53**	260	41			299.21	24.93	760			
106.30	**8.86**	270	67			303.15	25.26	770			
110.24	**9.18**	280	65			307.09	25.58	780			
114.17	**9.51**	290	97			311.02	25.91	790			
118.11	**9.84**	300	150			314.96	26.24	800			
122.05	**10.17**	310				318.90	26.57	810			
125.98	**10.50**	320				322.83	26.90	820			
129.92	**10.82**	330				326.77	27.22	830			
133.86	**11.15**	340				330.71	27.55	840			
137.80	**11.48**	350				334.65	27.88	850			
141.73	**11.81**	360				338.58	28.21	860			
145.67	**12.14**	370				342.52	28.54	870			
149.61	**12.46**	380				346.46	28.86	880			
153.54	**12.79**	390				350.39	29.19	890			
157.48	**13.12**	400				354.33	29.52	900			
161.42	**13.45**	410				358.27	29.85	910			
165.35	**13.78**	420				362.20	30.18	920			
169.29	**14.10**	430				366.14	30.50	930			
173.23	**14.43**	440				370.08	30.83	940			
177.17	**14.76**	450				374.02	31.16	950			
181.10	**15.09**	460				377.95	31.49	960			
185.04	**15.42**	470				381.89	31.82	970			
188.98	**15.74**	480				385.83	32.14	980			
192.91	**16.07**	490				389.76	32.47	990			
196.85	**16.40**	500				393.70	32.80	1000			

Dynamic Cone Penetrometer Log Sheet

Location Information

County		Pavement Material	
State		Pavement Thickness	
Roadway		Base Material	
Penetrometer Operators		Base Thickness	
Start Time		SubBase material	
Finish Time		SubBase Thickness	
Penetrometer test #	ODOT 7 SR4 North Bound Right Berm 5' of Date 11/18/16	Data Recorder	
Other Information			



DCP 3
Redesignated D-003-0-16

Dynamic Cone Penetrometer Log Sheet

Location Information

County
State
Roadway
Penetrometer Operators
Start Time
Finish Time
Penetrometer test # 3 Date 11/22/16
Other Information ODOT 7 SR4 North Bound Right Berm 4' off Pavement Edge Between Dips

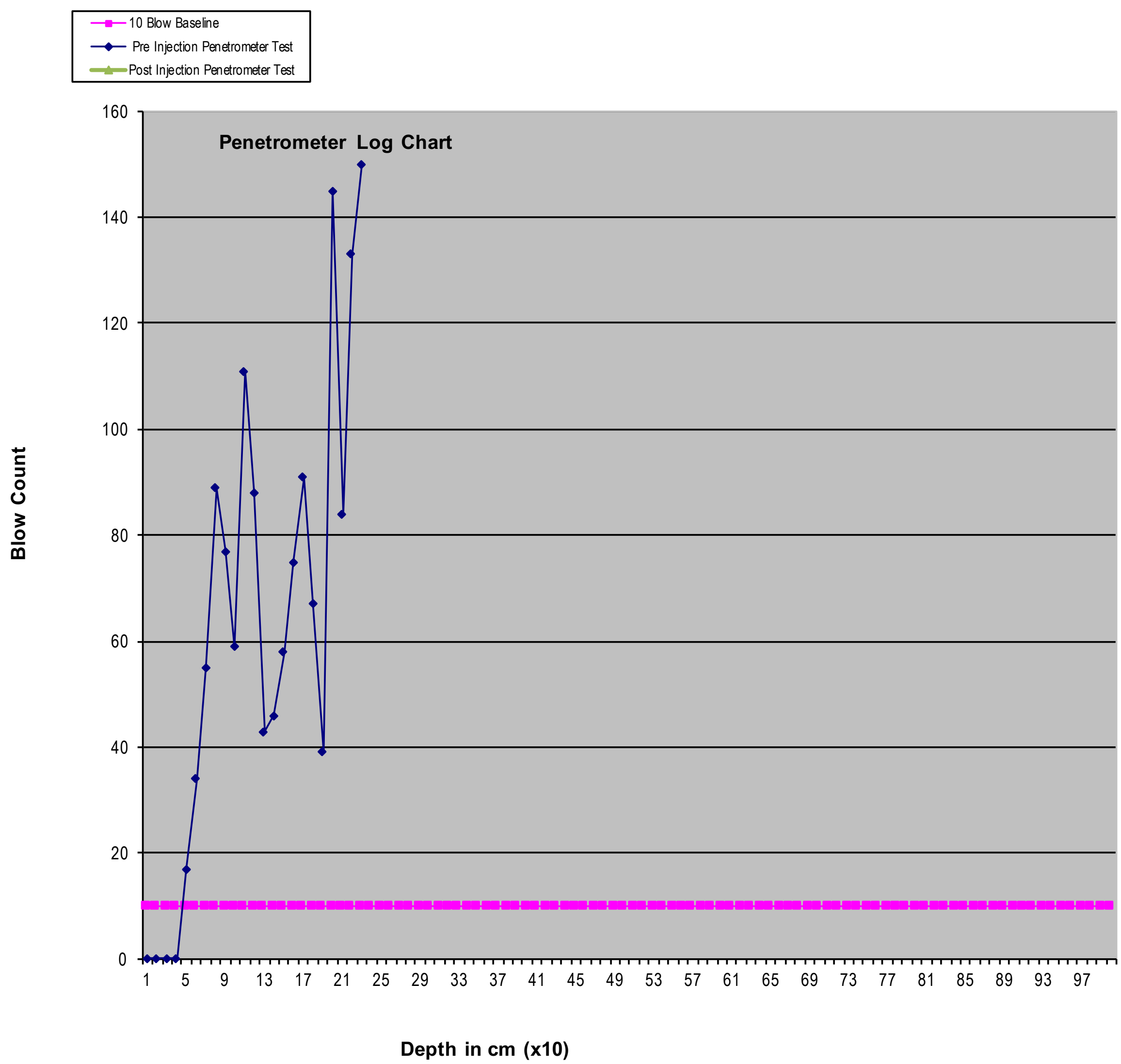
Pavement Material
Pavement Thickness
Base Material
Base Thickness
SubBase material
SubBase Thickness
Data Recorder

in depth	ft depth	cm depth	Pre Blows	Post Blows	in depth	ft depth	cm depth	Pre Blows	Post Blows
3.94	0.33	10	0.2		200.79	16.73	510		
7.87	0.66	20	0.2		204.72	17.06	520		
11.81	0.98	30	0.2		208.66	17.38	530		
15.75	1.31	40	0.2		212.60	17.71	540		
19.69	1.64	50	17		216.54	18.04	550		
23.62	1.97	60	34		220.47	18.37	560		
27.56	2.30	70	55		224.41	18.70	570		
31.50	2.62	80	89		228.35	19.02	580		
35.43	2.95	90	77		232.28	19.35	590		
39.37	3.28	100	59		236.22	19.68	600		
43.31	3.61	110	111		240.16	20.01	610		
47.24	3.94	120	88		244.09	20.34	620		
51.18	4.26	130	43		248.03	20.66	630		
55.12	4.59	140	46		251.97	20.99	640		
59.06	4.92	150	58		255.91	21.32	650		
62.99	5.25	160	75		259.84	21.65	660		
66.93	5.58	170	91		263.78	21.98	670		
70.87	5.90	180	67		267.72	22.30	680		
74.80	6.23	190	39		271.65	22.63	690		
78.74	6.56	200	145		275.59	22.96	700		
82.68	6.89	210	84		279.53	23.29	710		
86.61	7.22	220	133		283.46	23.62	720		
90.55	7.54	230	150		287.40	23.94	730		
94.49	7.87	240			291.34	24.27	740		
98.43	8.20	250			295.28	24.60	750		
102.36	8.53	260			299.21	24.93	760		
106.30	8.86	270			303.15	25.26	770		
110.24	9.18	280			307.09	25.58	780		
114.17	9.51	290			311.02	25.91	790		
118.11	9.84	300			314.96	26.24	800		
122.05	10.17	310			318.90	26.57	810		
125.98	10.50	320			322.83	26.90	820		
129.92	10.82	330			326.77	27.22	830		
133.86	11.15	340			330.71	27.55	840		
137.80	11.48	350			334.65	27.88	850		
141.73	11.81	360			338.58	28.21	860		
145.67	12.14	370			342.52	28.54	870		
149.61	12.46	380			346.46	28.86	880		
153.54	12.79	390			350.39	29.19	890		
157.48	13.12	400			354.33	29.52	900		
161.42	13.45	410			358.27	29.85	910		
165.35	13.78	420			362.20	30.18	920		
169.29	14.10	430			366.14	30.50	930		
173.23	14.43	440			370.08	30.83	940		
177.17	14.76	450			374.02	31.16	950		
181.10	15.09	460			377.95	31.49	960		
185.04	15.42	470			381.89	31.82	970		
188.98	15.74	480			385.83	32.14	980		
192.91	16.07	490			389.76	32.47	990		
196.85	16.40	500			393.70	32.80	1000		

Dynamic Cone Penetrometer Log Sheet

Location Information

County		Pavement Material	
State		Pavement Thickness	
Roadway		Base Material	
Penetrometer Operators		Base Thickness	
Start Time		SubBase material	
Finish Time		SubBase Thickness	
Penetrometer test #	ODOT 7 SR4 North Bound Right Berm 4' of Date 11/22/16	Data Recorder	
Other Information			



DCP 4
Redesignated D-004-0-16

Dynamic Cone Penetrometer Log Sheet

Location Information

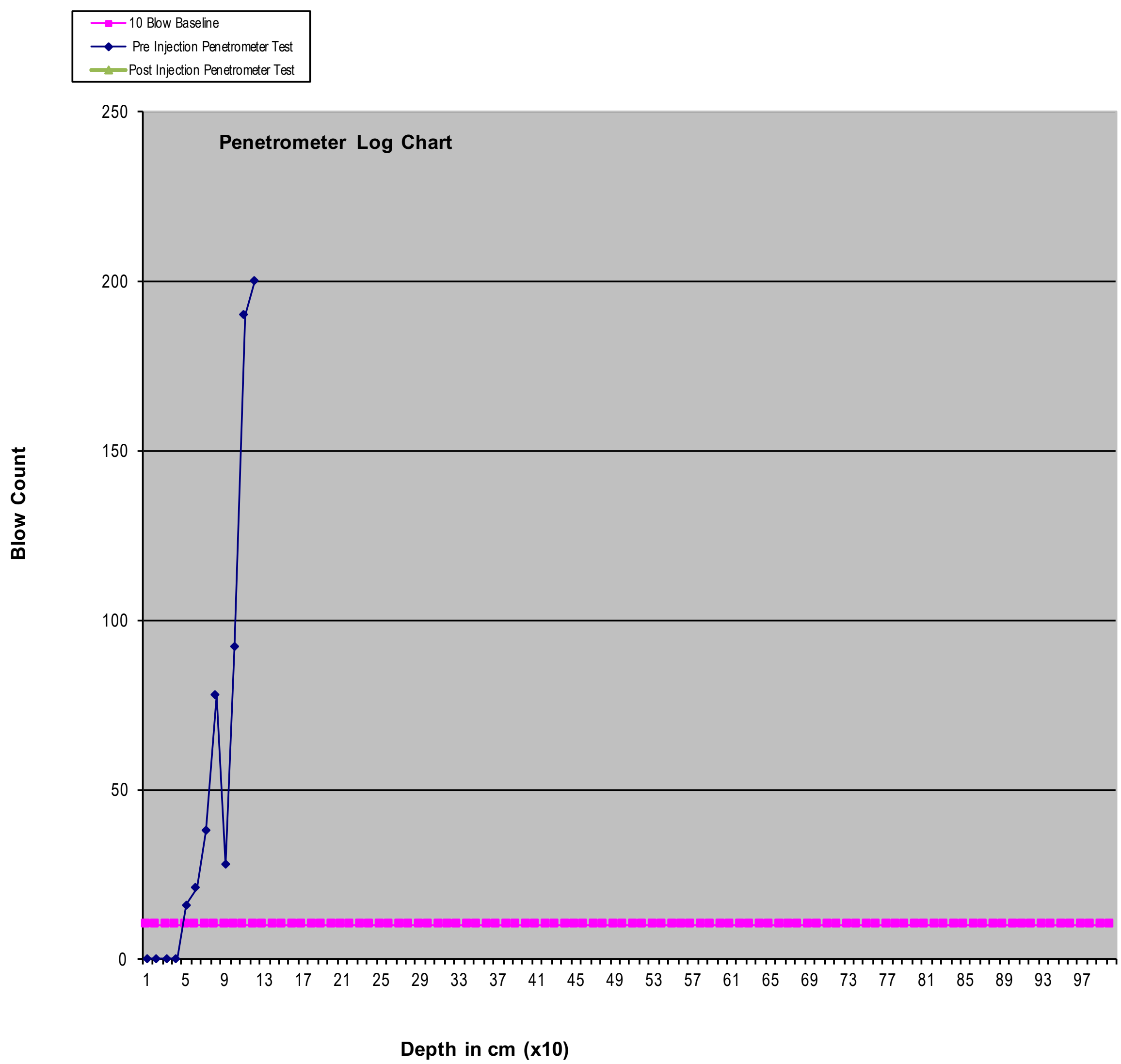
County		Pavement Material	
State		Pavement Thickness	
Roadway		Base Material	
Penetrometer Operators		Base Thickness	
Start Time		SubBase material	
Finish Time		SubBase Thickness	
Penetrometer test #	4 Date 11/22/16	Data Recorder	
Other Information	ODOT 7 SR4 North Bound Right Berm 4' off Pavement Edge Light Pole EF6		

in depth	ft depth	cm depth	Pre Blows		Post Blows	in depth	ft depth	cm depth	Pre Blows		Post Blows
3.94	0.33	10	0.2			200.79	16.73	510			
7.87	0.66	20	0.2			204.72	17.06	520			
11.81	0.98	30	0.2			208.66	17.38	530			
15.75	1.31	40	0.2			212.60	17.71	540			
19.69	1.64	50	16			216.54	18.04	550			
23.62	1.97	60	21			220.47	18.37	560			
27.56	2.30	70	38			224.41	18.70	570			
31.50	2.62	80	78			228.35	19.02	580			
35.43	2.95	90	28			232.28	19.35	590			
39.37	3.28	100	92			236.22	19.68	600			
43.31	3.61	110	190			240.16	20.01	610			
47.24	3.94	120	200			244.09	20.34	620			
51.18	4.26	130				248.03	20.66	630			
55.12	4.59	140				251.97	20.99	640			
59.06	4.92	150				255.91	21.32	650			
62.99	5.25	160				259.84	21.65	660			
66.93	5.58	170				263.78	21.98	670			
70.87	5.90	180				267.72	22.30	680			
74.80	6.23	190				271.65	22.63	690			
78.74	6.56	200				275.59	22.96	700			
82.68	6.89	210				279.53	23.29	710			
86.61	7.22	220				283.46	23.62	720			
90.55	7.54	230				287.40	23.94	730			
94.49	7.87	240				291.34	24.27	740			
98.43	8.20	250				295.28	24.60	750			
102.36	8.53	260				299.21	24.93	760			
106.30	8.86	270				303.15	25.26	770			
110.24	9.18	280				307.09	25.58	780			
114.17	9.51	290				311.02	25.91	790			
118.11	9.84	300				314.96	26.24	800			
122.05	10.17	310				318.90	26.57	810			
125.98	10.50	320				322.83	26.90	820			
129.92	10.82	330				326.77	27.22	830			
133.86	11.15	340				330.71	27.55	840			
137.80	11.48	350				334.65	27.88	850			
141.73	11.81	360				338.58	28.21	860			
145.67	12.14	370				342.52	28.54	870			
149.61	12.46	380				346.46	28.86	880			
153.54	12.79	390				350.39	29.19	890			
157.48	13.12	400				354.33	29.52	900			
161.42	13.45	410				358.27	29.85	910			
165.35	13.78	420				362.20	30.18	920			
169.29	14.10	430				366.14	30.50	930			
173.23	14.43	440				370.08	30.83	940			
177.17	14.76	450				374.02	31.16	950			
181.10	15.09	460				377.95	31.49	960			
185.04	15.42	470				381.89	31.82	970			
188.98	15.74	480				385.83	32.14	980			
192.91	16.07	490				389.76	32.47	990			
196.85	16.40	500				393.70	32.80	1000			

Dynamic Cone Penetrometer Log Sheet

Location Information

County		Pavement Material	
State		Pavement Thickness	
Roadway		Base Material	
Penetrometer Operators		Base Thickness	
Start Time		SubBase material	
Finish Time		SubBase Thickness	
Penetrometer test #	ODOT 7 SR4 North Bound Right Berm 4' of Date 11/22/16	Data Recorder	
Other Information			



DCP 5
Redesignated D-005-0-16

Dynamic Cone Penetrometer Log Sheet

Location Information

County		Pavement Material	
State		Pavement Thickness	
Roadway		Base Material	
Penetrometer Operators		Base Thickness	
Start Time		SubBase material	
Finish Time		SubBase Thickness	
Penetrometer test #	5	Date 11/22/16	Data Recorder
Other Information	ODOT 7 SR4 North Bound Right Berm 4' off Pavement Edge North Dip		

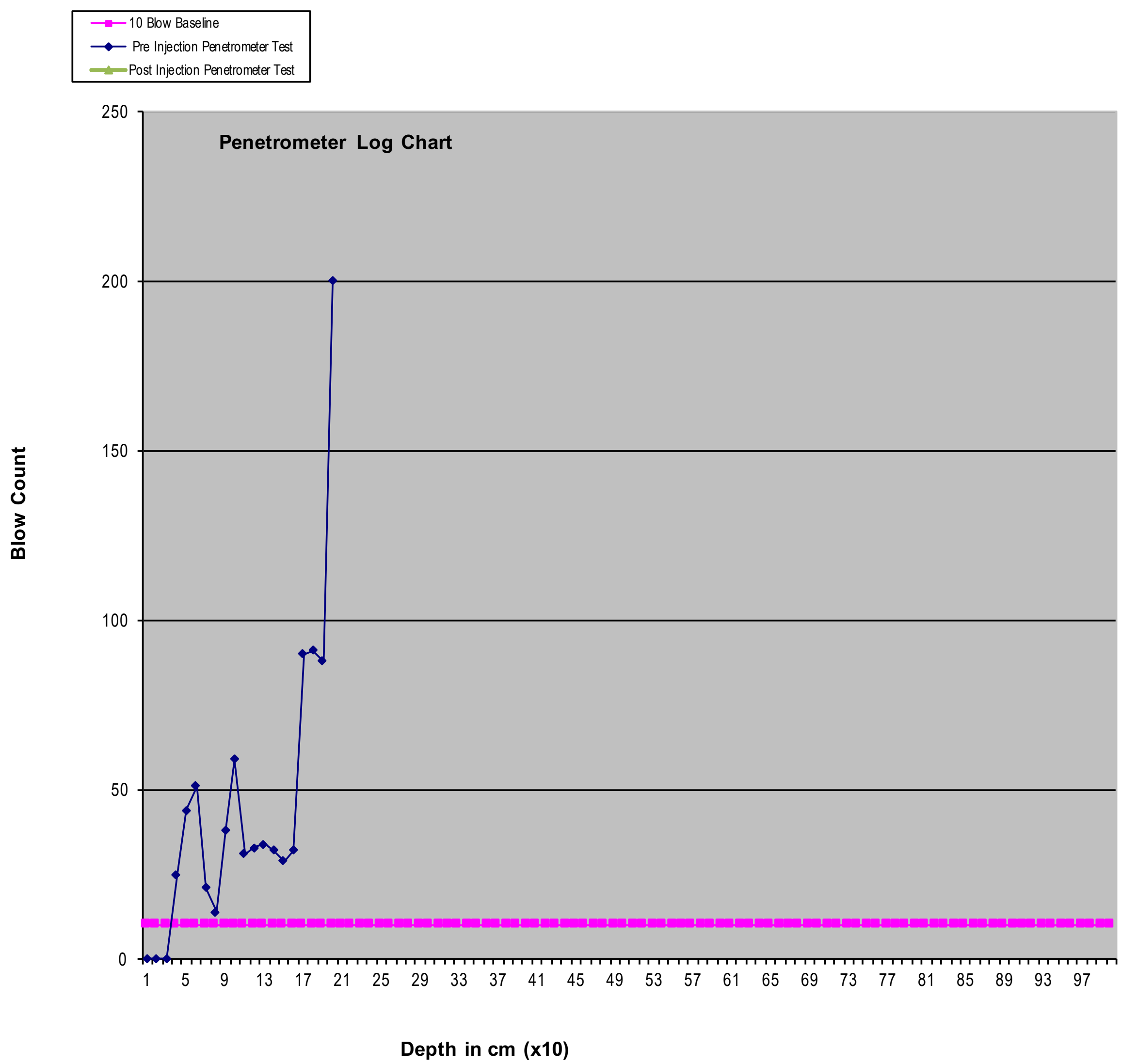
in depth	ft depth	cm depth	Pre Blows		Post Blows
3.94	0.33	10	0.2		
7.87	0.66	20	0.2		
11.81	0.98	30	0.2		
15.75	1.31	40	25		
19.69	1.64	50	44		
23.62	1.97	60	51		
27.56	2.30	70	21		
31.50	2.62	80	14		
35.43	2.95	90	38		
39.37	3.28	100	59		
43.31	3.61	110	31		
47.24	3.94	120	33		
51.18	4.26	130	34		
55.12	4.59	140	32		
59.06	4.92	150	29		
62.99	5.25	160	32		
66.93	5.58	170	90		
70.87	5.90	180	91		
74.80	6.23	190	88		
78.74	6.56	200	200		
82.68	6.89	210			
86.61	7.22	220			
90.55	7.54	230			
94.49	7.87	240			
98.43	8.20	250			
102.36	8.53	260			
106.30	8.86	270			
110.24	9.18	280			
114.17	9.51	290			
118.11	9.84	300			
122.05	10.17	310			
125.98	10.50	320			
129.92	10.82	330			
133.86	11.15	340			
137.80	11.48	350			
141.73	11.81	360			
145.67	12.14	370			
149.61	12.46	380			
153.54	12.79	390			
157.48	13.12	400			
161.42	13.45	410			
165.35	13.78	420			
169.29	14.10	430			
173.23	14.43	440			
177.17	14.76	450			
181.10	15.09	460			
185.04	15.42	470			
188.98	15.74	480			
192.91	16.07	490			
196.85	16.40	500			

in depth	ft depth	cm depth	Pre Blows		Post Blows
200.79	16.73	510			
204.72	17.06	520			
208.66	17.38	530			
212.60	17.71	540			
216.54	18.04	550			
220.47	18.37	560			
224.41	18.70	570			
228.35	19.02	580			
232.28	19.35	590			
236.22	19.68	600			
240.16	20.01	610			
244.09	20.34	620			
248.03	20.66	630			
251.97	20.99	640			
255.91	21.32	650			
259.84	21.65	660			
263.78	21.98	670			
267.72	22.30	680			
271.65	22.63	690			
275.59	22.96	700			
279.53	23.29	710			
283.46	23.62	720			
287.40	23.94	730			
291.34	24.27	740			
295.28	24.60	750			
299.21	24.93	760			
303.15	25.26	770			
307.09	25.58	780			
311.02	25.91	790			
314.96	26.24	800			
318.90	26.57	810			
322.83	26.90	820			
326.77	27.22	830			
330.71	27.55	840			
334.65	27.88	850			
338.58	28.21	860			
342.52	28.54	870			
346.46	28.86	880			
350.39	29.19	890			
354.33	29.52	900			
358.27	29.85	910			
362.20	30.18	920			
366.14	30.50	930			
370.08	30.83	940			
374.02	31.16	950			
377.95	31.49	960			
381.89	31.82	970			
385.83	32.14	980			
389.76	32.47	990			
393.70	32.80	1000			

Dynamic Cone Penetrometer Log Sheet

Location Information

County		Pavement Material	
State		Pavement Thickness	
Roadway		Base Material	
Penetrometer Operators		Base Thickness	
Start Time		SubBase material	
Finish Time		SubBase Thickness	
Penetrometer test #	ODOT 7 SR4 North Bound Right Berm 4' of Date 11/22/16	Data Recorder	
Other Information			



DCP 6
Redesignated D-006-0-16

Dynamic Cone Penetrometer Log Sheet

Location Information

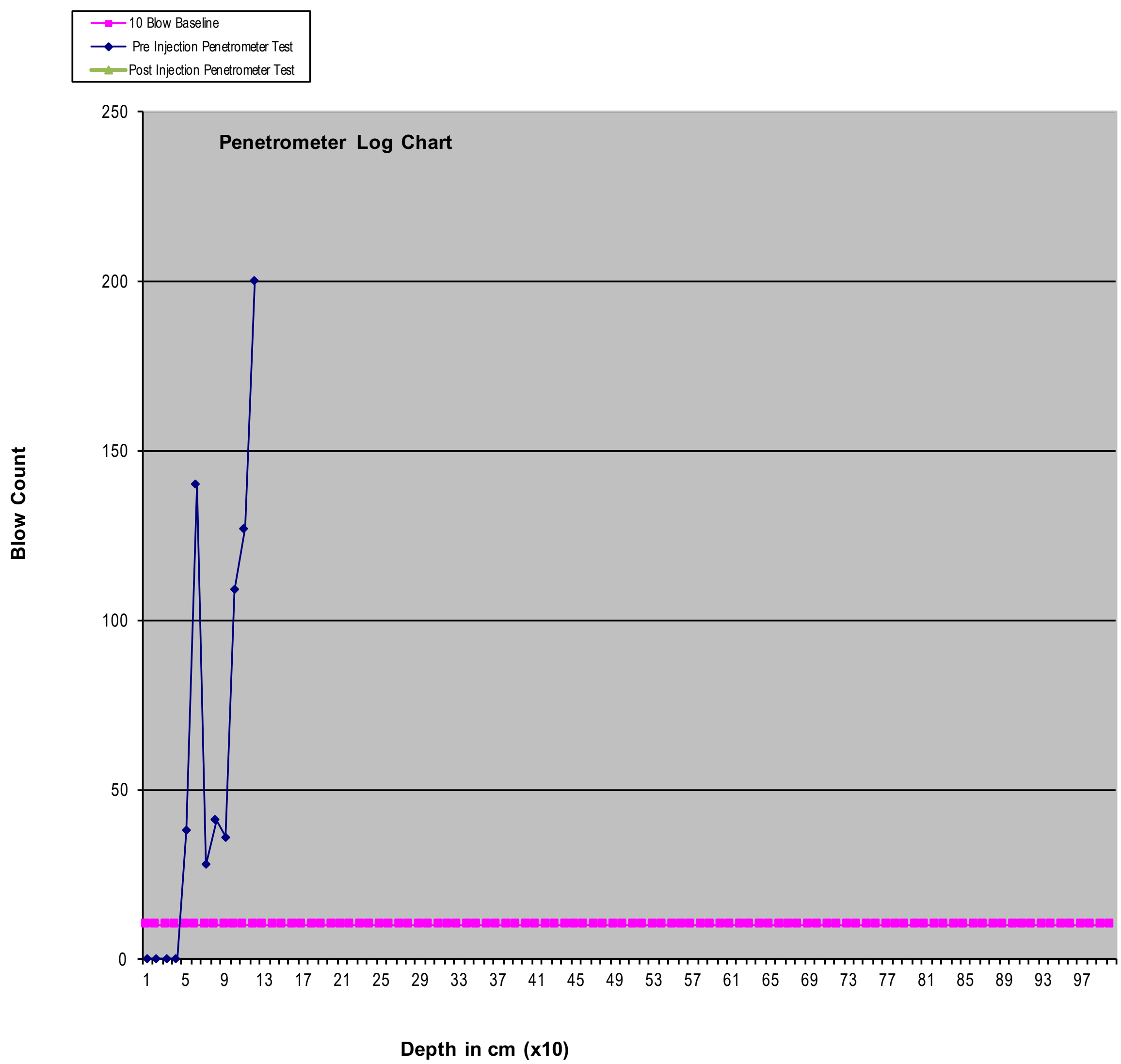
County ______ Pavement Material ______
State ______ Pavement Thickness ______
Roadway ______ Base Material ______
Penetrometer Operators ______ Base Thickness ______
Start Time ______ SubBase material ______
Finish Time ______ SubBase Thickness ______
Penetrometer test # 6 Date 11/22/16 Data Recorder ______
Other Information ODOT 7 SR4 North Bound Right Berm 4' off Pavement Edge Past North Dip

in depth	ft depth	cm depth	Pre Blows		Post Blows	in depth	ft depth	cm depth	Pre Blows		Post Blows
3.94	**0.33**	10	0.2			200.79	16.73	510			
7.87	**0.66**	20	0.2			204.72	17.06	520			
11.81	**0.98**	30	0.2			208.66	17.38	530			
15.75	**1.31**	40	0.2			212.60	17.71	540			
19.69	**1.64**	50	38			216.54	18.04	550			
23.62	**1.97**	60	140			220.47	18.37	560			
27.56	**2.30**	70	28			224.41	18.70	570			
31.50	**2.62**	80	41			228.35	19.02	580			
35.43	**2.95**	90	36			232.28	19.35	590			
39.37	**3.28**	100	109			236.22	19.68	600			
43.31	**3.61**	110	127			240.16	20.01	610			
47.24	**3.94**	120	200			244.09	20.34	620			
51.18	**4.26**	130				248.03	20.66	630			
55.12	**4.59**	140				251.97	20.99	640			
59.06	**4.92**	150				255.91	21.32	650			
62.99	**5.25**	160				259.84	21.65	660			
66.93	**5.58**	170				263.78	21.98	670			
70.87	**5.90**	180				267.72	22.30	680			
74.80	**6.23**	190				271.65	22.63	690			
78.74	**6.56**	200				275.59	22.96	700			
82.68	**6.89**	210				279.53	23.29	710			
86.61	**7.22**	220				283.46	23.62	720			
90.55	**7.54**	230				287.40	23.94	730			
94.49	**7.87**	240				291.34	24.27	740			
98.43	**8.20**	250				295.28	24.60	750			
102.36	**8.53**	260				299.21	24.93	760			
106.30	**8.86**	270				303.15	25.26	770			
110.24	**9.18**	280				307.09	25.58	780			
114.17	**9.51**	290				311.02	25.91	790			
118.11	**9.84**	300				314.96	26.24	800			
122.05	**10.17**	310				318.90	26.57	810			
125.98	**10.50**	320				322.83	26.90	820			
129.92	**10.82**	330				326.77	27.22	830			
133.86	**11.15**	340				330.71	27.55	840			
137.80	**11.48**	350				334.65	27.88	850			
141.73	**11.81**	360				338.58	28.21	860			
145.67	**12.14**	370				342.52	28.54	870			
149.61	**12.46**	380				346.46	28.86	880			
153.54	**12.79**	390				350.39	29.19	890			
157.48	**13.12**	400				354.33	29.52	900			
161.42	**13.45**	410				358.27	29.85	910			
165.35	**13.78**	420				362.20	30.18	920			
169.29	**14.10**	430				366.14	30.50	930			
173.23	**14.43**	440				370.08	30.83	940			
177.17	**14.76**	450				374.02	31.16	950			
181.10	**15.09**	460				377.95	31.49	960			
185.04	**15.42**	470				381.89	31.82	970			
188.98	**15.74**	480				385.83	32.14	980			
192.91	**16.07**	490				389.76	32.47	990			
196.85	**16.40**	500				393.70	32.80	1000			

Dynamic Cone Penetrometer Log Sheet

Location Information

County		Pavement Material	
State		Pavement Thickness	
Roadway		Base Material	
Penetrometer Operators		Base Thickness	
Start Time		SubBase material	
Finish Time		SubBase Thickness	
Penetrometer test #	ODOT 7 SR4 North Bound Right Berm 4' of Date 11/22/16	Data Recorder	
Other Information			



DCP 7
Redesignated D-007-0-16

Dynamic Cone Penetrometer Log Sheet

Location Information

County		Pavement Material	
State		Pavement Thickness	
Roadway		Base Material	
Penetrometer Operators		Base Thickness	
Start Time		SubBase material	
Finish Time		SubBase Thickness	
Penetrometer test #	7 Date 11/22/16	Data Recorder	
Other Information	ODOT 7 SR4 South Bound Left Berm 5' off Pavement Edge End of Gaurdrail		

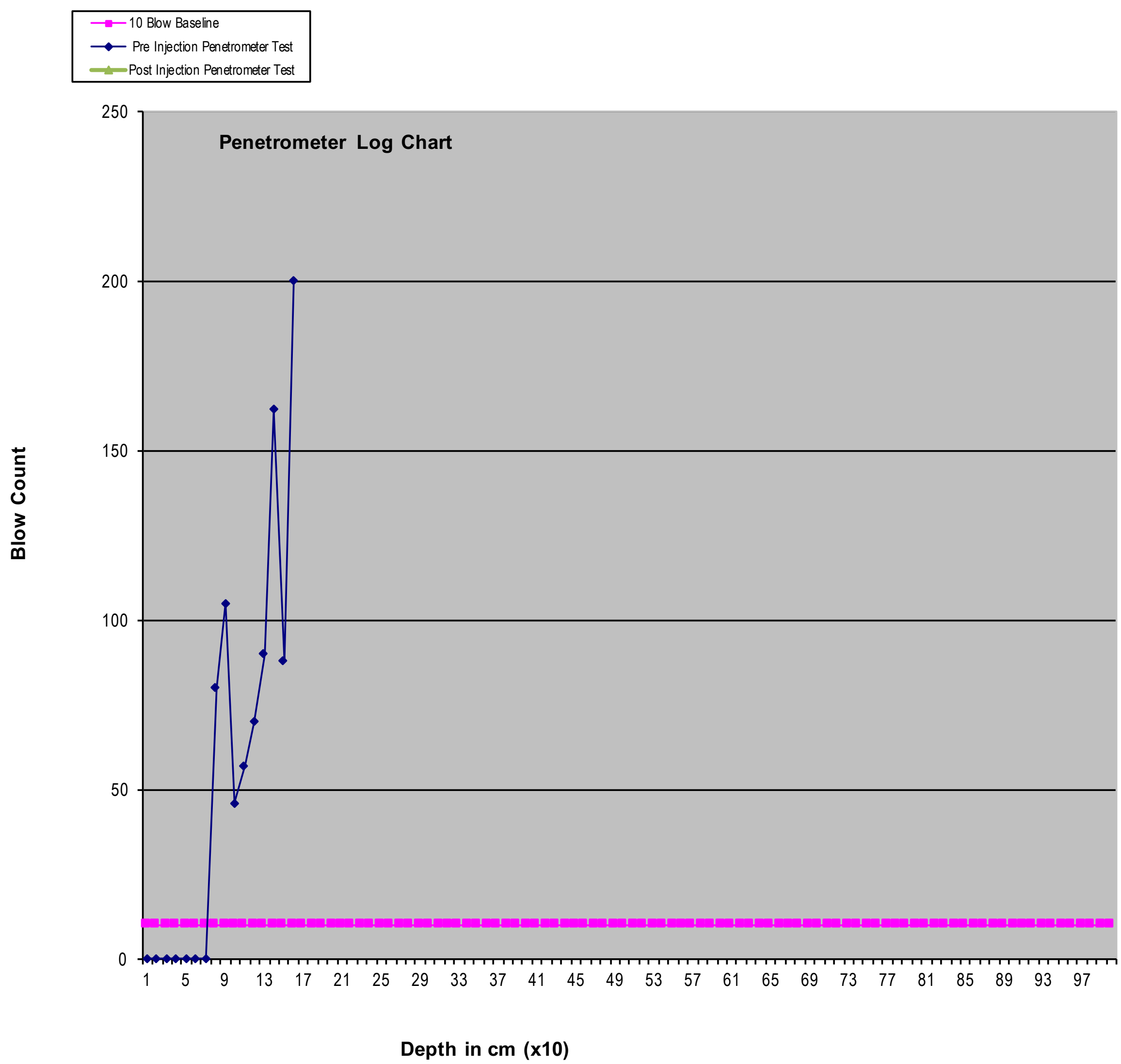
in depth	ft depth	cm depth	Pre Blows		Post Blows
3.94	0.33	10	0.2		
7.87	0.66	20	0.2		
11.81	0.98	30	0.2		
15.75	1.31	40	0.2		
19.69	1.64	50	0.2		
23.62	1.97	60	0.2		
27.56	2.30	70	0.2		
31.50	2.62	80	80		
35.43	2.95	90	105		
39.37	3.28	100	46		
43.31	3.61	110	57		
47.24	3.94	120	70		
51.18	4.26	130	90		
55.12	4.59	140	162		
59.06	4.92	150	88		
62.99	5.25	160	200		
66.93	5.58	170			
70.87	5.90	180			
74.80	6.23	190			
78.74	6.56	200			
82.68	6.89	210			
86.61	7.22	220			
90.55	7.54	230			
94.49	7.87	240			
98.43	8.20	250			
102.36	8.53	260			
106.30	8.86	270			
110.24	9.18	280			
114.17	9.51	290			
118.11	9.84	300			
122.05	10.17	310			
125.98	10.50	320			
129.92	10.82	330			
133.86	11.15	340			
137.80	11.48	350			
141.73	11.81	360			
145.67	12.14	370			
149.61	12.46	380			
153.54	12.79	390			
157.48	13.12	400			
161.42	13.45	410			
165.35	13.78	420			
169.29	14.10	430			
173.23	14.43	440			
177.17	14.76	450			
181.10	15.09	460			
185.04	15.42	470			
188.98	15.74	480			
192.91	16.07	490			
196.85	16.40	500			

in depth	ft depth	cm depth	Pre Blows		Post Blows
200.79	16.73	510			
204.72	17.06	520			
208.66	17.38	530			
212.60	17.71	540			
216.54	18.04	550			
220.47	18.37	560			
224.41	18.70	570			
228.35	19.02	580			
232.28	19.35	590			
236.22	19.68	600			
240.16	20.01	610			
244.09	20.34	620			
248.03	20.66	630			
251.97	20.99	640			
255.91	21.32	650			
259.84	21.65	660			
263.78	21.98	670			
267.72	22.30	680			
271.65	22.63	690			
275.59	22.96	700			
279.53	23.29	710			
283.46	23.62	720			
287.40	23.94	730			
291.34	24.27	740			
295.28	24.60	750			
299.21	24.93	760			
303.15	25.26	770			
307.09	25.58	780			
311.02	25.91	790			
314.96	26.24	800			
318.90	26.57	810			
322.83	26.90	820			
326.77	27.22	830			
330.71	27.55	840			
334.65	27.88	850			
338.58	28.21	860			
342.52	28.54	870			
346.46	28.86	880			
350.39	29.19	890			
354.33	29.52	900			
358.27	29.85	910			
362.20	30.18	920			
366.14	30.50	930			
370.08	30.83	940			
374.02	31.16	950			
377.95	31.49	960			
381.89	31.82	970			
385.83	32.14	980			
389.76	32.47	990			
393.70	32.80	1000			

Dynamic Cone Penetrometer Log Sheet

Location Information

County		Pavement Material	
State		Pavement Thickness	
Roadway		Base Material	
Penetrometer Operators		Base Thickness	
Start Time		SubBase material	
Finish Time		SubBase Thickness	
Penetrometer test #	ODOT 7 SR4 South Bound Left Berm 5' off Date 11/22/16	Data Recorder	
Other Information			



DCP 8
Redesignated D-008-0-16

Dynamic Cone Penetrometer Log Sheet

Location Information

County
State
Roadway
Penetrometer Operators
Start Time
Finish Time
Penetrometer test # 8
Date 11/22/16
Other Information ODOT 7 SR4 South Bound Right Berm 5' off Pavement Edge In Dip

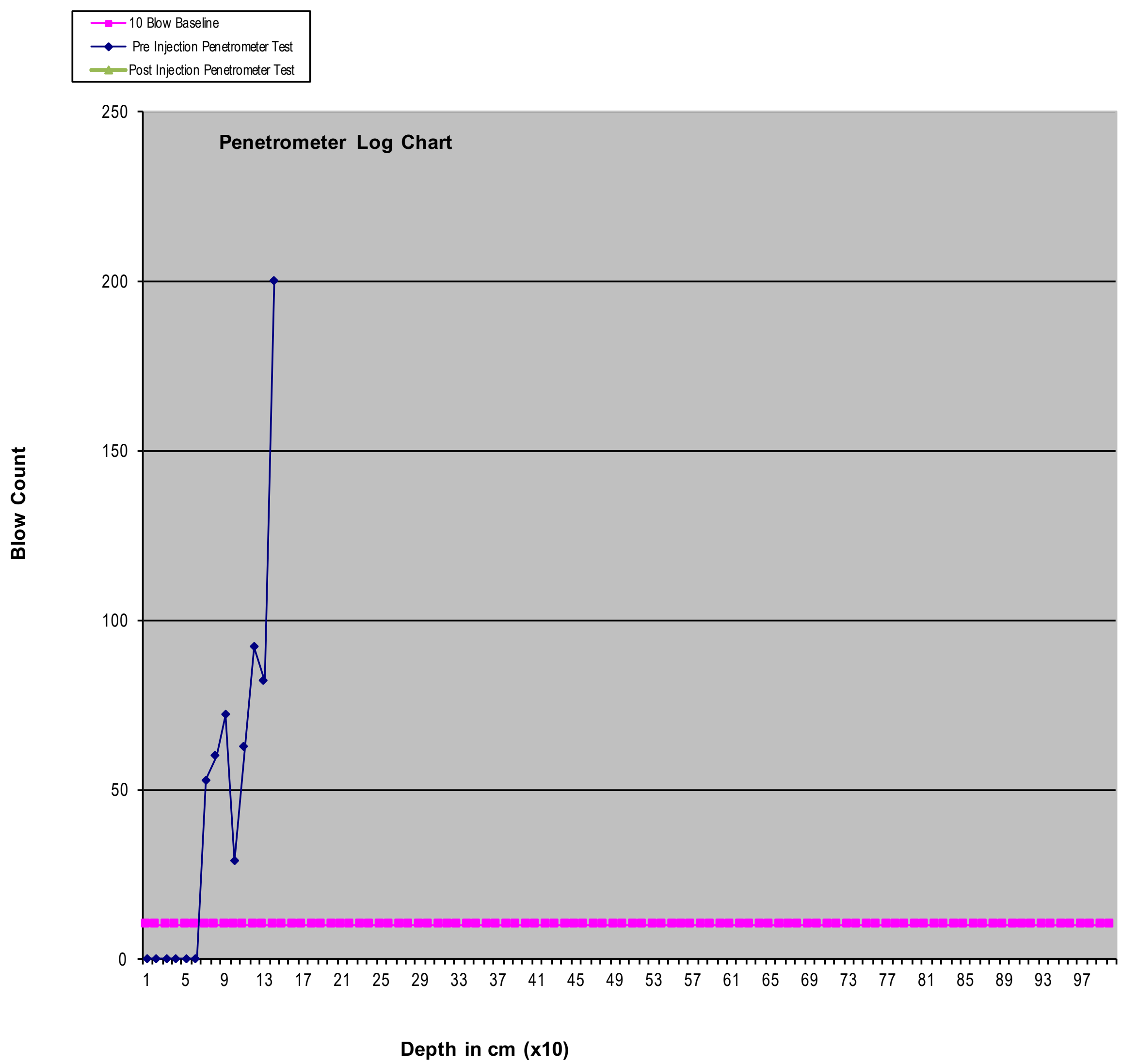
Pavement Material
Pavement Thickness
Base Material
Base Thickness
SubBase material
SubBase Thickness
Data Recorder

in depth	ft depth	cm depth	Pre Blows	Post Blows
3.94	**0.33**	10	**0.2**	
7.87	**0.66**	20	**0.2**	
11.81	**0.98**	30	**0.2**	
15.75	**1.31**	40	**0.2**	
19.69	**1.64**	50	**0.2**	
23.62	**1.97**	60	**0.2**	
27.56	**2.30**	70	**53**	
31.50	**2.62**	80	**60**	
35.43	**2.95**	90	**72**	
39.37	**3.28**	100	**29**	
43.31	**3.61**	110	**63**	
47.24	**3.94**	120	**92**	
51.18	**4.26**	130	**82**	
55.12	**4.59**	140	**200**	
59.06	**4.92**	150		
62.99	**5.25**	160		
66.93	**5.58**	170		
70.87	**5.90**	180		
74.80	**6.23**	190		
78.74	**6.56**	200		
82.68	**6.89**	210		
86.61	**7.22**	220		
90.55	**7.54**	230		
94.49	**7.87**	240		
98.43	**8.20**	250		
102.36	**8.53**	260		
106.30	**8.86**	270		
110.24	**9.18**	280		
114.17	**9.51**	290		
118.11	**9.84**	300		
122.05	**10.17**	310		
125.98	**10.50**	320		
129.92	**10.82**	330		
133.86	**11.15**	340		
137.80	**11.48**	350		
141.73	**11.81**	360		
145.67	**12.14**	370		
149.61	**12.46**	380		
153.54	**12.79**	390		
157.48	**13.12**	400		
161.42	**13.45**	410		
165.35	**13.78**	420		
169.29	**14.10**	430		
173.23	**14.43**	440		
177.17	**14.76**	450		
181.10	**15.09**	460		
185.04	**15.42**	470		
188.98	**15.74**	480		
192.91	**16.07**	490		
196.85	**16.40**	500		
200.79	16.73	510		
204.72	17.06	520		
208.66	17.38	530		
212.60	17.71	540		
216.54	18.04	550		
220.47	18.37	560		
224.41	18.70	570		
228.35	19.02	580		
232.28	19.35	590		
236.22	19.68	600		
240.16	20.01	610		
244.09	20.34	620		
248.03	20.66	630		
251.97	20.99	640		
255.91	21.32	650		
259.84	21.65	660		
263.78	21.98	670		
267.72	22.30	680		
271.65	22.63	690		
275.59	22.96	700		
279.53	23.29	710		
283.46	23.62	720		
287.40	23.94	730		
291.34	24.27	740		
295.28	24.60	750		
299.21	24.93	760		
303.15	25.26	770		
307.09	25.58	780		
311.02	25.91	790		
314.96	26.24	800		
318.90	26.57	810		
322.83	26.90	820		
326.77	27.22	830		
330.71	27.55	840		
334.65	27.88	850		
338.58	28.21	860		
342.52	28.54	870		
346.46	28.86	880		
350.39	29.19	890		
354.33	29.52	900		
358.27	29.85	910		
362.20	30.18	920		
366.14	30.50	930		
370.08	30.83	940		
374.02	31.16	950		
377.95	31.49	960		
381.89	31.82	970		
385.83	32.14	980		
389.76	32.47	990		
393.70	32.80	1000		

Dynamic Cone Penetrometer Log Sheet

Location Information

County		Pavement Material	
State		Pavement Thickness	
Roadway		Base Material	
Penetrometer Operators		Base Thickness	
Start Time		SubBase material	
Finish Time		SubBase Thickness	
Penetrometer test #	ODOT 7 SR4 South Bound Right Berm 5' o Date 11/22/16	Data Recorder	
Other Information			



DCP 9
Redesignated D-009-0-16

Dynamic Cone Penetrometer Log Sheet

Location Information

County
State
Roadway
Penetrometer Operators
Start Time
Finish Time
Penetrometer test # 9
Date 11/22/16
Other Information ODOT 7 SR4 South Bound Right Berm 5' off Pavement Edge In Southern Dip

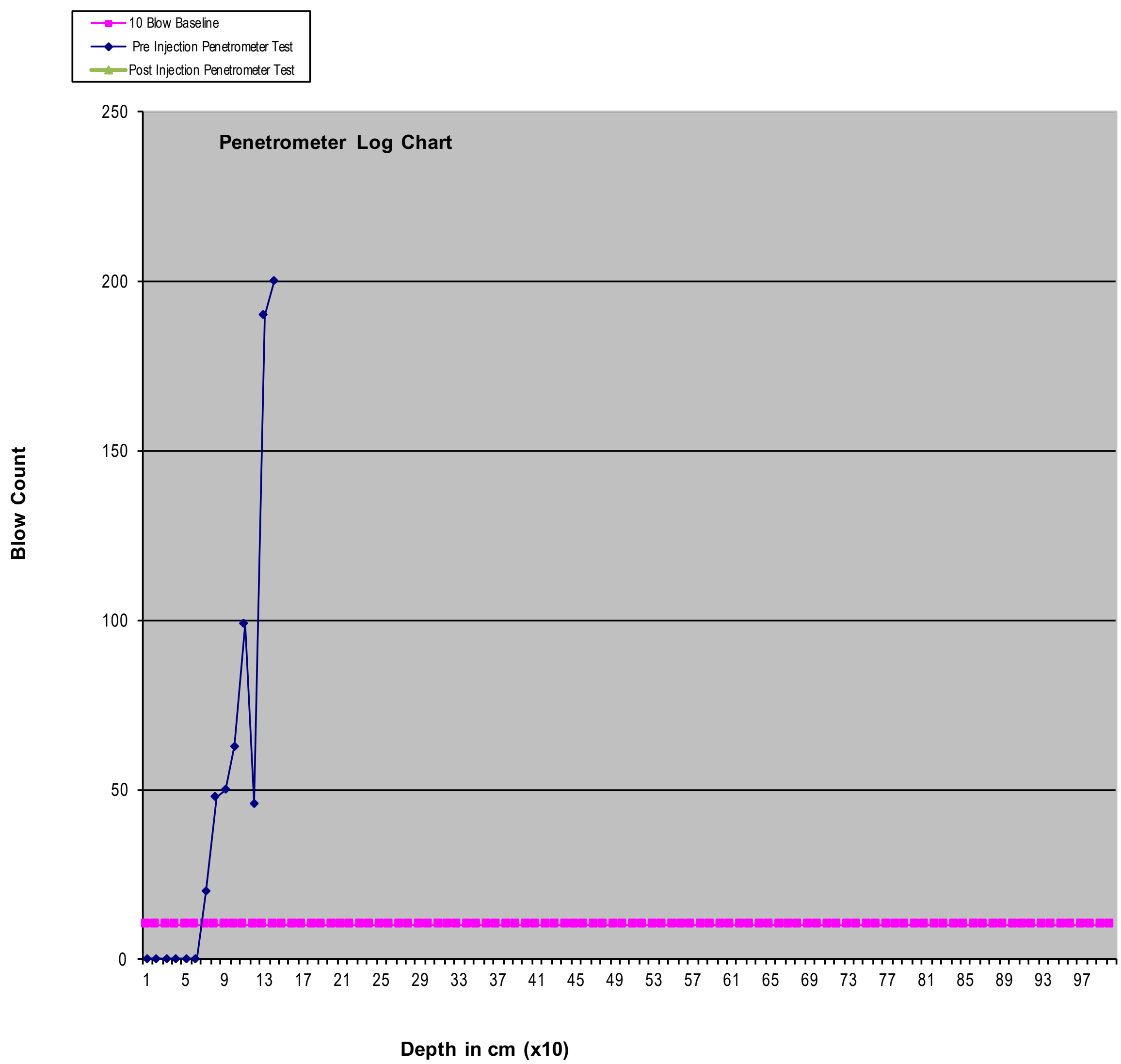
Pavement Material
Pavement Thickness
Base Material
Base Thickness
SubBase material
SubBase Thickness
Data Recorder

in depth	ft depth	cm depth	Pre Blows		Post Blows	in depth	ft depth	cm depth	Pre Blows		Post Blows
3.94	0.33	10	0.2			200.79	16.73	510			
7.87	0.66	20	0.2			204.72	17.06	520			
11.81	0.98	30	0.2			208.66	17.38	530			
15.75	1.31	40	0.2			212.60	17.71	540			
19.69	1.64	50	0.2			216.54	18.04	550			
23.62	1.97	60	0.2			220.47	18.37	560			
27.56	2.30	70	20			224.41	18.70	570			
31.50	2.62	80	48			228.35	19.02	580			
35.43	2.95	90	50			232.28	19.35	590			
39.37	3.28	100	63			236.22	19.68	600			
43.31	3.61	110	99			240.16	20.01	610			
47.24	3.94	120	46			244.09	20.34	620			
51.18	4.26	130	190			248.03	20.66	630			
55.12	4.59	140	200			251.97	20.99	640			
59.06	4.92	150				255.91	21.32	650			
62.99	5.25	160				259.84	21.65	660			
66.93	5.58	170				263.78	21.98	670			
70.87	5.90	180				267.72	22.30	680			
74.80	6.23	190				271.65	22.63	690			
78.74	6.56	200				275.59	22.96	700			
82.68	6.89	210				279.53	23.29	710			
86.61	7.22	220				283.46	23.62	720			
90.55	7.54	230				287.40	23.94	730			
94.49	7.87	240				291.34	24.27	740			
98.43	8.20	250				295.28	24.60	750			
102.36	8.53	260				299.21	24.93	760			
106.30	8.86	270				303.15	25.26	770			
110.24	9.18	280				307.09	25.58	780			
114.17	9.51	290				311.02	25.91	790			
118.11	9.84	300				314.96	26.24	800			
122.05	10.17	310				318.90	26.57	810			
125.98	10.50	320				322.83	26.90	820			
129.92	10.82	330				326.77	27.22	830			
133.86	11.15	340				330.71	27.55	840			
137.80	11.48	350				334.65	27.88	850			
141.73	11.81	360				338.58	28.21	860			
145.67	12.14	370				342.52	28.54	870			
149.61	12.46	380				346.46	28.86	880			
153.54	12.79	390				350.39	29.19	890			
157.48	13.12	400				354.33	29.52	900			
161.42	13.45	410				358.27	29.85	910			
165.35	13.78	420				362.20	30.18	920			
169.29	14.10	430				366.14	30.50	930			
173.23	14.43	440				370.08	30.83	940			
177.17	14.76	450				374.02	31.16	950			
181.10	15.09	460				377.95	31.49	960			
185.04	15.42	470				381.89	31.82	970			
188.98	15.74	480				385.83	32.14	980			
192.91	16.07	490				389.76	32.47	990			
196.85	16.40	500				393.70	32.80	1000			

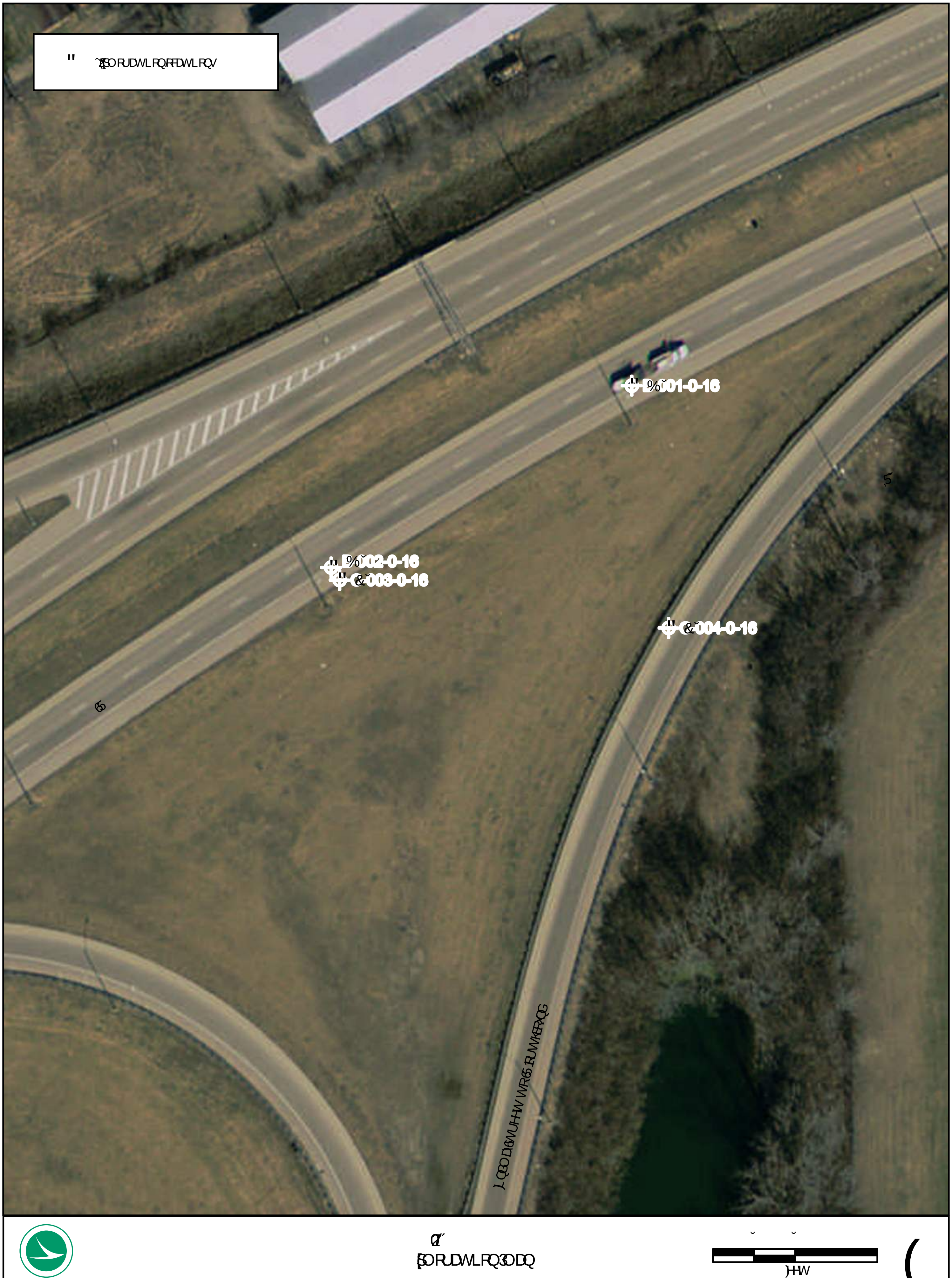
Dynamic Cone Penetrometer Log Sheet

Location Information

County		Pavement Material	
State		Pavement Thickness	
Roadway		Base Material	
Penetrometer Operators		Base Thickness	
Start Time		SubBase material	
Finish Time		SubBase Thickness	
Penetrometer test #	ODOT 7 SR4 South Bound Right Berm 5' o Date 11/22/16	Data Recorder	
Other Information			



MOT-4-19.30 (ODOT 2016) – Exploratory Borings



001-0-16
002-0-16
003-0-16
004-0-16

PROJECT: MOT-4-19.30	DRILLING FIRM / OPERATOR: ODOT / BINKLEY	DRILL RIG: CME 55 TRUCK	STATION / OFFSET:	EXPLORATION ID B-001-0-16
TYPE: UNCONTROLLED FILL	SAMPLING FIRM / LOGGER: ODOT / MCLEISH	HAMMER: CME AUTOMATIC	ALIGNMENT:	
PID: SFN: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 5/27/15	ELEVATION: 764.9 (MSL) EOB: 20.0 ft.	PAGE 1 OF 1
START: 1/31/17 END: 1/31/17	SAMPLING METHOD: SPT	ENERGY RATIO (%): 85	LAT / LONG: 39.780297, -84.158906	

MATERIAL DESCRIPTION AND NOTES	ELEV. 764.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
ASPHALT (15")	763.7																	
DENSE, GRAYISH BROWN, **STONE FRAGMENTS WITH SAND AND SILT**, LITTLE CLAY, (FILL), DAMP		4	8 18 23	58	100	SS-1A	-	-	-	-	-	-	-	-	-	6	A-2-4 (V)	
		9	15 18 14	45	44	SS-2A	-	-	-	-	-	-	-	-	-	5	A-2-4 (V)	
	751.4																	
LOOSE TO MEDIUM DENSE, VERY DARK BROWN, **UNCONTROLLED FILL**, REFUSE CONTAINING GLASS WITH SAND, SILT, CLAY AND STONE FRAGMENTS, DAMP		14	8 3 4	10	0	SS-3A	-	-	-	-	-	-	-	-	-	-	UCF (V)	
		17	8 12 14	37	6	SS-4A	-	-	-	-	-	-	-	-	-	17	UCF (V)	
@20.0'; ATTEMPTED CPT SOUNDING WITH MINIMAL PENETRATION.	744.9	19	4 4 12	23	11	SS-5A	-	-	-	-	-	-	-	-	-	15	UCF (V)	
		EOB 20																

NOTES: HOLE DRY UPON COMPLETION. LAT/LONG FROM OGE HANDHELD GPS UNIT. ELEV FROM OSIP DEM.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: AUGER CUTTINGS MIXED WITH 50 LB. BENTONITE CHIPS

PROJECT: MOT-4-19.30	DRILLING FIRM / OPERATOR: ODOT / BINKLEY	DRILL RIG: CME 55 TRUCK	STATION / OFFSET:
TYPE: UNCONTROLLED FILL	SAMPLING FIRM / LOGGER: ODOT / MCLEISH	HAMMER: CME AUTOMATIC	ALIGNMENT:
PID: SFN: N/A	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 5/27/15	ELEVATION: 767.8 (MSL) EOB: 12.5 ft.
START: 1/31/17 END: 1/31/17	SAMPLING METHOD: SPT	ENERGY RATIO (%): 85	LAT / LONG: 39.779984, -84.159550

EXPLORATION ID B-002-0-16

PAGE 1 OF 1

MATERIAL DESCRIPTION AND NOTES	ELEV. 767.8	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	BACK FILL
ASPHALT (36")	764.8	0–3																
VERY DENSE, BROWN, **STONE FRAGMENTS WITH SAND**, LITTLE SILT, TRACE CLAY, WITH CONCRETE FRAGMENTS (FILL), DAMP	759.8	6–7.5	21 28 26	76	100	SS-1A	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
MEDIUM DENSE, DARK GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT**, LITTLE CLAY, (FILL), DAMP	756.8	8.5–10	17 12 12	34	100	SS-2A	-	-	-	-	-	-	-	-	-	12	A-2-4 (V)	
DENSE, DARK GRAYISH BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, LITTLE SILT, TRACE CLAY, WITH CONCRETE FRAGMENTS (FILL), DAMP	755.3	11–12.5 EOB	12 14 31	64	100	SS-3A	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	

NOTES: HOLE DRY UPON COMPLETION. LAT/LONG FROM OGE HANDHELD GPS UNIT. ELEV FROM OSIP DEM.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: AUGER CUTTINGS MIXED WITH 50 LB. BENTONITE CHIPS

MOT-4-19.30 (ODOT, 2021) – DCP

DCP TEST DATA

OHIO DEPARTMENT OF TRANSPORTATION
OFFICE OF GEOTECHNICAL ENGINEERING

Project: MOT-4-1993
PID: 115661
Date: 9/15/2021

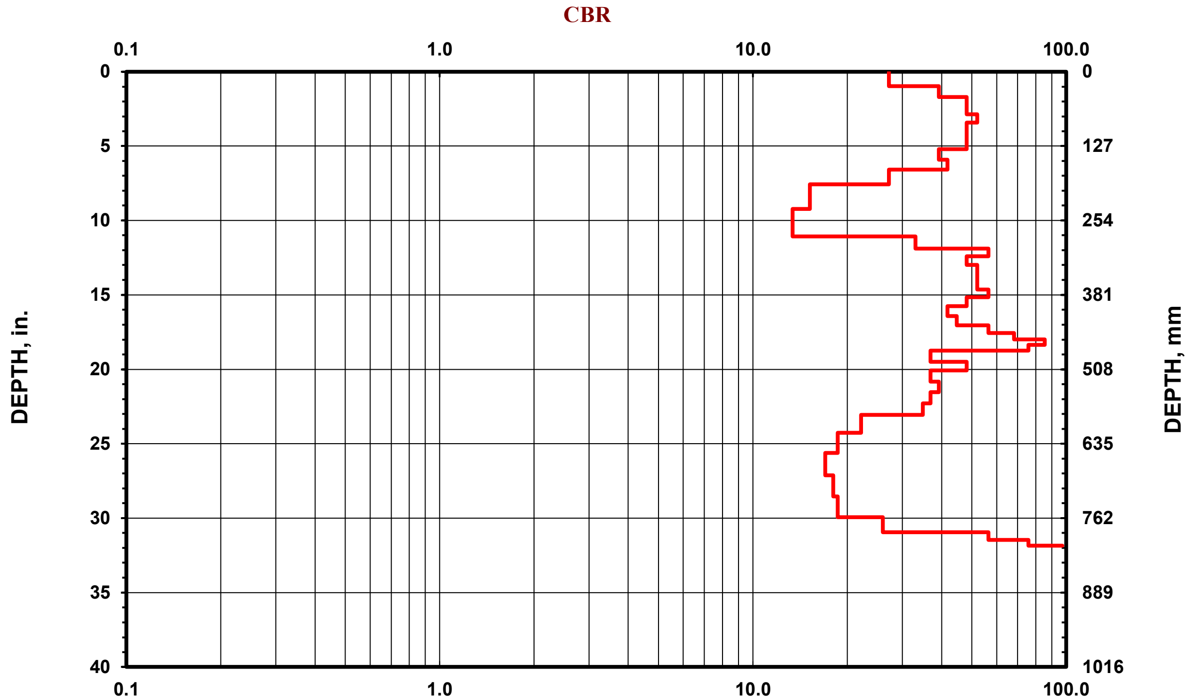
Soil Type
◉ All other soils

Crew: PPP

Hammer
○ 10.1 lbs.
◉ 17.6 lbs.

Office of Geotechnical Engineering
Geology, Exploration, and Laboratory Section

ID: D-001-0-21
Ground Elev. (ft): 774.2
Start Depth (ft:): 1.5
Latitude: 39.77918953
Longitude: 84.16103283



Ground Elev.	Depth (Ft)	Material Encountered
774.2	0.71	Asphalt (8.5-in)
	1.42	Concrete (8.5-in)
		Sand

Notes: Completed in eastbound passing lane

DCP TEST DATA

OHIO DEPARTMENT OF TRANSPORTATION
OFFICE OF GEOTECHNICAL ENGINEERING

Project: MOT-4-1993
PID: 115661
Date: 9/15/2021

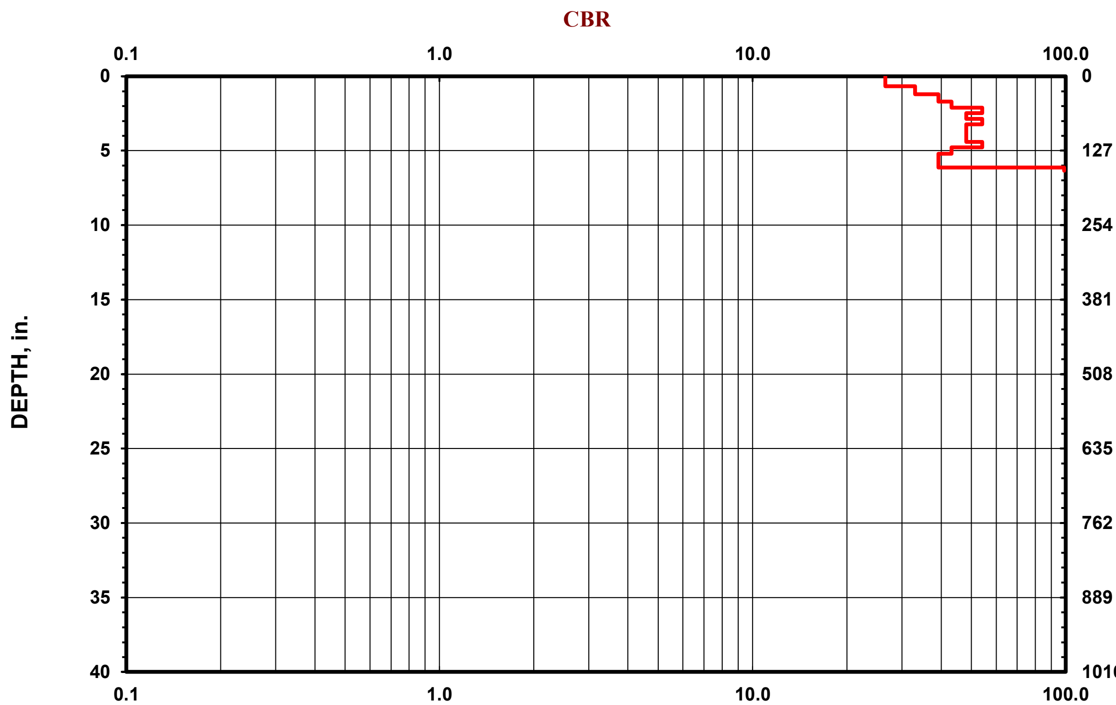
Soil Type
◉ All other soils

Crew: PPP

Hammer
○ 10.1 lbs.
◉ 17.6 lbs.

Office of Geotechnical Engineering
Geology, Exploration, and Laboratory Section

ID: D-002-0-21
Ground Elev. (ft): 774
Start Depth (ft:): 1.8
Latitude: 39.779638
Longitude: 84.160688



Ground Elev.	Depth (Ft)	Material Encountered
774	0.83	Asphalt (10-in)
	1.58	Concrete (9-in)
		Sand

Notes: Completed in westbound passing lane

DCP TEST DATA

OHIO DEPARTMENT OF TRANSPORTATION
OFFICE OF GEOTECHNICAL ENGINEERING

Project: MOT-4-1993
PID: 115661
Date: 9/15/2021

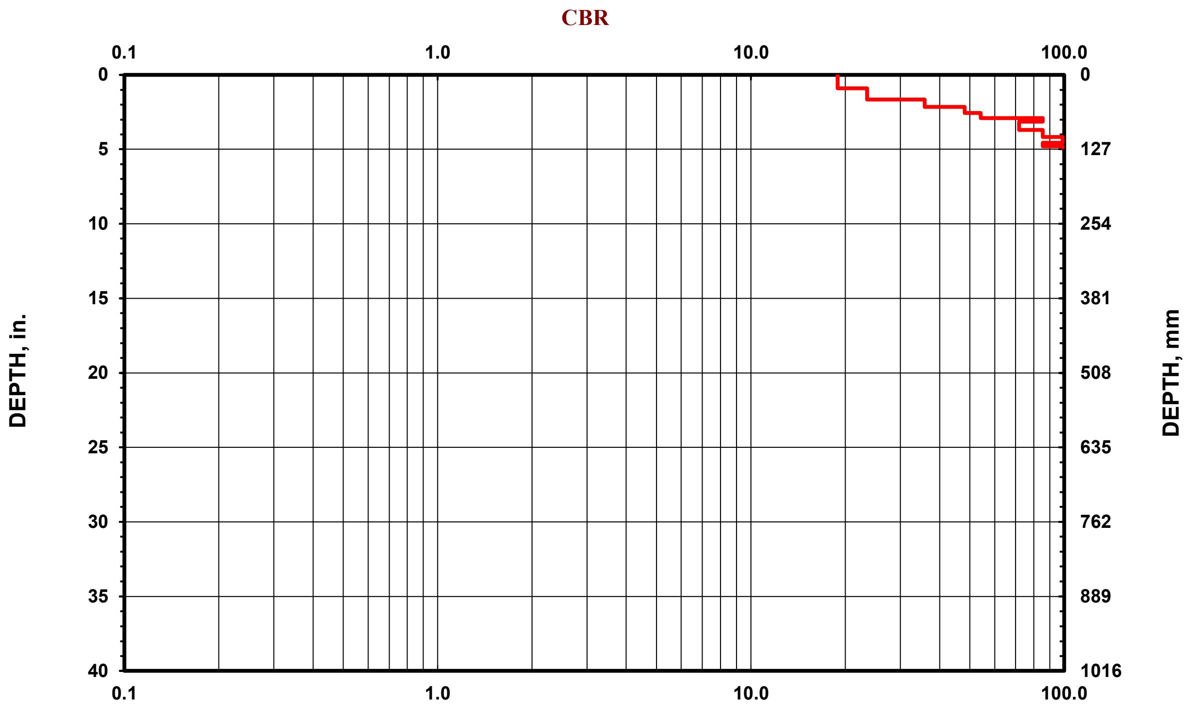
Soil Type
◉ All other soils

Crew: PPP

Hammer
○ 10.1 lbs.
◉ 17.6 lbs.

Office of Geotechnical Engineering
Geology, Exploration, and Laboratory Section

ID: D-003-0-21
Ground Elev. (ft): 771.5
Start Depth (ft:): 2.2
Latitude: 39.779537
Longitude: 84.160388



Ground Elev.	Depth (Ft)	Material Encountered
771.5	1.25	Asphalt (15-in)
	2.00	Concrete (9-in)
		Sand

Notes: Completed in eastbound travel lane

DCP TEST DATA

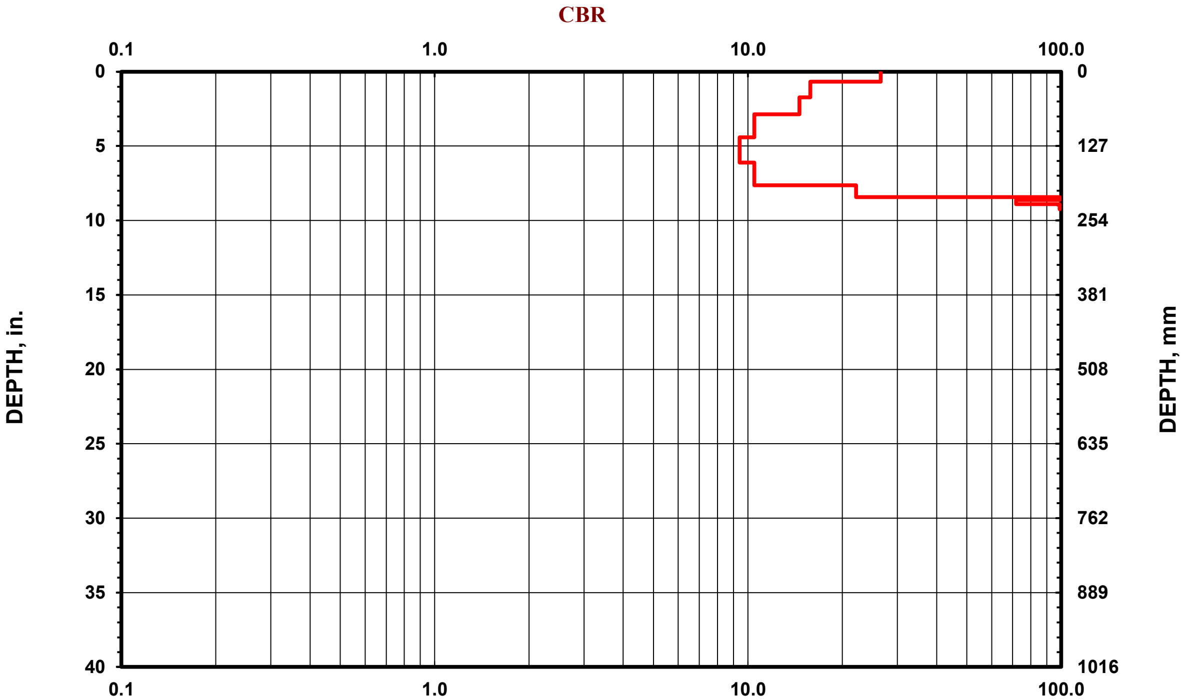
OHIO DEPARTMENT OF TRANSPORTATION
OFFICE OF GEOTECHNICAL ENGINEERING

Project:	MOT-4-1993	**Crew:**	PPP	**ID:**	D-004-0-21
PID:	115661			**Ground Elev. (ft):**	771.7
Date:	9/15/2021			**Start Depth (ft:):**	2.42
				Latitude:	39.779970
				Longitude:	84.160197

Soil Type
◉ All other soils

Hammer
○ 10.1 lbs.
◉ 17.6 lbs.

Office of Geotechnical Engineering
Geology, Exploration, and Laboratory Section



Ground Elev.	Depth (Ft)	Material Encountered
771.7	1.54	Asphalt (18.5-in)
	2.29	Concrete (9-in)
		Sand

Notes: Completed in westbound driving lane

DCP TEST DATA

OHIO DEPARTMENT OF TRANSPORTATION
OFFICE OF GEOTECHNICAL ENGINEERING

Project: MOT-4-1993
PID: 115661
Date: 9/15/2021

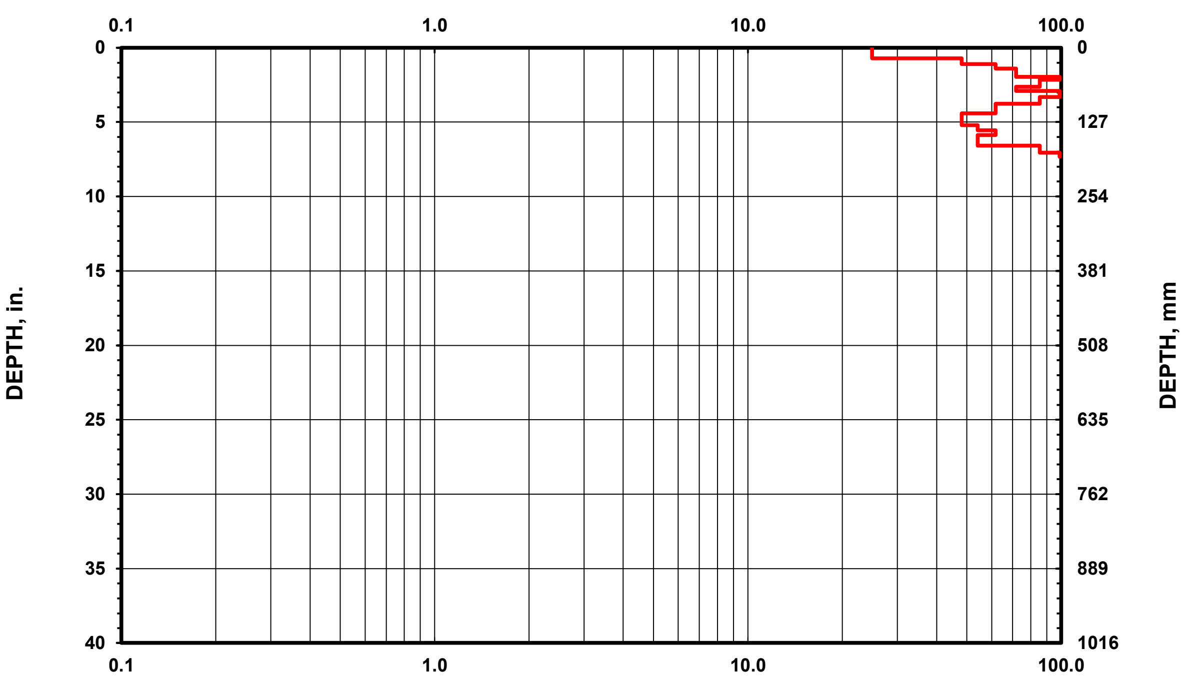
Soil Type
◉ All other soils

Crew: PPP

Hammer
○ 10.1 lbs.
◉ 17.6 lbs.

Office of Geotechnical Engineering
Geology, Exploration, and Laboratory Section

ID: D-005-0-21
Ground Elev. (ft): 769.2
Start Depth (ft:): 2.67
Latitude: 39.779903
Longitude: 84.159825



Ground Elev.	Depth (Ft)	Material Encountered
769.2	1.58	Asphalt (19-in)
	2.33	Concrete (9-in)
		Sand

Notes: Completed in eastbound passing lane

DCP TEST DATA

OHIO DEPARTMENT OF TRANSPORTATION
OFFICE OF GEOTECHNICAL ENGINEERING

Project: MOT-4-1993
PID: 115661
Date: 9/15/2021

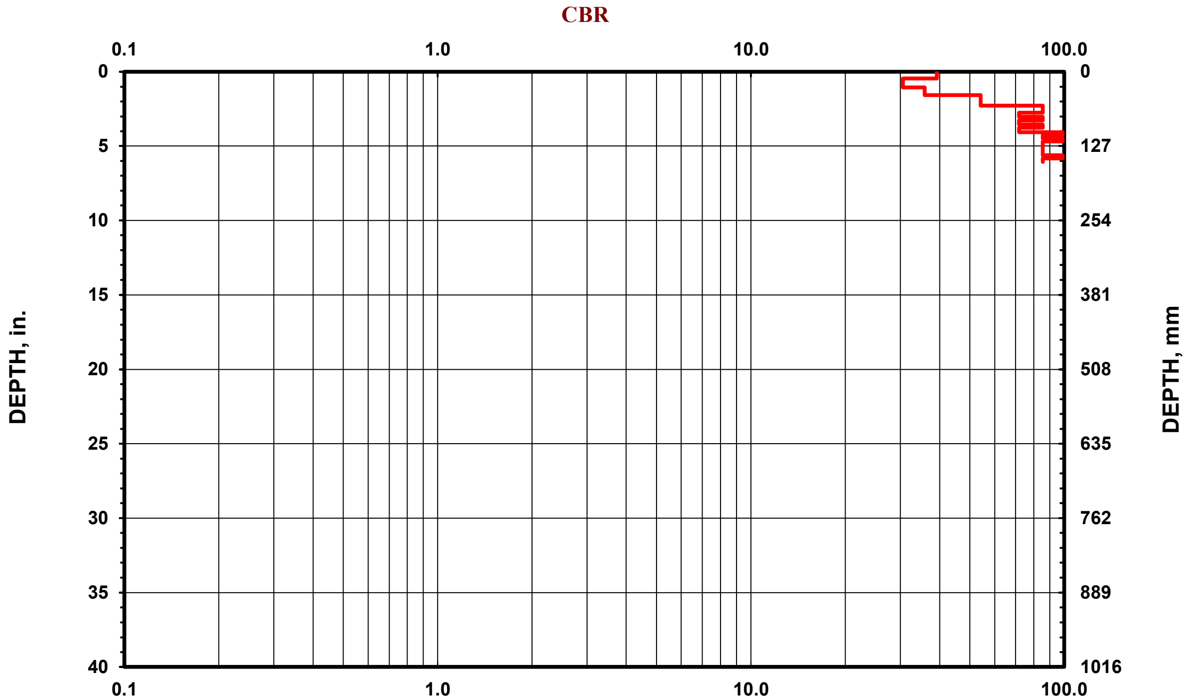
Soil Type
(●) All other soils

Crew: PPP

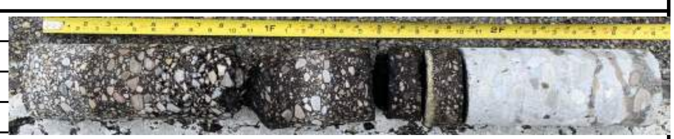
Hammer
( ) 10.1 lbs.
(●) 17.6 lbs.

Office of Geotechnical Engineering
Geology, Exploration, and Laboratory Section

ID: D-006-0-21
Ground Elev. (ft): 767.7
Start Depth (ft:): 2.75
Latitude: 39.780050
Longitude: 84.159459



Ground Elev.	Depth (Ft)	Material Encountered
767.7	2.00	Asphalt (24-in)
	2.75	Concrete (9-in)
		Sand



Notes: Completed in eastbound travel lane

DCP TEST DATA

OHIO DEPARTMENT OF TRANSPORTATION
OFFICE OF GEOTECHNICAL ENGINEERING

Project: MOT-4-1993
PID: 115661
Date: 9/15/2021

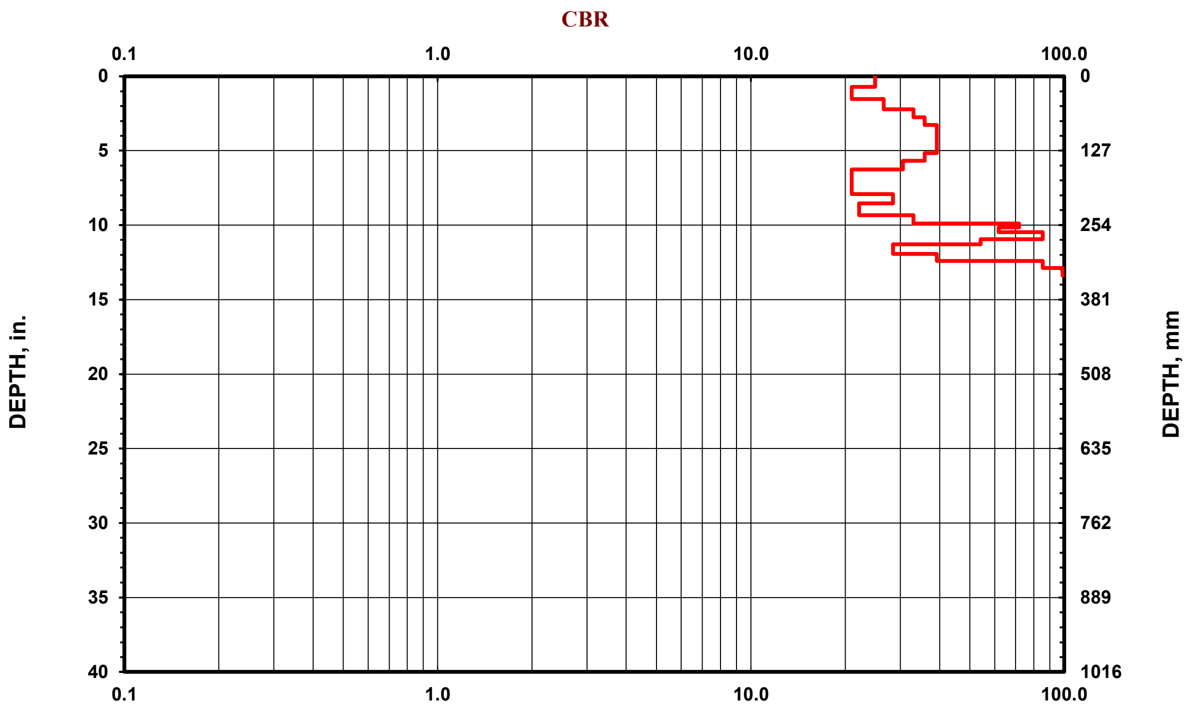
Soil Type
(◉) All other soils

Crew: PPP

Hammer
( ) 10.1 lbs.
(◉) 17.6 lbs.

Office of Geotechnical Engineering
Geology, Exploration, and Laboratory Section

ID: D-007-0-21
Ground Elev. (ft): 769.2
Start Depth (ft:): 1.67
Latitude: 39.780244
Longitude: 84.159594



Ground Elev.	Depth (Ft)	Material Encountered
767.7	0.75	Asphalt (9-in)
	1.50	Concrete (9-in)
	1.67	Void (2-in)
		Sand

Notes: Completed in westbound passing lane.

MOT-4-19.73 (ODOT, 1958) – Construction Drawings

FED. RD. DIVISION	STATE	PROJECT		
2	OHIO			

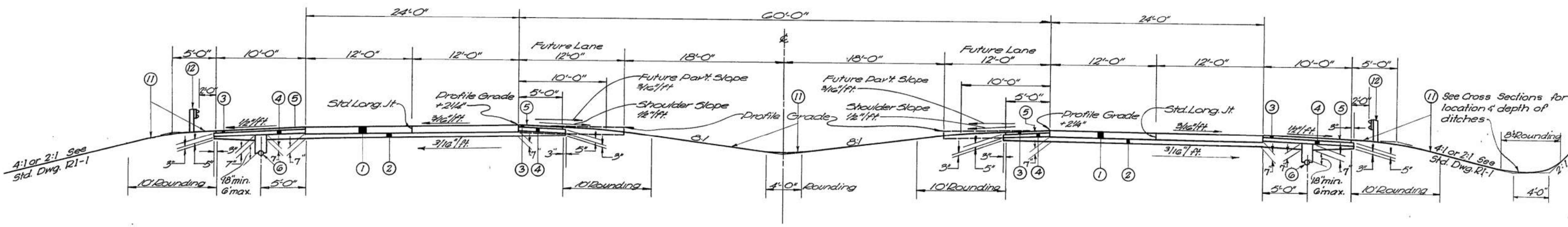
MOT-4-19.92

2/143

TYPICAL SECTIONS

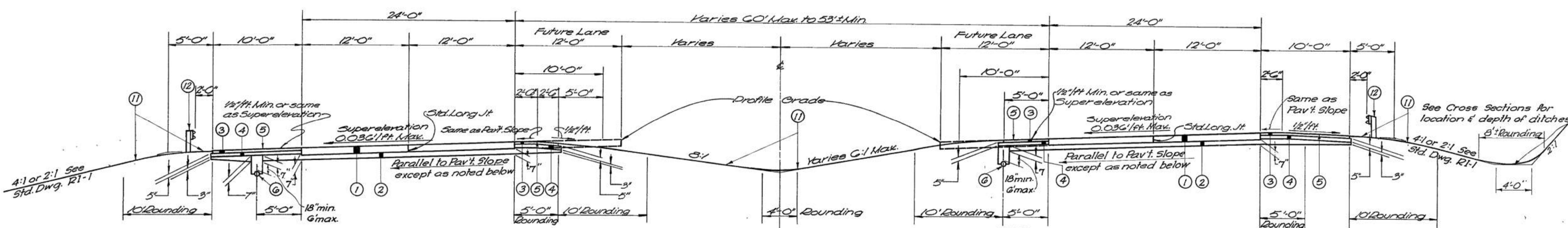
TYPE T-71

SCALE: 3/16"= 1'-0"



NORMAL SECTION

Sta. 97+96.21 to Sta. 112+65.30



NOTE:
In Superelevation transition sections, the subbase shall slope down and away from the ℄ at 3/16"/ft. until the pavement slope reaches 3/16"/ft. down and toward the ℄, at which place the subbase shall be constructed parallel to the pavement slope. (See cross sections)
Backslopes on all ditches shall be 2:1 unless otherwise indicated on the Plans or Cross Sections

SUPERELEVATED SECTION

Sta. 84+35.16 to Sta. 97+96.21

1. Item T-71 — 9" Reinforced Portland Cement Concrete Pavement
2. Item I-22 — 6" Subbase (except where otherwise noted)
3. Item B-219 — 3" Waterproofed Aggregate Base Course, Type "B"
4. Item I-18 — 5" Stabilized Crushed Aggregate Shoulder Material
5. Item T-31 — Bituminous Surface Treatment, See Proposal Note*
6. Item I-4 — 6" Underdrain
7. Item I-12 — Special Barrier Curb, as per plan
8. Item I-12 — Standard Type 6 Curb
9. Item I-12 — Standard Type 2A Curb
10. Item I-21 — 4" Portland Cement Concrete Median Pavement, Std. Type 1
11. Item L-9 — Seeding and Protecting
12. Item I-15 — Standard Type 2B Guard Rail (for Fills 10' and over)
13. Item I-12 — Standard Type 2B Curb
14. Item T-35 — 1½" Asphaltic Concrete Surface Course, Type "C" (70-85)
15. Item B-35 — 1½" Asphaltic Concrete Leveling Course (70-85)
16. Item B-35 — 3" Max. Asphaltic Concrete Base Course (70-85)
17. Item I-13 — 4" Plain Portland Cement Concrete Sidewalk
18. Item B-70 — 9" Plain Portland Cement Concrete Base Course, as per plan
19. Item I-23 — Standard Precast Traffic Divider

* Using 0.008 Cu. Yd. No. 6 Aggregate and 0.25 Gal. Bituminous Material per Sq. Yd

20. Item T-30 — Bituminous Tack Coat: Sec. M-5.5, MS-2 or RS-1 or Sec. M-5.2 RC-1 or RC-2, as per Sec. T-30.02 @ 0.10 gal. per sq. yd.

GENERAL NOTES

RANDOM FILL AREA

Between Sta. 72 + 00± and Sta. 96 + 00±, the Proposed Project passes over an area of active and abandoned dumps. All such areas between the construction limits (except as noted below) shall be thoroughly compacted with a 50 ton roller in accordance with the requirements of the Proposal Note dealing with "Fill Compaction Using Heavy Pneumatic Tired Rollers". A sufficient quantity of E-101.02 granular material modified as described below shall be provided to aid in leveling up the area as the rolling progresses. An amount estimated to be sufficient to form a one (1) foot compacted depth layer has been provided in the quantities.

Ran dom dump material from Sta. 81 + 25± to Sta. 82 + 60± shall be totally excavated to its full depth between the outside berm lines of S. R. 4. The cut slopes required to excavate this random material shall not be steeper than 1:1. The limits of excavation shown on the plans and cross sections are approximate only; the intent is to remove only ran dom dump material. Stable, natural foundation material other than that which must be removed to safely excavate the waste shall be left in place. Any material excavated from this area that will not meet the requirements of normal embankment materials shall be used outside the 1:1 slope lines shown on the cross sections in the pond areas to reduce the Special Borrow, Granular Material Sec. E-101.02 Modified as set forth below, provided that the said material is deposited to an elevation not to exceed the water surface of the existing ponds. See note for "Construction of Embankment in Existing Water Filled Gravel Pits".
The above excavation shall be paid for at the unit price bid for Item E-101, Roadway Excavation.

After random material has been excavated, the area shall be backfilled in accordance with Item E-101.

CONSTRUCTION OF EMBANKMENT IN EXISTING WATER FILLED GRAVEL PITS

Fill in these areas shall be constructed by end dumping granular material meeting the requirements of Section E-101.02, Granular Material, modified as follows: at least 75% of the grains or particles shall be retained on a No. 200 sieve.

The granular material shall be carried to an elevation two (2) feet above the surface of the existing water level. Prior to completing these fills, that portion which was placed by end dumping shall be rolled with the 50 ton roller in accordance with the requirements of the Proposal Note dealing with "Fill Compaction Using Heavy Pneumatic Tired Rollers".

Any excavated random dump material that is not satisfactory for normal embankment use shall be used to reduce the granular borrow when so ordered by the Engineer. If so used, it shall be placed outside the limits of the 1:1 slope lines shown on the cross sections and to an elevation not to exceed the elevation of the existing water surface.

Above an elevation two feet above water level, embankment construction in these areas shall be in accordance with Sec. E-101.08.

SUPERELEVATION

Superelevated curves shall be built without crown. The crown shall be worked out of the pavement in accordance with the elevations shown on the Pavement Details or in the Superelevation Tables.

EXPANSION AND CONTRACTION JOINTS

Where transverse joints are located closer than ten (10) feet to the regular blockout joints around catch basins or manholes, the blockout joints shall be continued to the transverse joint. Similarly, where longitudinal joints are located closer than two (2) feet to the regular blockout joints around catch basins or manholes, the blockout joints shall be continued to the longitudinal joints.

Although certain expansion and contraction joints have been detailed on this plan, no waiver of the specifications is intended. Standard expansion joints shall be provided at the major structure and the maximum spacing between contraction joints shall in all cases be in accordance with Standard Drawing T.J.

SANITARY CONNECTIONS PASSING OVER WATER MAINS

Any proposed house connection or lateral sewer, excluding inlet connections, which must of necessity pass over a proposed water main shall be encased in a minimum of six (6) inches of Class "E" Concrete for a distance of ten (10) feet on each side of the centerline of the water main. Payment for this work shall be included in the unit price bid for the pertinent pipe item.

PROPOSED HOUSE CONNECTIONS

The State will notify property owners in advance of construction that if they contemplate new house connections to the proposed sewers, the property owner must furnish, at his sole cost, tees or wyes of the proper size and material to the Contractor. The Contractor will then install the tees or wyes as he proceeds with laying the sewer and payment for the work involved will be at the same rate as though he were furnishing and laying straight pipe.

To obtain a house connection to either an existing sewer that is to remain or to a proposed sewer, the property owner or his agent, at his sole cost, shall furnish all material and labor required to install the house connection from the carrier sewer to a point beyond the limits of roadway construction. The property owner must display a City of Dayton Sewer Permit before the Contractor may install the tees or wyes.

COOPERATION BETWEEN CONTRACTOR AND PROPERTY OWNERS

The Contractor must cooperate with the property owner or his agent to give said property owner or his agent ample opportunity for extending said sewer connection from the tee branch or existing sewer to a point beyond the roadway construction limits. The necessary house connections shall be installed by the property owner or his agent at no cost to the Contractor, other than the cost of cooperation in scheduling his work, which cost shall be assumed by the Contractor and shall be included in the unit prices bid for various sewer items.

SEWER HOUSE DRAINS (EXISTING HOUSE CONNECTIONS)

All existing house drains, which include sanitary, yard, roof, basement or other similar house drains now in use which are disturbed because of the Highway improvement, shall be replaced by the Contractor. If the existing sewer is to be abandoned, then a satisfactory house connection shall be provided to the new sewer. Where an existing house is to be removed, the upgrade end of the existing house connection shall be plugged with a precast vitrified or concrete stopper, and accurately referenced if the existing house connection remains satisfactory for future use. Estimated quantities of 6" and 8" Class B Storm Sewers have been included in the General Summary, and all the above work, except plugging, shall be included with and paid for at the unit prices bid for the pipe items actually furnished and placed. Payment for plugging specified shall be included in the unit price bid for Item E-101, Roadway Excavation.

REMOVAL OF EXISTING HOUSE DRAINS

The removal of all existing house connections, which includes sanitary, yard, roof, basement or other similar pipe drains within the roadway construction limits shall be classified and paid for as Item E-101, Roadway Excavation, unless otherwise itemized for payment in the plans.

EXISTING SANITARY DRAINS OR SEWERS

Sanitary drains or sewers, which include leaching bed outlets, cellar drains, sink drains or polluted water of any kind, shall not be connected to the highway drainage system, either pipes or ditches. Any such drains encountered shall be plugged with Class "E" concrete at the Right-of-Way line. Payment for plugging shall be included in the unit price bid for Item E-101, Roadway Excavation.

CONNECTIONS TO EXISTING SEWERS

At places where the plans provide for proposed drainage pipe to be connected to existing pipes, it shall be the responsibility of the Contractor to locate the existing pipe both as to line and grade before he starts to lay the proposed sewer. The cost of this operation shall be included in the unit price bid for the pertinent Pipe Item.

UNRECORDED CONNECTIONS

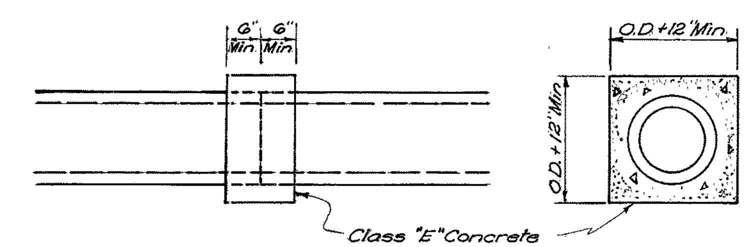
Any unrecorded active connections to a sewer through existing catch basins or manholes to be abandoned encountered during construction shall be reconnected to the sewer as the Engineer may direct. Payment for this work shall be included in the unit price bid for Item I-16.

LOCATION AND SIZE OF PIPES

The location, type, depth and size of all existing pipes are shown as near exact as the available information will permit. The State will not be responsible for any variations found during construction.

PIPE CULVERT

When bell and spigot pipe is used, any necessary pipe cut-offs will be made at the spigot end of the length of pipe adjacent to the end length. When tongue and groove pipe is used, the length of pipe next to the end length shall be cut and butt joint formed with a collar as shown. The cost of the joint and collar shall be included in the Contract Unit Price bid for the pertinent pipe item.



ITEM I-15 TEMPORARY GUARD RAIL

See note in Proposal for description of this item. An estimated quantity of 200 Lin. Ft. has been carried to the General Summary for use where directed by the Engineer for traffic control at ramp stubs on the west side of the Stanley Avenue extension or for other uses deemed essential by the Engineer.

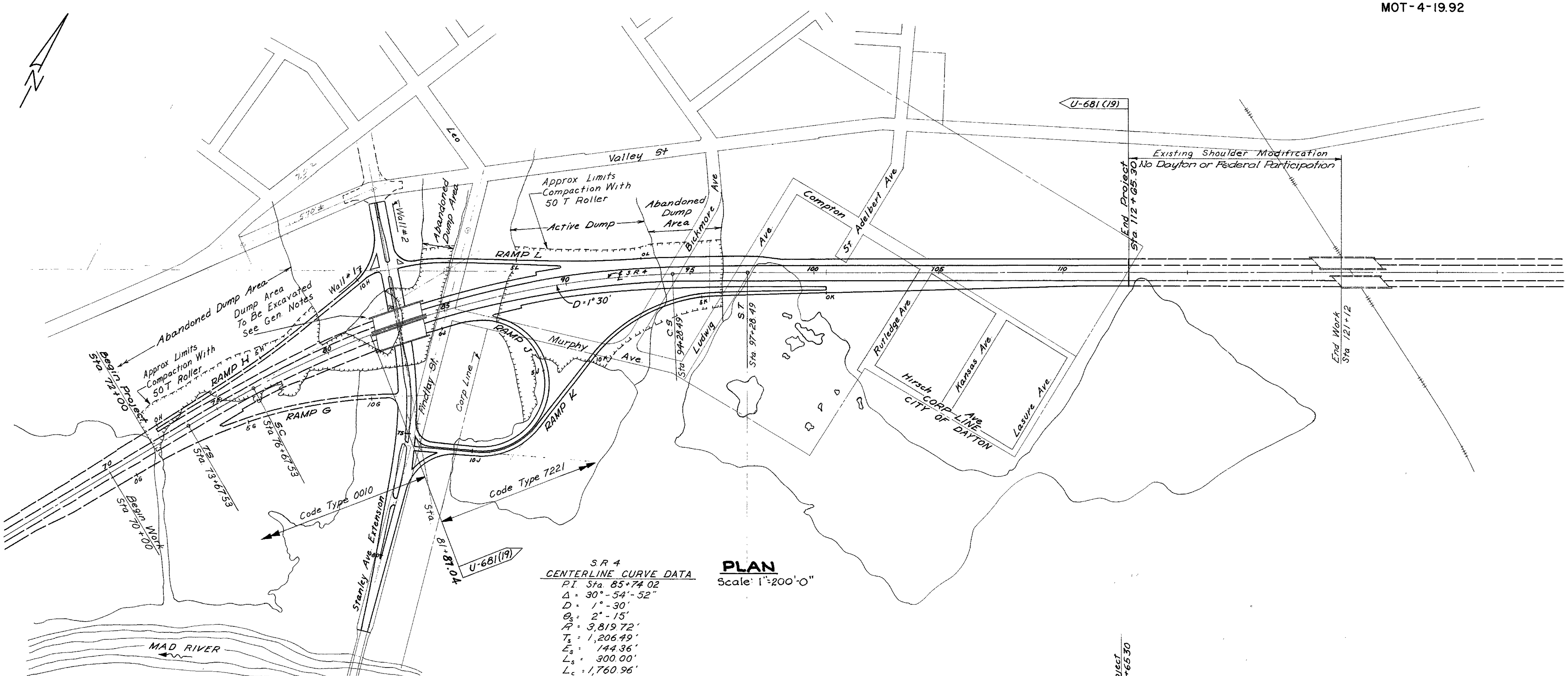
ESTIMATED QUANTITIES

Specific locations and usage of estimated quantities set up on this plan to be used "as directed by the Engineer" shall be made a matter of record by incorporation into the final change order governing completion of this project.

FED. RD. DIVISION	STATE	PROJECT		
2	OHIO			

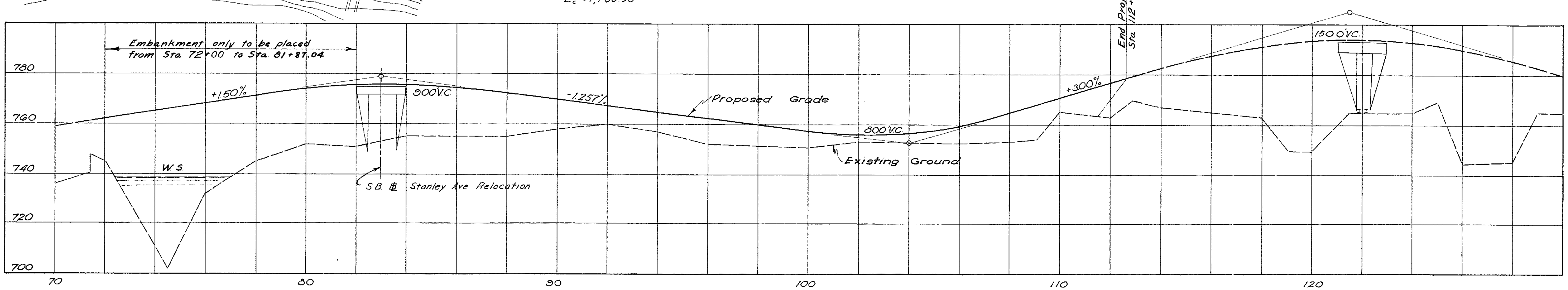
20/143

MOT-4-19.92



PLAN

Scale: 1"=200'-0"



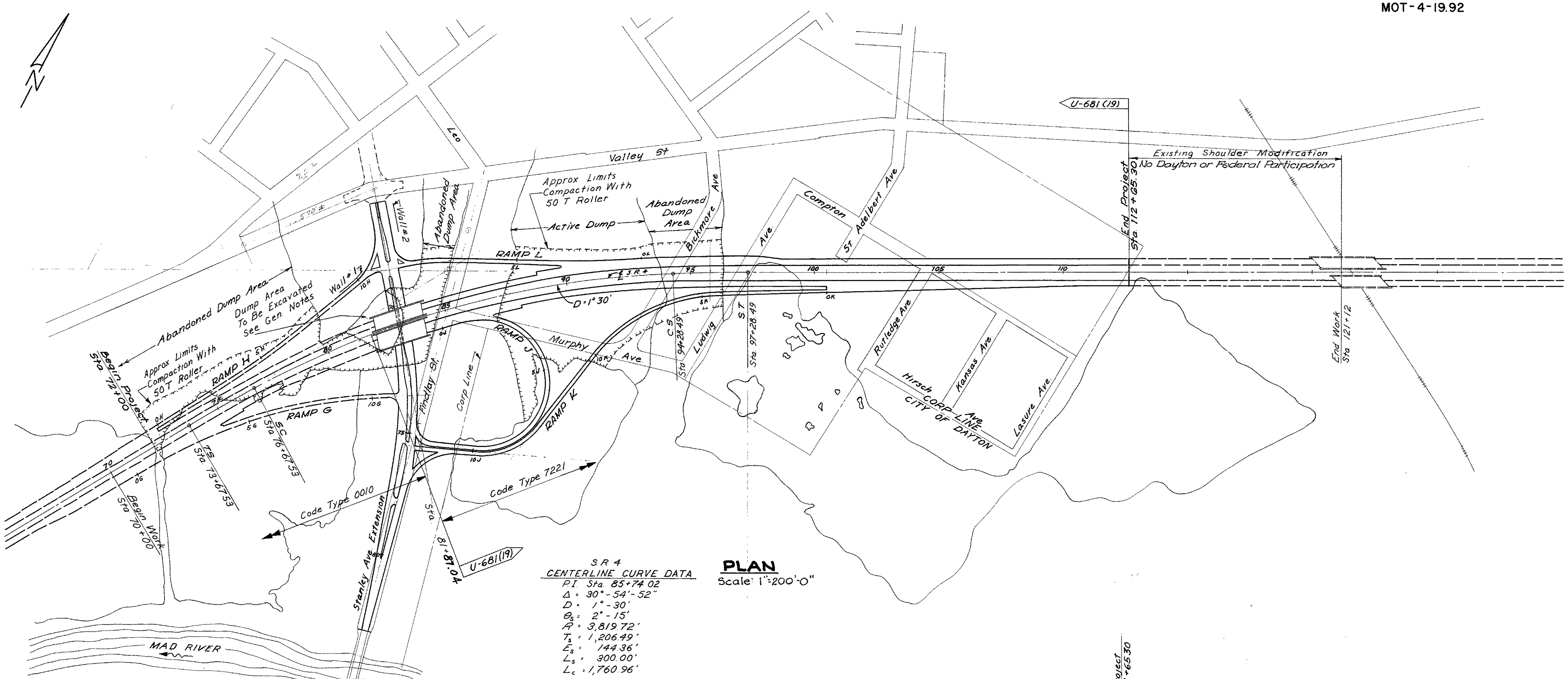
PROFILE

Scale: Horizontal- 1"=200'
Vertical- 1"=20'

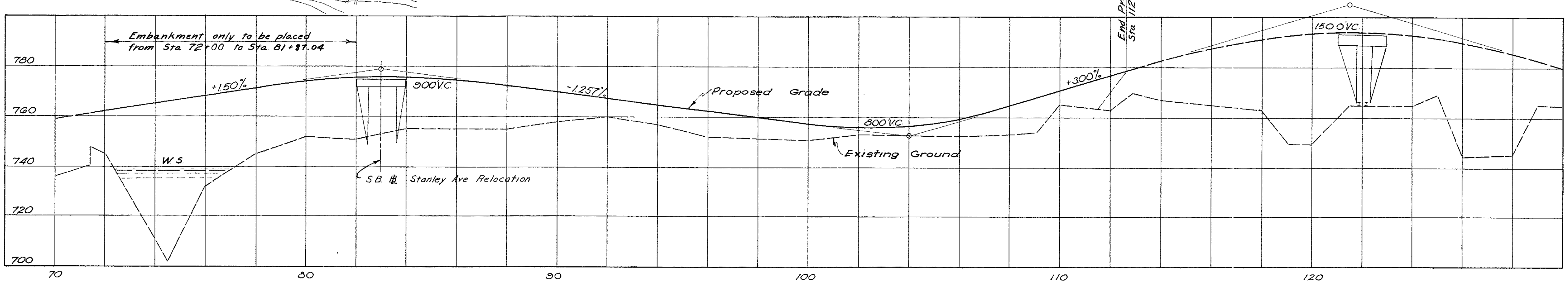
FED. RD. DIVISION	STATE	PROJECT		
2	OHIO			

20/143

MOT-4-19.92



PLAN
Scale: 1"=200'-0"



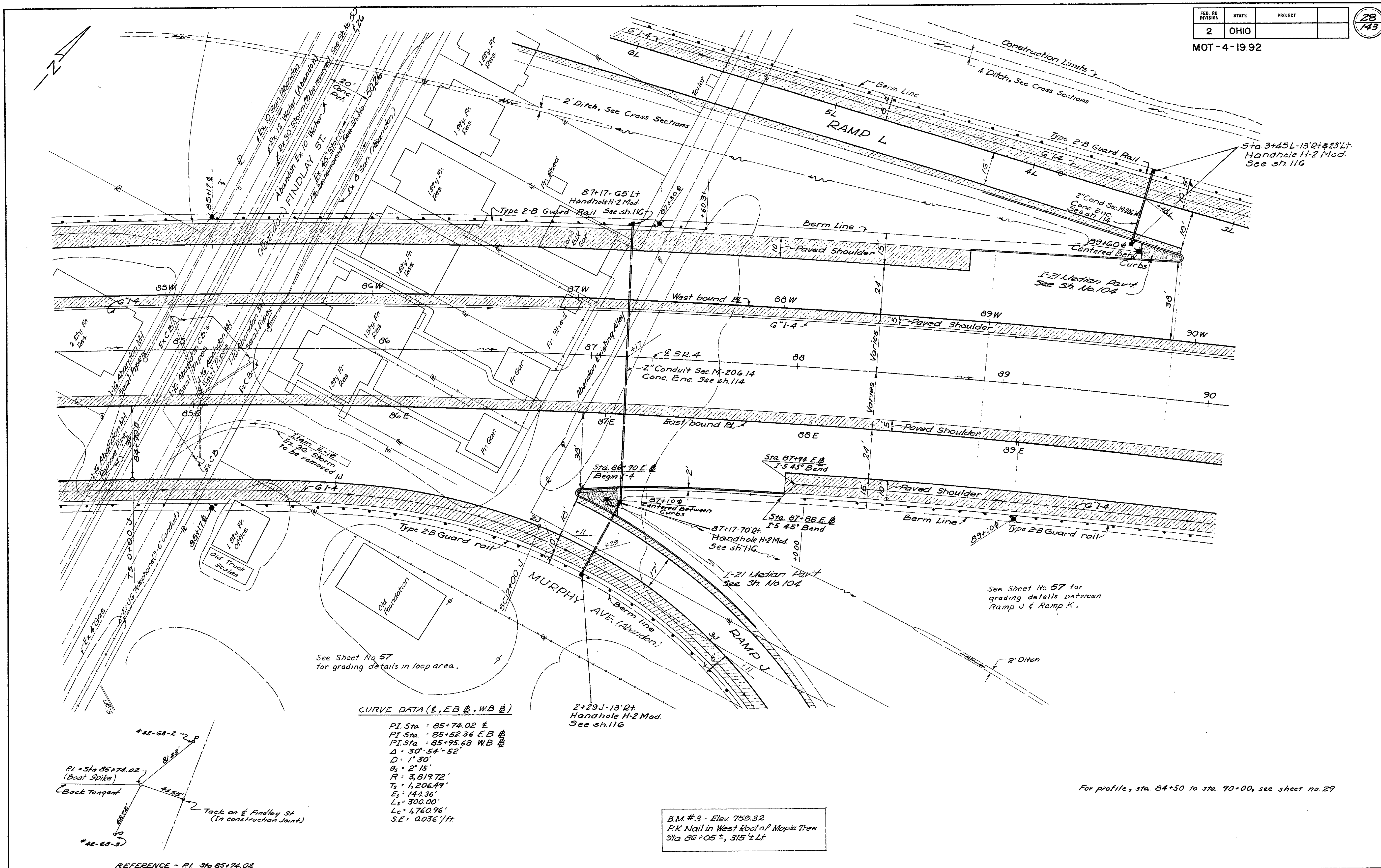
PROFILE
Scale: Horizontal- 1"=200'
Vertical- 1"=20'

SCHEMATIC PLAN

FED. RD DIVISION	STATE	PROJECT	
2	OHIO		

28/143

MOT-4-19.92



CURVE DATA (℄, E.B. ℄, W.B. ℄)

P.I. Sta. = 85+74.02 ℄
P.I. Sta. = 85+52.36 E.B. ℄
P.I. Sta. = 85+95.68 W.B. ℄
Δ = 30°-54'-52"
D = 1°30'
θ_s = 2°15'
R = 3,819.72'
T_s = 1,206.49'
E_s = 144.36'
L_s = 300.00'
L_c = 1,760.96'
S.E. = 0.036'/ft.

P.I. = Sta. 85+74.02 (Boat Spike)
Back Tangent
#42-68-2
#42-68-3
Tack on ℄ Findlay St. (In construction Joint)

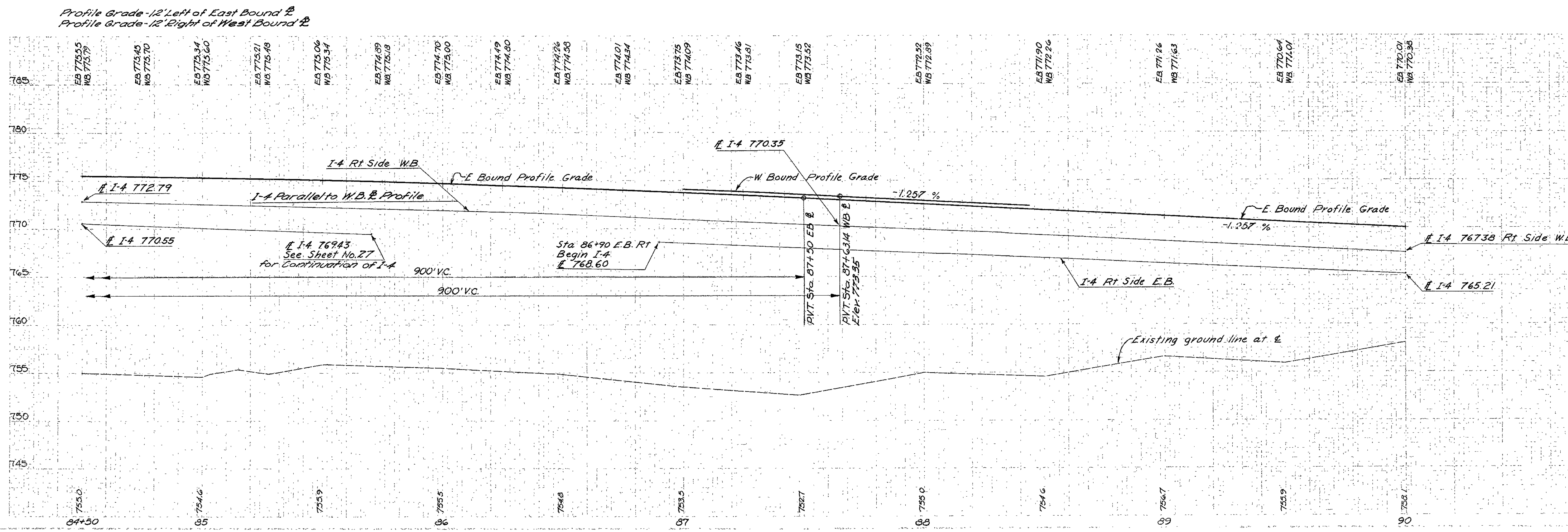
REFERENCE - P.I. Sta. 85+74.02

B.M. #3 - Elev. 759.32
P.K. Nail in West Root of Maple Tree
Sta. 86+05±, 315'± Lt.

For profile, sta. 84+50 to sta. 90+00, see sheet no. 29

STA. 84+50 to STA. 90+00

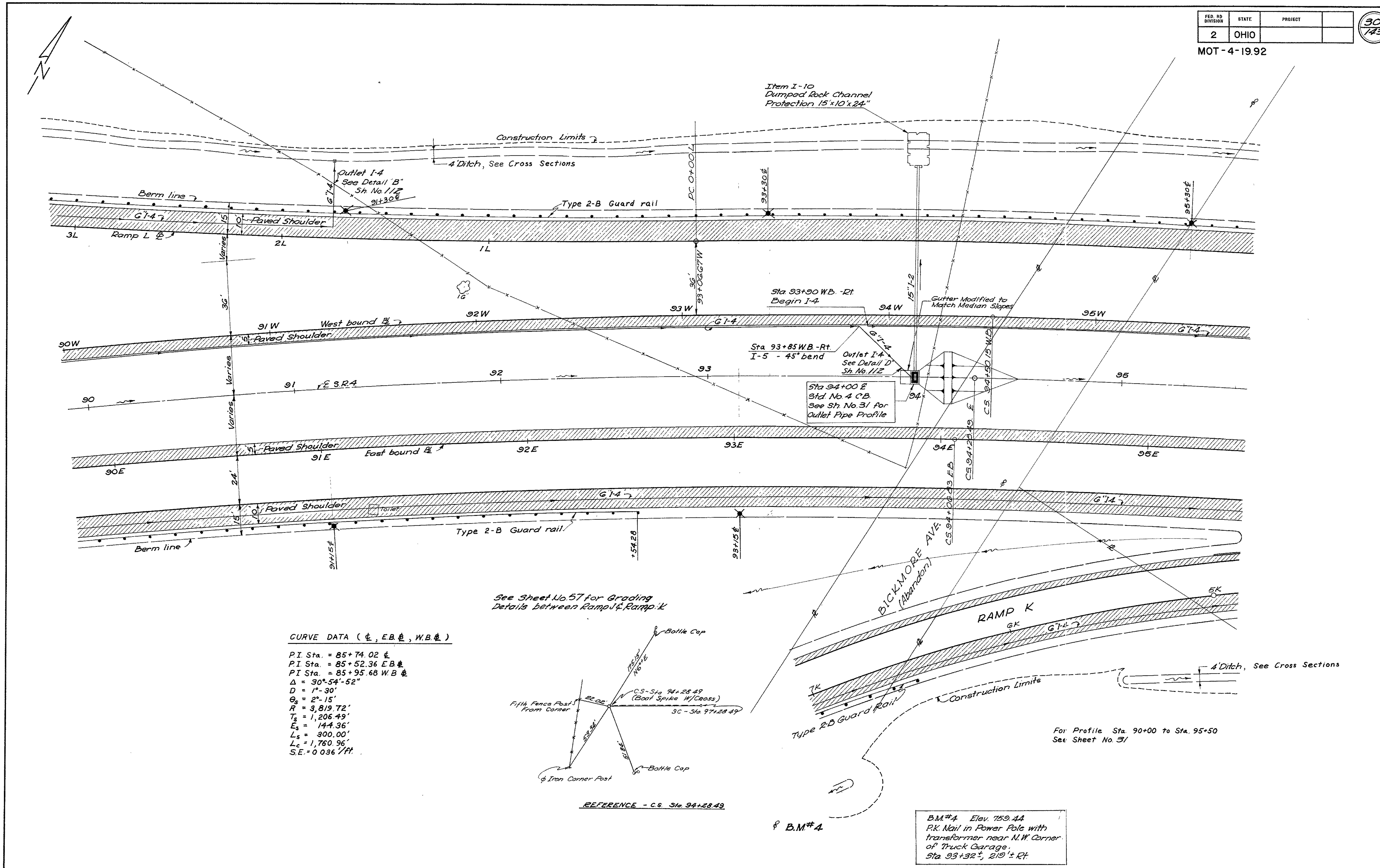
MOT-4-19.92



STA. 84+50 to STA. 90+00

FED. RD. DIVISION	STATE	PROJECT	
2	OHIO		

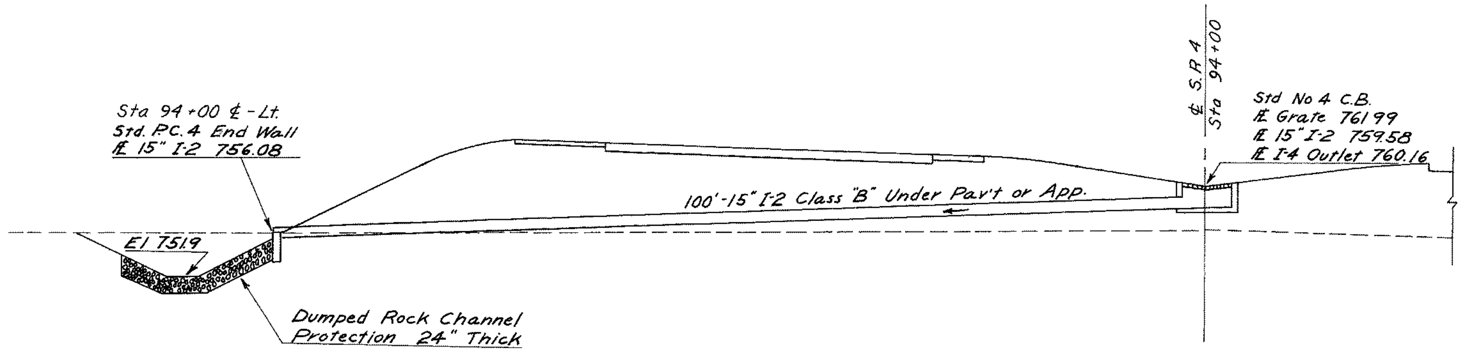
MOT-4-19.92



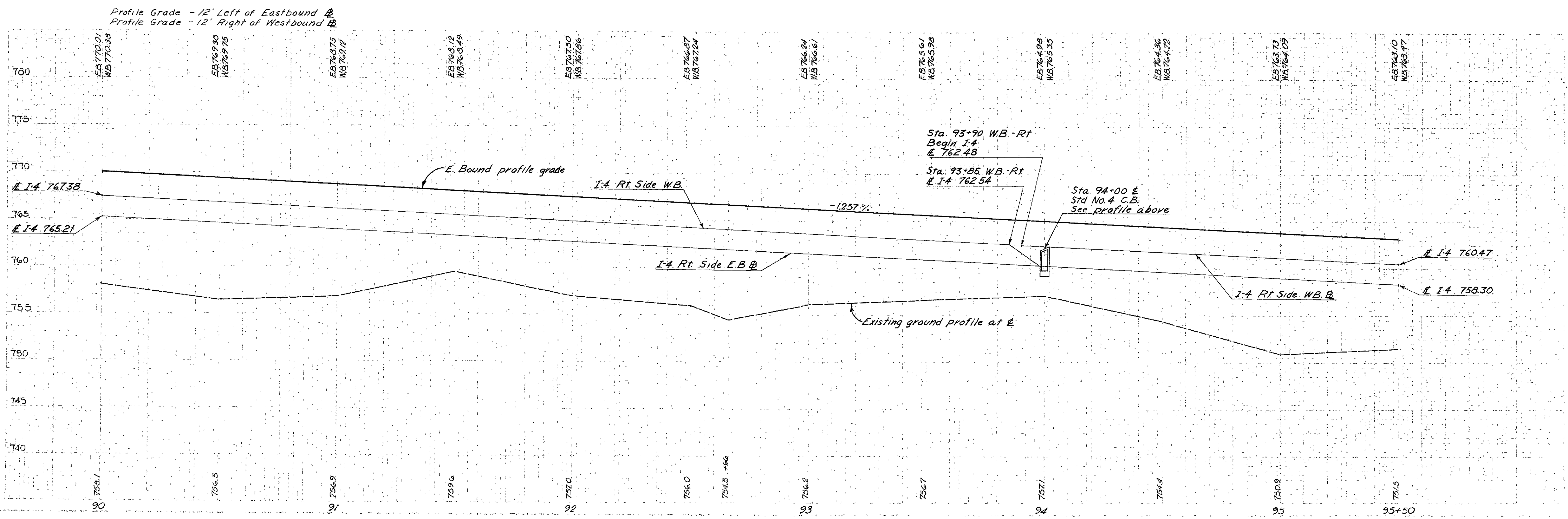
CURVE DATA (℄, E.B.℄, W.B.℄)

P.I. Sta. = 85+74.02 ℄
P.I. Sta. = 85+52.36 E.B.℄
P.I. Sta. = 85+95.68 W.B.℄
Δ = 30°-54'-52"
D = 1°-30'
θ_s = 2°-15'
R = 3,819.72'
T_s = 1,206.49'
E_s = 144.36'
L_s = 300.00'
L_c = 1,760.96'
S.E. = 0.036 '/ft.

B.M.#4 Elev. 759.44
P.K. Nail in Power Pole with
transformer near N.W. Corner
of Truck Garage.
Sta. 93+32±, 219'± Rt.

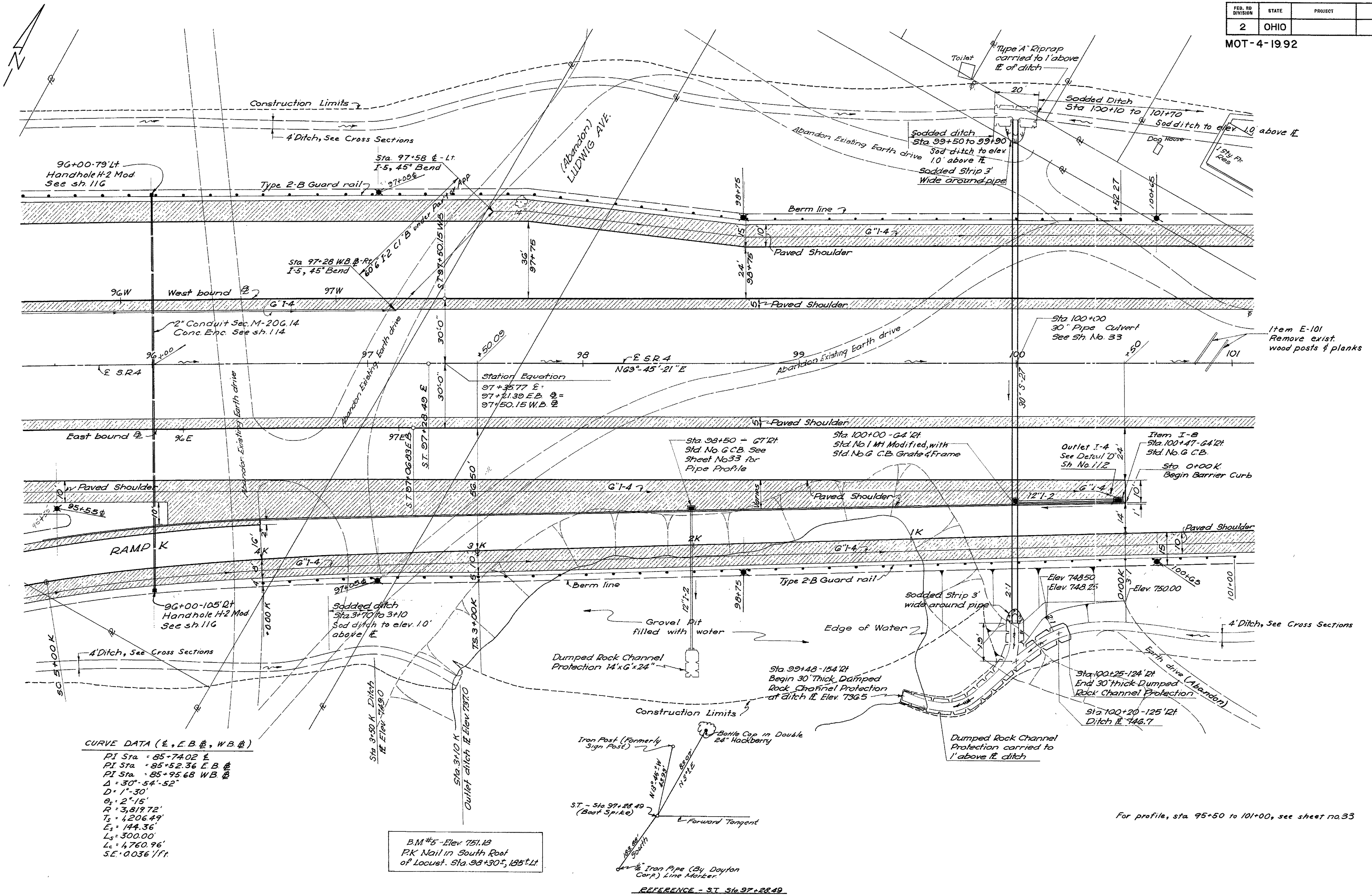


I-2 PROFILE
Sta. 94+00 ℄
Scale 1"=10'



FED. RD DIVISION	STATE	PROJECT		SHEET
2	OHIO			32/143

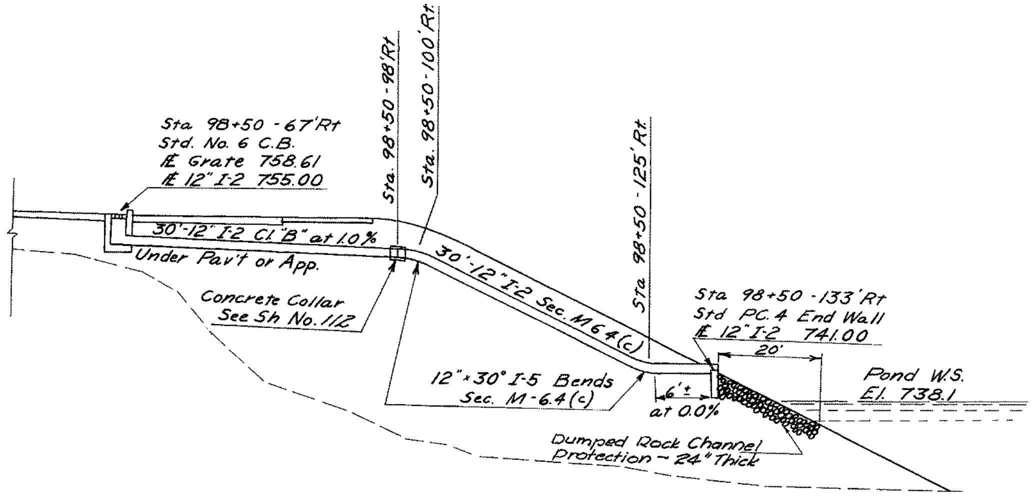
MOT-4-19.92



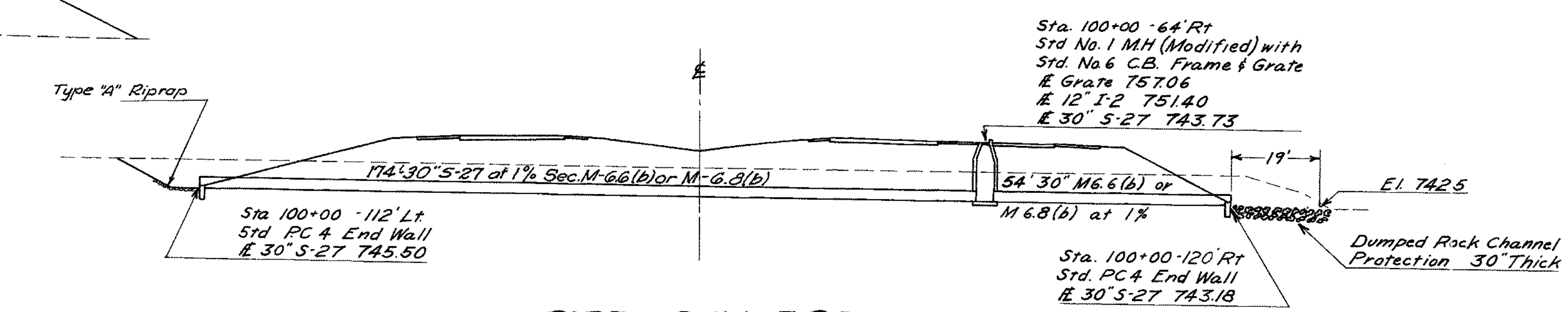
CURVE DATA (℄, E.B. ℄, W.B. ℄)
PI Sta. = 85+74.02 ℄
PI Sta. = 85+52.36 E.B. ℄
PI Sta. = 85+95.68 W.B. ℄
Δ = 30°-54'-52"
D = 1°-30'
θ_s = 2°-15'
R = 3,819.72'
T_s = 1,206.49'
E_s = 144.36'
L_s = 300.00'
L_c = 1,760.96'
S.E. = 0.036'/ft.

B.M. #5 - Elev. 751.18
P.K. Nail in South Root of Locust. Sta. 98+30±, 185'± Lt.

For profile, sta. 95+50 to 101+00, see sheet no. 33



1-2 PROFILE
Sta. 98+50
Scale 1"=10'



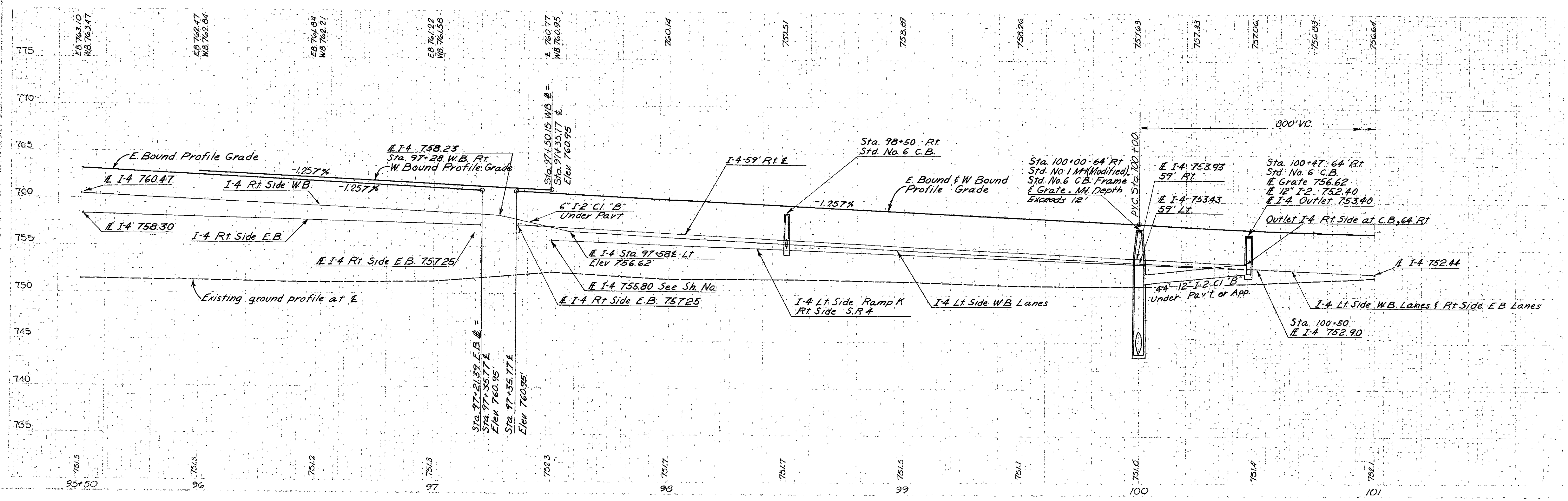
PIPE CULVERT
Sta. 100+00
Scale 1"=20'

Drainage Area = 25.8 Acres

Q_{50} = 38 c.f.s

Estimate of Quantities

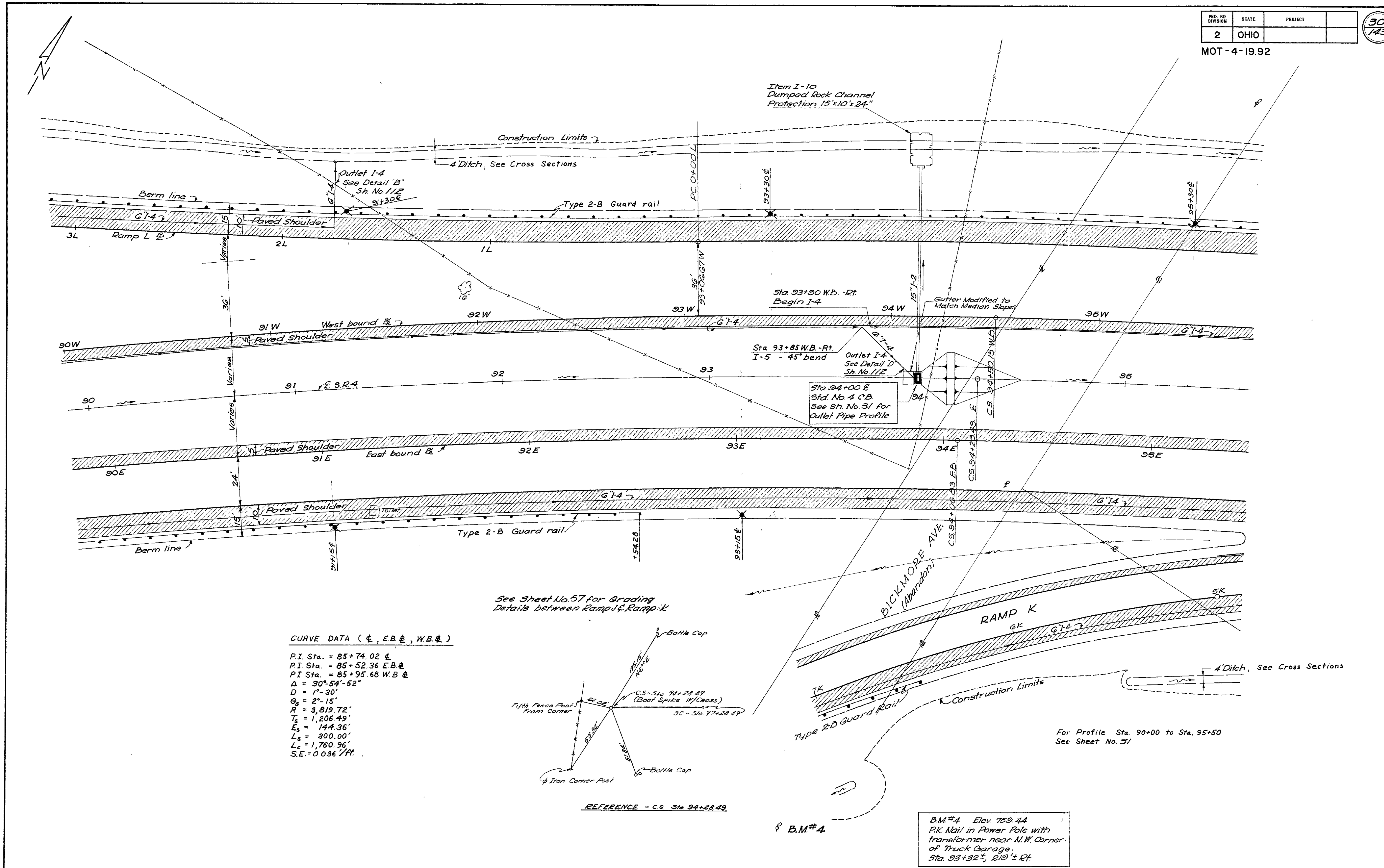
Description	Item	Quantity
Excavation for Structures	E-2	266 cu. yd.
Type "A" Riprap	I-10	22 sq. yd.
Dumped Rock Channel Protection	I-10	82 cu. yd.
Sodding	L-10	9 sq. yd.
Concrete for Structures - Class "E"	S-1	1.02 cu. yd.
Pipe for Roadway Culverts 30"	S-27	228 lin. ft.
Std No. 1 M.H. (Modified)	I-8	1 ea.



FED. RD DIVISION	STATE	PROJECT	
2	OHIO		

30/143

MOT-4-19.92

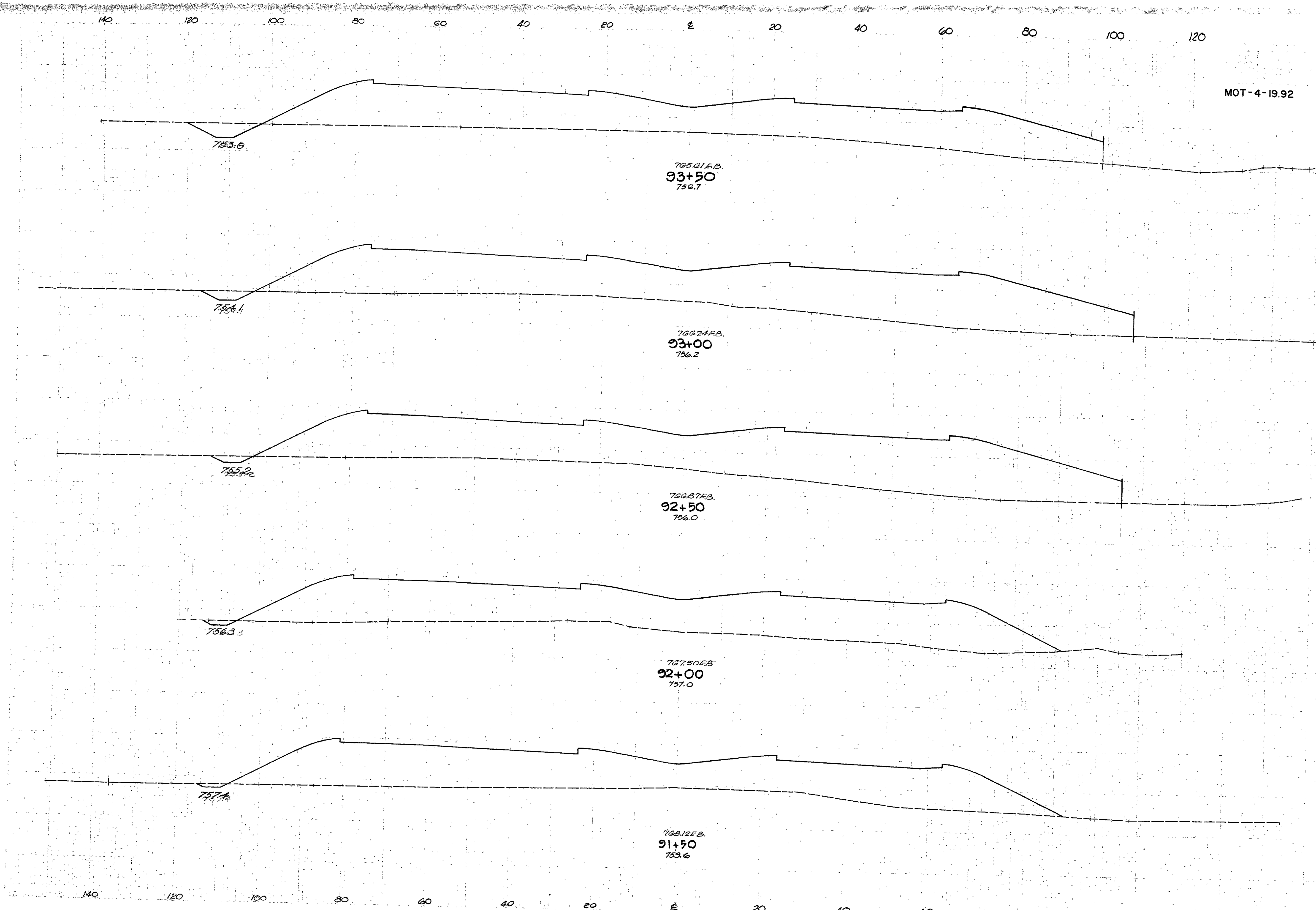


CURVE DATA (¢, E.B. ℄, W.B. ℄)

P.I. Sta. = 85+74.02 ¢
P.I. Sta. = 85+52.36 E.B. ℄
P.I. Sta. = 85+95.68 W.B. ℄
Δ = 30°-54'-52"
D = 1°-30'
θ_s = 2°-15'
R = 3,819.72'
T_s = 1,206.49'
E_s = 144.36'
L_s = 300.00'
L_c = 1,760.96'
S.E. = 0.036 '/ft.

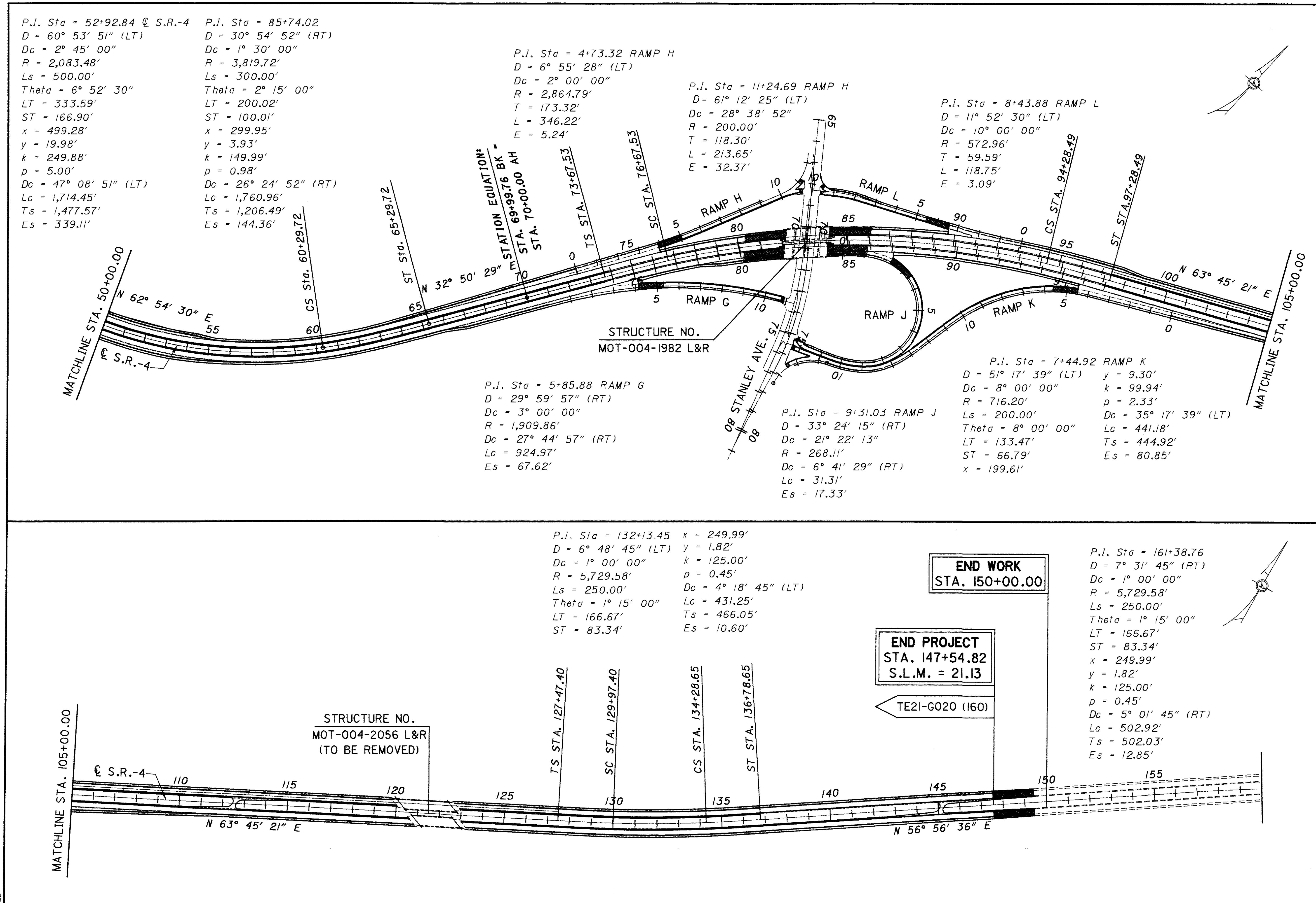
B.M. #4 Elev. 759.44
P.K. Nail in Power Pole with transformer near N.W. Corner of Truck Garage.
Sta. 93+32±, 219'± Rt.

MOT-4-19.92



END AREA CUT	END AREA FILL	CU. YDS. CUT	CU. YDS. FILL
32	1690		
		47	3441
19	2030		
		29	3802
12	2080		
		16	3534
5	1740		
		7	3151
3	1665		
		7	3437

MOT-4-19.14 (ODOT, 2003) – Construction Drawings



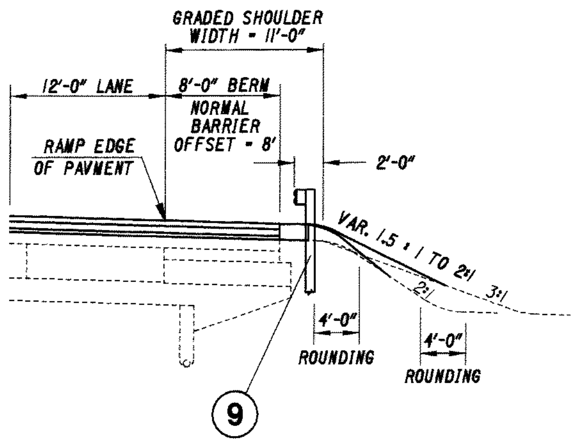
0 100 200 400
HORIZONTAL SCALE IN FEET

CALCULATED
CHECKED

SCHEMATIC PLAN FROM STA. 50+00.00 TO STA. 160+00

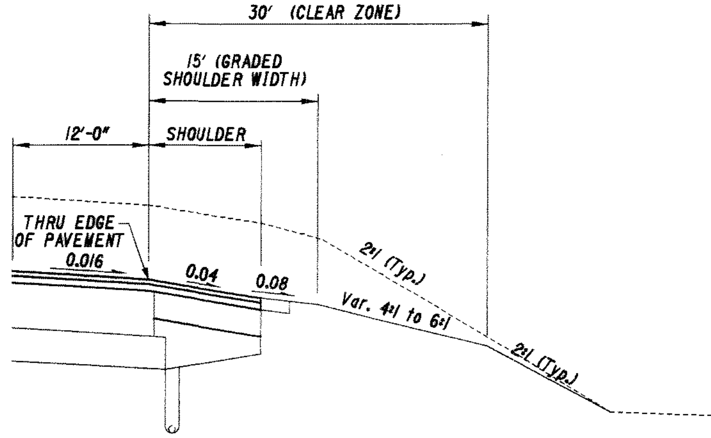
MOT-4-19.14

3/585



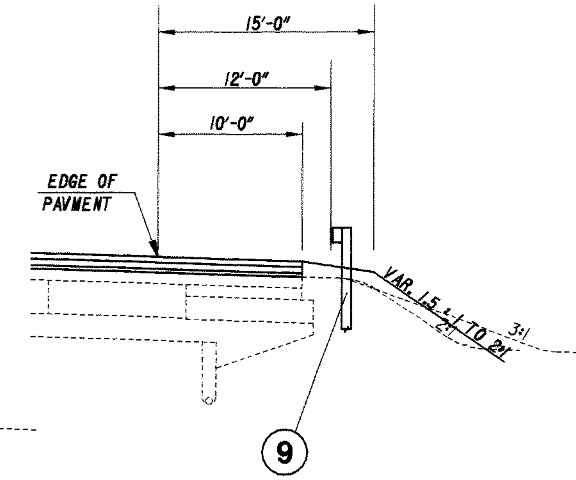
RAMP TYPICAL PLACEMENT OF GUARDRAIL AND BARRIER GRADING

OPPOSITE HAND ALSO, SEE CROSS SECTIONS

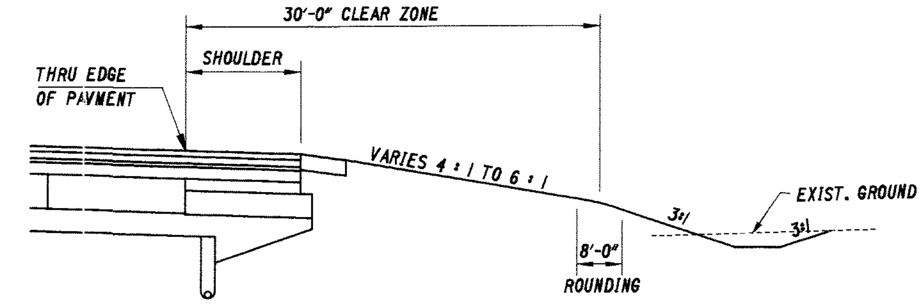


SAFETY GRADING FILL SECTION

OPPOSITE HAND ALSO, SEE CROSS SECTIONS

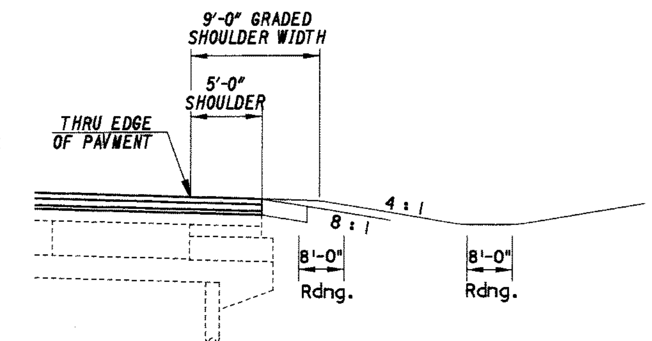


MAINLINE TYPICAL PLACEMENT OF GUARDRAIL



SAFETY GRADING MEDIUM FILL SECTION

OPPOSITE HAND ALSO, SEE CROSS SECTIONS



MEDIAN GRADING SECTION

OPPOSITE HAND ALSO, SEE CROSS SECTIONS

Pavement Legend

1. ITEM 858 - 1½" ASPHALT CONCRETE, SURFACE COURSE, 12.5mm, TYPE B (446), AS PER PLAN w/1059 WARRANTY
2. ITEM 858 - 1¾" ASPHALT CONCRETE, INTERMEDIATE COURSE, 19mm, TYPE B (446), AS PER PLAN

2A. ITEM 858 - 1½" ASPHALT CONCRETE, INTERMEDIATE COURSE, 19mm, TYPE B (446), AS PER PLAN

3. ITEM 858 - 0" TO 1" ASPHALT CONCRETE, INTERMEDIATE COURSE, 9.5mm, TYPE B (448), AS PER PLAN

3A. ITEM 858 - VARIABLE DEPTH ASPHALT CONCRETE, INTERMEDIATE COURSE, 19mm, TYPE B (448), AS PER PLAN

4. ITEM 301 - 7½" BITUMINOUS AGGREGATE BASE, PG 64-22

4A. ITEM 301 - 7¼" BITUMINOUS AGGREGATE BASE, PG 64-22

4B. ITEM 301 - Var. 11½" to 13¾" BITUMINOUS AGGREGATE BASE, PG 64-22

4C. ITEM 301 - 13¾" BITUMINOUS AGGREGATE BASE, PG 64-22

5. ITEM 304 - 6" AGGREGATE BASE

5A. ITEM 304 - 10" AGGREGATE BASE

5B. ITEM 304 - 12" AGGREGATE BASE

6. ITEM 407 - TACK COAT

7. ITEM 407 - TACK COAT FOR INTERMEDIATE COURSE

8. ITEM 622 - CONCRETE BARRIER, TYPE D

9. ITEM 606 - GUARDRAIL, TYPE 5

10. ITEM 202 - WEARING COURSE REMOVED

11. ITEM 605 - 6" SHALLOW PIPE UNDERDRAIN

12. THIS BALLOON IS NOT USED

13. THIS BALLOON IS NOT USED

14. ITEM 830 - CONCRETE MEDIAN AS PER PLAN (SEE SHEET NO. 24)

15. ITEM 830 - CURB, TYPE 6

A. EXISTING 4-1/4" OF ASPHALT CONCRETE ON 9" REINFORCED PORTLAND CEMENT CONCRETE ON 6" SUBBASE

A1. EXISTING ASPHALT CONCRETE VARIES FROM 2-1/2" TO 5" ON 9" REINFORCED PORTLAND CEMENT CONCRETE ON 6" SUBBASE

A2. EXISTING ASPHALT CONCRETE VARIES FROM 2-1/2" TO 6" ON 9" REINFORCED PORTLAND CEMENT CONCRETE ON 6" SUBBASE

B. EXISTING ASPHALT CONCRETE VARIES FROM 8" TO 4-1/4" ON 9" REINFORCED PORTLAND CEMENT CONCRETE ON 6" SUBBASE

C. EXISTING ASPHALT CONCRETE VARIES FROM 4-1/4" TO 9" ON 9" REINFORCED PORTLAND CEMENT CONCRETE ON 6" SUBBASE

D. EXISTING 4-1/4" OF ASPHALT CONCRETE ON 9" REINFORCED PORTLAND CEMENT CONCRETE ON VARIABLE DEPTH SUBBASE

E. EXISTING 4-1/4" ASPHALT CONCRETE ON 3" BITUMINOUS AGGREGATE BASE ON 6" STABILIZED CRUSHED AGGREGATE ON VARIABLE SUBBASE

F. EXISTING 4-1/4" ASPHALT CONCRETE ON 3" WATERPROOFED AGGREGATE BASE ON 5" CRUSHED AGGREGATE ON VARIABLE DEPTH SUBBASE

G. EXISTING 6" UNDERDRAIN

H. EXISTING GUARDRAIL

I. EXISTING LONGITUDINAL JOINT

J. EXISTING WALL

K. EXISTING CURB AND GUTTER, TYPE 2

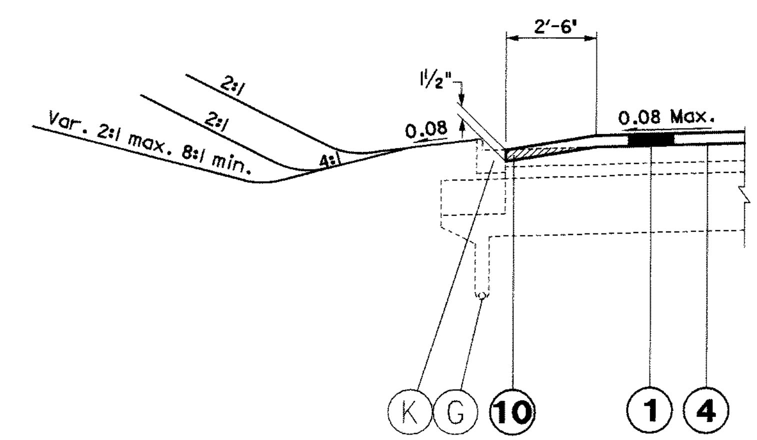
L. EXISTING CURB, TYPE 2-A

M. EXISTING CURB, TYPE 6

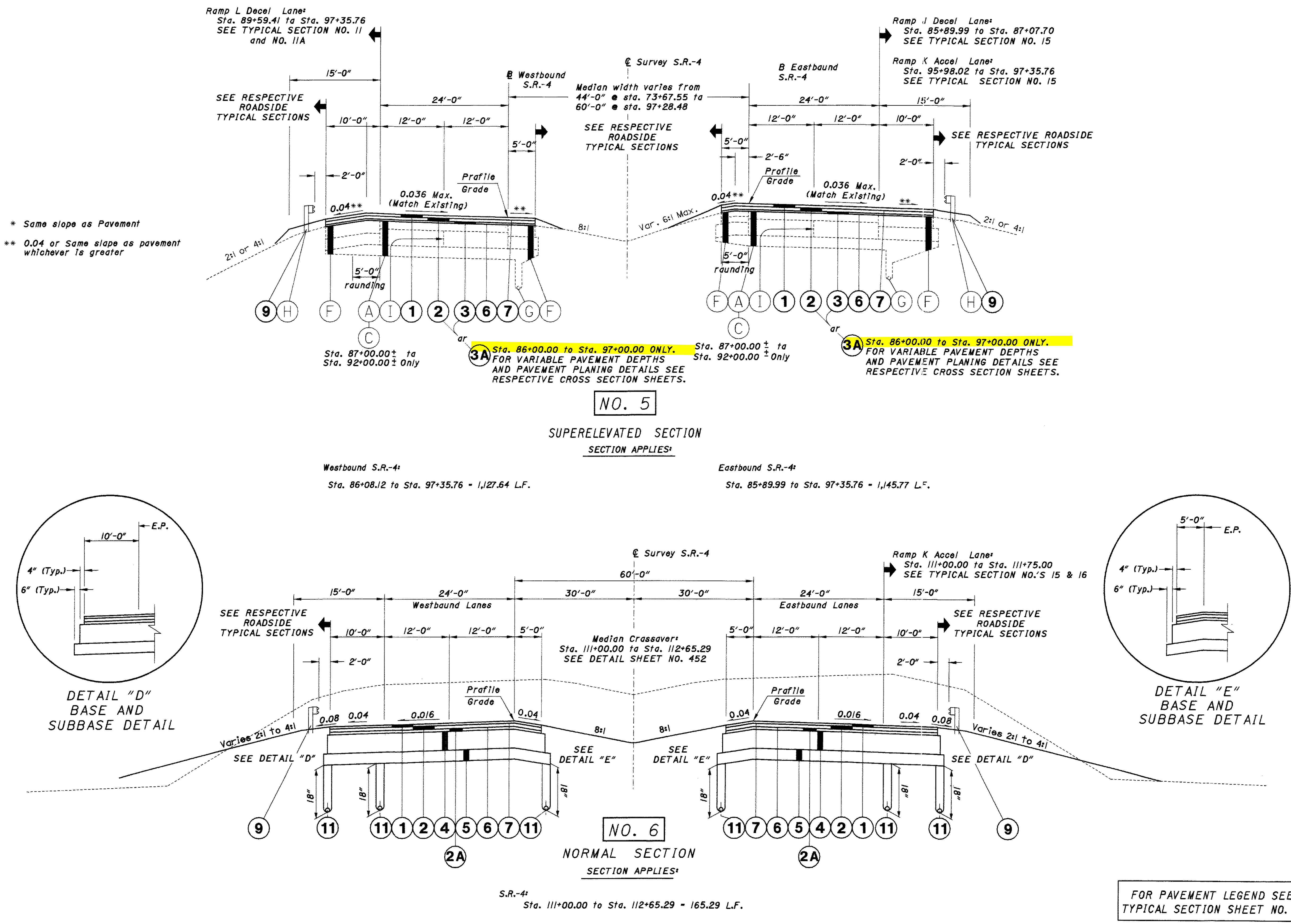
N. EXISTING 4" CONCRETE MEDIAN

Plan Symbols

- PAVEMENT TRANSITION AREAS: See respective Plan Sheets and Transition Detail Sheets for specific transition details and dimensions
- PAVEMENT WIDENING AREAS: See Respective Plan Sheets and Typical Sections for specific details and dimensions
- FOR REMOVAL OF GRAVEL TURN AROUND DETAILS SEE GENERAL NOTES ITEM-203 EXCAVATION AS PER PLAN
- CURB REMOVAL AND PAVEMENT REPLACEMENT, SEE TYPICAL SECTION 18A FOR MORE DETAILS



CURB DETAIL



FOR PAVEMENT LEGEND SEE TYPICAL SECTION SHEET NO. 5

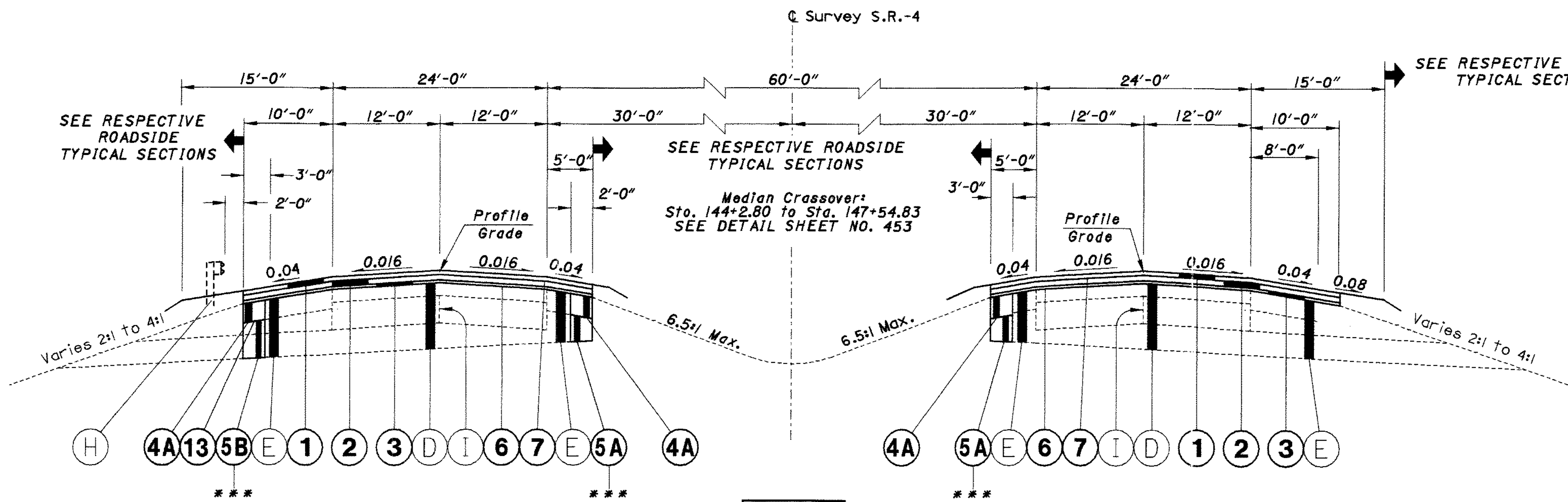
TYPICAL SECTIONS

MOT-4-19.14

* Same slope as Pavement

** 0.04 or Same slope as pavement whichever is greater

*** Field adjust thickness of the 304 course. The bottoms of the existing and proposed aggregate base should align for positive drainage.



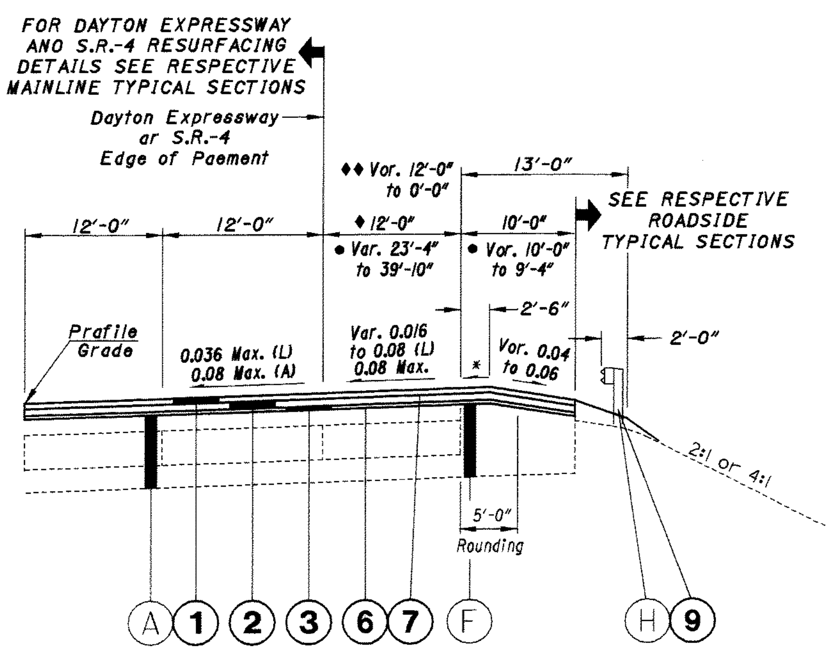
NO. 10

NORMAL SECTION

SECTION APPLIES:

S.R.-4:

Sta. 138+35.00 to Sta. 147+54.82 = 919.82 L.F.



NO. 11

TERMINAL SECTION

Dayton Expressway

SECTION APPLIES:

Ramp A :

● Sta. 12+11.80 ℄ to Sta. 13+51.22 ℄ - 140.07 L.F.

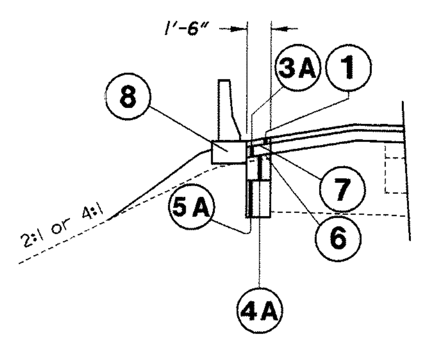
Total - 140.07 L.F.

Ramp L :

♦ Sta. 97+00.00 ℄ to Sta. 97+75.00 ℄ - 75.00 L.F. (Opposite Hand)

♦♦ Sta. 97+75.00 ℄ to Sta. 98+75.00 ℄ - 100.00 L.F. (Opposite Hand)

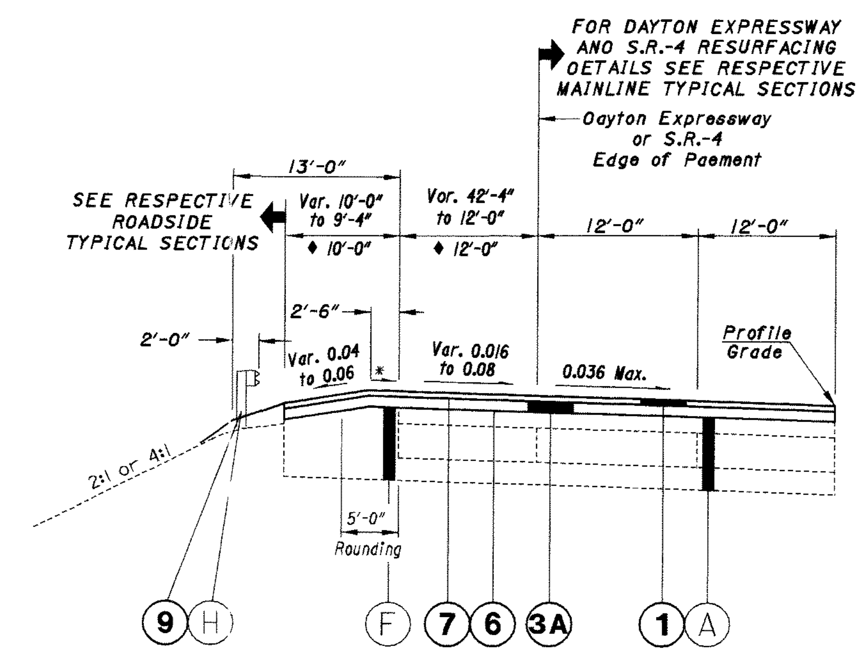
Total - 175.00 L.F.



SECTION APPLIES:

Ramp L:

Sta. 0+59.31 to Sta. 1+11.00 - 51.69 L.F.



NO. 11A

TERMINAL SECTION

Dayton Expressway

SECTION APPLIES:

Ramp L :

Sta. 89+59.41 ℄ to Sta. 92+94.50 ℄ - 335.09 L.F.

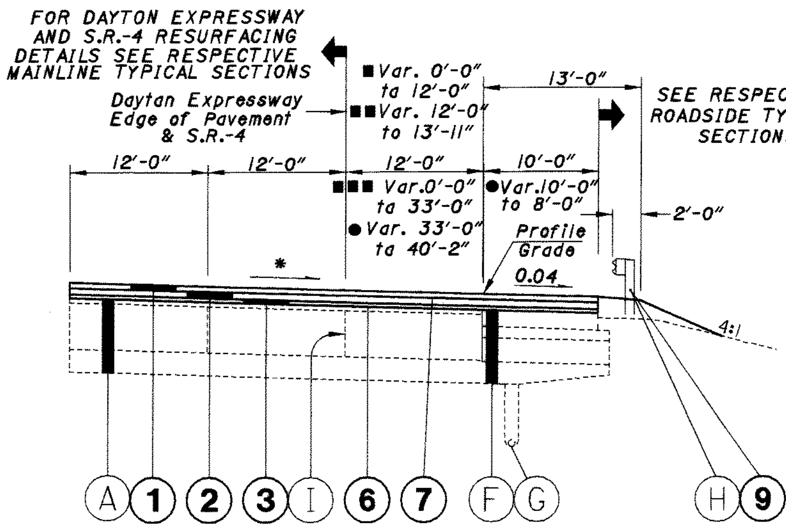
♦ Sta. 92+94.50 ℄ to Sta. 97+00.00 ℄ - 405.50 L.F.

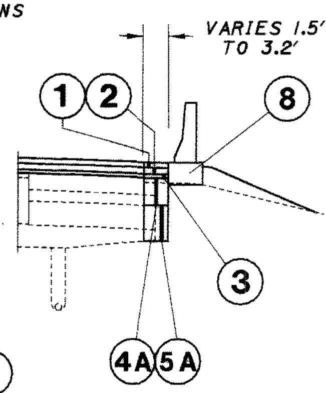
Total - 740.59 L.F.

FOR PAVEMENT LEGEND SEE TYPICAL SECTION SHEET NO. 5

TYPICAL SECTIONS

MOT-4-19.14





SECTION APPLIES:
Sta. 65+76.09 to Sta. 66+28.41 - 52.32 L.F.

NO. 14

TERMINAL SECTION

SECTION APPLIES:
S.R.-4

Ramp G:
■Sta. 66+00.00 ℄ to Sta. 67+00.00 ℄ - 100.00 L.F.
Sta. 67+00.00 ℄ to Sta. 69+99.76 ℄ - 299.76 L.F.

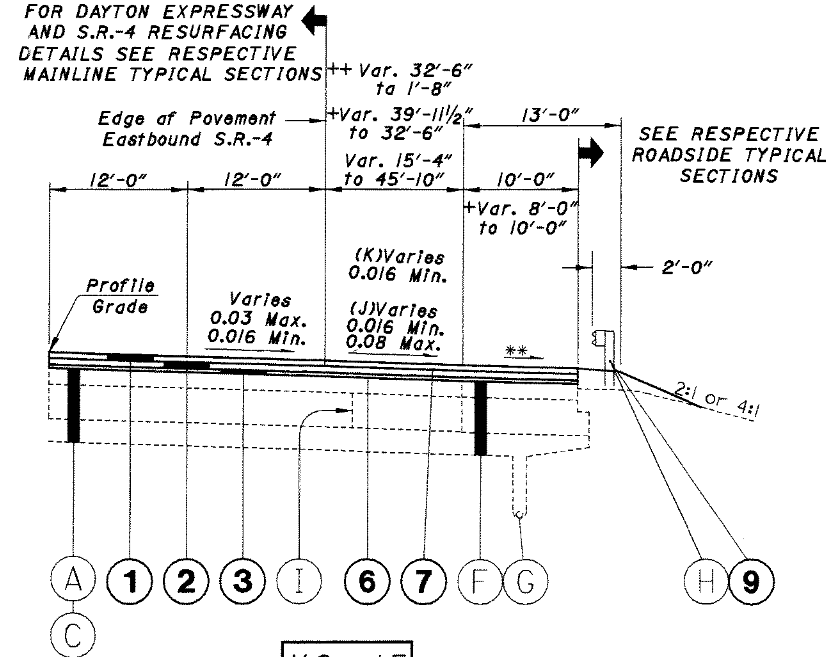
STATION EQUATION:
Sta. 69+99.76 ℄ BACK =
Sta. 70+00.00 ℄ AHEAD

Sta. 70+00.00 ℄ to Sta. 70+90.85 ℄ - 90.85 L.F.
■■ Sta. 70+90.85 ℄ to Sta. 72+39.21 ℄ - 148.36 L.F.
Total - 638.97 L.F.

Ramp H:
■■■Sta. 62+49.77 ℄ to Sta. 69+99.76 ℄ - 749.99 L.F. (Opposite Hand)

STATION EQUATION:
Sta. 69+99.76 ℄ BACK =
Sta. 70+00.00 ℄ AHEAD

Sta. 70+00.00 ℄ to Sta. 75+51.33 ℄ - 551.33 L.F. (Opposite Hand)
● Sta. 75+51.33 ℄ to Sta. 76+40.52 ℄ - 89.19 L.F. (Opposite Hand)
Total - 1390.51 L.F.



Sta. 87+00.00 to
Sta. 92+00.00 Only

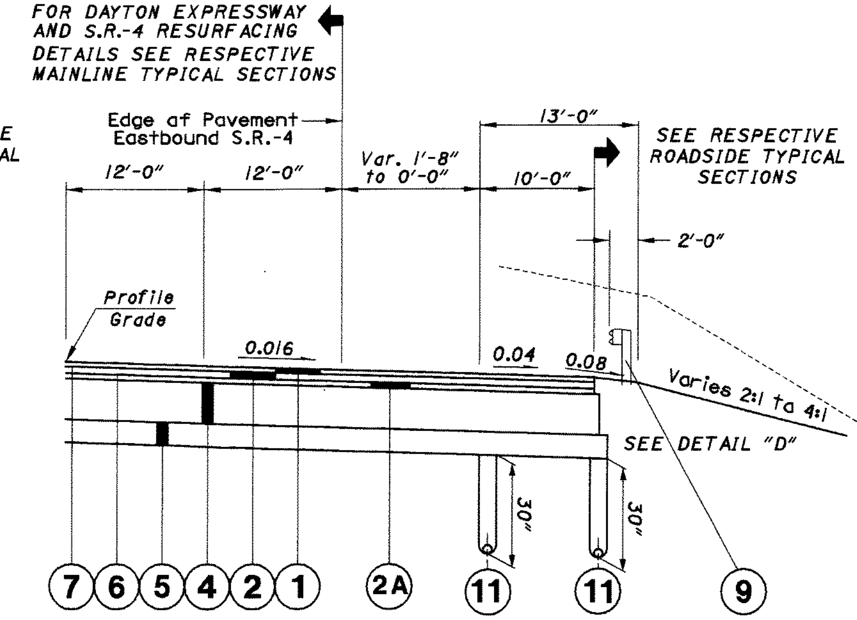
NO. 15

TERMINAL SECTION

SECTION APPLIES:
S.R.-4

Ramp J:
Sta. 85+89.99 ℄ to Sta. 87+07.70 ℄ - 117.71 L.F.
Total - 117.71 L.F.

Ramp K:
+ Sta. 95+98.02 ℄ to Sta. 97+50.09 ℄ - 152.07 L.F.
++ Sta. 97+50.09 ℄ to Sta. 111+00.00 ℄ - 1,349.91 L.F.
Total - 1,501.98 L.F.



NO. 16

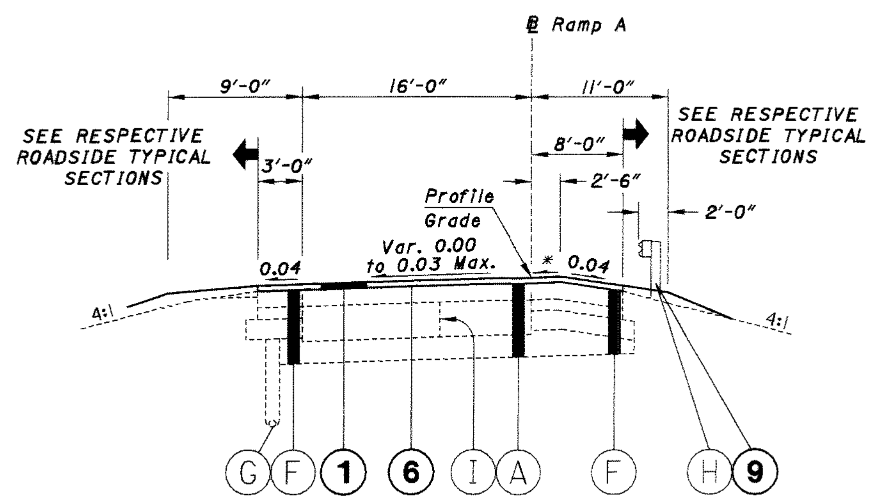
TERMINAL SECTION

SECTION APPLIES:
S.R.-4

Ramp K:
Sta. 111+00.00 ℄ to Sta. 111+75.00 ℄ = 75.00 L.F.
Total - 75.00 L.F.

* Same slope as Pavement

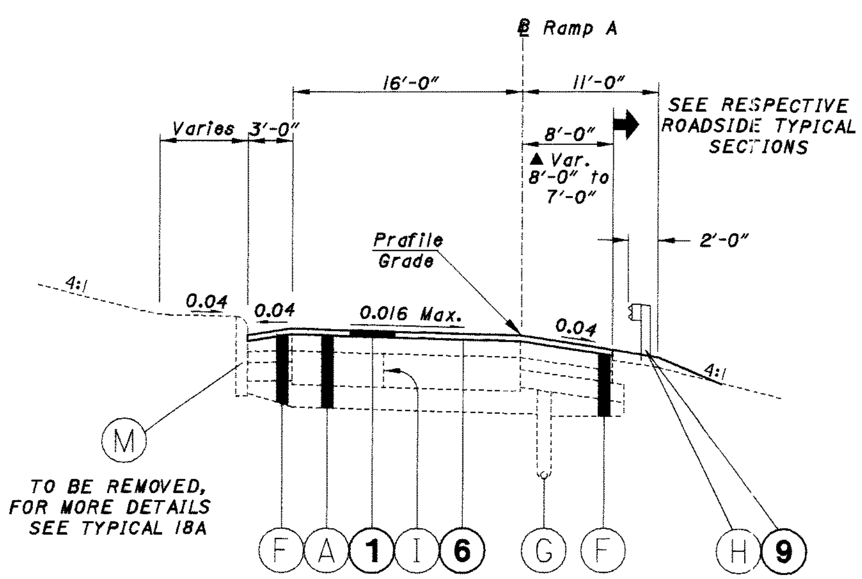
** 0.04 or Same slope as pavement whichever is greater



NO. 17

SUPERELEVATED RAMP SECTION
Dayton Expressway
SECTION APPLIES:

Ramp A:
Sta. 4+52.23(A) to Sta. 7+87.95(A) = 335.72 L.F.
Total - 335.72 L.F.

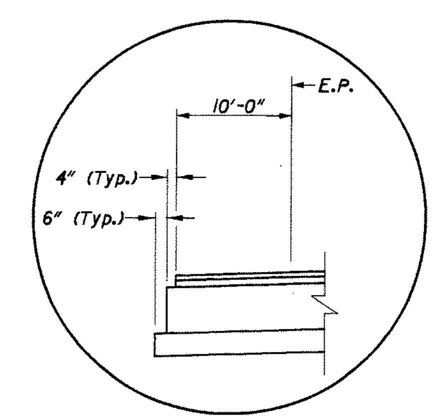


TO BE REMOVED,
FOR MORE DETAILS
SEE TYPICAL 18A

NO. 18

SUPERELEVATED RAMP SECTION
Dayton Expressway
SECTION APPLIES:

Ramp A:
Sta. 7+87.95(A) to Sta. 8+75.00(A) = 87.05 L.F.
▲Sta. 8+75.00(A) to Sta. 9+23.92(A) = 48.92 L.F.
Total - 135.97 L.F.



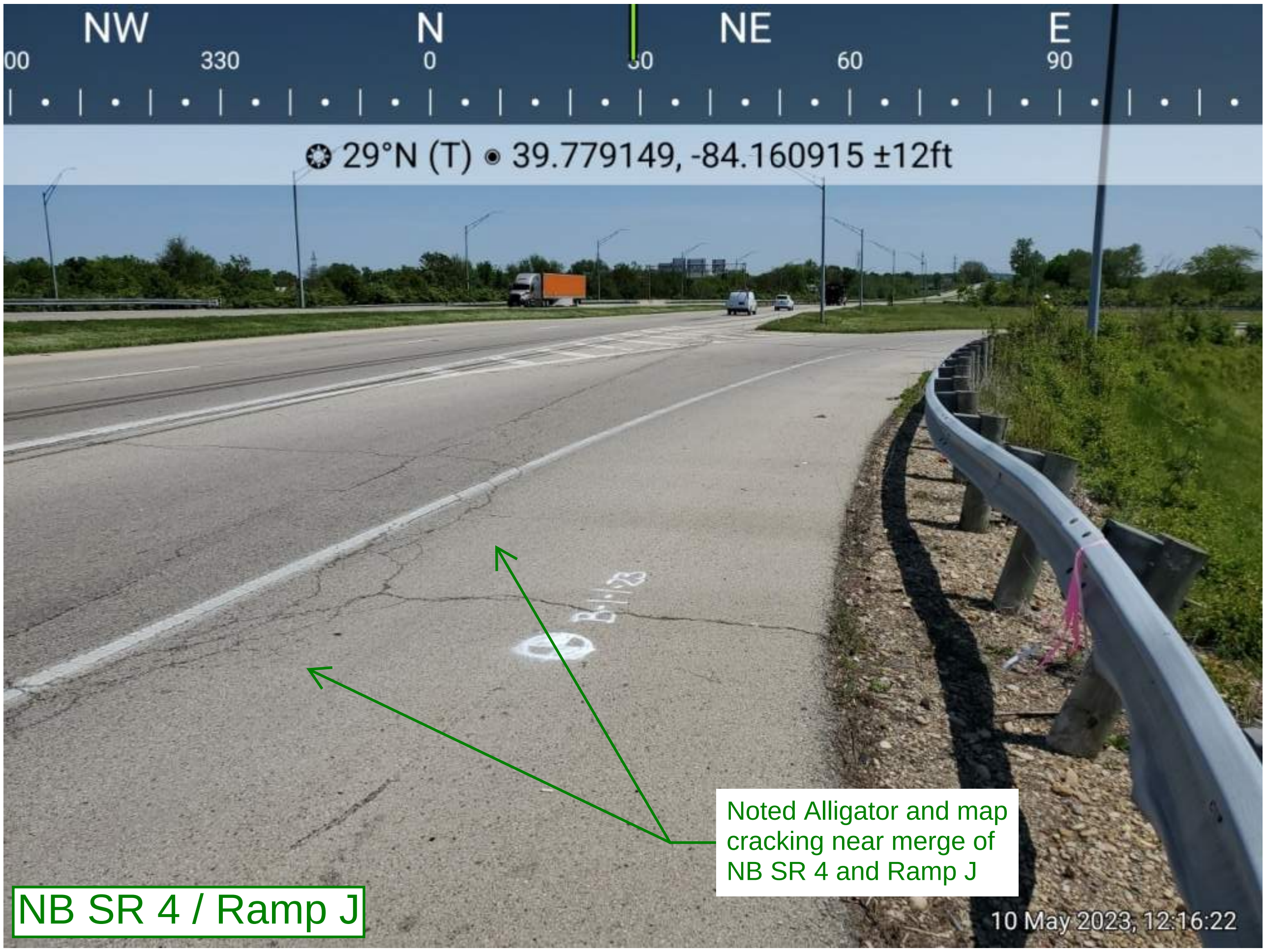
DETAIL "D"
BASE AND
SUBBASE DETAIL

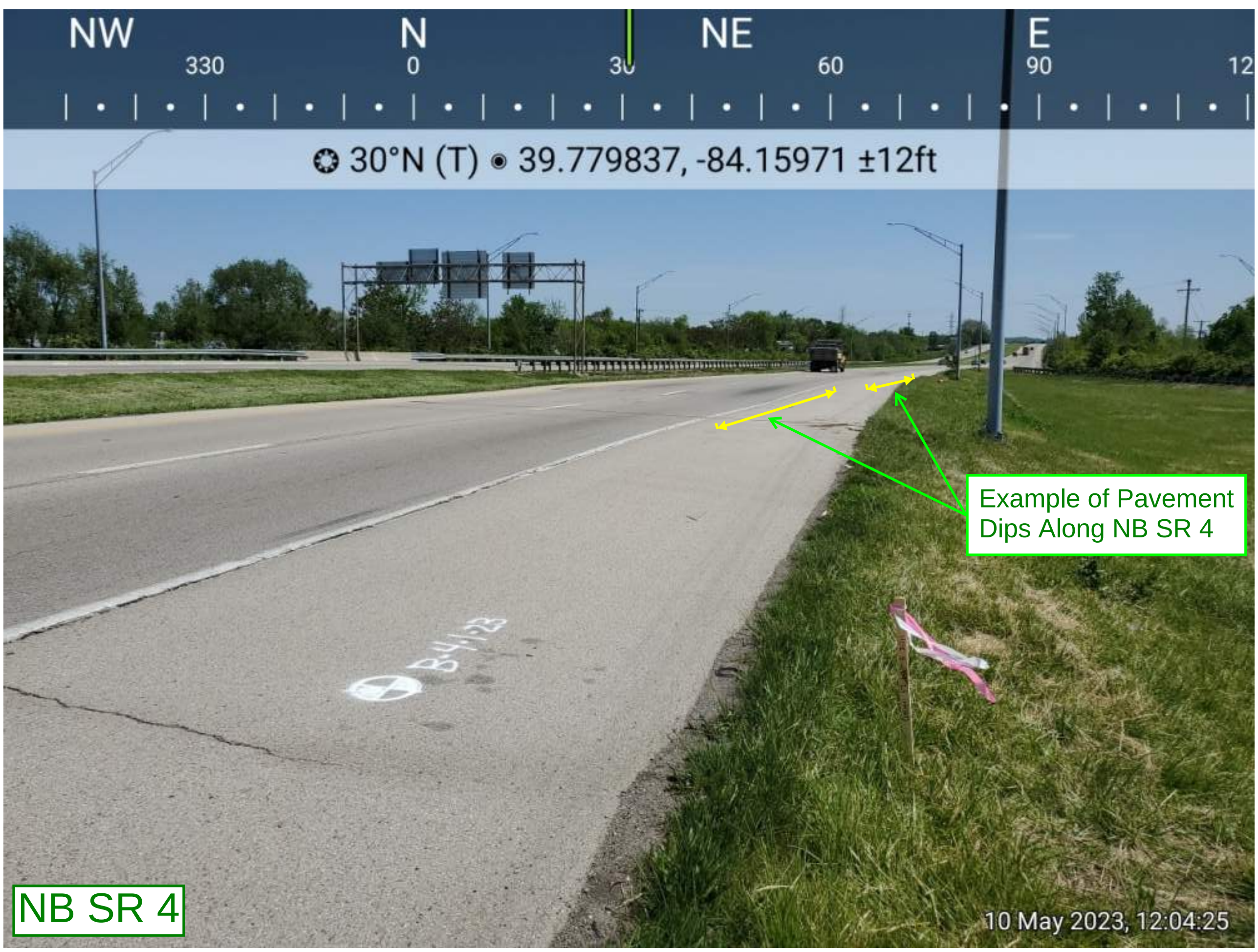
FOR PAVEMENT LEGEND SEE
TYPICAL SECTION SHEET NO. 5

TYPICAL SECTIONS

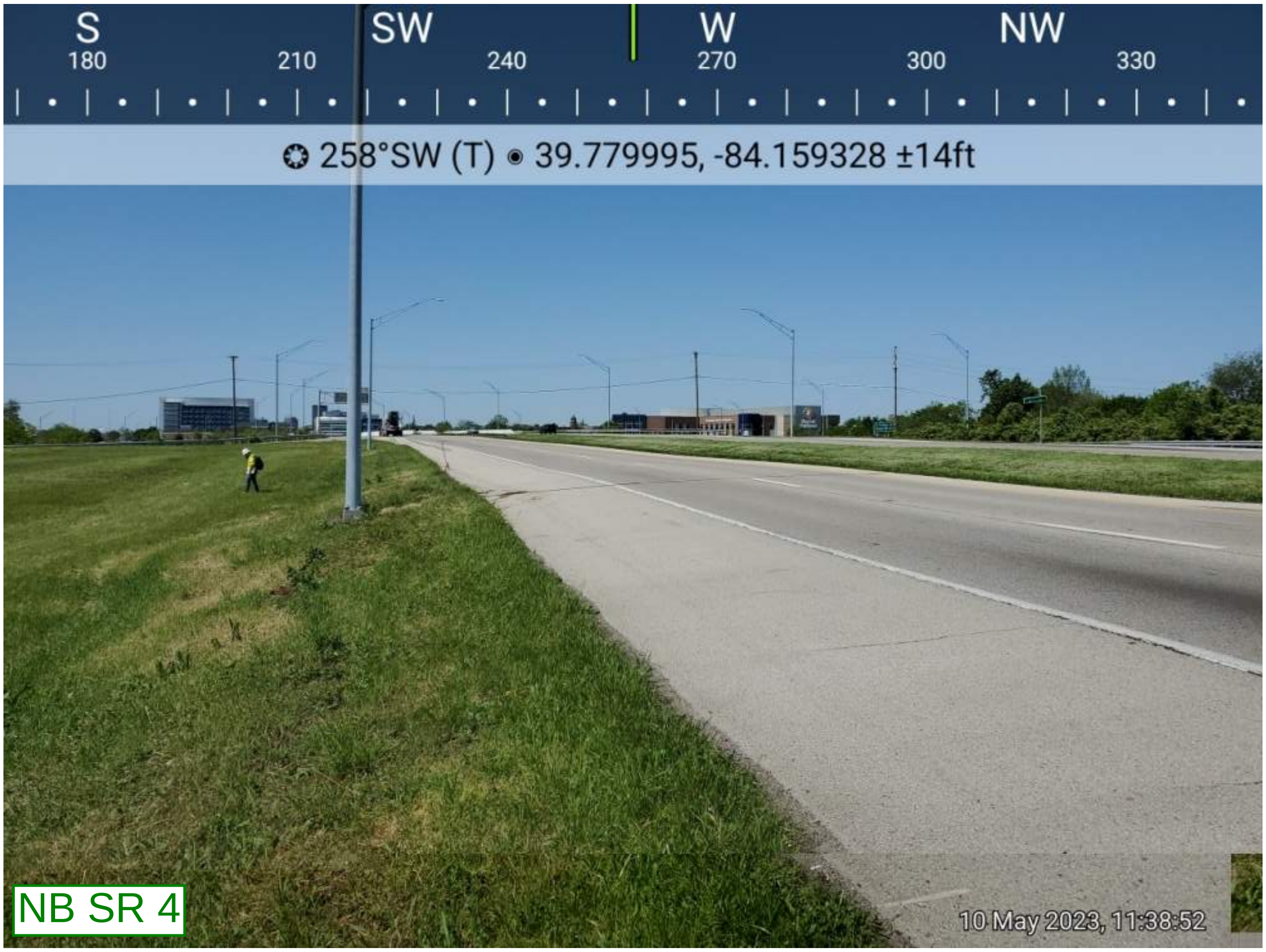
MOT-4-19.14

Appendix C. Site Photos


NW
N
NE
E
00
330
0
30
60
90
29°N (T) 39.779149, -84.160915 ±12ft
B-1-1-23
Noted Alligator and map cracking near merge of NB SR 4 and Ramp J
NB SR 4 / Ramp J
10 May 2023, 12:16:22


NW
N
NE
E
330
0
30
60
90
12
30°N (T) 39.779837, -84.15971 ±12ft
B-4-1-23
Example of Pavement Dips Along NB SR 4
NB SR 4
10 May 2023, 12:04:25


N
NE
E
SE
330
0
30
60
90
120
54°NE (T) ● 39.78022, -84.159394 ±28ft
Example of Pavement Dip Along NB SR 4
NB SR 4
16 May 2023, 1:19:03 PM

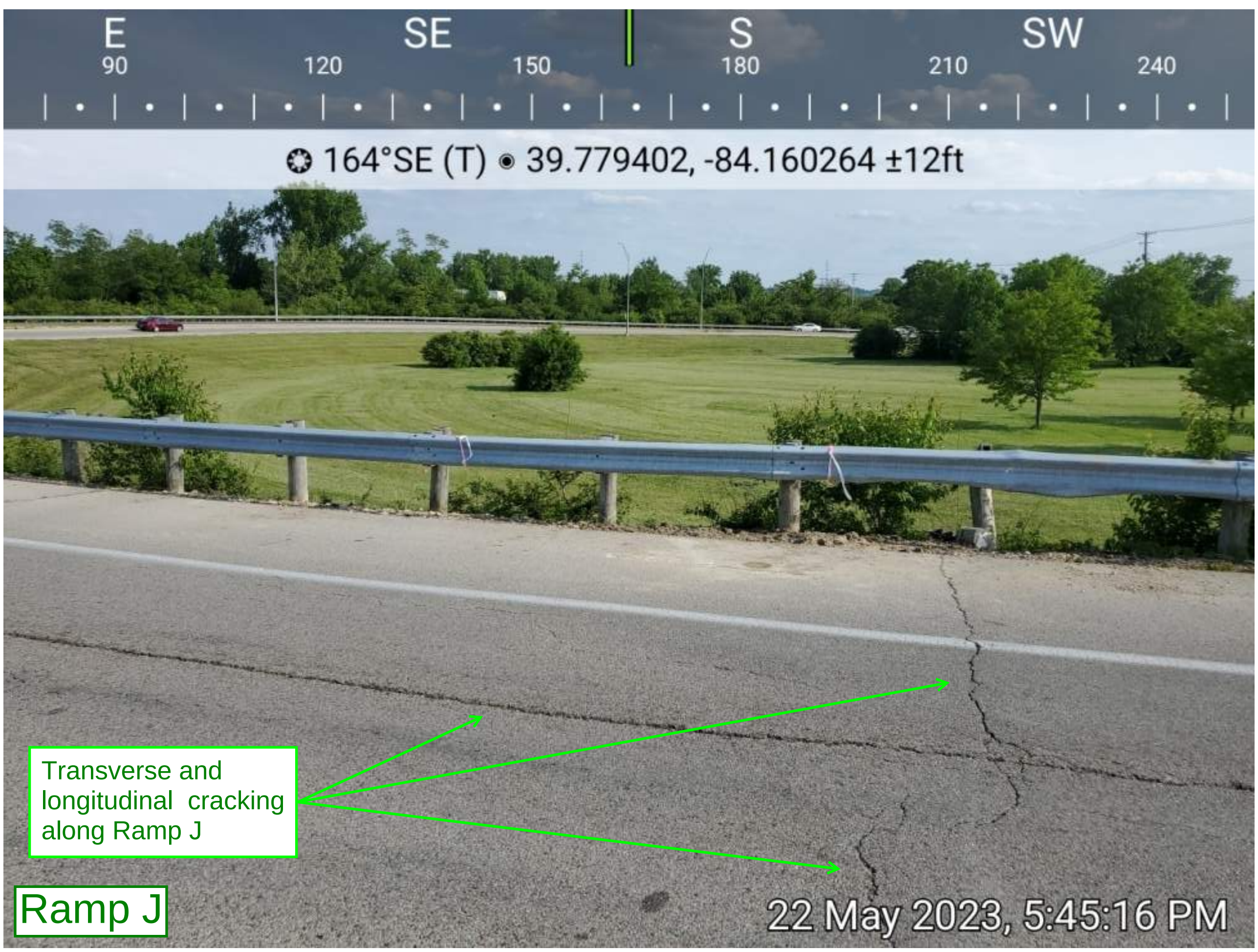

S
SW
W
NW
180
210
240
270
300
330
258°SW (T) ● 39.779995, -84.159328 ±14ft
NB SR 4
10 May 2023, 11:38:52

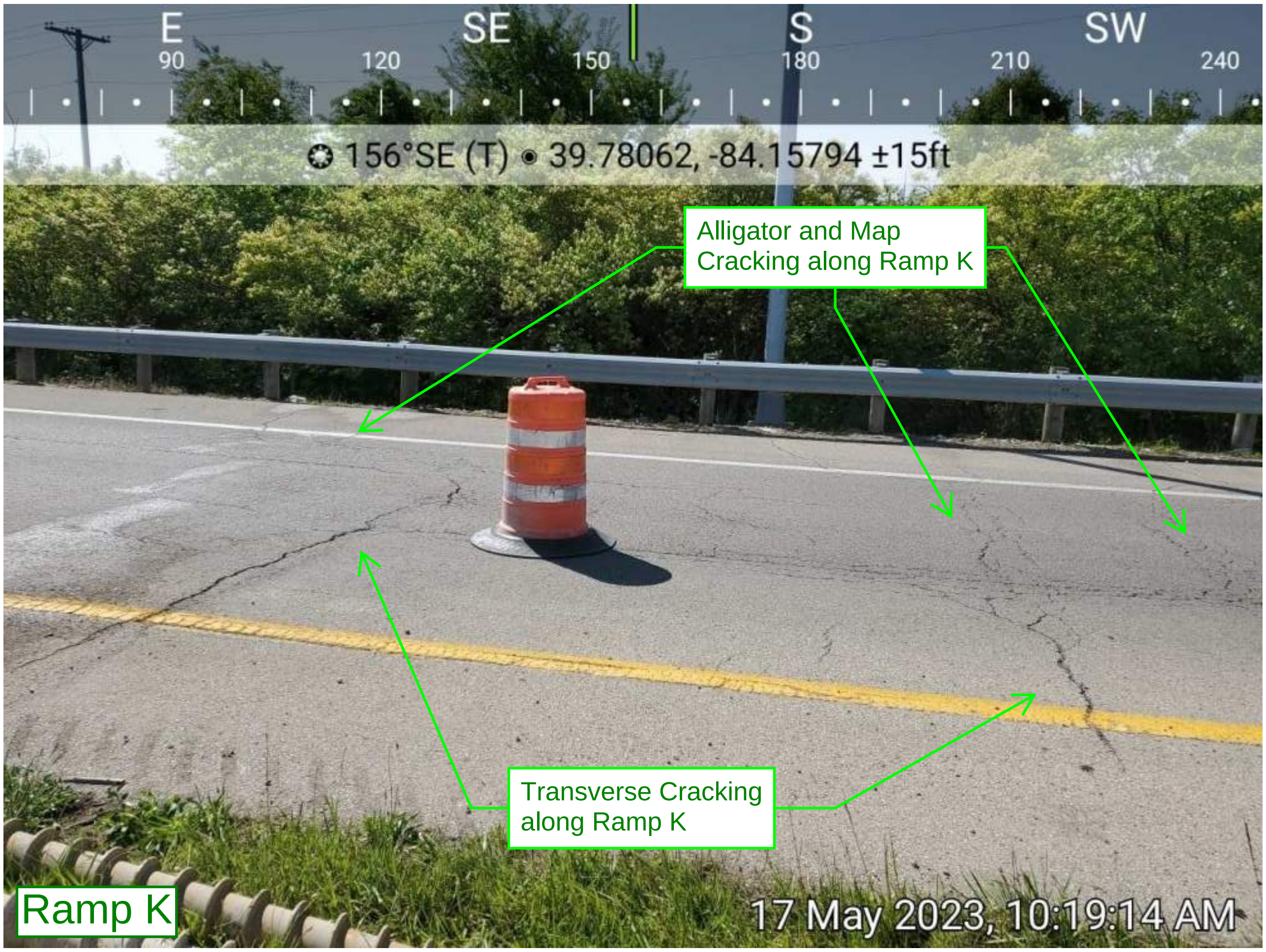

N
NE
E
SE
0
30
60
90
120
150
72°NE (T) 39.779969, -84.160337 ±15ft
Example of Pavement Dip Along SB SR 4
SB SR 4 / Ramp L
10 May 2023, 12:52:44

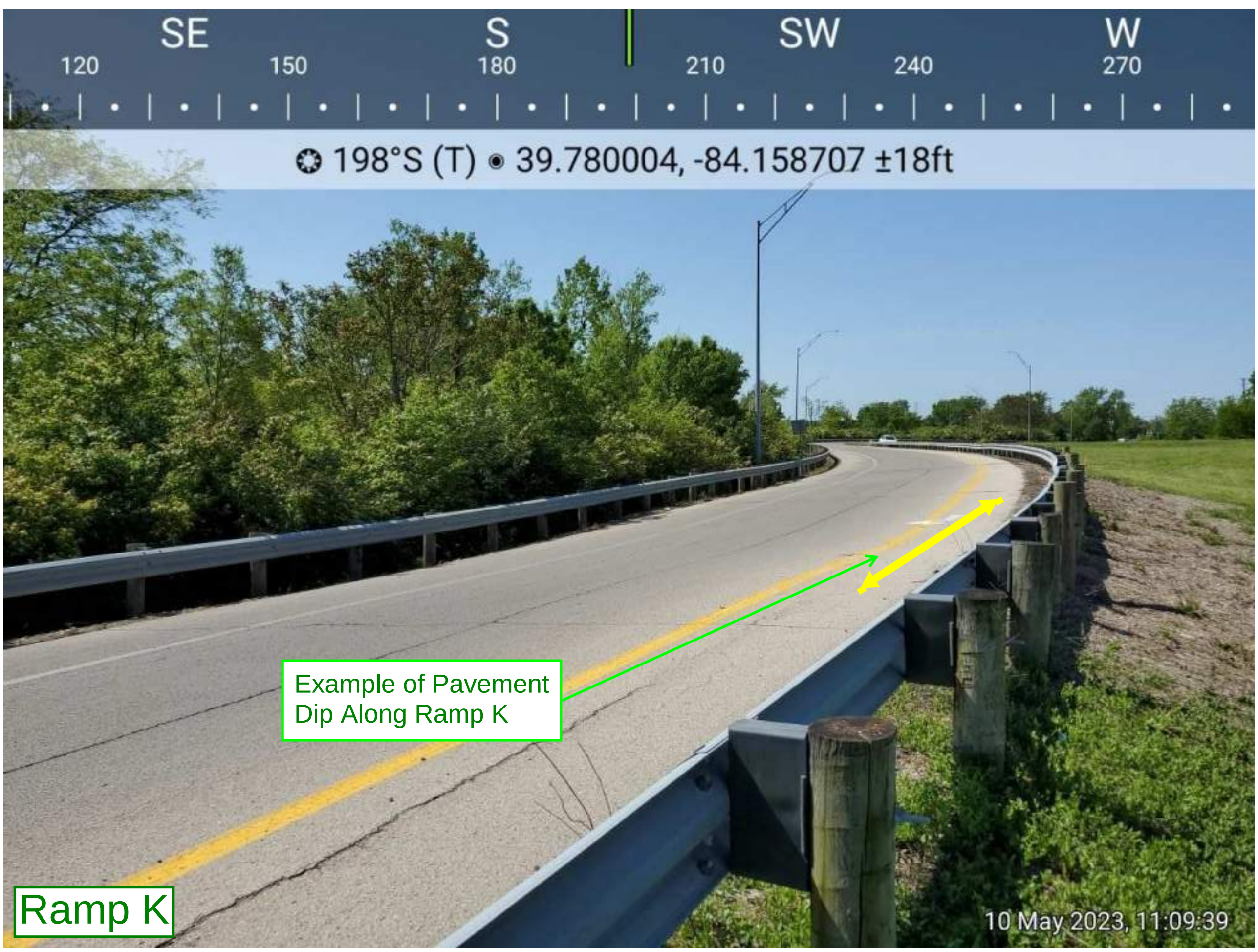

N
NE
E
SE
330
0
30
60
90
120
49°NE (T) 39.780244, -84.159533 ±20ft
SB SR 4
22 May 2023, 6:08:57 PM

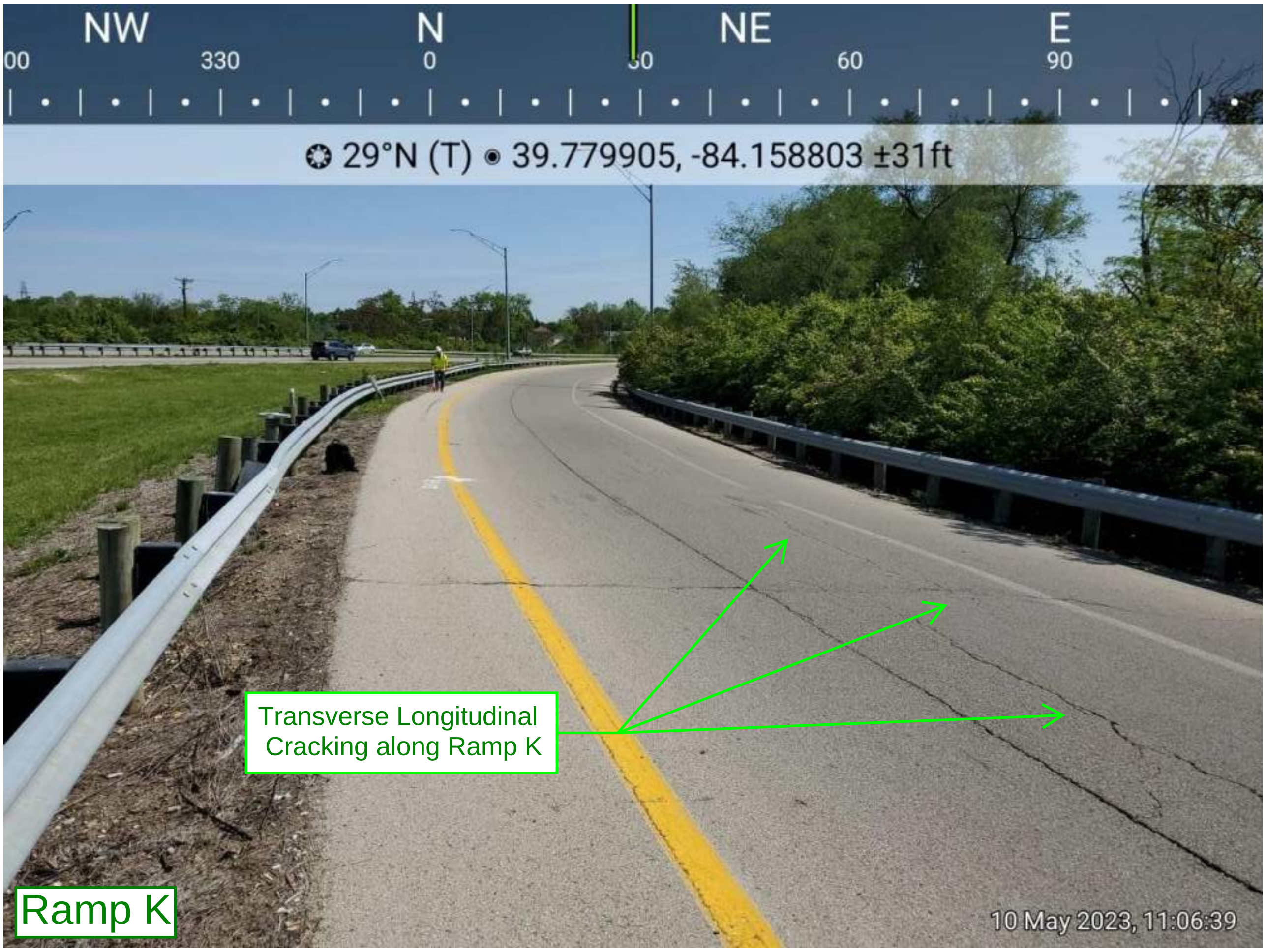
N
NE
E
SE
0
30
60
90
120
150
82°E (T) 39.780144, -84.160129 ±13ft
Example of Pavement Dip Along SB SR-4
SB SR 4 / Ramp L
10 May 2023, 12:41:46


SW
W
NW
N
210
240
270
300
330
0
291°W (T) 39.778855, -84.159253 ±12ft
Ramp J
10 May 2023, 10:33:39

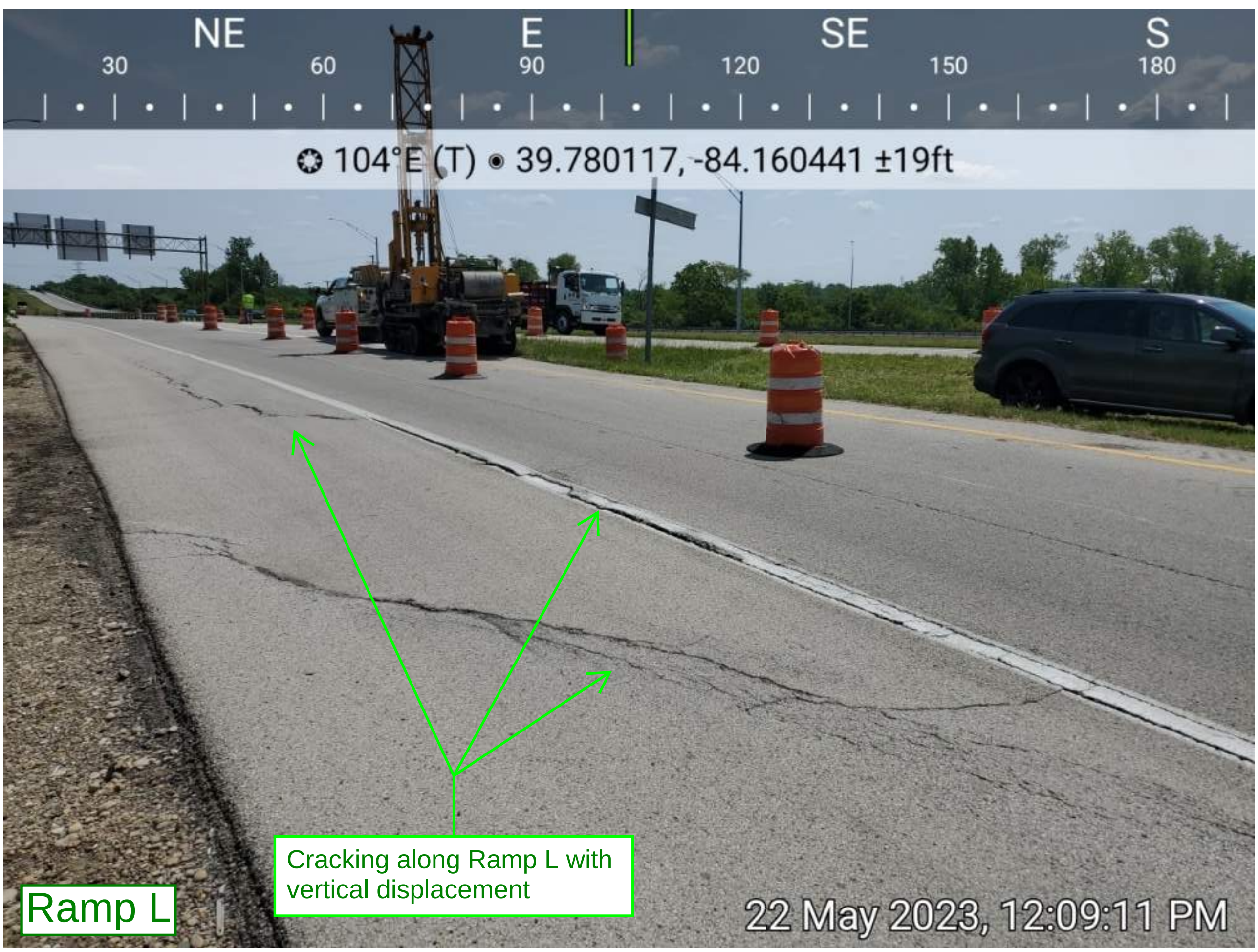

E
SE
S
SW
90
120
150
180
210
240
164°SE (T) 39.779402, -84.160264 ±12ft
Transverse and longitudinal cracking along Ramp J
Ramp J
22 May 2023, 5:45:16 PM


E
SE
S
SW
90
120
150
180
210
240
156°SE (T) 39.78062, -84.15794 ±15ft
Alligator and Map Cracking along Ramp K
Transverse Cracking along Ramp K
Ramp K
17 May 2023, 10:19:14 AM


SE
S
SW
W
120
150
180
210
240
270
198°S (T) 39.780004, -84.158707 ±18ft
Example of Pavement Dip Along Ramp K
Ramp K
10 May 2023, 11:09:39


NW
N
NE
E
330
0
30
60
90
29°N (T) 39.779905, -84.158803 ±31ft
Transverse Longitudinal Cracking along Ramp K
Ramp K
10 May 2023, 11:06:39


N
NE
E
SE
0
30
60
90
120
150
64°NE (T) 39.779999, -84.160524 ±15ft
Ramp L
10 May 2023, 12:48:36


NE
E
SE
S
30
60
90
120
150
180
104°E (T) 39.780117, -84.160441 ±19ft
Cracking along Ramp L with vertical displacement
Ramp L
22 May 2023, 12:09:11 PM

Appendix D. Boring Logs and Pavement Core Photos

PROJECT: MOT-4-19.30	DRILLING FIRM / OPERATOR: CENTRAL STAR / TS	DRILL RIG: DIEDRICH D-50 TRACK	STATION / OFFSET: 0+99, 6' RT.	EXPLORATION ID B-001-1-23
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: HDR / DCM	HAMMER: AUTOMATIC HAMMER	ALIGNMENT: RAMP J	
PID: 117239 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 3/7/22	ELEVATION: 772.6 (MSL) EOB: 42.0 ft.	PAGE 1 OF 2
START: 5/18/23 END: 5/18/23	SAMPLING METHOD: SPT	ENERGY RATIO (%): 86.8	LAT / LONG: 39.779141, -84.160889	

MATERIAL DESCRIPTION AND NOTES	ELEV. 772.6	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
12 inches Asphalt @ 3 inches - 5 inches : Delaminated and Degraded	771.6																
LOOSE TO MEDIUM DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP (FILL)		6-9-11	29	61	SS-1	-	68	21	4	5	2	15	15	NP	7	A-1-a (0)	
	768.1	3-3-2	7	44	SS-2	-	-	-	-	-	-	-	-	-	5	A-1-a (V)	
MEDIUM DENSE, BLACK AND DARK BROWN, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, MOIST (FILL)		4-10-7	25	56	SS-3	-	54	17	6	13	10	27	21	6	15	A-1-b (0)	
	765.1	4-6-7	19	39	SS-4	-	-	-	-	-	-	-	-	-	16	A-1-b (V)	
MEDIUM DENSE, BROWN, **GRAVEL WITH SAND AND SILT**, TRACE CLAY, DAMP (FILL)	763.6	6-8-4	17	56	SS-5	-	67	20	8	1	4	25	18	7	8	A-2-4 (0)	
LOOSE, BROWN, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP TO MOIST (FILL)		3-2-4	9	33	SS-6	-	-	-	-	-	-	-	-	-	16	A-1-b (V)	
		4-2-5	10	67	SS-7	-	42	37	4	14	3	26	NP	NP	14	A-1-b (0)	
	759.6	4-10-9	27	78	SS-8A	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP (FILL)					SS-8B	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
		6-15-26	59	56	SS-9	-	-	-	-	-	-	-	-	-	6	A-1-a (V)	
	756.1	10-9-10	27	67	SS-10	-	67	21	4	5	3	20	15	5	5	A-1-a (0)	
DENSE TO VERY DENSE, DARK BROWN TO BLACK, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, NOTED BRICK, PLASTIC, AND ASPHALT, DAMP (FILL - Refuse Material)		10-18-16	49	33	SS-11	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
		6-18-21	56	67	SS-12	-	61	17	6	11	5	25	20	5	9	A-1-b (0)	
	751.6	50/5"	-	100	SS-13	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
MEDIUM DENSE, BLACK, **FINE SAND**, MOIST (FILL - Refuse Material)	750.6	6-4-4	12	100	SS-14A	-	-	-	-	-	-	-	-	-	18	A-3 (V)	
STIFF TO VERY STIFF, BLACK TO DARK BROWN, **SILTY CLAY**, LITTLE SAND, TRACE GRAVEL, NOTED GLASS AND ASPHALT, DAMP (FILL - Refuse Material)	750.1				SS-14B	1.00	-	-	-	-	-	-	-	-	21	A-6a (V)	
DENSE TO VERY DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP		8-10-12	32	100	SS-15	-	-	-	-	-	-	-	-	-	6	A-1-a (V)	
		18-15-15	43	100	SS-16	-	67	26	2	4	1	18	NP	NP	4	A-1-a (0)	
		12-16-18	49	100	SS-17	-	-	-	-	-	-	-	-	-	4	A-1-a (V)	
@ 28.0' : Added Water To Assist Drilling Below 28.0' : Wet Samples		12-15-16	45	100	SS-18	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	

PID: 117239	SFN:	PROJECT: MOT-4-19.30	STATION / OFFSET: 0+99, 6' RT.	START: 5/18/23	END: 5/18/23	PG 2 OF 2	B-001-1-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 742.6	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
DENSE TO VERY DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP *(continued)* @ 30.0' : Added Water To Assist Drilling		31 – 32	19 20 22	61	89	SS-19	-	60	32	3	4	1	16	NP	NP	5	A-1-a (0)	
		33 – 34	8 17 23	58	78	SS-20	-	-	-	-	-	-	-	-	-	7	A-1-a (V)	
Below 35.5' : Trace Broken Stone Fragments	734.6	35 – 37	12 15 13	41	67	SS-21	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SLT, TRACE CLAY, DAMP		38 – 40	9 7 11	26	78	SS-22	-	62	31	1	4	2	14	NP	NP	8	A-1-a (0)	
	730.6	41 – 42 (EOB)	12 21 14	51	67	SS-23	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 150 LB. BENTONITE CHIPS; TREMIED 25 LB. BENTONITE POWDER; 94 LB. CEMENT; PLACED 25 LB. QUICKCRETE; 55 GAL. WATER

PROJECT: MOT-4-19.30	DRILLING FIRM / OPERATOR: CENTRAL STAR / TS	DRILL RIG: DIEDRICH D-50 TRACK	STATION / OFFSET: 87+64, 58' RT.	EXPLORATION ID B-002-1-23
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: HDR / DCM	HAMMER: AUTOMATIC HAMMER	ALIGNMENT: SR 4	
PID: 117239 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 3/7/22	ELEVATION: 770.9 (MSL) EOB: 47.5 ft.	PAGE 1 OF 2
START: 5/15/23 END: 5/16/23	SAMPLING METHOD: SPT	ENERGY RATIO (%): 86.8	LAT / LONG: 39.779490, -84.160409	

MATERIAL DESCRIPTION AND NOTES	ELEV. 770.9
12 inches of Asphalt 9 inches of Reinforced Concrete	769.2
MEDIUM DENSE, BROWN, **COARSE AND FINE SAND**, DAMP (FILL)	768.7
DENSE TO VERY DENSE, BROWN AND DARK BROWN, **GRAVEL WITH SAND AND SILT**, TRACE CLAY, DAMP (FILL)	761.4
VERY DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP (FILL) Below 14.0' : Water Added to Assist in Drilling	755.4
VERY DENSE, BLACK WITH DARK BROWN, **GRAVEL**, "AND" SAND, TRACE SILT, TRACE CLAY, WITH CONSTRUCTION DEBRIS (GLASS, METAL, WASTED ASPHALT), PETROLEUM ODOR, DAMP TO MOIST (FILL - Refuse Material) Below 18.5' : Medium Dense to Dense	747.9
MEDIUM DENSE, BLACK WITH DARK BROWN, **SANDY SILT**, SOME GRAVEL, TRACE CLAY, WITH CONSTRUCTION DEBRIS (GLASS, METAL, WASTED ASPHALT), PETROLEUM ODOR, DAMP TO MOIST (FILL - Refuse Material)	743.4
DENSE TO VERY DENSE, BROWN, **GRAVEL WITH SAND AND SILT**, TRACE CLAY, DRY	

DEPTHS: 1 – 29

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)
7 10 17	39	89	SS-1A	-	-	-	-	-	-	-	-	-	15	A-3a (V)
			SS-1B	-	-	-	-	-	-	-	-	-	11	A-2-4 (V)
8 19 13	46	89	SS-2	-	-	-	-	-	-	-	-	-	10	A-2-4 (V)
11 15 21	52	100	SS-3	-	39	26	10	19	6	21	13	8	8	A-2-4 (0)
11 15 24	56	100	SS-4	-	-	-	-	-	-	-	-	-	5	A-2-4 (V)
11 17 34	74	100	SS-5	-	-	-	-	-	-	-	-	-	7	A-2-4 (V)
21 35 29	93	100	SS-6	-	-	-	-	-	-	-	-	-	4	A-1-a (V)
23 24 26	72	100	SS-7	-	60	29	4	5	2	20	15	5	5	A-1-a (0)
22 26 26	75	89	SS-8	-	-	-	-	-	-	-	-	-	5	A-1-a (V)
20 26 41	97	89	SS-9	-	-	-	-	-	-	-	-	-	10	A-1-a (V)
50/5"	-	100	SS-10	-	-	-	-	-	-	-	-	-	10	A-1-a (V)
13 48 43	132	89	SS-11	-	50	30	8	7	5	21	15	6	9	A-1-a (0)
5 9 13	32	33	SS-12	-	-	-	-	-	-	-	-	-	13	A-1-a (V)
5 7 9	23	89	SS-13	-	-	-	-	-	-	-	-	-	19	A-1-a (V)
9 5 6	16	56	SS-14	-	-	-	-	-	-	-	-	-	12	A-1-a (V)
10 3 4	10	89	SS-15	-	25	18	15	- 42 -		NP	NP	NP	12	A-4a (1)
8 4 4	12	17	SS-16	-	-	-	-	-	-	-	-	-	20	A-4a (V)
4 7 13	29	56	SS-17	-	-	-	-	-	-	-	-	-	10	A-4a (V)
12 13 20	48	100	SS-18	-	-	-	-	-	-	-	-	-	4	A-2-4 (V)
10 14	43	100	SS-19	-	35	27	9	- 29 -		NP	NP	NP	4	A-2-4 (0)

HOLE SEALED

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 8/16/23 12:58 - C:\PWWORKING\EAST01\D3330003\20230514_MOT-4-19.30_BORING LOGS - REV 1_WITH LAB DATA.GPJ

PID: 117239	SFN:	PROJECT: MOT-4-19.30	STATION / OFFSET: 87+64, 58' RT.	START: 5/15/23	END: 5/16/23	PG 2 OF 2	B-002-1-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 740.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
DENSE TO VERY DENSE, BROWN, **GRAVEL WITH SAND AND SILT**, TRACE CLAY, DRY *(continued)*			16															
@ 31.5' - 33.0' : Gray and Black	738.4	31, 32	16 20 19	56	83	SS-20	-	-	-	-	-	-	-	-	-	4	A-2-4 (V)	
MEDIUM DENSE, BROWN, **GRAVEL WITH SAND AND SILT**, TRACE CLAY, DAMP		33, 34, 35	10 12 13	36	100	SS-21	-	-	-	-	-	-	-	-	-	8	A-2-4 (V)	
		36, 37	9 9 8	25	67	SS-22	-	-	-	-	-	-	-	-	-	10	A-2-4 (V)	
	729.9	38, 39, 40	3 11 14	36	67	SS-23	-	-	-	-	-	-	-	-	-	9	A-2-4 (V)	
DENSE TO VERY DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP		41, 42	7 11 14	36	78	SS-24	-	66	21	8	4	1	15	NP	NP	9	A-1-a (0)	
		43, 44, 45	3 16 19	51	89	SS-25	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
	723.4	46, 47, EOB	12 12 14	38	100	SS-26	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 200 LB. BENTONITE CHIPS; TREMIED 25 LB. BENTONITE POWDER; 94 LB. CEMENT; PLACED 50 LB. QUICKCRETE; 55 GAL. WATER

PROJECT: MOT-4-19.30	DRILLING FIRM / OPERATOR: CENTRAL STAR / TS	DRILL RIG: DIEDRICH D-50 TRACK	STATION / OFFSET: 3+07, 4' RT.	EXPLORATION ID B-002-2-23
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: HDR / DCM	HAMMER: AUTOMATIC HAMMER	ALIGNMENT: RAMP J	
PID: 117239 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 3/7/22	ELEVATION: 768.2 (MSL) EOB: 33.5 ft.	PAGE 1 OF 2
START: 5/18/23 END: 5/18/23	SAMPLING METHOD: SPT	ENERGY RATIO (%): 86.8	LAT / LONG: 39.779310, -84.160214	

MATERIAL DESCRIPTION AND NOTES	ELEV. 768.2	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
9 inches Asphalt	767.5																	
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, (ROUNDED GRAVEL), DAMP (FILL)		1, 2	9 14 11	36	78	SS-1	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
	764.2	3, 4	4 4 8	17	56	SS-2	-	79	2	1	- 18 -		NP	NP	NP	6	A-1-b (0)	
VERY STIFF, BLACK AND DARK BROWN, **SANDY SILT**, "AND" GRAVEL, LITTLE CLAY, DAMP (FILL - Refuse Material)		5	8 6 8	20	89	SS-3	-	-	-	-	-	-	-	-	-	12	A-4a (V)	
		6, 7	6 6 10	23	89	SS-4	-	38	17	8	17	20	28	18	10	13	A-4a (0)	
		8	7 8 6	20	89	SS-5	-	-	-	-	-	-	-	-	-	12	A-4a (V)	
		9, 10	3 5 8	19	89	SS-6	-	-	-	-	-	-	-	-	-	15	A-4a (V)	
	757.2	11	7 16 21	54	89	SS-7	-	-	-	-	-	-	-	-	-	14	A-4a (V)	
DENSE TO VERY DENSE, BROWN, **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, DAMP		12, 13	8 12 16	41	100	SS-8	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
		14	14 22 25	68	89	SS-9	-	-	-	-	-	-	-	-	-	5	A-1-b (V)	
		15, 16	20 21 21	61	100	SS-10	-	46	35	12	4	3	13	NP	NP	8	A-1-b (0)	
		17	17 24 34	84	100	SS-11	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
		18	21 32 50/4"	-	100	SS-12	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
@ 19.0' : Water Added to Assist in Drilling @ 19.5' - 20.0' : Black		19, 20	20 25 14	56	89	SS-13	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
	746.2	21, 22	16 17 25	61	78	SS-14	-	-	-	-	-	-	-	-	-	5	A-1-b (V)	
MEDIUM DENSE TO DENSE, BROWN AND ORANGE BROWN, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP		23	6 7 5	17	67	SS-15	-	43	22	15	19	1	21	18	3	9	A-1-b (0)	
	743.7	24, 25	6 10 20	43	89	SS-16	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	
VERY DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP		26	6 14 21	51	89	SS-17	-	-	-	-	-	-	-	-	-	5	A-1-a (V)	
		27, 28	17 22 27	71	100	SS-18	-	-	-	-	-	-	-	-	-	5	A-1-a (V)	
		29	27 21 30	74	89	SS-19	-	61	33	2	3	1	14	NP	NP	5	A-1-a (0)	
			13															

PID: 117239	SFN:	PROJECT: MOT-4-19.30	STATION / OFFSET: 3+07, 4' RT.	START: 5/18/23	END: 5/18/23	PG 2 OF 2	B-002-2-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 738.2	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
VERY DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP *(continued)*		31	21 24	65	89	SS-20	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
		32																
	734.7	EOB 33	17 22 23	65	89	SS-21	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: TREMIED 25 LB. BENTONITE POWDER; 94 LB. CEMENT; PLACED 25 LB. QUICKCRETE; 55 GAL. WATER

PROJECT: MOT-4-19.30	DRILLING FIRM / OPERATOR: CENTRAL STAR / TS	DRILL RIG: DIEDRICH D-50 TRACK	STATION / OFFSET: 4+53, 15' LT.	EXPLORATION ID B-003-1-23
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: HDR / DCM	HAMMER: AUTOMATIC HAMMER	ALIGNMENT: RAMP L	
PID: 117239 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 3/7/22	ELEVATION: 770.3 (MSL) EOB: 46.0 ft.	PAGE 1 OF 2
START: 5/19/23 END: 5/22/23	SAMPLING METHOD: SPT	ENERGY RATIO (%): 86.8	LAT / LONG: 39.779984, -84.160505	

MATERIAL DESCRIPTION AND NOTES	ELEV. 770.3
14 inches Asphalt	769.1
MEDIUM DENSE, GRAY AND BROWN, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP (FILL)	762.8
MEDIUM DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP (FILL)	761.3
DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP (FILL - Refuse Material)	759.8
MEDIUM DENSE TO DENSE, DARK BROWN TO BLACK, **GRAVEL WITH SAND AND SILT**, TRACE CLAY, DAMP (FILL - Refuse Material)	755.3
MEDIUM DENSE, BLACK, **GRAVEL WITH SAND AND SILT**, TRACE CLAY, NOTED GLASS, PLASTIC, AND WASTED ASPHALT, MOIST TO WET (FILL - Refuse Material) @ 17.5' - 22.5' : Trace Clay	752.3
LOOSE TO MEDIUM DENSE, BLACK, **GRAVEL**, "AND" SAND, TRACE SILT, TRACE CLAY, NOTED GLASS, PLASTIC, AND WASTED ASPHALT, MOIST (FILL - Refuse Material) @ 22.5' : Noted Wood Fragment	746.3
LOOSE TO MEDIUM DENSE, BROWN, **GRAVEL WITH SAND AND SILT**, TRACE CLAY, NOTED ASPHALT AND GLASS, DAMP (FILL - Refuse Material) @ 26.0' - 26.5' : Asphalt Fragments	743.1
VERY DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP	

DEPTHS: 1 – 29

SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
3-6-9	22	44	SS-1	-	48	23	12	15	2	16	NP	NP	10	A-1-b (0)	
7-6-7	19	6	SS-2	-	-	-	-	-	-	-	-	-	4	A-1-b (V)	
10-10-10	29	17	SS-3	-	-	-	-	-	-	-	-	-	4	A-1-b (V)	
9-10-13	33	44	SS-4	-	-	-	-	-	-	-	-	-	5	A-1-b (V)	
9-10-9	27	56	SS-5	-	62	15	11	8	4	15	NP	NP	5	A-1-a (0)	
9-13-10	33	61	SS-6	-	70	16	5	6	3	17	17	NP	5	A-1-a (0)	
9-10-18	41	78	SS-7	-	-	-	-	-	-	-	-	-	7	A-2-4 (V)	
12-11-21	46	22	SS-8	-	-	-	-	-	-	-	-	-	9	A-2-4 (V)	
5-12-9	30	78	SS-9	-	33	25	15	23	4	17	NP	NP	10	A-2-4 (0)	
3-8-6	20	33	SS-10	-	-	-	-	-	-	-	-	-	23	A-2-4 (V)	
4-3-10	19	44	SS-11	-	32	19	14	- 35 -		NP	NP	NP	31	A-2-4 (0)	
7-6-8	20	33	SS-12	-	52	29	8	8	3	28	NP	NP	11	A-1-a (0)	
15-11-6	25	17	SS-13	-	-	-	-	-	-	-	-	-	11	A-1-a (V)	
2-2-3	7	17	SS-14	-	-	-	-	-	-	-	-	-	25	A-1-a (V)	
3-3-6	13	33	SS-15	-	-	-	-	-	-	-	-	-	18	A-1-a (V)	
6-6-4	14	56	SS-16	-	64	6	3	- 27 -		NP	NP	NP	6	A-2-4 (0)	
7-3-4	10	44	SS-17	-	-	-	-	-	-	-	-	-	16	A-2-4 (V)	
9-15-21	52	78	SS-18	-	-	-	-	-	-	-	-	-	5	A-1-a (V)	
11															

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 8/16/23 12:59 - C:\PWWORKING\EAST01\D3330003\20230514_MOT-4-19.30_BORING LOGS - REV 1_WITH LAB DATA.GPJ

PID: 117239	SFN:	PROJECT: MOT-4-19.30	STATION / OFFSET: 4+53, 15' LT.	START: 5/19/23	END: 5/22/23	PG 2 OF 2	B-003-1-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 740.3	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
VERY DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP *(continued)*	738.3	30–32	22 25	68	100	SS-19	-	57	22	11	7	3	15	NP	NP	4	A-1-a (0)	
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL WITH SAND**, TRACE CLAY, TRACE SILT, WET	W 737.3	32–33.5	9 9 19	41	89	SS-20	-	-	-	-	-	-	-	-	-	15	A-1-b (V)	
	733.3	35–36.5	7 6 4	14	100	SS-21	-	46	49	1	1	3	14	NP	NP	10	A-1-b (0)	
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL**, "AND" SAND, TRACE CLAY, WET		37–38.5	7 8 10	26	89	SS-22	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
	727.8	40–41.5	9 15 16	45	67	SS-23	-	52	45	1	0	2	13	NP	NP	9	A-1-a (0)	
DENSE, BROWN, **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, WET		42–43.5	7 10 12	32	89	SS-24	-	-	-	-	-	-	-	-	-	12	A-1-b (V)	
	724.3	45–46 EOB	10 12 12	35	78	SS-25	-	46	25	21	6	2	15	NP	NP	12	A-1-b (0)	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: TREMIED 25 LB. BENTONITE POWDER; 94 LB. CEMENT; PLACED 50 LB. QUICKCRETE; 55 GAL. WATER

PROJECT: MOT-4-19.30	DRILLING FIRM / OPERATOR: CENTRAL STAR / TS	DRILL RIG: DIEDRICH D-50 TRACK	STATION / OFFSET: 89+91, 58' RT.	EXPLORATION ID B-004-1-23
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: HDR / DCM	HAMMER: AUTOMATIC HAMMER	ALIGNMENT: SR 4	
PID: 117239 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 3/7/22	ELEVATION: 768.1 (MSL) EOB: 50.0 ft.	PAGE 1 OF 2
START: 5/15/23 END: 5/15/23	SAMPLING METHOD: SPT	ENERGY RATIO (%): 86.8	LAT / LONG: 39.779853, -84.159766	

MATERIAL DESCRIPTION AND NOTES	ELEV. 768.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
27 inches Asphalt	765.8																	
MEDIUM DENSE, BROWN AND GRAY, **GRAVEL**, TRACE SAND, TRACE SILT, TRACE CLAY, (ROUNDED GRAVEL), DAMP (FILL)			7 8 6	20	67	SS-1	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	
	762.6		4 3 5	12	56	SS-2	-	84	1	1	- 14 -		NP	NP	NP	5	A-1-a (0)	
DENSE TO VERY DENSE, BROWN, **GRAVEL WITH SAND AND SILT**, TRACE CLAY, DAMP (FILL)			19 14 11	36	78	SS-3	-	-	-	-	-	-	-	-	-	8	A-2-4 (V)	
			10 12 10	32	78	SS-4	-	64	6	3	- 27 -		NP	NP	NP	7	A-2-4 (0)	
			11 16 13	42	78	SS-5	-	-	-	-	-	-	-	-	-	6	A-2-4 (V)	
	756.1		6 25 26	74	78	SS-6	-	-	-	-	-	-	-	-	-	5	A-2-4 (V)	
VERY DENSE, BROWN, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, WITH CONSTRUCTION DEBRIS, GLASS AND METAL FRAGMENTS, DAMP (FILL - Refuse Material)	754.6		17 25 41	95	100	SS-7	-	56	15	11	15	3	23	20	3	7	A-1-b (0)	
MEDIUM DENSE, BROWN AND BLACK, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, WITH CONSTRUCTION DEBRIS, GLASS AND METAL FRAGMENTS, PETROLEUM ODOR, MOIST (FILL - Refuse Material) @ 15.5' - 16.0' : Wet Seam			15 4 3	10	78	SS-8	-	-	-	-	-	-	-	-	-	5	A-1-b (V)	
			2 3 3	9	56	SS-9	-	42	28	15	14	1	32	NP	NP	19	A-1-b (0)	
			8 8 6	20	44	SS-10	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
	749.1		6 5 6	16	44	SS-11	-	-	-	-	-	-	-	-	-	12	A-1-b (V)	
MEDIUM DENSE TO DENSE, BLACK, **COARSE AND FINE SAND**, SOME GRAVEL, LITTLE SILT, MOIST (FILL - Refuse Material) 20.5' - 22.0' : Trace Root Hairs			6 5 6	16	22	SS-12	-	30	16	37	17	0	21	NP	NP	12	A-3a (0)	
	746.1		11 12 10	32	22	SS-13	-	-	-	-	-	-	-	-	-	12	A-3a (V)	
MEDIUM DENSE, BROWN AND BLACK, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, WITH CONSTRUCTION DEBRIS, PETROLEUM ODOR, DAMP (FILL - Refuse Material)	744.6		13 11 8	27	50	SS-14	-	62	16	9	8	5	20	18	2	8	A-1-a (0)	
MEDIUM DENSE TO DENSE, BLACK, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, PETROLEUM ODOR, DAMP TO MOIST (FILL - Refuse Material) 26.5' - 28.0' : Glass and Wood Fragments			5 5 6	16	17	SS-15	-	38	25	15	17	5	24	NP	NP	19	A-1-b (0)	
			3 13 12	36	17	SS-16	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	
			7 9 7	23	17	SS-17	-	-	-	-	-	-	-	-	-	17	A-1-b (V)	
	738.6		6 3 6	13	17	SS-18	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
			12			SS-19	-	-	-	-	-	-	-	-	-	7	A-1-a (V)	

MATERIAL DESCRIPTION AND NOTES	ELEV. 738.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
DENSE, GRAY AND BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, WET *(continued)*		W 737.1	13 10	33	78													
		31 – 32	7 11 13	35	78	SS-20	-	60	20	12	7	1	16	NP	NP	11	A-1-a (0)	
	734.1	33																
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, WET		34 – 35	8 10 12	32	100	SS-21	-	-	-	-	-	-	-	-	-	12	A-1-b (V)	
		36 – 37	4 5 10	22	100	SS-22	-	41	37	16	5	1	13	NP	NP	14	A-1-b (0)	
@ 38.5' : 6 inches of Sand Heave. Begin Using Betonite Slurry in Drilling.		38 – 40	5 5 7	17	100	SS-23	-	-	-	-	-	-	-	-	-	15	A-1-b (V)	
	727.1																	
MEDIUM DENSE, BROWN, **GRAVEL**, "AND" SAND, TRACE CLAY, (ROUNDED TO SUB ROUNDED GRAVEL), WET		41 – 42	9 8 9	25	78	SS-24	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
		43																
		44 – 45	6 5 5	14	44	SS-25	-	56	42	1	0	1	13	NP	NP	13	A-1-a (0)	
		46 – 47	6 6 5	16	44	SS-26	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
		48 – 49	11 9 8	25	56	SS-27	-	-	-	-	-	-	-	-	-	14	A-1-a (V)	
	718.1	EOB 50																

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 75 LB. BENTONITE CHIPS; TREMIED 25 LB. BENTONITE POWDER; 94 LB. CEMENT; PLACED 25 LB. QUICKCRETE; 55 GAL. WATER

PROJECT: MOT-4-19.30	DRILLING FIRM / OPERATOR: CENTRAL STAR / TS	DRILL RIG: DIEDRICH D-50 TRACK	STATION / OFFSET: 89+60, 64' LT.	EXPLORATION ID B-004-2-23
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: HDR / DCM	HAMMER: AUTOMATIC HAMMER	ALIGNMENT: SR 4	
PID: 117239 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 3/7/22	ELEVATION: 771.5 (MSL) EOB: 46.5 ft.	PAGE 1 OF 2
START: 5/19/23 END: 5/19/23	SAMPLING METHOD: SPT	ENERGY RATIO (%): 86.8	LAT / LONG: 39.780079, -84.160106	

MATERIAL DESCRIPTION AND NOTES	ELEV. 771.5
10 inches Asphalt 8 inches Reinforced Concrete	770.0
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP (FILL) @ 5.5' - 6.5' : Trace Gray	761.0
DENSE TO VERY DENSE, GRAY, BROWN AND BLACK, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, NOTED ASPHALT, DAMP (FILL - Refuse Material)	757.5
VERY STIFF TO HARD, BLACK TO DARK GRAY, **SILTY CLAY**, LITTLE SAND, TRACE GRAVEL, DAMP (FILL - Refuse Material) @ 15.0' : Water Added to Assist Drilling	756.0
DENSE, BLACK, **GRAVEL**, "AND" SAND, TRACE SILT, TRACE CLAY, NOTED ASPHALT AND METAL FRAGMENTS, MOIST (FILL - Refuse Material)	754.5
MEDIUM DENSE TO DENSE, BLACK, **GRAVEL WITH SAND AND SILT**, TRACE CLAY, NOTED ASPHALT AND METAL FRAGMENTS, WET (FILL - Refuse Material)	750.5
DENSE, BROWN AND BLACK, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, NOTED GLASS AND ASPHALT, DAMP (FILL - Refuse Material) @ 21.0' - 21.5' : Wet	747.2
HARD, BLACK, **SANDY SILT**, SOME CLAY, LITTLE GRAVEL, NOTED ASPHALT, DAMP TO MOIST (FILL - Refuse Material)	746.0
DENSE TO VERY DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP	

SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
6 7 7	20	56	SS-1	-	-	-	-	-	-	-	-	-	11	A-1-a (V)
7 8 11	27	56	SS-2	-	63	27	3	5	2	21	16	5	9	A-1-a (0)
7 10 13	33	78	SS-3	-	-	-	-	-	-	-	-	-	6	A-1-a (V)
13 13 8	30	56	SS-4	-	-	-	-	-	-	-	-	-	6	A-1-a (V)
13 11 11	32	67	SS-5	-	-	-	-	-	-	-	-	-	4	A-1-a (V)
13 17 19	52	78	SS-6	-	-	-	-	-	-	-	-	-	7	A-1-a (V)
8 20 12	46	89	SS-7	-	60	22	6	8	4	23	18	5	7	A-1-a (0)
21 23 17	58	100	SS-8	-	-	-	-	-	-	-	-	-	8	A-1-a (V)
7 8 13	30	89	SS-9	4.50	-	-	-	-	-	-	-	-	17	A-6b (V)
16 18 11	42	89	SS-10	-	51	30	9	7	3	27	NP	NP	14	A-1-a (0)
9 8 10	26	100	SS-11	-	40	28	11	15	6	31	24	7	19	A-2-4 (0)
8 3 10	19	17	SS-12	-	-	-	-	-	-	-	-	-	24	A-2-4 (V)
11 13 15	41	89	SS-13A	-	-	-	-	-	-	-	-	-	14	A-2-4 (V)
			SS-13B	-	-	-	-	-	-	-	-	-	35	A-1-b (V)
6 12 12	35	44	SS-14	-	-	-	-	-	-	-	-	-	10	A-1-b (V)
8 10 17	39	89	SS-15	-	68	10	6	13	3	22	17	5	11	A-1-b (0)
6 10 15	36	100	SS-16	-	14	9	7	37	33	35	25	10	25	A-4a (7)
10 14 21	51	67	SS-17	-	-	-	-	-	-	-	-	-	8	A-1-a (V)
27 12 10	32	78	SS-18	-	55	26	6	10	3	22	17	5	7	A-1-a (0)

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 8/16/23 13:51 - C:\PWWORKING\EAST01\D3330003\20230514_MOT-4-19.30_BORING LOGS - REV 1_WITH LAB DATA.GPJ

PID: 117239	SFN:	PROJECT: MOT-4-19.30	STATION / OFFSET: 89+60, 64' LT.	START: 5/19/23	END: 5/19/23	PG 2 OF 2	B-004-2-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 741.5	DEPTHS	SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
DENSE TO VERY DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP *(continued)*	739.0	31, 32	50/5"	-	60	SS-19	-	-	-	-	-	-	-	-	-	12	A-1-a (V)	
VERY DENSE, BROWN AND GRAY-BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP @ 32.5' : Water Added to Assist Drilling		33, 34	14 15 28	62	89	SS-20	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
	734.0	35, 36, 37	13 26 20	67	83	SS-21	-	65	15	12	6	2	13	NP	NP	10	A-1-a (0)	
VERY DENSE, BROWN, **GRAVEL**, "AND" SAND, TRACE CLAY, MOIST TO WET @ 37.5' : Water Added to Assist Drilling		38, 39	12 19 28	68	100	SS-22	-	-	-	-	-	-	-	-	-	11	A-1-a (V)	
		40, 41, 42	8 26 21	68	100	SS-23	-	53	43	2	0	2	14	11	3	10	A-1-a (0)	
@ 42.5' : 1-foot of Sand Heave		43, 44	12 21 34	80	100	SS-24	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
@ 45.0' : 2-feet of Sand Heave	725.0	45, 46 EOB	8 17 17	49	100	SS-25	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 100 LB. BENTONITE CHIPS; TREMIED 25 LB. BENTONITE POWDER; 94 LB. CEMENT; PLACED 50 LB. QUICKCRETE; 55 GAL. WATER

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 8/16/23 12:59 - C:\PWWORKING\EAST01\D3330003\20230514_MOT-4-19.30_BORING LOGS - REV 1_WITH LAB DATA.GPJ

PROJECT: MOT-4-19.30	DRILLING FIRM / OPERATOR: CENTRAL STAR / TS	DRILL RIG: DIEDRICH D-50 TRACK	STATION / OFFSET: 91+52, -26' LT.	EXPLORATION ID B-008-0-23
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: HDR / DCM	HAMMER: AUTOMATIC HAMMER	ALIGNMENT: SR 4	
PID: 117239 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 3/7/22	ELEVATION: 768.1 (MSL) EOB: 41.5 ft.	PAGE 1 OF 2
START: 5/22/23 END: 5/22/23	SAMPLING METHOD: SPT	ENERGY RATIO (%): 86.8	LAT / LONG: 39.780288, -84.159458	

MATERIAL DESCRIPTION AND NOTES	ELEV. 768.1
13 inches Asphalt	767.0
DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP (FILL)	762.6
VERY DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP (FILL)	759.1
DENSE, DARK BROWN, BLACK, AND GRAY, **GRAVEL WITH SAND AND SILT**, TRACE CLAY, DAMP (FILL - Refuse Material)	756.6
LOOSE TO MEDIUM DENSE, BLACK, **SANDY SILT**, SOME GRAVEL, TRACE CLAY, NOTED CINDERS, ASPHALT, GLASS, METAL, ELECTRIC PLUG, MOIST (FILL - Refuse Material) @ 15.5' - 17.0' : Organic Odor	751.1
STIFF TO VERY STIFF, BLACK, **SANDY SILT**, LITTLE CLAY, LITTLE GRAVEL, NOTED GLASS, METAL, DAMP (FILL - Refuse Material)	746.6
DENSE, BROWN, **GRAVEL**, LITTLE SAND, TRACE SILT, TRACE CLAY, DAMP	742.6
DENSE TO VERY DENSE, BROWN, **GRAVEL**, "AND" SAND, TRACE SILT, TRACE CLAY, DAMP	

DEPTHS: 1 – 29; W 738.1

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
					GRADATION (%)					ATTERBERG					
8 14 11	36	89	SS-1	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
2 14 14	41	89	SS-2	-	66	16	7	10	1	19	15	4	9	A-1-a (0)	
17 20 30	72	83	SS-3	-	-	-	-	-	-	-	-	-	5	A-1-a (V)	
27 30 37	97	100	SS-4	-	63	26	4	5	2	17	NP	NP	6	A-1-a (0)	
17 24 22	67	100	SS-5	-	-	-	-	-	-	-	-	-	7	A-1-a (V)	
10 18 14	46	100	SS-6	-	42	16	14	19	9	29	20	9	12	A-2-4 (0)	
4 6 13	27	89	SS-7A	-	-	-	-	-	-	-	-	-	13	A-2-4 (V)	
			SS-7B	-	31	28	5	33	3	25	NP	NP	19	A-4a (0)	
6 4 3	10	56	SS-8	-	-	-	-	-	-	-	-	-	16	A-4a (V)	
3 4 3	10	0	SS-9	-	-	-	-	-	-	-	-	-	27	A-4a (V)	
2 3 2	7	89	SS-10	-	-	-	-	-	-	-	-	-	24	A-4a (V)	
2 1 4	7	44	SS-11	-	-	-	-	-	-	-	-	-	30	A-4a (V)	
3 6 6	17	67	SS-12	2.50	16	13	23	29	19	32	26	6	21	A-4a (3)	
3 4 8	17	33	SS-13	1.00	-	-	-	-	-	-	-	-	20	A-4a (V)	
11 12 11	33	67	SS-14	-	-	-	-	-	-	-	-	-	4	A-1-a (V)	
7 11 12	33	89	SS-15	-	70	14	6	7	3	15	18	NP	5	A-1-a (0)	
12 17 22	56	78	SS-16	-	-	-	-	-	-	-	-	-	4	A-1-a (V)	
7 12 11	33	67	SS-17	-	-	-	-	-	-	-	-	-	5	A-1-a (V)	

PID: 117239	SFN:	PROJECT: MOT-4-19.30	STATION / OFFSET: 91+52, -26' LT.	START: 5/22/23	END: 5/22/23	PG 2 OF 2	B-008-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 738.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
DENSE TO VERY DENSE, BROWN, **GRAVEL**, "AND" SAND, TRACE SILT, TRACE CLAY, DAMP *(continued)* Below 30' : Wet		31	9 11 15	38	78	SS-18	-	57	26	10	5	2	13	NP	NP	9	A-1-a (0)	
		32																
		33	7 12 13	36	89	SS-19	-	-	-	-	-	-	-	-	-	11	A-1-a (V)	
		34																
		35																
		36	7 16 28	64	89	SS-20	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	
		37																
		38	4 14 16	43	67	SS-21	-	-	-	-	-	-	-	-	-	12	A-1-a (V)	
	728.1	39																
HARD, GRAY, **SANDY SILT**, SOME CLAY, LITTLE GRAVEL, DAMP (Glacial Till)		40																
	726.6	41 EOB	19 19 27	67	89	SS-22	4.50	16	8	17	36	23	18	12	6	10	A-4a (5)	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 100 LB. BENTONITE CHIPS; TREMIED 25 LB. BENTONITE POWDER; 94 LB. CEMENT; PLACED 25 LB. QUICKCRETE; 55 GAL. WATER

PROJECT: MOT-4-19.30	DRILLING FIRM / OPERATOR: CENTRAL STAR / TS	DRILL RIG: DIEDRICH D-50 TRACK	STATION / OFFSET: 92+90, 59' RT.	EXPLORATION ID B-009-0-23
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: HDR / DCM	HAMMER: AUTOMATIC HAMMER	ALIGNMENT: SR 4	
PID: 117239 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 3/7/22	ELEVATION: 764.2 (MSL) EOB: 43.0 ft.	PAGE 1 OF 2
START: 5/16/23 END: 5/16/23	SAMPLING METHOD: SPT	ENERGY RATIO (%): 86.8	LAT / LONG: 39.780282, -84.158882	

MATERIAL DESCRIPTION AND NOTES	ELEV. 764.2	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
17 inches Asphalt	762.8	1																
MEDIUM DENSE, BROWN, **GRAVEL**, LITTLE SAND, TRACE SILT, TRACE CLAY, DAMP (FILL)		2	7-8-9	25	67	SS-1	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
		3 4	5-9-11	29	67	SS-2	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
Below 4.5' : Very Dense		5	10-14-21	51	56	SS-3	-	73	11	6	7	3	18	16	2	9	A-1-a (0)	
	756.2	6 7	15-19-20	56	100	SS-4	-	-	-	-	-	-	-	-	-	7	A-1-a (V)	
MEDIUM DENSE TO DENSE, BLACK AND DARK BROWN, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, WITH GLASS, METAL, ASPHALT FRAGMENTS, PLASTIC, DAMP (FILL - Refuse Material)		8	22-17-12	42	78	SS-5	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
		9 10	4-10-10	29	67	SS-6	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	
		11	7-5-10	22	0	SS-7	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
	750.7	12 13	8-5-4	13	67	SS-8	-	58	9	15	15	3	28	26	2	10	A-1-b (0)	
LOOSE, BLACK, **SANDY SILT**, SOME GRAVEL, TRACE CLAY, WITH GLASS, METAL, ASPHALT FRAGMENTS, PLASTIC, WET (FILL - Refuse Material)		14	1-2-4	9	17	SS-9	-	33	16	15	- 36 -		NP	NP	NP	39	A-4a (0)	
		15 16	2-3-4	10	11	SS-10	-	-	-	-	-	-	-	-	-	34	A-4a (V)	
		17	4-2-3	7	6	SS-11	-	-	-	-	-	-	-	-	-	24	A-4a (V)	
		18 19	3-5-2	10	6	SS-12	-	-	-	-	-	-	-	-	-	47	A-4a (V)	
	743.2	20	3-2-3	7	6	SS-13	-	-	-	-	-	-	-	-	-	17	A-4a (V)	
DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP		21 22	10-13-13	38	44	SS-14	-	-	-	-	-	-	-	-	-	6	A-1-a (V)	
		23																
		24 25	13-13-8	30	78	SS-15	-	-	-	-	-	-	-	-	-	4	A-1-a (V)	
		26																
		27	13-14-10	35	89	SS-16	-	59	23	10	7	1	15	NP	NP	8	A-1-a (0)	
		28																
		29	13-10-12	32	100	SS-17	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 8/16/23 12:59 - C:\PWWORKING\EAST01\D3330003\20230514_MOT-4-19.30_BORING LOGS - REV 1_WITH LAB DATA.GPJ

PID: 117239	SFN:	PROJECT: MOT-4-19.30	STATION / OFFSET: 92+90, 59' RT.	START: 5/16/23	END: 5/16/23	PG 2 OF 2	B-009-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 734.2	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, DAMP *(continued)* Below 30.0' : Water/Bentonite Slurry Added to Assist Drilling and Prevent Heave	730.7	31–32	13 12 11	33	89	SS-18	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
MEDIUM DENSE, BROWN, **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, MOIST	728.2	34–35	9 9 9	26	100	SS-19	-	16	42	32	8	2	12	NP	NP	16	A-1-b (0)	
STIFF, GRAY, **SANDY SILT**, SOME GRAVEL, TRACE CLAY, DAMP		36–37	6 8 12	29	100	SS-20	-	-	-	-	-	-	-	-	-	12	A-4a (V)	
		39–40	2 2 5	10	89	SS-21	-	25	14	21	32	8	17	11	6	11	A-4a (1)	
@ 41.0' - 43.0' : Sample Slipped Out of Shelby Tube (Disturbed). Sample Jarred for Testing.	721.2	41–43 EOB			50	ST-22	-	30	13	21	26	10	15	10	5	11	A-4a (0)	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 100 LB. BENTONITE CHIPS; TREMIED 25 LB. BENTONITE POWDER; 94 LB. CEMENT; PLACED 50 LB. QUICKCRETE; 55 GAL. WATER

PROJECT: MOT-4-19.30	DRILLING FIRM / OPERATOR: CENTRAL STAR / TS	DRILL RIG: DIEDRICH D-50 TRACK	STATION / OFFSET: 8+21, 14' RT.	EXPLORATION ID B-010-0-23
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: HDR / DCM	HAMMER: AUTOMATIC HAMMER	ALIGNMENT: RAMP K	
PID: 117239 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 3/7/22	ELEVATION: 763.0 (MSL) EOB: 45.3 ft.	PAGE 1 OF 2
START: 5/17/23 END: 5/17/23	SAMPLING METHOD: SPT	ENERGY RATIO (%): 86.8	LAT / LONG: 39.779882, -84.158816	

MATERIAL DESCRIPTION AND NOTES	ELEV. 763.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
30 inches Asphalt 12 inches Concrete	759.5	1 2 3																
DENSE, BROWN, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, DAMP (FILL)	758.0	4	38 12 14	38	78	SS-1	-	38	29	15	13	5	12	NP	NP	7	A-1-b (0)	
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL**, SOME SAND, LITTLE SILT, TRACE CLAY, DAMP (FILL)		5 6	12 11 28	56	78	SS-2	-	58	20	8	11	3	21	19	2	7	A-1-a (0)	
	755.5	7	8 9 6	22	100	SS-3A	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
VERY DENSE, BLACK TO DARK BROWN, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, NOTED GLASS BRICK, AND ASPHALT, DAMP (FILL - Refuse Material)						SS-3B	-	-	-	-	-	-	-	-	-	21	A-1-b (V)	
	753.5	8 9	8 14 21	51	78	SS-4	-	57	14	9	16	4	23	17	6	9	A-1-b (0)	
LOOSE TO MEDIUM DENSE, DARK BROWN, **GRAVEL**, SOME SAND, TRACE SILT, TRACE CLAY, NOTED GLASS AND ASPHALT, MOIST (FILL - Refuse Material)		10	11 8 6	20	100	SS-5	-	-	-	-	-	-	-	-	-	13	A-1-a (V)	
		11 12	5 2 1	4	0	SS-6	-	-	-	-	-	-	-	-	-	-	A-1-a (V)	
		13	1 0 3	4	44	SS-7	-	60	23	10	6	1	38	NP	NP	18	A-1-a (0)	
@ 14.0' - 15.5' : Cinders	747.5	14 15	4 1 1	3	33	SS-8	-	-	-	-	-	-	-	-	-	15	A-1-a (V)	
LOOSE, DARK BROWN, ORANGE, AND BLACK, **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, NOTED ASPHALT, GLASS, WOOD, METAL WIRE, AND PETROLEUM ODOR, WET (FILL - Refuse Material)		16	3 1 2	4	44	SS-9	-	-	-	-	-	-	-	-	-	22	A-1-b (V)	
		17 18	2 1 4	7	78	SS-10	-	34	50	9	5	2	24	NP	NP	17	A-1-b (0)	
@ 18.5' - 20.0' : Missed Sample (Over-Augered and Miscounted Tooling)		19 20																
	741.5	21	2 1 2	4	17	SS-11	-	-	-	-	-	-	-	-	-	20	A-1-b (V)	
MEDIUM DENSE TO DENSE, DARK BROWN, ORANGE, AND BLACK, **GRAVEL WITH SAND AND SILT**, TRACE CLAY, NOTED ASPHALT, GLASS, WOOD, METAL WIRE, AND PETROLEUM ODOR, MOIST TO WET (FILL - Refuse Material)		22 23	3 3 3	9	67	SS-12	-	48	42	7	0	3	38	29	9	22	A-2-4 (0)	
	738.0	24	8 12 20	46	22	SS-13	-	-	-	-	-	-	-	-	-	16	A-2-4 (V)	
LOOSE TO MEDIUM DENSE, BROWN, **GRAVEL**, "AND" SAND, TRACE SILT, TRACE CLAY, MOIST TO WET		25	11 8 6	20	44	SS-14	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
		W 736.0 26 27																
		28	2 3 3	9	67	SS-15	-	59	29	9	2	1	14	NP	NP	13	A-1-a (0)	
		29																
			5															

STANDARD ODOT SOIL BORING LOG (8.5 X 11) - OH DOT.GDT - 8/16/23 12:59 - C:\PWWORKING\EAST01\D3330003\20230514_MOT-4-19.30_BORING LOGS - REV 1_WITH LAB DATA.GPJ

PID: 117239	SFN:	PROJECT: MOT-4-19.30	STATION / OFFSET: 8+21, 14' RT.	START: 5/17/23	END: 5/17/23	PG 2 OF 2	B-010-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 733.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
LOOSE TO MEDIUM DENSE, BROWN, **GRAVEL**, "AND" SAND, TRACE SILT, TRACE CLAY, MOIST TO WET *(continued)*	730.5	31, 32	5 5	14	78	SS-16	-	-	-	-	-	-	-	-	-	11	A-1-a (V)	
STIFF TO VERY STIFF, GRAY, **SANDY SILT**, LITTLE CLAY, LITTLE GRAVEL, MOIST @ 34.5' - 35.0' : Attempted Shelby Tube. Pushed 6 inches to Refusal. Zero Recovery.	728.0	33, 34, 35	4 5 9	20	67	SS-17	-	14	9	19	38	20	16	11	5	12	A-4a (5)	
MEDIUM DENSE, BROWN, **GRAVEL WITH SAND**, LITTLE SILT, LITTLE CLAY, WET	726.0	36, 37	8 7 6	19	44	SS-18	-	71	4	2	- 23 -		NP	NP	NP	9	A-1-b (0)	
MEDIUM DENSE TO DENSE, BROWN TO GRAYISH-BROWN, **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, WET		38, 39	9 8 5	19	44	SS-19	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	
		40, 41, 42	4 3 4	10	67	SS-20	-	48	48	2	1	1	14	NP	NP	9	A-1-b (0)	
		43, 44	6 6 19	36	33	SS-21	-	-	-	-	-	-	-	-	-	22	A-1-b (V)	
	717.7	EOB 45	40 50/3"	-	89	SS-22	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: POURED 200 LB. BENTONITE CHIPS; TREMIED 25 LB. BENTONITE POWDER; 94 LB. CEMENT; PLACED 25 LB. QUICKCRETE; 55 GAL. WATER

PROJECT: MOT-4-19.30	DRILLING FIRM / OPERATOR: CENTRAL STAR / TS	DRILL RIG: DIEDRICH D-50 TRACK	STATION / OFFSET: 96+04, 65' RT.	EXPLORATION ID B-011-0-23
TYPE: ROADWAY	SAMPLING FIRM / LOGGER: HDR / DCM	HAMMER: AUTOMATIC HAMMER	ALIGNMENT: SR 4	
PID: 117239 SFN:	DRILLING METHOD: 3.25" HSA	CALIBRATION DATE: 3/7/22	ELEVATION: 761.5 (MSL) EOB: 34.0 ft.	PAGE 1 OF 2
START: 5/17/23 END: 5/17/23	SAMPLING METHOD: SPT	ENERGY RATIO (%): 86.8	LAT / LONG: 39.780667, -84.157898	

MATERIAL DESCRIPTION AND NOTES	ELEV. 761.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
12 inches Asphalt	760.5																	
DENSE TO VERY DENSE, BROWN, **GRAVEL**, LITTLE SAND, TRACE SILT, TRACE CLAY, DAMP (FILL)		1–2	10 14 16	43	89	SS-1	-	-	-	-	-	-	-	-	-	7	A-1-a (V)	
		3	12 13 24	54	83	SS-2	-	63	25	5	5	2	18	15	3	7	A-1-a (0)	
		4–5	24 27 28	80	100	SS-3	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	
		6	14 17 26	62	78	SS-4	-	-	-	-	-	-	-	-	-	5	A-1-a (V)	
@ 7.0' - 10.0' : Wet / Saturated		7–8	15 22 17	56	44	SS-5	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	
	751.5	9	16 18 37	80	44	SS-6	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	
MEDIUM DENSE TO DENSE, BLACK, **GRAVEL WITH SAND**, LITTLE SILT, TRACE CLAY, WITH ASPHALT FRAGMENTS, MOIST TO WET (FILL - Refuse Material)		10–11	15 21 25	67	89	SS-7	-	-	-	-	-	-	-	-	-	12	A-1-b (V)	
	748.0	12	6 7 9	23	89	SS-8	-	53	22	7	13	5	26	21	5	16	A-1-b (0)	
DENSE TO VERY DENSE, BROWN, **GRAVEL**, SOME SAND, TRACE CLAY, TRACE SILT, DAMP		13–14	12 12 12	35	89	SS-9	-	-	-	-	-	-	-	-	-	4	A-1-a (V)	
		15	11 15 15	43	89	SS-10	-	64	19	7	1	9	16	NP	NP	5	A-1-a (0)	
		16–17	11 14 17	45	78	SS-11	-	-	-	-	-	-	-	-	-	4	A-1-a (V)	
		18	12 18 22	58	78	SS-12	-	-	-	-	-	-	-	-	-	6	A-1-a (V)	
		19–20																
		21	11 17 9	38	78	SS-13	-	-	-	-	-	-	-	-	-	5	A-1-a (V)	
	739.0	W 739.0 22																
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL**, "AND" SAND, TRACE SILT, WET @ 23.0' - 24.0' : Orange-Brown		23	10 9 12	30	78	SS-14	-	-	-	-	-	-	-	-	-	11	A-1-a (V)	
		24–25																
		26	6 6 10	23	78	SS-15	-	55	43	1	1	0	13	NP	NP	13	A-1-a (0)	
		27																
		28–29	11 16 17	48	78	SS-16	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	

PID: 117239	SFN:	PROJECT: MOT-4-19.30	STATION / OFFSET: 96+04, 65' RT.	START: 5/17/23	END: 5/17/23	PG 2 OF 2	B-011-0-23

MATERIAL DESCRIPTION AND NOTES	ELEV. 731.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL**, "AND" SAND, TRACE SILT, WET *(continued)* @ 31.0' - 31.5' : Orange Brown	729.0	31 – 32	14 16 13	42	89	SS-17	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
DENSE, BROWN, **COARSE AND FINE SAND**, LITTLE SILT, TRACE CLAY, TRACE GRAVEL, WET	727.5	33 – 34 EOB	11 15 13	41	78	SS-18	-	1	11	75	12	1	13	NP	NP	20	A-3a (0)	

NOTES: NONE

ABANDONMENT METHODS, MATERIALS, QUANTITIES: TREMIED 25 LB. BENTONITE POWDER; 94 LB. CEMENT; PLACED 25 LB. QUICKCRETE; 55 GAL. WATER

B-001-1-23

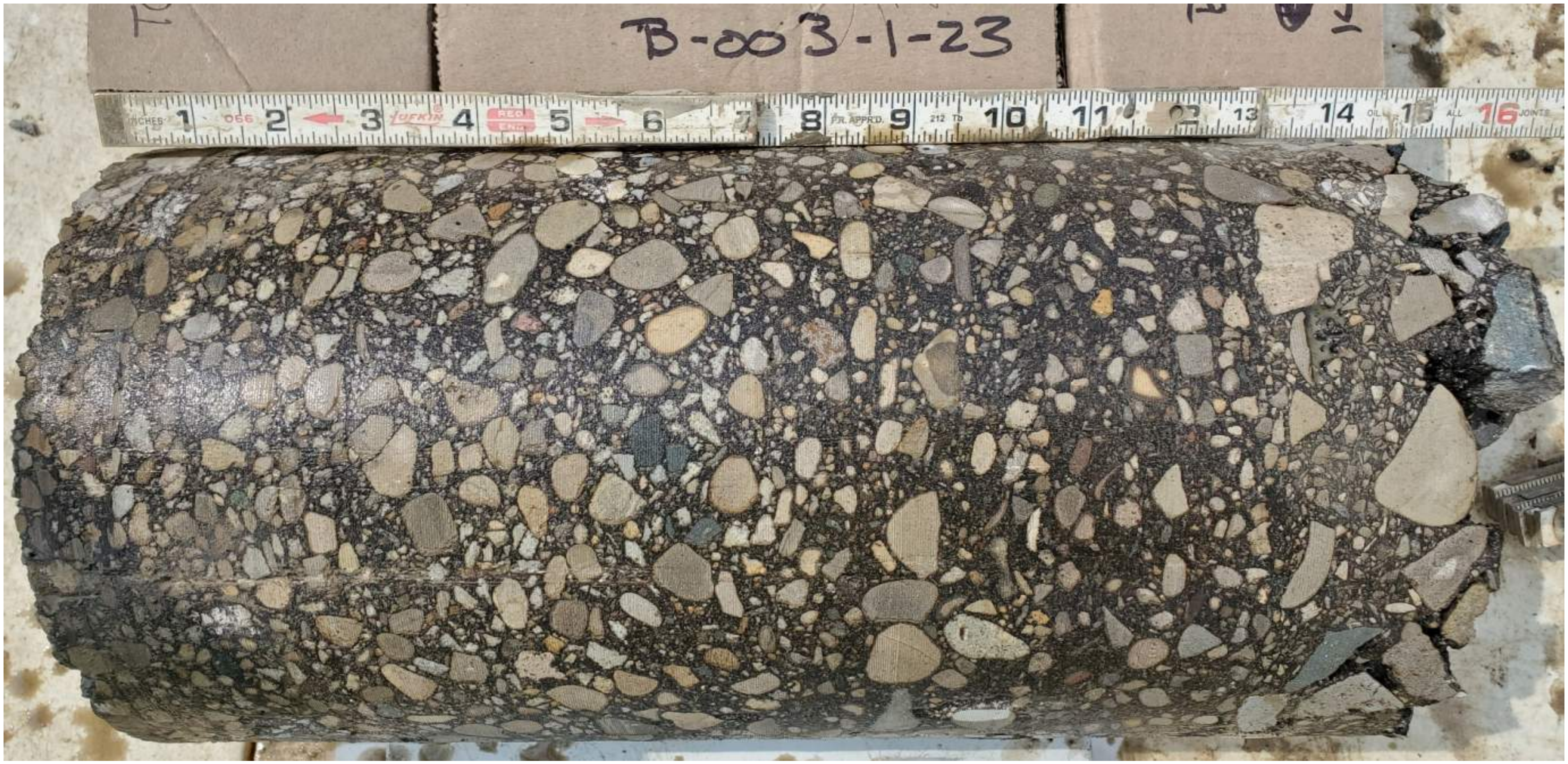
Top		Bottom

Asphalt 12 inches

Total Thickness: 12 inches

MOT-4-19.30 PID 117239

B-003-1-23



Top	Bottom

Asphalt 14 inches

Total Thickness: 14 inches

B-004-1-23

Top | Bottom

Asphalt 27 inches

Total Thickness: 27 inches

MOT-4-19.30 PID 117239

B-004-2-23

Top				Bottom
Asphalt	10 inches	**Portland Cement**	8 inches	

Total Thickness: 18 inches

MOT-4-19.30 PID 117239

B-008-0-23

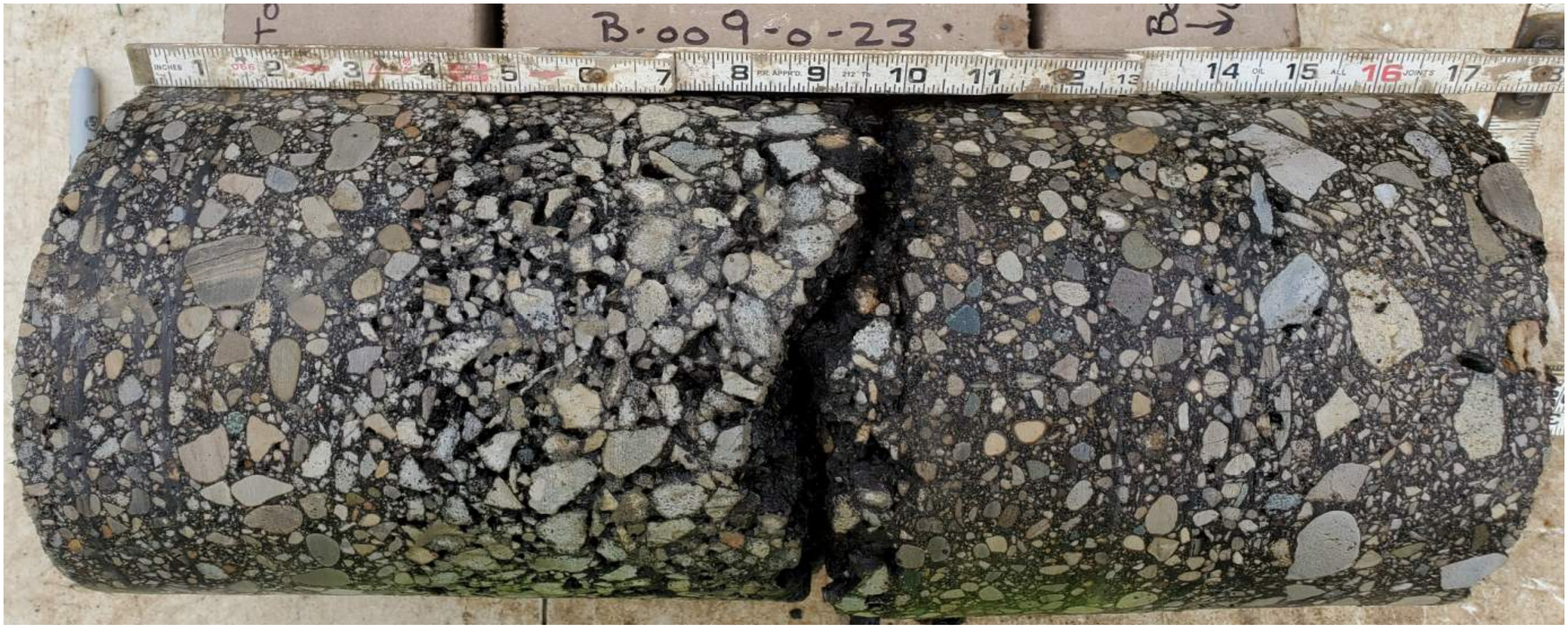
Top

Bottom

Asphalt 13 inches

Total Thickness: 13 inches

B-009-0-23



Top		Bottom

Asphalt 17 inches

Total Thickness: 17 inches

B-011-0-23

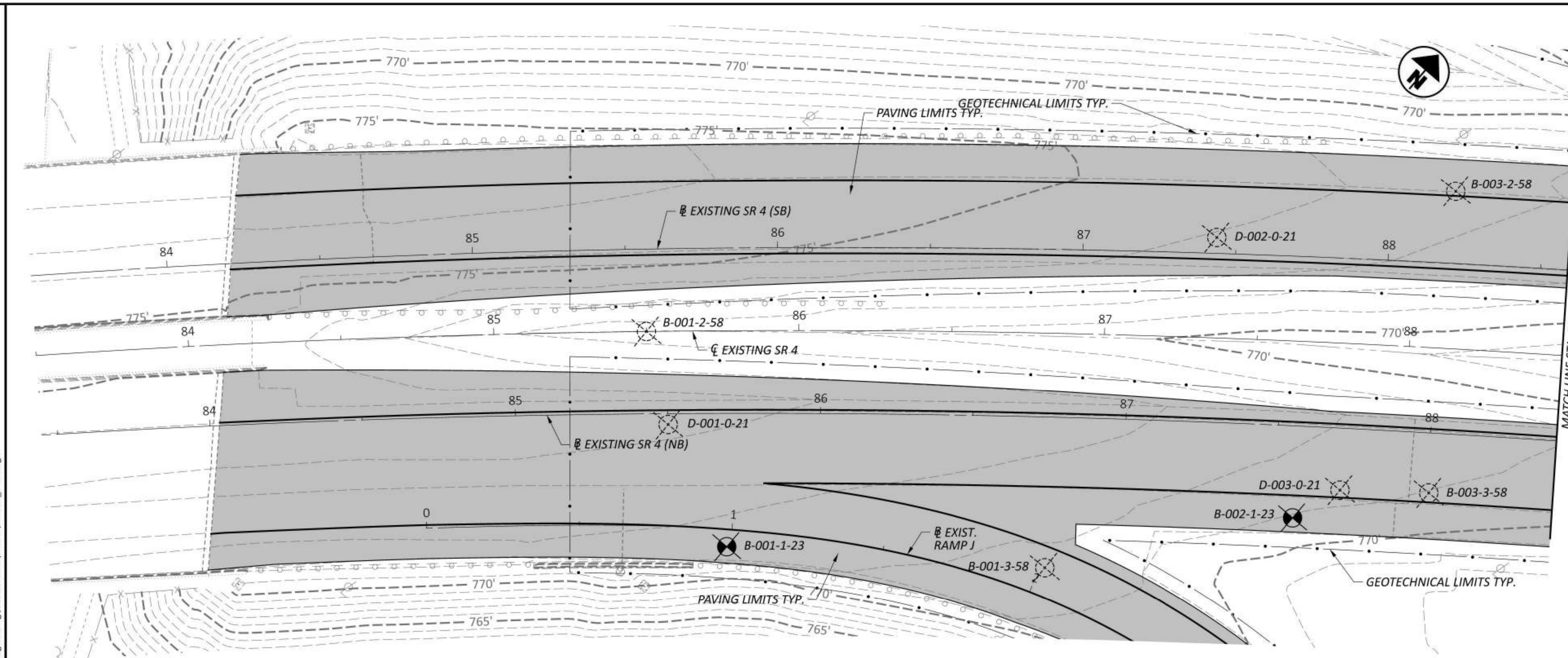
Top	Bottom

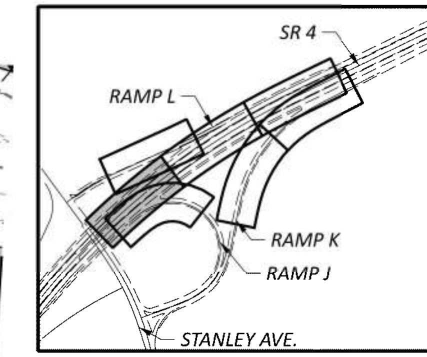
Asphalt 12 inches

Total Thickness: 12 inches

MOT-4-19.30 PID 117239

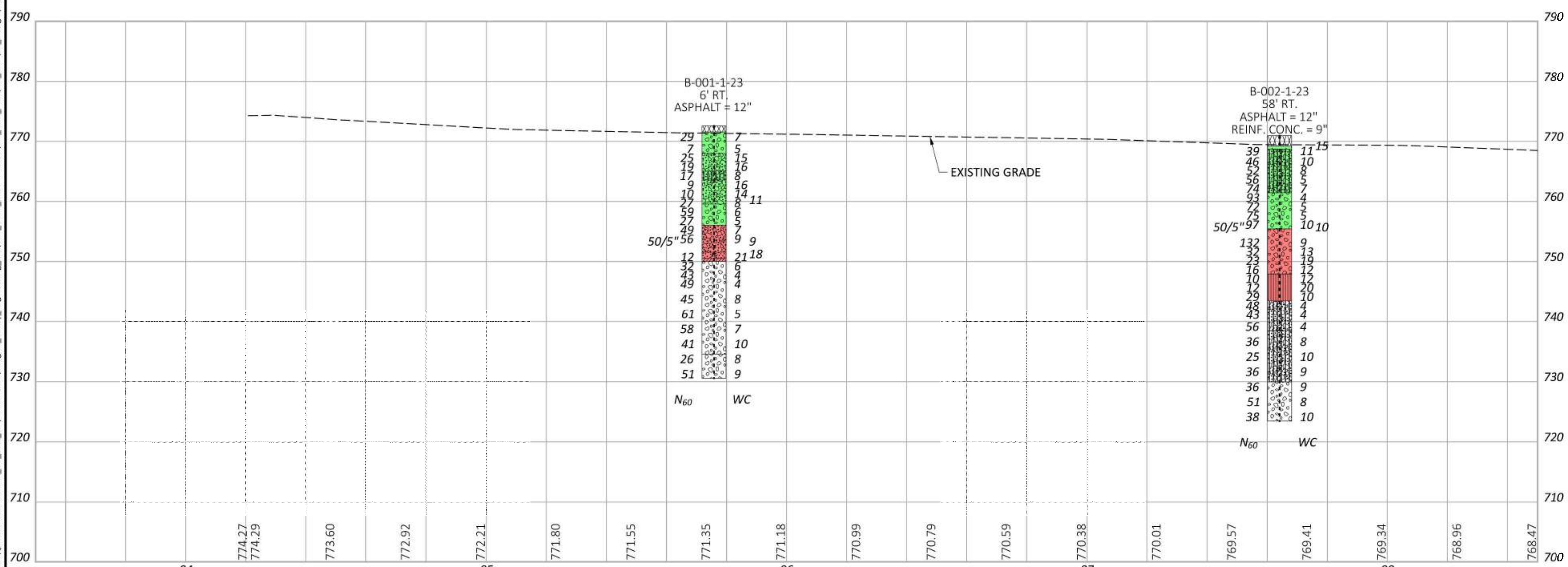
Appendix E. Geotechnical Plan and Profile Sheets





KEY MAP

HORIZONTAL SCALE IN FEET
0 10 20 40



LEGEND

GRANULAR FILL

REFUSE MATERIAL

GEOTECHNICAL PROFILE - ROADWAY
SR4 - STA. 83+50 TO STA. 88+50

DESIGN AGENCY

HDR

DESIGNER
DCM

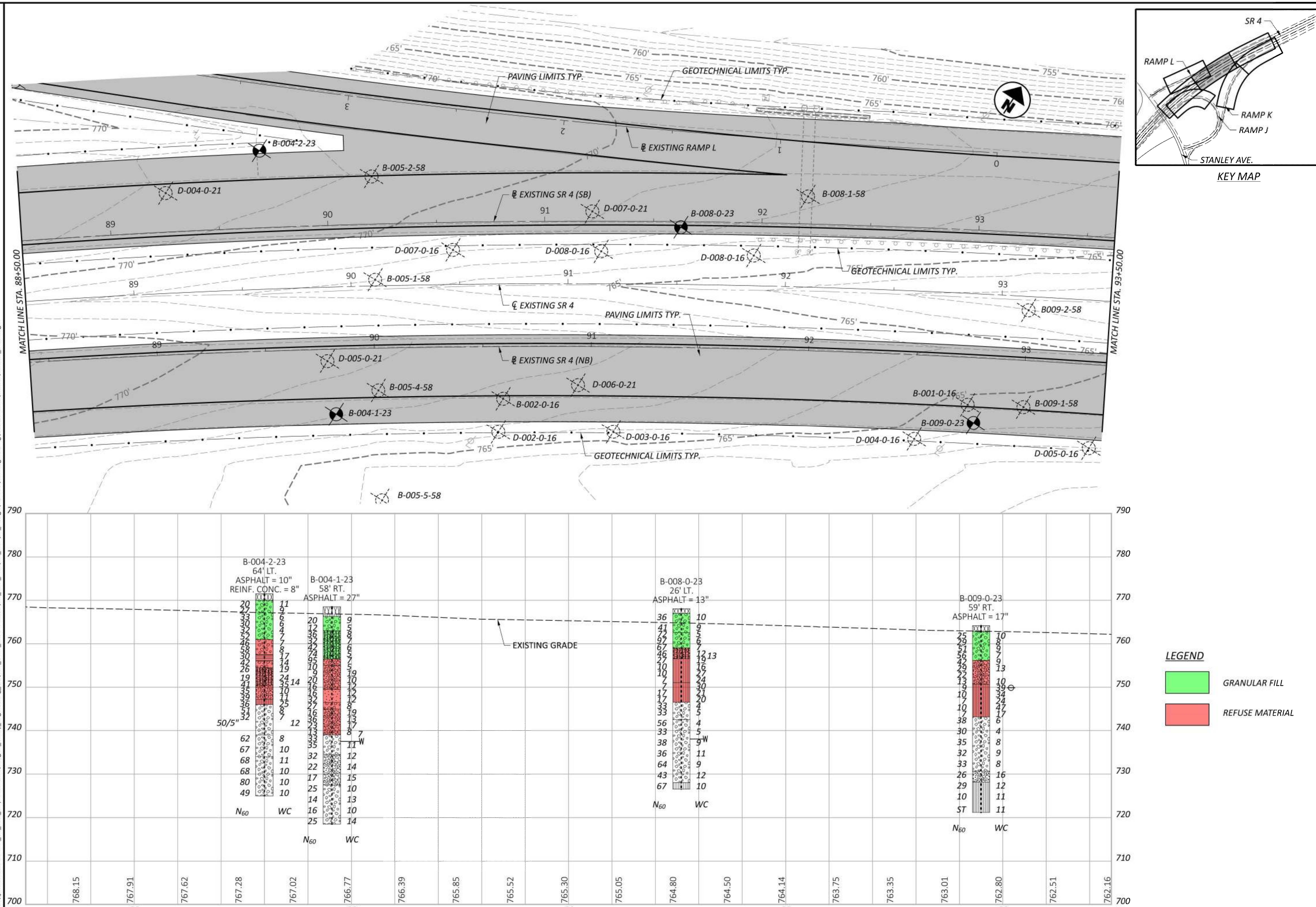
REVIEWER
DVM 10/12/2023

PROJECT ID
117239

SHEET P.1 TOTAL 7

MOT-4-19.30

MODEL: 117239_IP001 PAPERSIZE: 34x22 (in.) DATE: 10/13/2023 TIME: 3:55:46 PM USER: DMATCHIS
pw:\\pwhdruseast01:HDR_US_East_01\Documents\Chagrin_Valley_Engineering_Ltd\CVE_ODT_MOT-4-19.30\6.0_CAD_BIM\6.2_WIP\01_Design\117239\400 Engineering\Geotechnical\Sheets\117239_IP001.dgn



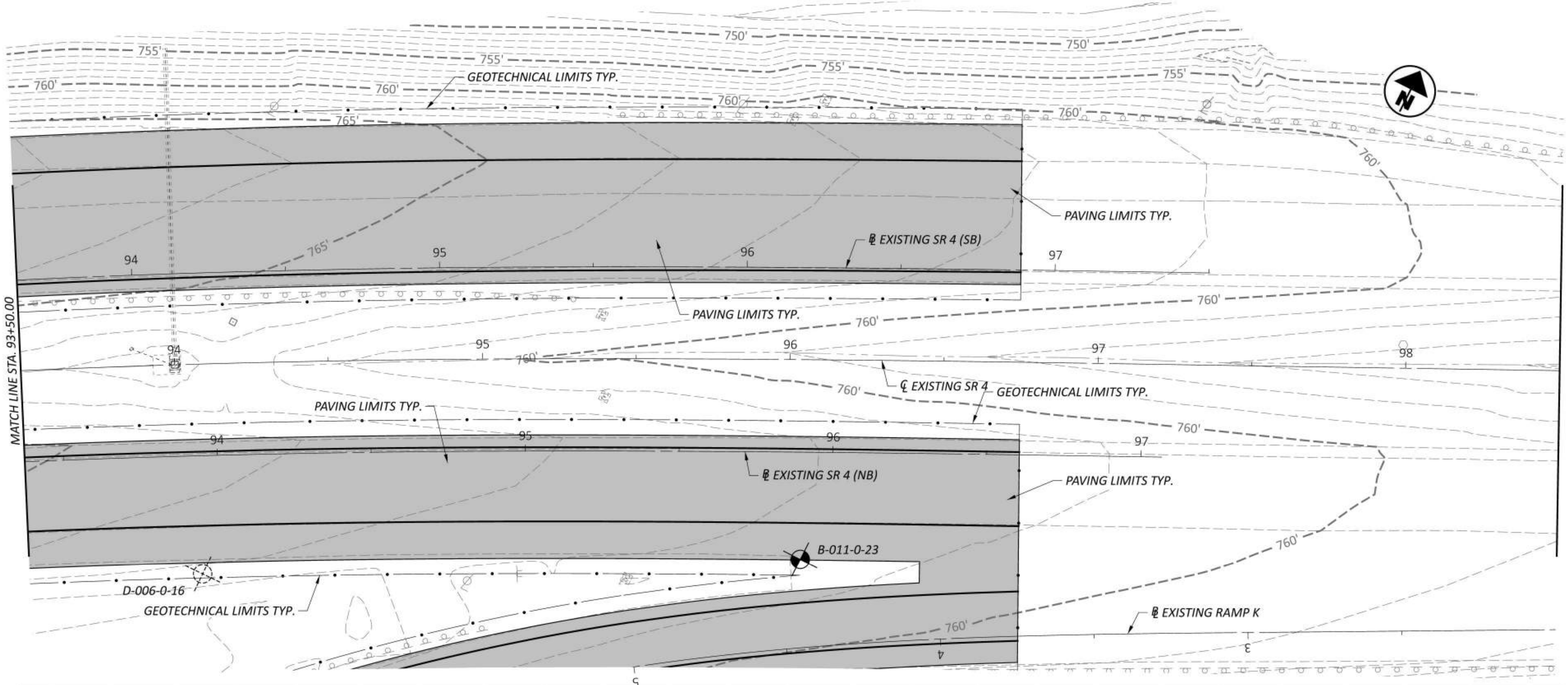
GEOTECHNICAL PROFILE - ROADWAY
SR4 - STA. 88+50 TO STA. 93+50

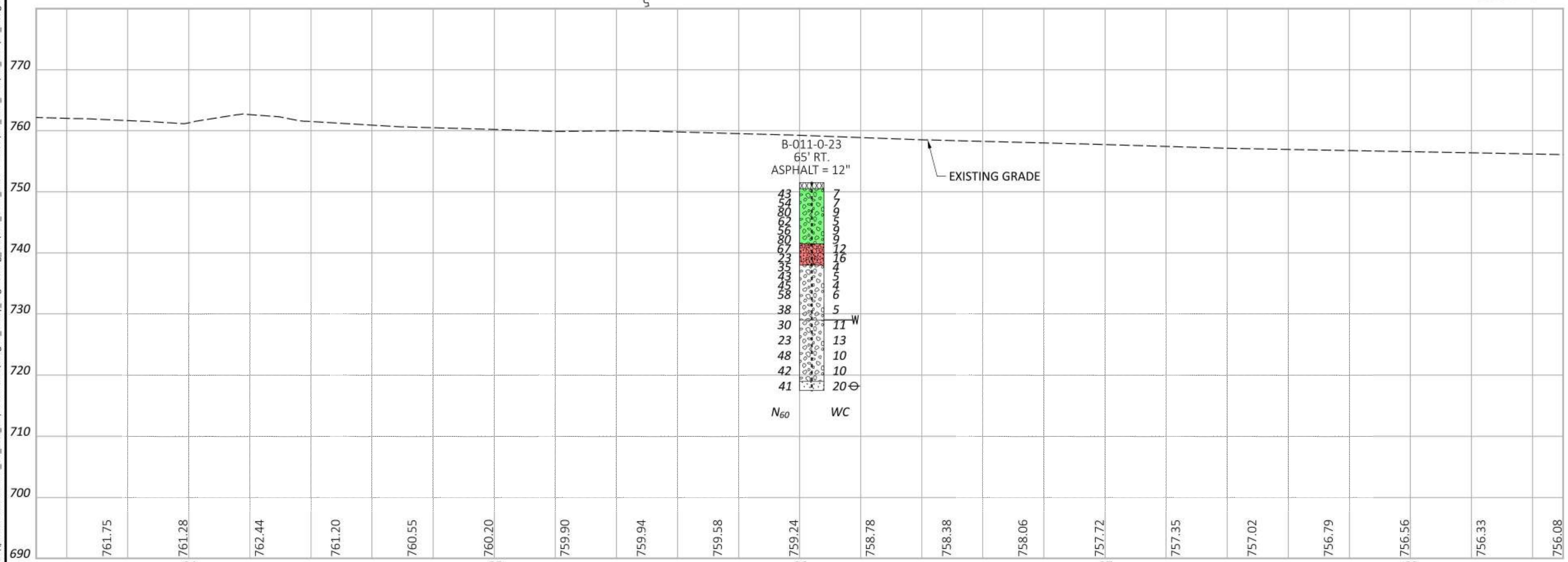
DESIGN AGENCY
HDR

DESIGNER
DCM
REVIEWER
DVM 10/12/2023
PROJECT ID
117239
SHEET P.2
TOTAL 7

MOT-4-19.30

MODEL: 117239_JP002 PAPERSIZE: 34x22 (in.) DATE: 10/13/2023 TIME: 3:56:43 PM USER: DMATCHIS
pw:\\pwhdruseas01:HDR_US_East_01\Documents\Chagrin_Valley_Engineering_Ltd\CVE_ODT_MOT-4-19.30\6.0_CAD_BIM\6.2_WIP\01_Design\117239\400 Engineering\Geotechnical\Sheets\117239_JP002.dgn

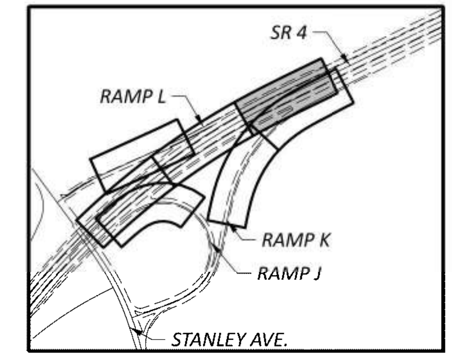




LEGEND

GRANULAR FILL

REFUSE MATERIAL



KEY MAP

HORIZONTAL SCALE IN FEET
0 10 20 40

GEOTECHNICAL PROFILE - ROADWAY
SR4 - STA. 93+50 TO STA. 95+50

DESIGN AGENCY

HDR

DESIGNER DCM

REVIEWER DVM 10/12/2023

PROJECT ID 117239

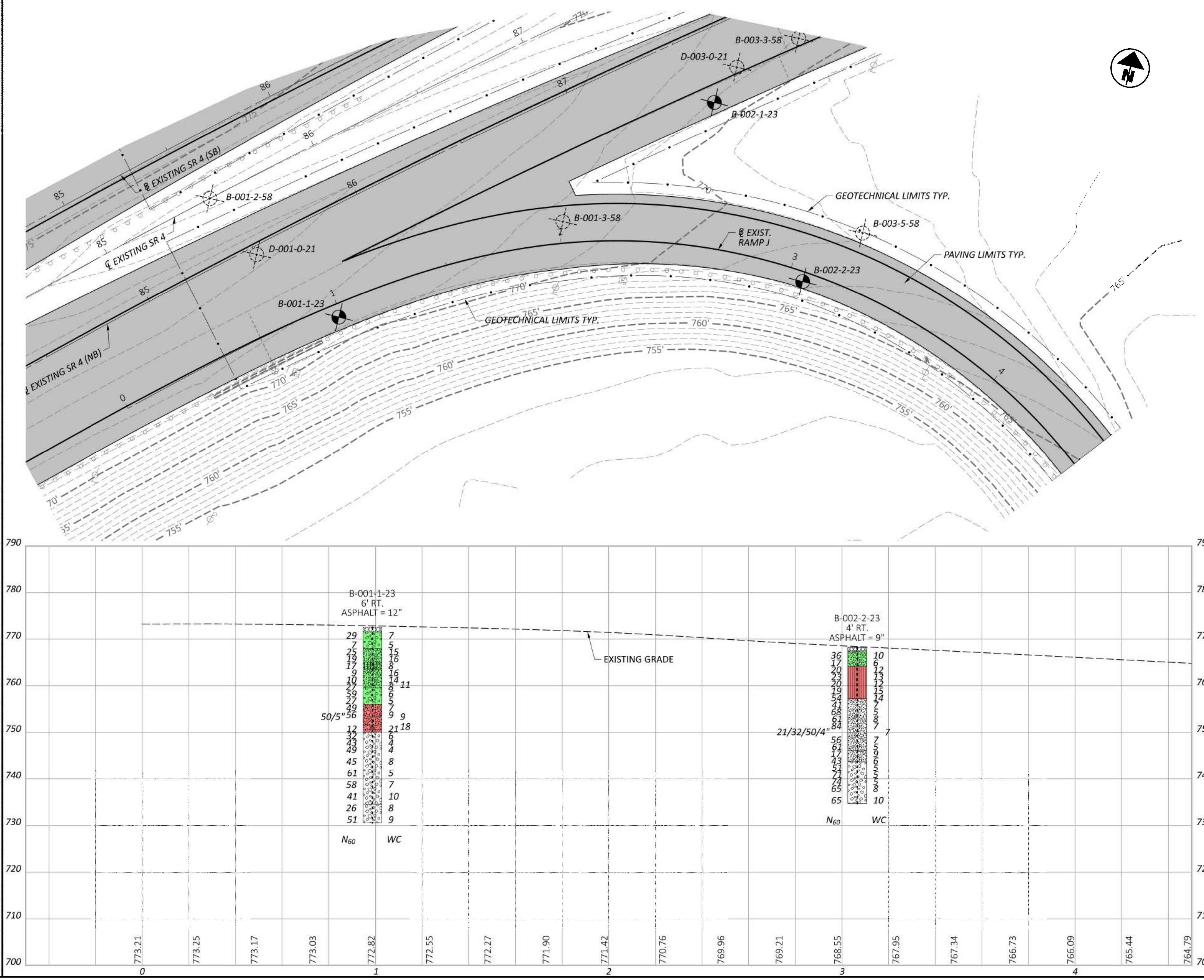
SHEET P.3 TOTAL 7

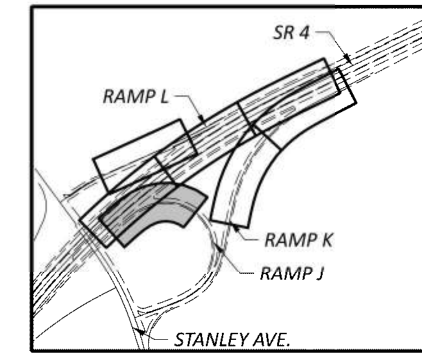
MOT-4-19.30

MODEL: 117239_JP003 PAPERSIZE: 34x22 (in.) DATE: 10/13/2023 TIME: 3:57:24 PM USER: DMATCHIS
pw:\\pwhdruseas01:HDR_US_East_01\Documents\Chagrin_Valley_Engineering_Ltd\CVE_ODT_MOT-4-19.30\6.0_CAD_BIM\6.2_WIP\01_Design\117239\400 Engineering\Geotechnical\Sheets\117239_JP003.dgn

MOT-4-19.30

MODEL: 117239_JP005 PAPERSIZE: 34x22 (in.) DATE: 10/13/2023 TIME: 3:57:59 PM USER: DMATCHIS
pw:\\pwhdruseas01:HDR_US_East_01\Documents\Chagrin_Valley_Engineering_Ltd\CVE_ODT_MOT-4-19.30\6.0_CAD_BIM\6.2_WIP\01_Design\117239\400 Engineering\Geotechnical\Sheets\117239_JP005.dgn





KEY MAP

HORIZONTAL SCALE IN FEET
0 10 20 40

GEOTECHNICAL PROFILE - ROADWAY
RAMP J - STA. 0+00 TO STA. 4+50

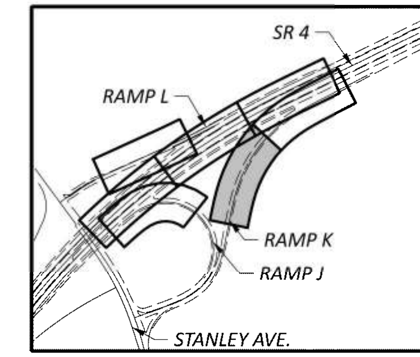
DESIGN AGENCY
HDR

DESIGNER
DCM

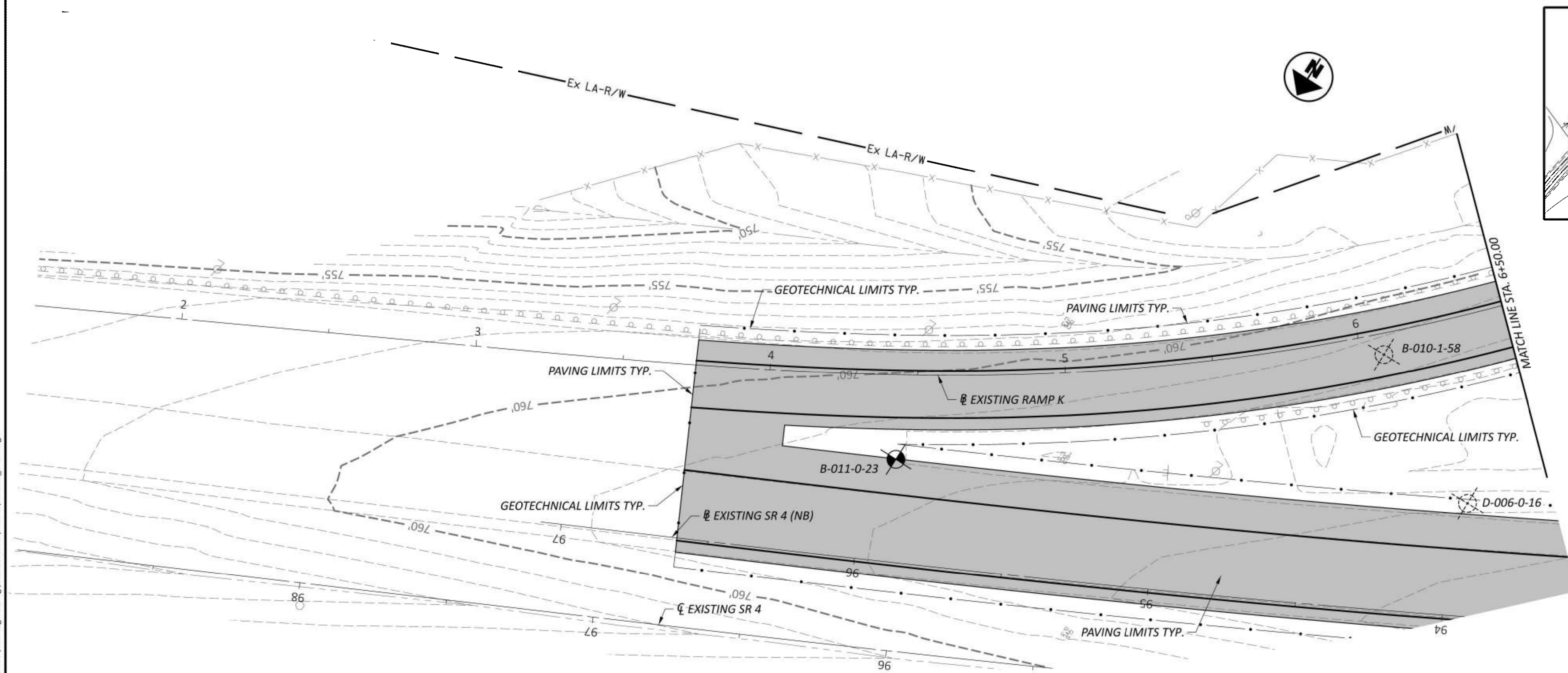
REVIEWER
DVM 10/12/2023

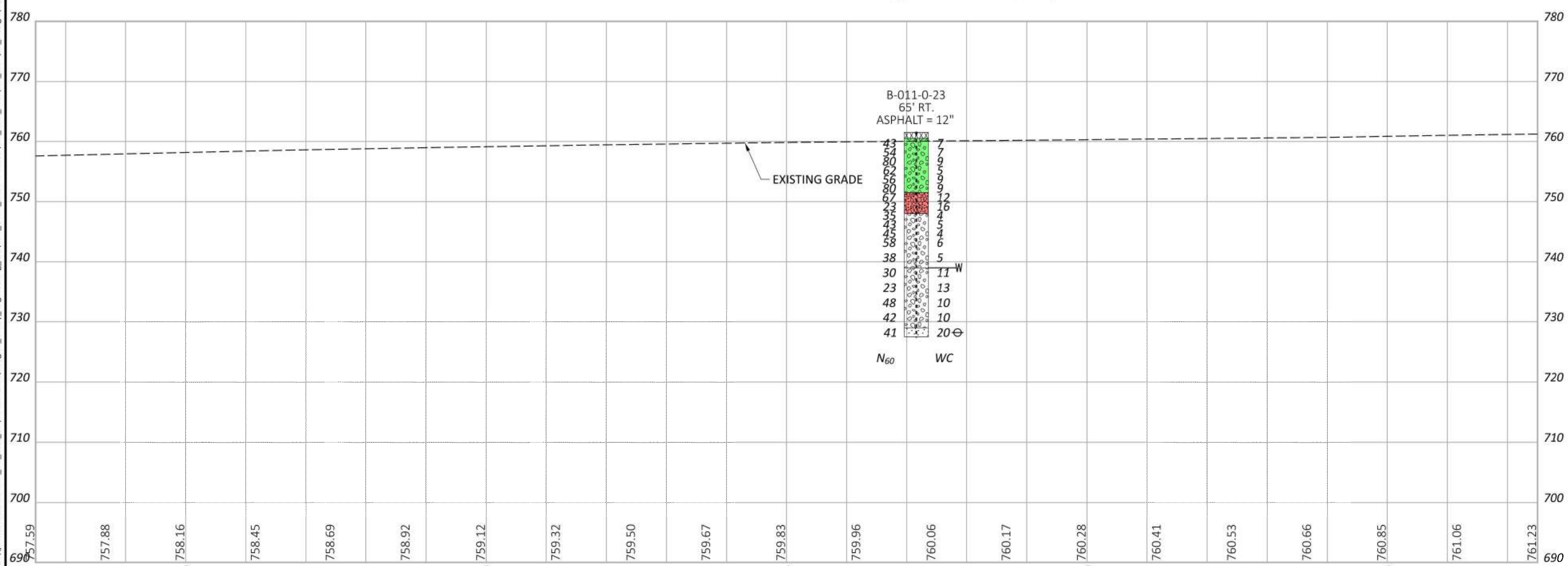
PROJECT ID
117239

SHEET P.4 | TOTAL 7



KEY MAP





LEGEND

- GRANULAR FILL
- REFUSE MATERIAL

HORIZONTAL SCALE IN FEET
0 10 20 40

GEOTECHNICAL PROFILE - ROADWAY
RAMP K - STA. 1+50 TO STA. 6+50

DESIGN AGENCY

HDR

DESIGNER
DCM

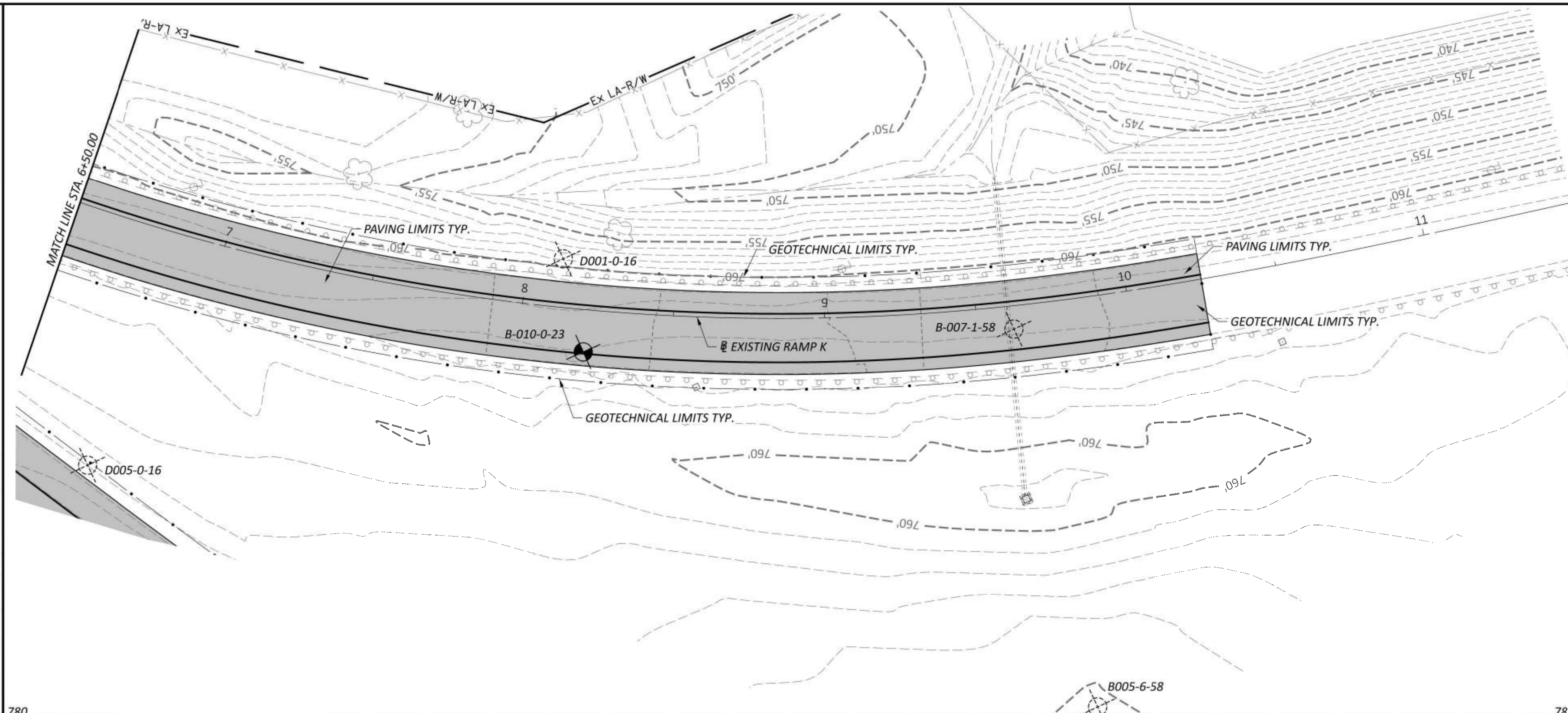
REVIEWER
DVM 10/12/2023

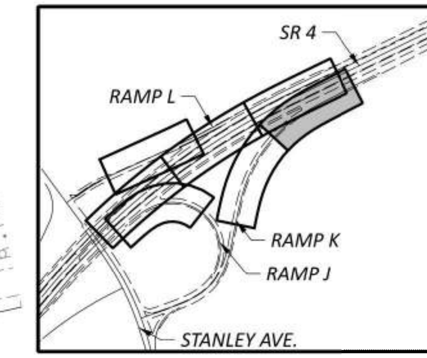
PROJECT ID
117239

SHEET P.5 TOTAL 7

MOT-4-19.30

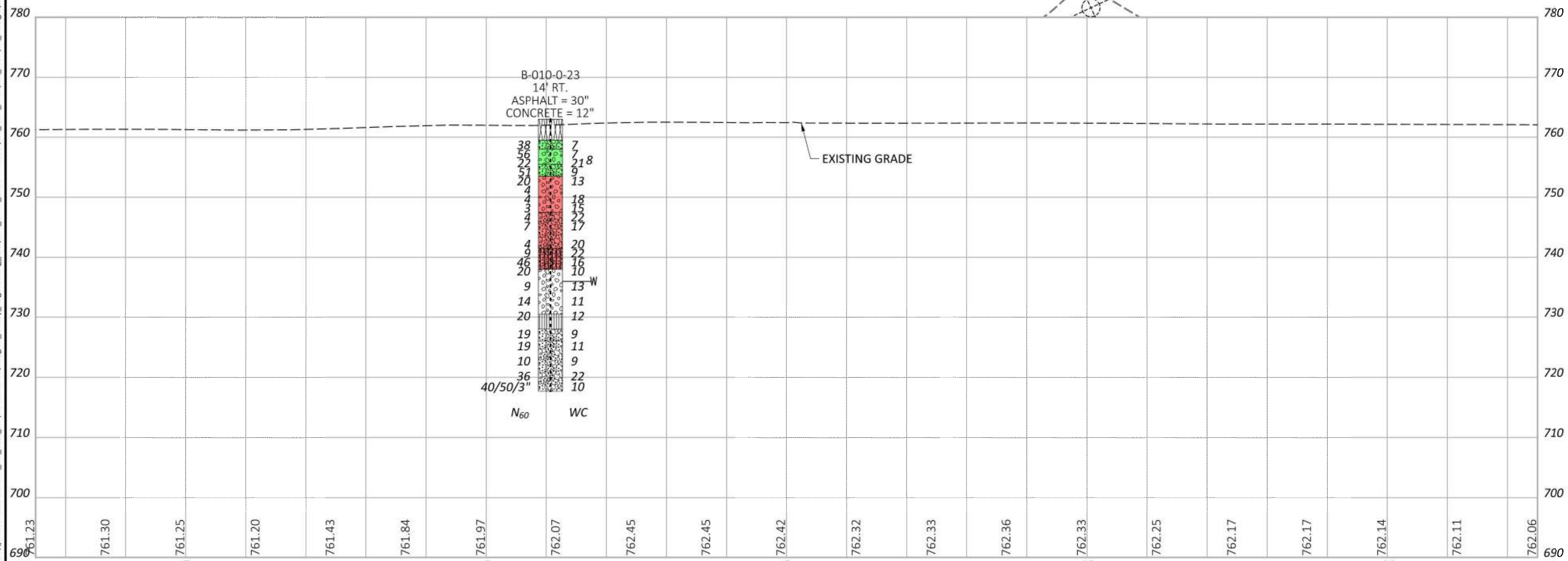
MODEL: 117239_JP007 PAPERSIZE: 34x22 (in.) DATE: 10/13/2023 TIME: 3:58:36 PM USER: DMATCHIS
pw:\\pwhdruseas01:HDR_US_East_01\Documents\Chagrin_Valley_Engineering_Ltd\CVE_ODT_MOT-4-19.30\6.0_CAD_BIM\6.2_WIP\01_Design\117239\400 Engineering\Geotechnical\Sheets\117239_JP007.dgn





KEY MAP

HORIZONTAL SCALE IN FEET
0 10 20 40



LEGEND

(green)	GRANULAR FILL
(red)	REFUSE MATERIAL

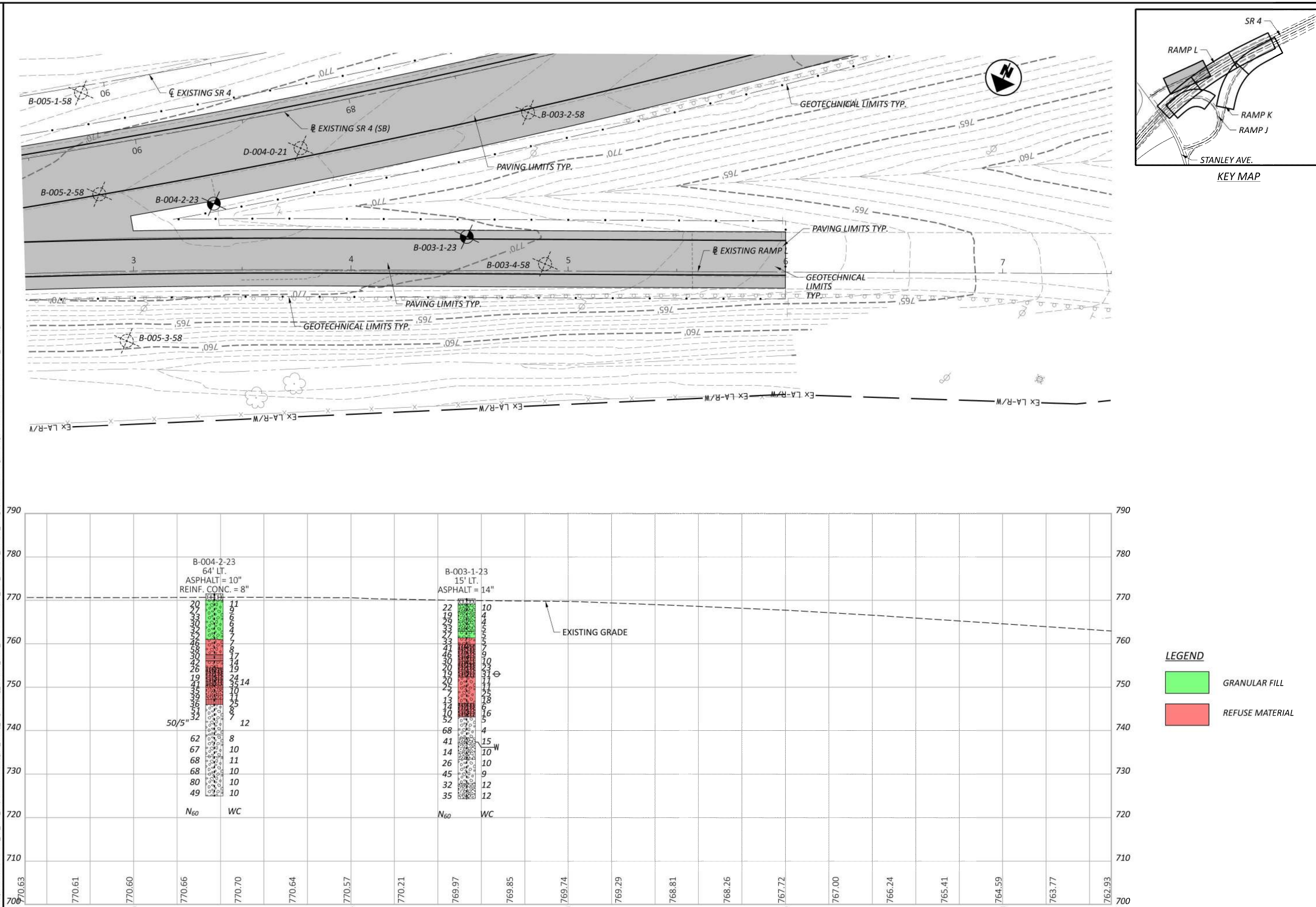
GEOTECHNICAL PROFILE - ROADWAY
RAMP K - STA. 6+50 TO STA. 11+00

DESIGN AGENCY
HDR

DESIGNER DCM
REVIEWER DVM 10/12/2023
PROJECT ID 117239
SHEET P.6 TOTAL 7

MOT-4-19.30

MODEL: 117239_JP008 PAPERSIZE: 34x22 (in.) DATE: 10/13/2023 TIME: 3:59:11 PM USER: DMATCHIS
pw:\\pwhdruseast01:HDR_US_East_01\Documents\Chagrin_Valley_Engineering_Ltd\CVE_ODT_MOT-4-19.30\6.0_CAD_BIM\6.2_WIP\01_Design\117239\400 Engineering\Geotechnical\Sheets\117239_JP008.dgn



HORIZONTAL SCALE IN FEET

0 10 20 40

GEOTECHNICAL PROFILE - ROADWAY
RAMP L - STA. 2+50 TO STA. 7+00

DESIGN AGENCY

HDR

DESIGNER DCM

REVIEWER DVM 10/12/2023

PROJECT ID 117239

SHEET P.7 TOTAL 7

MOT-4-19.30

Appendix F. Analyses

Current Areas of Noted Pavement Deflection

Determination of Soil Strength Parameters

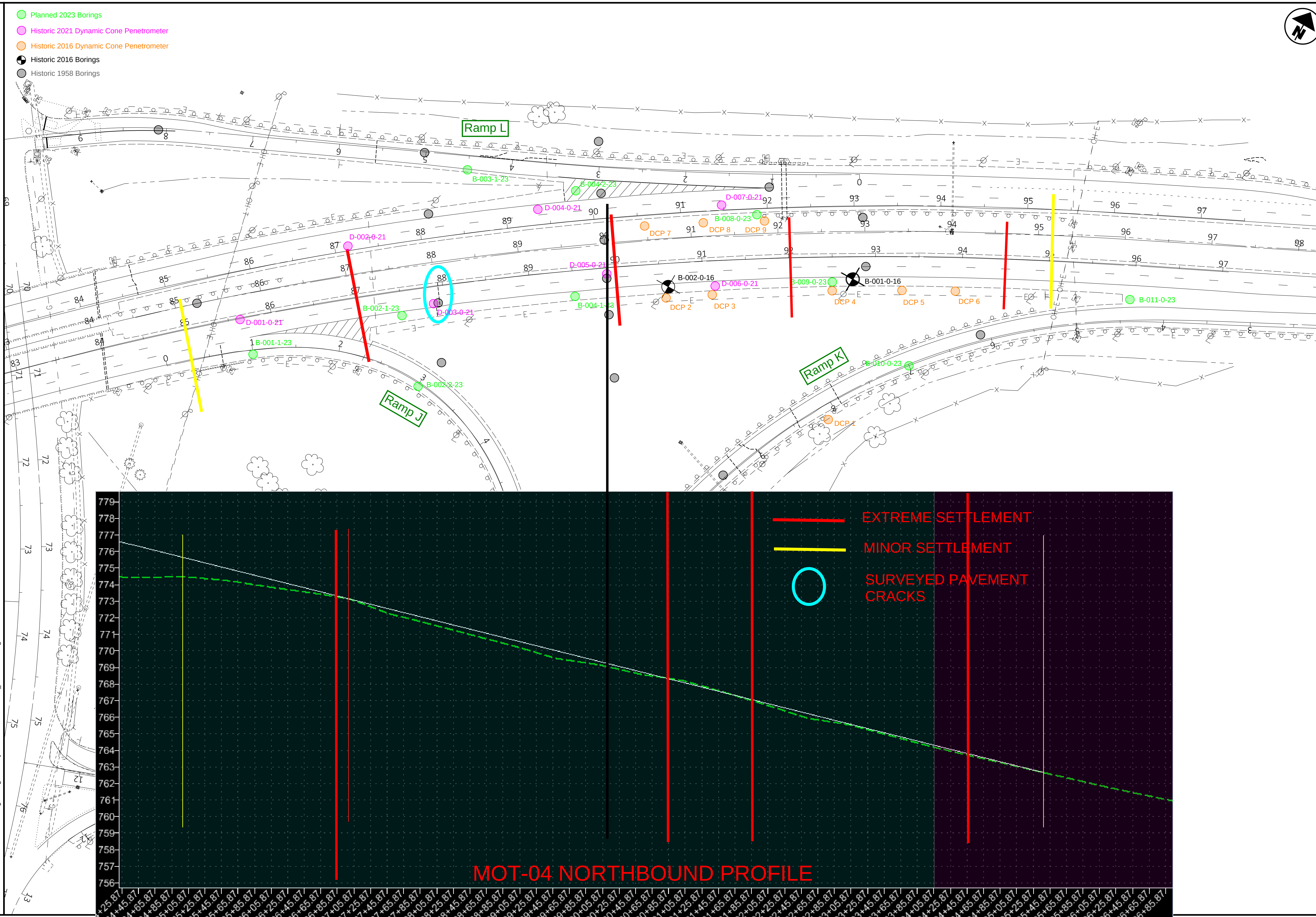
Grain Size of Existing Embankment Material (approximately upper 6 feet)

Potential Void Analysis

Undercut and Replacement with Geogrid Reinforcement

CBR Determination

Current Areas of Noted Pavement Deflection



BORING LOCATION PLAN

MOT-4-19.30

MODEL: Design - Geotech PAPERSIZE: 34x22 (in.) DATE: 4/13/2023 TIME: 12:52:57 PM USER: thomas
P:\23125 MOT-4-19.30\117239\400-Engineering\Roadway\Sheets\Exhibits\117239_Quick Exhibit.dgn

DESIGN AGENCY

CVE
CHAGRIN VALLEY ENGINEERING, LTD.

DESIGNER
SDK

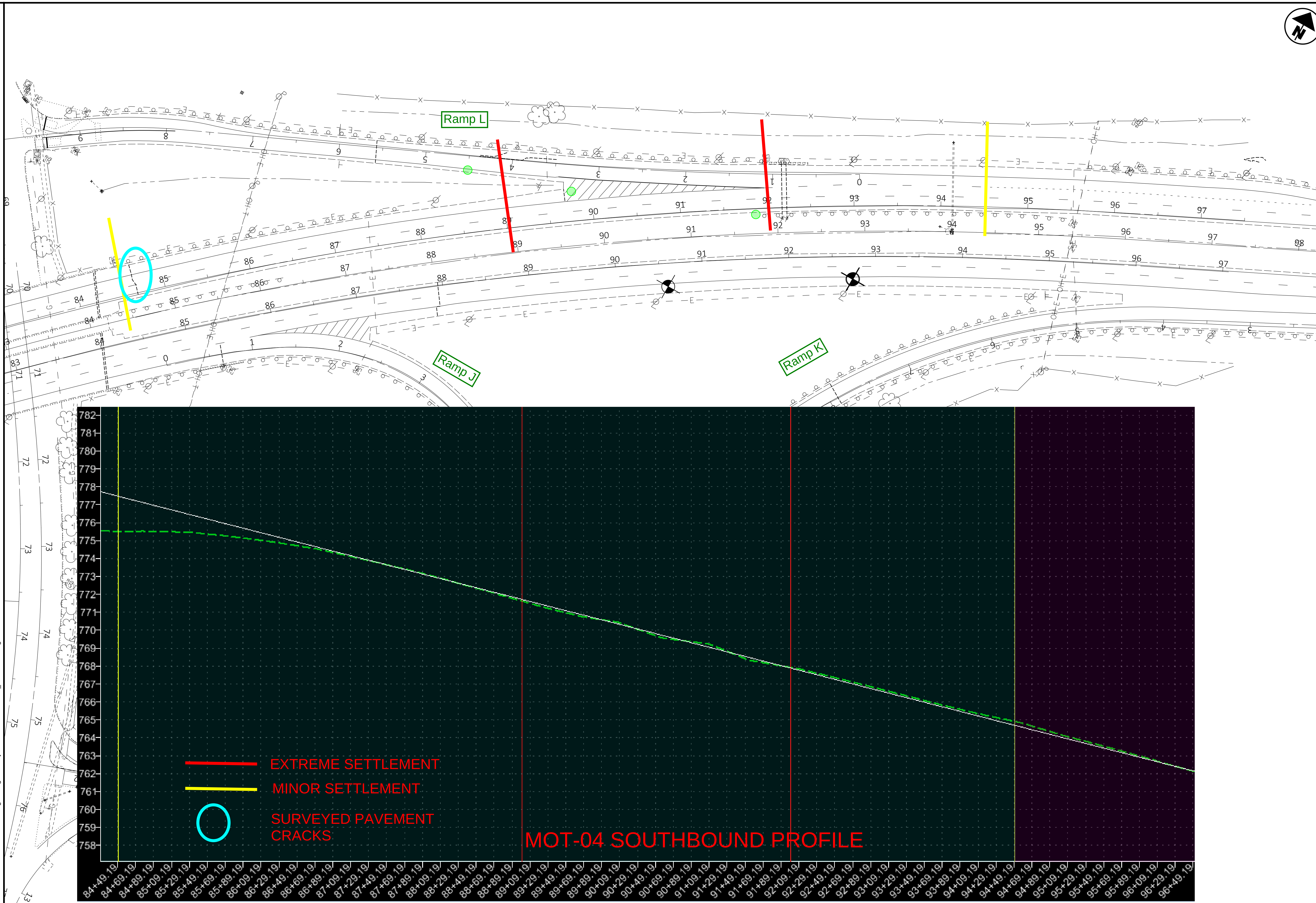
REVIEWER
JTS 03-23-23

PROJECT ID
117239

SHEET 1 TOTAL 1

MOT-4-19.30

MODEL: Design - Geotech PAPERSIZE: 34x22 (in.) DATE: 4/13/2023 TIME: 12:52:57 PM USER: thomas
P:\23125 MOT-4-19.30\117239\400-Engineering\Roadway\Sheets\Exhibits\117239_Quick Exhibit.dgn

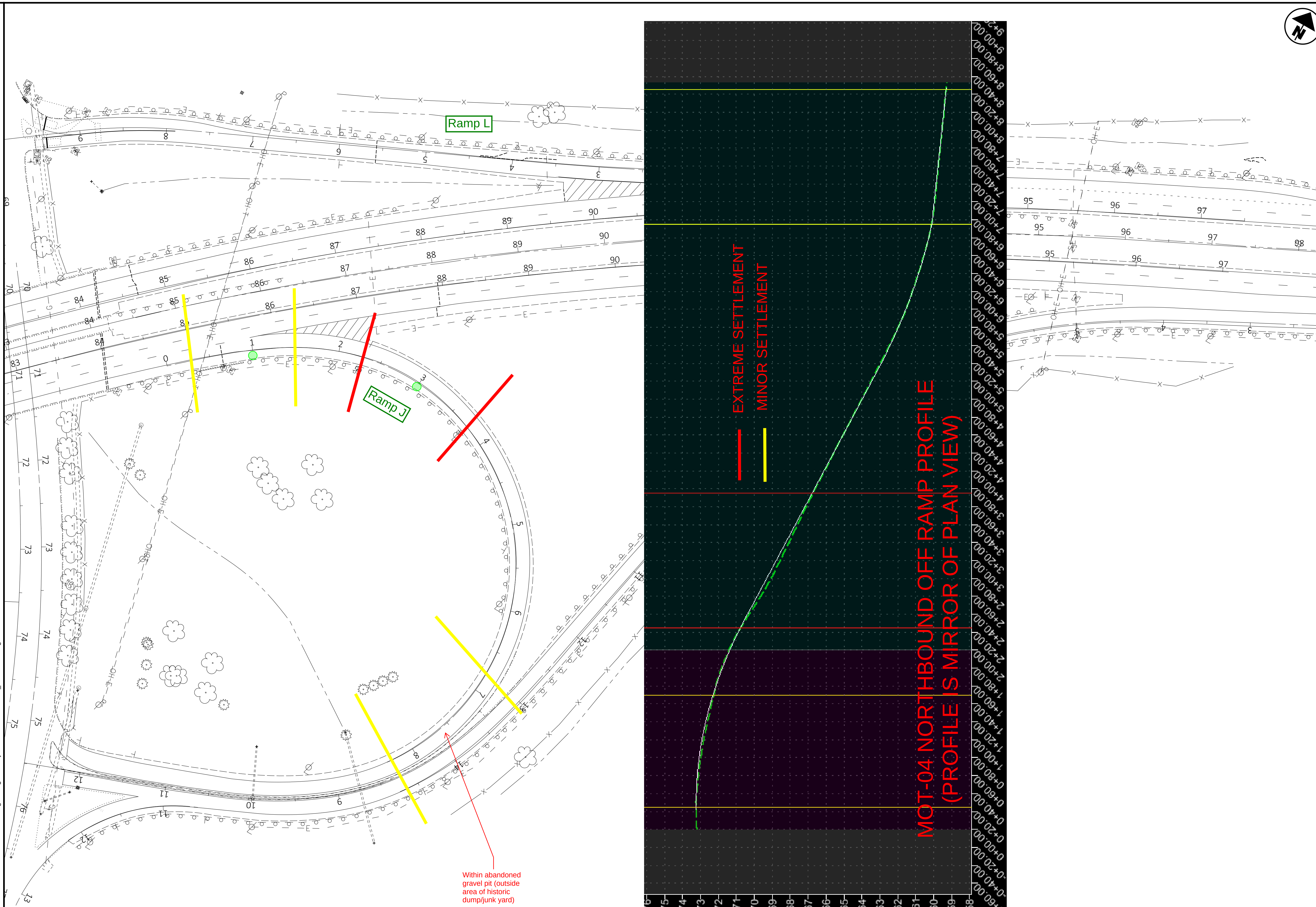


PLAN SHEET

DESIGN AGENCY	CVE CHAGRIN VALLEY ENGINEERING, LTD.
DESIGNER	SDK
REVIEWER	JTS 03-23-23
PROJECT ID	117239
SHEET 1	TOTAL 1

MOT-4-19.30

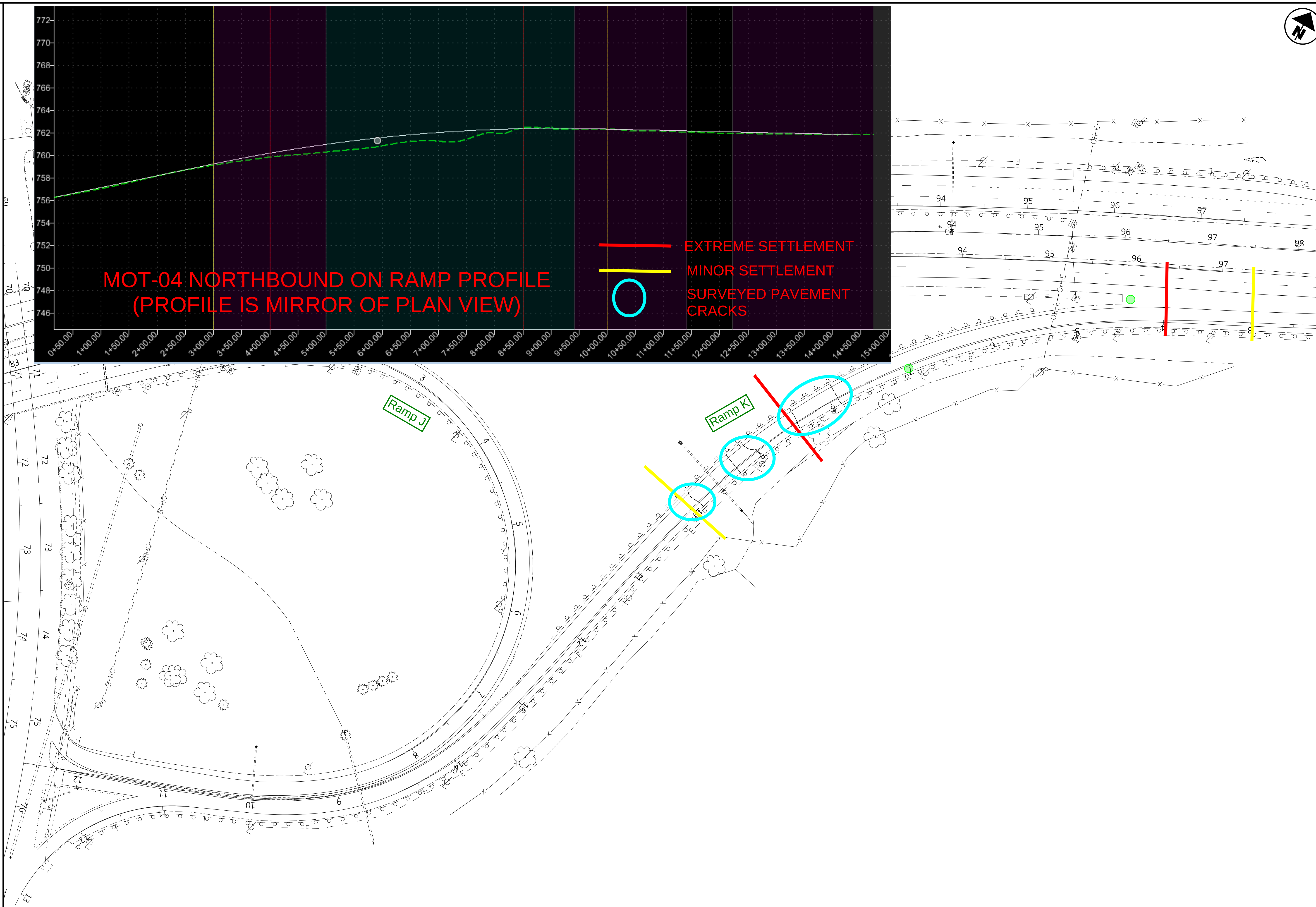
MODEL: Design - Geotech PAPERSIZE: 34x22 (in.) DATE: 4/13/2023 TIME: 12:52:57 PM USER: thomas
P:\23125 MOT-4-19.30\117239\400-Engineering\Roadway\Sheets\Exhibits\117239_Quick Exhibit.dgn



PLAN SHEET

DESIGN AGENCY
CHAGRIN VALLEY ENGINEERING, LTD.

DESIGNER	SDK
REVIEWER	JTS 03-23-23
PROJECT ID	117239
SHEET	1
TOTAL	1



PLAN SHEET

DESIGN AGENCY

CVE
CHAGRIN VALLEY ENGINEERING, LTD.

DESIGNER
SDK

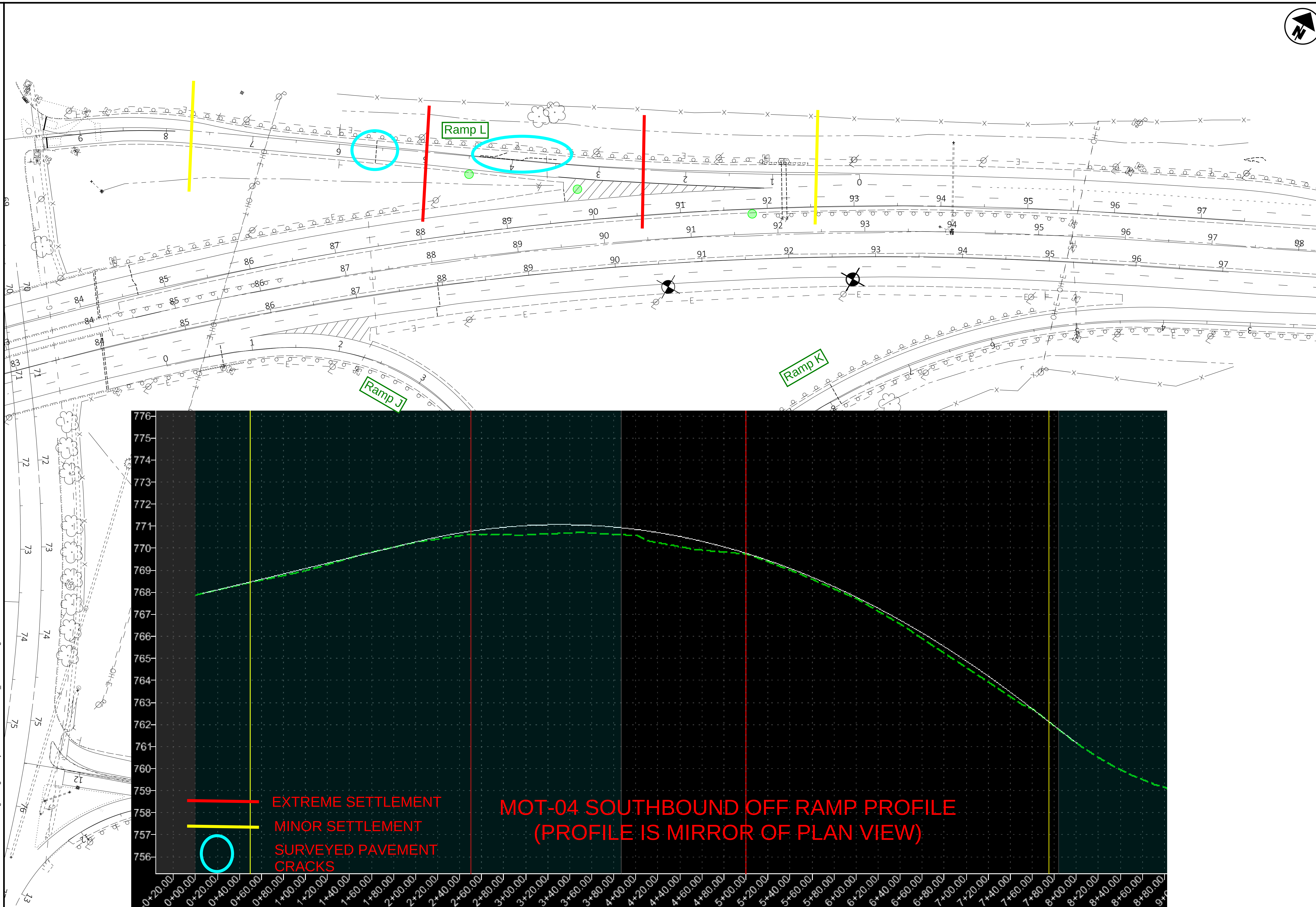
REVIEWER
JTS 03-23-23

PROJECT ID
117239

SHEET 1
TOTAL 1

MOT-4-19.30

MODEL: Design - Geotech PAPERSIZE: 34x22 (in.) DATE: 4/13/2023 TIME: 12:52:57 PM USER: thomas
P:\23125 MOT-4-19.30\117239\400-Engineering\Roadway\Sheets\Exhibits\117239_Quick Exhibit.dgn



PLAN SHEET

MOT-4-19.30

MODEL: Design - Geotech PAPERSIZE: 34x22 (in.) DATE: 4/13/2023 TIME: 12:52:57 PM USER: thomas
P:\23125 MOT-4-19.30\117239\400-Engineering\Roadway\Sheets\Exhibits\117239_Quick Exhibit.dgn

DESIGN AGENCY	CVE CHAGRIN VALLEY ENGINEERING, LTD.
DESIGNER	SDK
REVIEWER	JTS 03-23-23
PROJECT ID	117239
SHEET	1
TOTAL	1

Determination of Soil Strength Parameters

Layer		Undrained Shear Strength (Su) (psf)				Dry Unit Weight (pcf)		Moist Unit Wt. (pcf)		Adopted Short Term Parameters	Long-Term Strength Values				Adopted Long Term Strength Parameters
		PPR	N-values		Tested							N_{60} Value	ODOT GB-7 Correlations		
			Sowers	T and P	Values	Correlation	Tested	Correlation	Tested				Cohesion (psf)	phi (deg)	
	Max	N/A	N/A	N/A		125		140			**Max**	97	N/A	41	
	Min	N/A	N/A	N/A		95		115		S_u = **0** psf	**Min**	7	N/A	29	c' = **0** psf
Layer 1	**Average**	N/A	N/A	N/A		112		128		Φ = **36** deg	**Average**	43	N/A	36	Φ' = **36** deg
	Std Dev	N/A	N/A	N/A		9		7			**Std Dev**	23	N/A	4	
GRANULAR EMBANKMENT FILL										Y_{dry} = 110 pcf					Y_{dry} = 110 pcf
	Avg + Std	N/A	N/A	N/A		120		135		Y_{moist} = **130** pcf	**Avg + Std**	66	N/A	39	Y_{moist} = **130** pcf
	Avg - Std	N/A	N/A	N/A		103		121			**Avg - Std**	19	N/A	32	
	Max	N/A	N/A	N/A		125		140			**Max**	132	N/A	40	
	Min	N/A	N/A	N/A		100		120		S_u = **0** psf	**Min**	3	N/A	24	c' = **0** psf
Layer 2	**Average**	N/A	N/A	N/A		113		132		Φ = **24** deg	**Average**	26	N/A	32	Φ' = **24** deg
	Std Dev	N/A	N/A	N/A		7		6			**Std Dev**	22	N/A	4	
REFUSE MATERIAL										Y_{dry} = 110 pcf					Y_{dry} = 110 pcf
	Avg + Std	N/A	N/A	N/A		120		137		Y_{moist} = **130** pcf	**Avg + Std**	48	N/A	36	Y_{moist} = **130** pcf
	Avg - Std	N/A	N/A	N/A		106		126			**Avg - Std**	4	N/A	28	
	Max	N/A	N/A	N/A		130		150			**Max**	84	N/A	41	
	Min	N/A	N/A	N/A		110		130		S_u = **0** psf	**Min**	9	N/A	30	c' = **0** psf
Layer 3	**Average**	N/A	N/A	N/A		121		139		Φ = **36** deg	**Average**	41	N/A	36	Φ' = **36** deg
	Std Dev	N/A	N/A	N/A		4		4			**Std Dev**	17	N/A	3	
GRANULAR SOILS										Y_{dry} = 120 pcf					Y_{dry} = 120 pcf
	Avg + Std	N/A	N/A	N/A		125		143		Y_{moist} = **140** pcf	**Avg + Std**	58	N/A	39	Y_{moist} = **140** pcf
	Avg - Std	N/A	N/A	N/A		116		136			**Avg - Std**	24	N/A	33	
	Max	4500	4000	4000		135		145			**Max**	67	250	28	
	Min	4500	750	1330		115		130		S_u = **2500** psf	**Min**	10	114	23	c' = **180** psf
Layer 4	**Average**	4500	2308	3062		123		137		Φ = **0** deg	**Average**	35	187	25	Φ' = **25** deg
	Std Dev	N/A	1629	1502		10		8			**Std Dev**	29	69	3	
COHESIVE SOILS										Y_{dry} = 125 pcf					Y_{dry} = 125 pcf
	Avg + Std	N/A	3937	4564		134		144		Y_{moist} = **135** pcf	**Avg + Std**	64	256	28	Y_{moist} = **135** pcf
	Avg - Std	N/A	679	1560		113		129			**Avg - Std**	6	118	23	

Values for Soil Strength Correlation	
Reference	Value
HI PI (Sowers)	0.25
MD PI (Sowers)	0.175
LO PI (Sowers)	0.075
T&P	0.133

Omitted: B-001-1-23, SS-8A

Layer 1

Layer 1	N_{60}	% Rec	HP	% Gr	% CS	% FS	% Silt	% Clay	LL	PL	PI	% WC		Short-Term Cohesion (psf) N-values PPR	Sowers	T & P	Correlated LT Cohesion (psf) per GB-7	phi (deg)	Midpoint Sample Depth (ft.)	Midpoint Sample Elevation (ft.)	Correlated Dry Unit Wt. (pcf) per GB-7	Correlated Moist Unit Wt. (pcf) per GB-7	Correlated C_c	Assumed Specific Gravity (G_s)	Computed Void Ratio (e)
Max	97	100	2.0	84	37	15	19	10	27	21	8	16	Max	N/A	N/A	N/A	N/A	41	16.0	770.6	125	140	0.153	2.71	0.780
Min	7	6	0.5	38	1	1	1	1	12	13	2	4	Min	N/A	N/A	N/A	N/A	29	2.0	752.5	95	115	0.018	2.65	0.353
Average	43	71	1.4	61	20	6	9	3	19	16	5	8	Average	N/A	N/A	N/A	N/A	36	6.7	762.1	112	128	0.085	2.71	0.525
Std Dev	23	22	0.7	12	9	4	5	2	4	2	2	3	Std Dev	N/A	N/A	N/A	N/A	4	3.6	4.3	9	7	0.037	0.01	0.122
Avg + Std	66	93	2.1	73	29	10	14	6	24	18	7	11	Avg + Std	N/A	N/A	N/A	N/A	39	10.4	766.4	120	135	0.122	2.72	0.647
Avg - Std	19	49	0.7	49	10	2	4	1	15	14	3	5	Avg - Std	N/A	N/A	N/A	N/A	32	3.1	757.8	103	121	0.048	2.70	0.403

Alignment	Surface Elevation	Exploration ID	From		To	Sample ID	N_{60}	% Rec	HP	% Gr	% CS	% FS	% Silt	% Clay	LL	PL	PI	% WC	ODOT Class.	Soil Type	Layer	Short-Term Cohesion (psf) N-values PPR	Sowers	T & P	Correlated LT Cohesion (psf) per GB-7	phi (deg)	Midpoint Sample Depth (ft.)	Midpoint Sample Elevation (ft.)	Correlated Dry Unit Wt. (pcf) per GB-7	Correlated Moist Unit Wt. (pcf) per GB-7	Correlated C_c	Assumed Specific Gravity (G_s)	Computed Void Ratio (e)
Ramp J	772.6	B-001-1-23	1	-	2.5	SS-1	29	61	-	68	21	4	5	2	15	15	NP	7	A-1-a	Granular	1	N/A				34	2.0	770.6	100	120	0.045	2.71	0.691
Ramp J	772.6	B-001-1-23	3	-	4.5	SS-2	7	44	-	-	-	-	-	-	-	-	-	5	A-1-a	Granular	1	N/A				29	4.0	768.6	95	115		2.71	0.780
Ramp J	772.6	B-001-1-23	4.5	-	6	SS-3	25	56	1.5	54	17	6	13	10	27	21	6	15	A-1-b	Granular	1	N/A				33	5.0	767.6	100	120	0.153	2.71	0.691
Ramp J	772.6	B-001-1-23	6	-	7.5	SS-4	19	39	2	-	-	-	-	-	-	-	-	16	A-1-b	Granular	1	N/A				32	7.0	765.6	105	125		2.71	0.611
Ramp J	772.6	B-001-1-23	7.5	-	9	SS-5	17	56	-	67	20	8	1	4	25	18	7	8	A-2-4	Granular	1	N/A				31	8.0	764.6	105	125	0.135	2.71	0.611
Ramp J	772.6	B-001-1-23	9	-	10.5	SS-6	9	33	2	-	-	-	-	-	-	-	-	16	A-1-b	Granular	1	N/A				30	10.0	762.6	100	120		2.71	0.691
Ramp J	772.6	B-001-1-23	10.5	-	12	SS-7	10	67	0.5	42	37	4	14	3	26	NP	NP	14	A-1-b	Granular	1	N/A				30	11.0	761.6	105	125	0.144	2.71	0.611
Ramp J	772.6	B-001-1-23	12	-	13	SS-8A	27	78	1	-	-	-	-	-	-	-	-	11	A-1-b	Granular	1	N/A				33	13.0	759.6	110	130		2.71	0.537
Ramp J	772.6	B-001-1-23	13	-	13.5	SS-8B	-	-	-	-	-	-	-	-	-	-	-	8	A-1-a	Granular	1	N/A					13.0	759.6	125			2.71	0.353
Ramp J	772.6	B-001-1-23	13.5	-	15	SS-9	59	56	-	-	-	-	-	-	-	-	-	6	A-1-a	Granular	1	N/A				39	14.0	758.6	125	140		2.71	0.353
Ramp J	772.6	B-001-1-23	15	-	16.5	SS-10	27	67	-	67	21	4	5	3	20	15	5	5	A-1-a	Granular	1	N/A				33	16.0	756.6	110	130	0.09	2.71	0.537
EB SR 4	770.9	B-002-1-23	1.75	-	2.25	SS-1A	39	89	-	-	-	-	-	-	-	-	-	15	A-3a	Granular	1	N/A				35	2.0	768.9	110	125		2.65	0.503
EB SR 4	770.9	B-002-1-23	2.25	-	3.25	SS-1B	-	-	-	-	-	-	-	-	-	-	-	11	A-2-4	Granular	1	N/A					3.0	767.9	115			2.71	0.470
EB SR 4	770.9	B-002-1-23	3.5	-	5	SS-2	46	89	-	-	-	-	-	-	-	-	-	10	A-2-4	Granular	1	N/A				36	4.0	766.9	110	125		2.71	0.537
EB SR 4	770.9	B-002-1-23	5	-	6.5	SS-3	52	100	-	39	26	10	19	6	21	13	8	8	A-2-4	Granular	1	N/A				39	6.0	764.9	120	135	0.099	2.71	0.409
EB SR 4	770.9	B-002-1-23	6.5	-	8	SS-4	56	100	-	-	-	-	-	-	-	-	-	5	A-2-4	Granular	1	N/A				39	7.0	763.9	120	135		2.71	0.409
EB SR 4	770.9	B-002-1-23	8	-	9.5	SS-5	74	100	-	-	-	-	-	-	-	-	-	7	A-2-4	Granular	1	N/A				40	9.0	761.9	120	135		2.71	0.409
EB SR 4	770.9	B-002-1-23	9.5	-	11	SS-6	93	100	-	-	-	-	-	-	-	-	-	4	A-1-a	Granular	1	N/A				40	10.0	760.9	120	135		2.71	0.409
EB SR 4	770.9	B-002-1-23	11	-	12.5	SS-7	72	100	-	60	29	4	5	2	20	15	5	5	A-1-a	Granular	1	N/A				40	12.0	758.9	125	140	0.09	2.71	0.353
EB SR 4	770.9	B-002-1-23	12.5	-	14	SS-8	75	89	-	-	-	-	-	-	-	-	-	5	A-1-a	Granular	1	N/A				41	13.0	757.9	125	140		2.71	0.353
EB SR 4	770.9	B-002-1-23	14	-	15.5	SS-9	97	89	-	-	-	-	-	-	-	-	-	10	A-1-a	Granular	1	N/A				40	15.0	755.9	125	140		2.71	0.353
Ramp J	768.2	B-002-2-23	0.75	-	2.25	SS-1	36	78	-	-	-	-	-	-	-	-	-	10	A-1-b	Granular	1	N/A				35	2.0	766.2	110	125		2.71	0.537
Ramp J	768.2	B-002-2-23	2.5	-	4	SS-2	17	56	-	79	2	1	-	-	NP	NP	NP	6	A-1-b	Granular	1	N/A				31	3.0	765.2	100	120	N/A	2.71	0.691
Ramp L	770.3	B-003-1-23	1.2	-	2.7	SS-1	22	44	-	48	23	12	15	2	16	NP	NP	10	A-1-b	Granular	1	N/A				32	2.0	768.3	100	120	0.054	2.71	0.691
Ramp L	770.3	B-003-1-23	3	-	4.5	SS-2	19	6	-	-	-	-	-	-	-	-	-	4	A-1-b	Granular	1	N/A				32	4.0	766.3	100	120		2.71	0.691
Ramp L	770.3	B-003-1-23	4.5	-	6	SS-3	29	17	-	-	-	-	-	-	-	-	-	4	A-1-b	Granular	1	N/A				34	5.0	765.3	100	120		2.71	0.691
Ramp L	770.3	B-003-1-23	6	-	7.5	SS-4	33	44	-	-	-	-	-	-	-	-	-	5	A-1-b	Granular	1	N/A				34	7.0	763.3	115	130		2.71	0.470
Ramp L	770.3	B-003-1-23	7.5	-	9	SS-5	27	56	-	62	15	11	8	4	15	NP	NP	5	A-1-a	Granular	1	N/A				33	8.0	762.3	105	125	0.045	2.71	0.611
EB SR 4	768.1	B-004-1-23	2.25	-	3.75	SS-1	20	67	-	-	-	-	-	-	-	-	-	9	A-1-a	Granular	1	N/A				32	3.0	765.1	100	120		2.71	0.691
EB SR 4	768.1	B-004-1-23	4	-	5.5	SS-2	12	56	-	84	1	1	-	-	NP	NP	NP	5	A-1-a	Granular	1	N/A				30	5.0	763.1	100	120	N/A	2.71	0.691
EB SR 4	768.1	B-004-1-23	5.5	-	7	SS-3	36	78	-	-	-	-	-	-	-	-	-	8	A-2-4	Granular	1	N/A				35	6.0	762.1	115	130		2.71	0.470
EB SR 4	768.1	B-004-1-23	7	-	8.5	SS-4	32	78	-	64	6	3	-	-	NP	NP	NP	7	A-2-4	Granular	1	N/A				34	8.0	760.1	115	130	N/A	2.71	0.470
EB SR 4	768.1	B-004-1-23	8.5	-	10	SS-5	42	78	-	-	-	-	-	-	-	-	-	6	A-2-4	Granular	1	N/A				35	9.0	759.1	115	130		2.71	0.470
EB SR 4	768.1	B-004-1-23	10	-	11.5	SS-6	74	78	-	-	-	-	-	-	-	-	-	5	A-2-4	Granular	1	N/A				40	11.0	757.1	125	140		2.71	0.353
WB SR 4	771.5	B-004-2-23	1.5	-	3	SS-1	20	56	-	-	-	-	-	-	-	-	-	11	A-1-a	Granular	1	N/A				32	2.0	769.5	100	120		2.71	0.691
WB SR 4	771.5	B-004-2-23	3	-	4.5	SS-2	27	56	-	63	27	3	5	2	21	16	5	9	A-1-a	Granular	1	N/A				33	4.0	767.5	100	120	0.099	2.71	0.691
WB SR 4	771.5	B-004-2-23	4.5	-	6	SS-3	33	78	-	-	-	-	-	-	-	-	-	6	A-1-a	Granular	1	N/A				34	5.0	766.5	110	125		2.71	0.537
WB SR 4	771.5	B-004-2-23	6	-	7.5	SS-4	30	56	-	-	-	-	-	-	-	-	-	6	A-1-a	Granular	1	N/A				34	7.0	764.5	105	125		2.71	0.611
WB SR 4	771.5	B-004-2-23	7.5	-	9	SS-5	32	67	-	-	-	-	-	-	-	-	-	4	A-1-a	Granular	1	N/A				34	8.0	763.5	115	130		2.71	0.470
WB SR 4	771.5	B-004-2-23	9	-	10.5	SS-6	52	78	-	-	-	-	-	-	-	-	-	7	A-1-a	Granular	1	N/A				39	10.0	761.5	120	135		2.71	0.409
WB SR 4	768.1	B-008-0-23	1.1	-	2.6	SS-1	36	89	-	-	-	-	-	-	-	-	-	10	A-1-a	Granular	1	N/A				35	2.0	766.1	110	125		2.71	0.537
WB SR 4	768.1	B-008-0-23	3.5	-	5	SS-2	41	89	-	66	16	7	10	1	19	15	4	9	A-1-a	Granular	1	N/A				35	4.0	764.1	110	125	0.081	2.71	0.537
WB SR 4	768.1	B-008-0-23	5	-	6.5	SS-3	72	83	-	-	-	-	-	-	-	-	-	5	A-1-a	Granular	1	N/A				40	6.0	762.1	120	135		2.71	0.409
WB SR 4	768.1	B-008-0-23	6.5	-	8	SS-4	97	100	-	63	26	4	5	2	17	NP	NP	6	A-1-a	Granular	1	N/A				40	7.0	761.1	120	135	0.063	2.71	0.409
WB SR 4	768.1	B-008-0-23	8	-	9.5	SS-5	67	100	-	-	-	-	-	-	-	-	-	7	A-1-a	Granular	1	N/A				40	9.0	759.1	120	135		2.71	0.409
EB SR 4	764.2	B-009-0-23	1.4	-	2.9	SS-1	25	67	-	-	-	-	-	-	-	-	-	10	A-1-a	Granular	1	N/A				33	2.0	762.2	100	120		2.71	0.691
EB SR 4	764.2	B-009-0-23	3	-	4.5	SS-2	29	67	-	-	-	-	-	-	-	-	-	8	A-1-a	Granular	1	N/A				34	4.0	760.2	100	120		2.71	0.691
EB SR 4	764.2	B-009-0-23	4.5	-	6	SS-3	51	56	-	73	11	6	7	3	18	16	2	9	A-1-a	Granular	1	N/A				38	5.0	759.2	115	130	0.072	2.71	0.470
EB SR 4	764.2	B-009-0-23	6	-	7.5	SS-4	56	100	-	-	-	-	-	-	-	-	-	7	A-1-a	Granular	1	N/A				39	7.0	757.2	120	135		2.71	0.409
Ramp K	763.0	B-010-0-23	3.5	-	5	SS-1	38	78	-	38	29	15	13	5	12	NP	NP	7	A-1-b	Granular	1	N/A				35	4.0	759.0	110	125	0.018	2.71	0.537
Ramp K	763.0	B-010-0-23	5	-	6.5	SS-2	56	78	-	58	20	8	11	3	21	19	2	7	A-1-a	Granular	1	N/A				39	6.0	757.0	120	135	0.099	2.71	0.409
Ramp K	763.0	B-010-0-23	6.5	-	7.5	SS-3A	22	100	-	-	-	-	-	-	-	-	-	8	A-1-a	Granular	1	N/A				32	7.0	756.0	105	125		2.71	0.611
EB SR 4	761.5	B-011-0-23	1	-	2.5	SS-1	43	89	-	-	-	-	-	-	-	-	-	7	A-1-a	Granular	1	N/A				35	2.0	759.5	110	125		2.71	0.537
EB SR 4	761.5	B-011-0-23	2.5	-	4	SS-2	54	83	-	63	25	5	5	2	18	15	3	7	A-1-a	Granular	1	N/A				39	3.0	758.5	115	130	0.072	2.71	0.470
EB SR 4	761.5	B-011-0-23	4	-	5.5	SS-3	80	100	-	-	-	-	-	-	-	-	-	9	A-1-a	Granular	1	N/A				41	5.0	756.5	115	130		2.71	0.470
EB SR 4	761.5	B-011-0-23	5.5	-	7	SS-4	62	78	-	-	-	-	-	-	-	-	-	5	A-1-a	Granular	1	N/A				40	6.0	755.5	120	135		2.71	0.409
EB SR 4	761.5	B-011-0-23	7	-	8.5	SS-5	56	44	-	-	-	-	-	-	-	-	-	9	A-1-a	Granular	1	N/A				39	8.0	753.5	120	135		2.71	0.409
EB SR 4	761.5	B-011-0-23	8.5	-	10	SS-6	80	44	-	-	-	-	-	-	-	-	-	9	A-1-a	Granular	1	N/A				41	9.0	752.5	120	135		2.71	0.409

Values for Soil Strength Correlation	
Reference	Value
HI PI (Sowers)	0.25
MD PI (Sowers)	0.175
LO PI (Sowers)	0.075
T&P	0.133

Omitted:		
	B-001-1-23	SS-14B
	B-002-2-23	SS-3, SS-4, SS-7
	B-004-2-23	SS-16
	B-008-0-23	SS-11, SS-12, SS-13

Layer 2

	N_{60}	% Rec	HP	% Gr	% CS	% FS	% Silt	% Clay	LL	PL	PI	% WC		Short-Term Cohesion (psf) N-values PPR	Sowers	T & P	Correlated LT Cohesion (psf) per GB-7	phi (deg)	Midpoint Sample Depth (ft.)	Midpoint Sample Elevation (ft.)	Correlated Dry Unit Wt. (pcf) per GB-7	Correlated Moist Unit Wt. (pcf) per GB-7	Correlated C_c	Assumed Specific Gravity (G_s)	Computed Void Ratio (e)
Max	132	100	4.5	70	50	37	33	9	38	29	9	47	Max	N/A	N/A	N/A	N/A	40	27.0	760.5	125	140	0.252	2.72	0.691
Min	3	0	0.3	25	6	3	0	0	17	15	2	5	Min	N/A	N/A	N/A	N/A	24	8.0	739.0	100	120	0.063	2.65	0.353
Average	26	54	2.4	48	22	11	13	4	26	20	5	16	Average	N/A	N/A	N/A	N/A	32	17.2	750.8	113	132	0.140	2.71	0.500
Std Dev	22	33	2.1	13	10	7	7	2	6	4	2	8	Std Dev	N/A	N/A	N/A	N/A	4	5.1	4.9	7	6	0.051	0.01	0.097
Avg + Std	48	86	4.5	61	32	17	20	6	31	24	8	24	Avg + Std	N/A	N/A	N/A	N/A	36	22.3	755.8	120	137	0.190	2.72	0.598
Avg - Std	4	21	0.3	35	12	4	5	2	20	16	3	8	Avg - Std	N/A	N/A	N/A	N/A	28	12.1	745.9	106	126	0.089	2.70	0.403

Alignment	Surface Elevation	Exploration ID	From		To	Sample ID	N_{60}	% Rec	HP	% Gr	% CS	% FS	% Silt	% Clay	LL	PL	PI	% WC	ODOT Class.	Soil Type	Layer	Short-Term Cohesion (psf) N-values PPR	Sowers	T & P	Correlated LT Cohesion (psf) per GB-7	phi (deg)	Midpoint Sample Depth (ft.)	Midpoint Sample Elevation (ft.)	Correlated Dry Unit Wt. (pcf) per GB-7	Correlated Moist Unit Wt. (pcf) per GB-7	Correlated C_c	Assumed Specific Gravity (G_s)	Computed Void Ratio (e)
Ramp J	772.6	B-001-1-23	16.5	-	18	SS-11	49	33	-	-	-	-	-	-	-	-	-	7	A-1-b	Granular	2	N/A				36	17.0	755.6	120	135		2.71	0.409
Ramp J	772.6	B-001-1-23	18	-	19.5	SS-12	56	67	-	61	17	6	11	5	25	20	5	9	A-1-b	Granular	2	N/A				39	19.0	753.6	125	140	0.135	2.71	0.353
Ramp J	772.6	B-001-1-23	19.5	-	19.9	SS-13	50/5"	100	-	-	-	-	-	-	-	-	-	9	A-1-b	Granular	2	N/A				40	20.0	752.6	125	140		2.71	0.353
Ramp J	772.6	B-001-1-23	21	-	22	SS-14A	12	100	-	-	-	-	-	-	-	-	-	18	A-3	Granular	2	N/A				30	22.0	750.6	115	135		2.65	0.438
EB SR 4	770.9	B-002-1-23	15.5	-	15.92	SS-10	50/5"	100	-	-	-	-	-	-	-	-	-	10	A-1-a	Granular	2	N/A				40	16.0	754.9	125	140		2.71	0.353
EB SR 4	770.9	B-002-1-23	17	-	18.5	SS-11	132	89	-	50	30	8	7	5	21	15	6	9	A-1-a	Granular	2	N/A				40	18.0	752.9	125	140	0.099	2.71	0.353
EB SR 4	770.9	B-002-1-23	18.5	-	20	SS-12	32	33	-	-	-	-	-	-	-	-	-	13	A-1-a	Granular	2	N/A				34	19.0	751.9	120	135		2.71	0.409
EB SR 4	770.9	B-002-1-23	20	-	21.5	SS-13	23	89	-	-	-	-	-	-	-	-	-	19	A-1-a	Granular	2	N/A				33	21.0	749.9	115	135		2.71	0.470
EB SR 4	770.9	B-002-1-23	21.5	-	23	SS-14	16	56	-	-	-	-	-	-	-	-	-	12	A-1-a	Granular	2	N/A				31	22.0	748.9	115	135		2.71	0.470
EB SR 4	770.9	B-002-1-23	23	-	24.5	SS-15	10	89	-	25	18	15	-	-	NP	NP	NP	12	A-4a	NP SILT	2	N/A				25	24.0	746.9	110	130	N/A	2.72	0.543
EB SR 4	770.9	B-002-1-23	24.5	-	26	SS-16	12	17	-	-	-	-	-	-	-	-	NP	20	A-4a	NP SILT	2	N/A				25	25.0	745.9	115	135		2.72	0.476
EB SR 4	770.9	B-002-1-23	26	-	27.5	SS-17	29	56	-	-	-	-	-	-	-	-	NP	10	A-4a	NP SILT	2	N/A				29	27.0	743.9	115	135		2.72	0.476
Ramp L	770.3	B-003-1-23	9	-	10.5	SS-6	33	61	-	70	16	5	6	3	17	17	NP	5	A-1-a	Granular	2	N/A				34	10.0	760.3	115	130	0.063	2.71	0.470
Ramp L	770.3	B-003-1-23	10.5	-	12	SS-7	41	78	-	-	-	-	-	-	-	-	-	7	A-2-4	Granular	2	N/A				35	11.0	759.3	120	135		2.71	0.409
Ramp L	770.3	B-003-1-23	12	-	13.5	SS-8	46	22	-	-	-	-	-	-	-	-	-	9	A-2-4	Granular	2	N/A				36	13.0	757.3	120	135		2.71	0.409
Ramp L	770.3	B-003-1-23	13.5	-	15	SS-9	30	78	-	33	25	15	23	4	17	NP	NP	10	A-2-4	Granular	2	N/A				34	14.0	756.3	110	130	0.063	2.71	0.537
Ramp L	770.3	B-003-1-23	15	-	16.5	SS-10	20	33	-	-	-	-	-	-	-	-	-	23	A-2-4	Granular	2	N/A				32	16.0	754.3	110	130		2.71	0.537
Ramp L	770.3	B-003-1-23	16.5	-	18	SS-11	19	44	-	32	19	14	-	-	NP	NP	NP	31	A-2-4	Granular	2	N/A				32	17.0	753.3	110	130	N/A	2.71	0.537
Ramp L	770.3	B-003-1-23	18	-	19.5	SS-12	20	33	-	52	29	8	8	3	28	NP	NP	11	A-1-a	Granular	2	N/A				32	19.0	751.3	110	130	0.162	2.71	0.537
Ramp L	770.3	B-003-1-23	19.5	-	21	SS-13	25	17	-	-	-	-	-	-	-	-	-	11	A-1-a	Granular	2	N/A				33	20.0	750.3	110	130		2.71	0.537
Ramp L	770.3	B-003-1-23	21	-	22.5	SS-14	7	17	-	-	-	-	-	-	-	-	-	25	A-1-a	Granular	2	N/A				29	22.0	748.3	110	130		2.71	0.537
Ramp L	770.3	B-003-1-23	22.5	-	24	SS-15	13	33	-	-	-	-	-	-	-	-	-	18	A-1-a	Granular	2	N/A				31	23.0	747.3	115	135		2.71	0.470
Ramp L	770.3	B-003-1-23	24	-	25.5	SS-16	14	56	-	64	6	3	-	-	NP	NP	NP	6	A-2-4	Granular	2	N/A				31	25.0	745.3	115	135	N/A	2.71	0.470
Ramp L	770.3	B-003-1-23	25.5	-	27	SS-17	10	44	-	-	-	-	-	-	-	-	-	16	A-2-4	Granular	2	N/A				30	26.0	744.3	110	130		2.71	0.537
EB SR 4	768.1	B-004-1-23	11.5	-	13	SS-7	95	100	-	56	15	11	15	3	23	20	3	7	A-1-b	Granular	2	N/A				40	12.0	756.1	125	140	0.117	2.71	0.353
EB SR 4	768.1	B-004-1-23	13	-	14.5	SS-8	10	78	-	-	-	-	-	-	-	-	-	5	A-1-b	Granular	2	N/A				30	14.0	754.1	105	125		2.71	0.611
EB SR 4	768.1	B-004-1-23	14.5	-	16	SS-9	9	56	-	42	28	15	14	1	32	NP	NP	19	A-1-b	Granular	2	N/A				30	15.0	753.1	105	125	0.198	2.71	0.611
EB SR 4	768.1	B-004-1-23	16	-	17.5	SS-10	20	44	-	-	-	-	-	-	-	-	-	10	A-1-b	Granular	2	N/A				32	17.0	751.1	110	130		2.71	0.537
EB SR 4	768.1	B-004-1-23	17.5	-	19	SS-11	14	44	-	-	-	-	-	-	-	-	-	12	A-1-b	Granular	2	N/A				31	18.0	750.1	110	130		2.71	0.537
EB SR 4	768.1	B-004-1-23	19	-	20.5	SS-12	16	22	-	30	16	37	17	0	21	NP	NP	12	A-3a	Granular	2	N/A				31	20.0	748.1	110	130	0.099	2.65	0.503
EB SR 4	768.1	B-004-1-23	20.5	-	22	SS-13	32	22	-	-	-	-	-	-	-	-	-	12	A-3a	Granular	2	N/A				34	21.0	747.1	120	140		2.65	0.378
EB SR 4	768.1	B-004-1-23	22	-	23.5	SS-14	27	50	-	62	16	9	8	5	20	18	2	8	A-1-a	Granular	2	N/A				33	23.0	745.1	115	135	0.09	2.71	0.470
EB SR 4	768.1	B-004-1-23	23.5	-	25	SS-15	16	17	-	38	25	15	17	5	24	NP	NP	19	A-1-b	Granular	2	N/A				31	24.0	744.1	115	135	0.126	2.71	0.470
EB SR 4	768.1	B-004-1-23	25	-	26.5	SS-16	36	17	-	-	-	-	-	-	-	-	-	13	A-1-b	Granular	2	N/A				35	26.0	742.1	120	140		2.71	0.409
EB SR 4	768.1	B-004-1-23	26.5	-	28	SS-17	23	17	-	-	-	-	-	-	-	-	-	17	A-1-b	Granular	2	N/A				33	27.0	741.1	115	135		2.71	0.470
WB SR 4	771.5	B-004-2-23	10.5	-	12	SS-7	46	89	-	60	22	6	8	4	23	18	5	7	A-1-a	Granular	2	N/A				36	11.0	760.5	120	135	0.117	2.71	0.409
WB SR 4	771.5	B-004-2-23	12	-	13.5	SS-8	58	100	-	-	-	-	-	-	-	-	-	8	A-1-a	Granular	2	N/A				39	13.0	758.5	125	140		2.71	0.353
WB SR 4	771.5	B-004-2-23	13.5	-	15	SS-9	30	89	4.5	-	-	-	-	-	-	-	-	17	A-1-a	Granular	2	N/A				34	14.0	757.5	110	130		2.71	0.537
WB SR 4	771.5	B-004-2-23	15	-	16.5	SS-10	42	89	-	51	30	9	7	3	27	NP	NP	14	A-1-a	Granular	2	N/A				35	16.0	755.5	120	135	0.153	2.71	0.409
WB SR 4	771.5	B-004-2-23	16.5	-	18	SS-11	26	100	0.25	40	28	11	15	6	31	24	7	19	A-2-4	Granular	2	N/A				33	17.0	754.5	110	130	0.189	2.71	0.537
WB SR 4	771.5	B-004-2-23	18.5	-	20	SS-12	19	17	-	-	-	-	-	-	-	-	-	24	A-2-4	Granular	2	N/A				32	19.0	752.5	110	130		2.71	0.537
WB SR 4	771.5	B-004-2-23	20	-	21	SS-13A	41	89	-	-	-	-	-	-	-	-	-	14	A-2-4	Granular	2	N/A				35	21.0	750.5	120	140		2.71	0.409
WB SR 4	771.5	B-004-2-23	21	-	21.5	SS-13B	-	-	-	-	-	-	-	-	-	-	-	35	A-1-b	Granular	2	N/A					21.0	750.5	125			2.71	0.353
WB SR 4	771.5	B-004-2-23	21.5	-	23	SS-14	35	44	-	-	-	-	-	-	-	-	-	10	A-1-b	Granular	2	N/A				35	22.0	749.5	120	140		2.71	0.409
WB SR 4	771.5	B-004-2-23	23	-	24.5	SS-15	39	89	-	68	10	6	13	3	22	17	5	11	A-1-b	Granular	2	N/A				35	24.0	747.5	120	140	0.108	2.71	0.409
WB SR 4	768.1	B-008-0-23	9.5	-	11	SS-6	46	100	2.5	42	16	14	19	9	29	20	9	12	A-2-4	Granular	2	N/A				36	10.0	758.1	115	130	0.171	2.71	0.470
WB SR 4	768.1	B-008-0-23	11	-	11.5	SS-7A	27	89	-	-	-	-	-	-	-	-	-	13	A-2-4	Granular	2	N/A				33	11.0	757.1	110	130		2.71	0.537
WB SR 4	768.1	B-008-0-23	11.5	-	12.5	SS-7B	-	-	-	31	28	5	33	3	25	NP	NP	19	A-4a	NP SILT	2	N/A					12.0	756.1	125		0.135	2.72	0.358
WB SR 4	768.1	B-008-0-23	12.5	-	14	SS-8	10	56	-	-	-	-	-	-	-	-	NP	16	A-4a	NP SILT	2	N/A				25	13.0	755.1	105	125		2.72	0.616
WB SR 4	768.1	B-008-0-23	14	-	15.5	SS-9	10	0	-	-	-	-	-	-	-	-	NP	27	A-4a	NP SILT	2	N/A				25	15.0	753.1	105	125		2.72	0.616
WB SR 4	768.1	B-008-0-23	15.5	-	17	SS-10	7	89	-	-	-	-	-	-	-	-	NP	24	A-4a	NP SILT	2	N/A				24	16.0	752.1	105	125		2.72	0.616
EB SR 4	764.2	B-009-0-23	7.5	-	9	SS-5	42	78	-	-	-	-	-	-	-	-	-	9	A-1-b	Granular	2	N/A				35	8.0	756.2	115	130		2.71	0.470
EB SR 4	764.2	B-009-0-23	9	-	10.5	SS-6	29	67	-	-	-	-	-	-	-	-	-	13	A-1-b	Granular	2	N/A				34	10.0	754.2	105	125		2.71	0.611
EB SR 4	764.2	B-009-0-23	10.5	-	12	SS-7	22	0	-	-	-	-	-	-	-	-	-	-	A-1-b	Granular	2	N/A				32	11.0	753.2	110	130		2.71	0.537
EB SR 4	764.2	B-009-0-23	12	-	13.5	SS-8	13	67	-	58	9	15	15	3	28	26	2	10	A-1-b	Granular	2	N/A				31	13.0	751.2	110	130	0.162	2.71	0.537
EB SR 4	764.2	B-009-0-23	13.5	-	15	SS-9	9	17	-	33	16	15	-	-	NP	NP	NP	39	A-4a	NP SILT	2	N/A				25	14.0	750.2	105	125	N/A	2.72	0.616
EB SR 4	764.2	B-009-0-23	15	-	16.5	SS-10	10	11	-	-	-	-	-	-	-	-	NP	34	A-4a	NP SILT	2	N/A				25	16.0	748.2	105	125		2.72	0.616
EB SR 4	764.2	B-009-0-23	16.5	-	18	SS-11	7	6	-	-	-	-	-	-	-	-	NP	24	A-4a	NP SILT	2	N/A				24	17.0	747.2	105	125		2.72	0.616
EB SR 4	764.2	B-009-0-23	18	-	19.5	SS-12	10	6	-	-	-	-	-	-	-	-	NP	47	A-4a	NP SILT	2	N/A				25	19.0	745.2	105	125		2.72	0.616
EB SR 4	764.2	B-009-0-23	19.5	-	21	SS-13	7	6	-	-	-	-	-	-	-	-	NP	17	A-4a	NP SILT	2	N/A				24	20.0	744.2	105	125		2.72	0.616
Ramp K	763.0	B-010-0-23	7.5	-	8	SS-3B	-	-	-	-	-	-	-	-	-	-	-	21	A-1-b	Granular	2	N/A					8.0	755.0	120			2.71	0.409
Ramp K	763.0	B-010-0-23	8	-	9.5	SS-4	51	78	-	57	14	9	16	4	23	17	6	9	A-1-b	Granular	2	N/A				38	9.0	754.0	120	135	0.117	2.71	0.409
Ramp K	763.0	B-010-0-23	9.5	-	11	SS-5	20	100	-	-	-	-	-	-	-	-	-	13	A-1-a	Granular	2	N/A				32	10.0	753.0	105	125		2.71	0.611
Ramp K	763.0	B-010-0-23	11	-	12.5	SS-6	4	0	-	-	-	-	-	-	-	-	-	-	A-1-a	Granular	2	N/A				28	12.0	751.0	100	120		2.71	0.691
Ramp K	763.0	B-010-0-23	12.5	-	14	SS-7	4	44	-	60	23	10	6	1	38	NP	NP	18	A-1-a	Granular	2	N/A				28	13.0	750.0	100	120	0.252	2.71	0.691
Ramp K	763.0	B-010-0-23	14	-	15.5	SS-8	3	33	-	-	-	-	-	-	-	-	-	15	A-1-a	Granular	2	N/A				28	15.0	748.0	100	120		2.71	0.691
Ramp K	763.0	B-010-0-23	15.5	-	17	SS-9	4	44	-	-	-	-	-	-	-	-	-	22	A-1-b	Granular	2	N/A				28	16.0	747.0	100	120		2.71	0.691
Ramp K	763.0	B-010-0-23	17	-	18.5	SS-10	7	78	-	34	50	9	5	2	24	NP	NP	17	A-1-b	Granular	2	N/A				29	18.0	745.0	105	125	0.126	2.71	0.611
Ramp K	763.0	B-010-0-23	20	-	21.5	SS-11	4	17	-	-	-	-	-	-	-	-	-	20	A-1-b	Granular	2	N/A				28	21.0	742.0	105	125		2.71	0.611
Ramp K	763.0	B-010-0-23	21.5	-	23	SS-12	9	67	-	48	42	7	0	3	38	29	9	22	A-2-4	Granular	2	N/A				30	22.0	741.0	110	130	0.252	2.71	0.537
Ramp K	763.0	B-010-0-23	23	-	24.5	SS-13	46	22	-	-	-	-	-	-	-	-	-	16	A-2-4	Granular	2	N/A				36	24.0	739.0	120	140		2.71	0.409
EB SR 4	761.5	B-011-0-23	10	-	11.5	SS-7	67	89	-	-	-	-	-	-	-	-	-	12	A-1-b	Granular	2	N/A				40	11.0	750.5	125	140		2.71	0.353

Values for Soil Strength Correlation	
Reference	Value
HI PI (Sowers)	0.25
MD PI (Sowers)	0.175
LO PI (Sowers)	0.075
T&P	0.133

Omitted:	B-002-2-23	SS-6, SS-7
	B-010-0-23	SS-17

Layer 3

	N_{60}	% Rec	HP	% Gr	% CS	% FS	% Silt	% Clay	LL	PL	PI	% WC		Short-Term Cohesion (psf) N-values PPR	Sowers	T & P	Correlated LT Cohesion (psf) per GB-7	phi (deg)	Midpoint Sample Depth (ft.)	Midpoint Sample Elevation (ft.)	Correlated Dry Unit Wt. (pcf) per GB-7	Correlated Moist Unit Wt. (pcf) per GB-7	Correlated C_c	Assumed Specific Gravity (G_s)	Computed Void Ratio (e)
Max	84	100	N/A	71	49	75	19	9	22	18	5	22	Max	N/A	N/A	N/A	N/A	41	49.0	756.2	130	150	0.108	2.71	0.537
Min	9	17	N/A	1	4	1	0	0	12	11	3	4	Min	N/A	N/A	N/A	N/A	30	12.0	718.0	110	130	0.018	2.65	0.301
Average	41	81	N/A	53	29	10	5	2	15	16	4	9	Average	N/A	N/A	N/A	N/A	36	31.2	736.7	121	139	0.043	2.71	0.404
Std Dev	17	18	N/A	15	12	15	4	2	2	3	1	4	Std Dev	N/A	N/A	N/A	N/A	3	8.7	8.3	4	4	0.021	0.01	0.052
Avg + Std	58	99	N/A	68	41	25	9	4	17	19	5	12	Avg + Std	N/A	N/A	N/A	N/A	39	39.9	745.0	125	143	0.064	2.72	0.456
Avg - Std	24	64	N/A	37	17	-4	1	0	12	13	3	5	Avg - Std	N/A	N/A	N/A	N/A	33	22.5	728.4	116	136	0.021	2.70	0.352

Alignment	Surface Elevation	Exploration ID	From		To	Sample ID	N_{60}	% Rec	HP	% Gr	% CS	% FS	% Silt	% Clay	LL	PL	PI	% WC	ODOT Class.	Soil Type	Layer	Short-Term Cohesion (psf) N-values PPR	Sowers	T & P	Correlated LT Cohesion (psf) per GB-7	phi (deg)	Midpoint Sample Depth (ft.)	Midpoint Sample Elevation (ft.)	Correlated Dry Unit Wt. (pcf) per GB-7	Correlated Moist Unit Wt. (pcf) per GB-7	Correlated C_c	Assumed Specific Gravity (G_s)	Computed Void Ratio (e)
Ramp J	772.6	B-001-1-23	22.5	-	24	SS-15	32	100	-	-	-	-	-	-	-	-	-	6	A-1-a	Granular	3	N/A				34	23.0	749.6	120	140		2.71	0.409
Ramp J	772.6	B-001-1-23	24	-	25.5	SS-16	43	100	-	67	26	2	4	1	18	NP	NP	4	A-1-a	Granular	3	N/A				35	25.0	747.6	120	140	0.072	2.71	0.409
Ramp J	772.6	B-001-1-23	25.5	-	27	SS-17	49	100	-	-	-	-	-	-	-	-	-	4	A-1-a	Granular	3	N/A				36	26.0	746.6	120	140		2.71	0.409
Ramp J	772.6	B-001-1-23	28	-	29.5	SS-18	45	100	-	-	-	-	-	-	-	-	-	8	A-1-a	Granular	3	N/A				36	29.0	743.6	120	140		2.71	0.409
Ramp J	772.6	B-001-1-23	30.5	-	32	SS-19	61	89	-	60	32	3	4	1	16	NP	NP	5	A-1-a	Granular	3	N/A				40	31.0	741.6	125	140	0.054	2.71	0.353
Ramp J	772.6	B-001-1-23	33	-	34.5	SS-20	58	78	-	-	-	-	-	-	-	-	-	7	A-1-a	Granular	3	N/A				39	34.0	738.6	125	140		2.71	0.353
Ramp J	772.6	B-001-1-23	35.5	-	37	SS-21	41	67	-	-	-	-	-	-	-	-	-	10	A-1-a	Granular	3	N/A				35	36.0	736.6	120	140		2.71	0.409
Ramp J	772.6	B-001-1-23	38	-	39.5	SS-22	26	78	-	62	31	1	4	2	14	NP	NP	8	A-1-a	Granular	3	N/A				33	39.0	733.6	115	135	0.036	2.71	0.470
Ramp J	772.6	B-001-1-23	40.5	-	42	SS-23	51	67	-	-	-	-	-	-	-	-	-	9	A-1-a	Granular	3	N/A				38	41.0	731.6	130	150		2.71	0.301
EB SR 4	770.9	B-002-1-23	27.5	-	29	SS-18	48	100	-	-	-	-	-	-	-	-	-	4	A-2-4	Granular	3	N/A				36	28.0	742.9	120	140		2.71	0.409
EB SR 4	770.9	B-002-1-23	29	-	30.5	SS-19	43	100	-	35	27	9	-	-	NP	NP	NP	4	A-2-4	Granular	3	N/A				35	30.0	740.9	120	140	N/A	2.71	0.409
EB SR 4	770.9	B-002-1-23	31	-	32.5	SS-20	56	83	-	-	-	-	-	-	-	-	-	4	A-2-4	Granular	3	N/A				39	32.0	738.9	125	140		2.71	0.353
EB SR 4	770.9	B-002-1-23	33.5	-	35	SS-21	36	100	-	-	-	-	-	-	-	-	-	8	A-2-4	Granular	3	N/A				35	34.0	736.9	120	140		2.71	0.409
EB SR 4	770.9	B-002-1-23	36	-	37.5	SS-22	25	67	-	-	-	-	-	-	-	-	-	10	A-2-4	Granular	3	N/A				33	37.0	733.9	115	135		2.71	0.470
EB SR 4	770.9	B-002-1-23	38.5	-	40	SS-23	36	67	-	-	-	-	-	-	-	-	-	9	A-2-4	Granular	3	N/A				35	39.0	731.9	120	140		2.71	0.409
EB SR 4	770.9	B-002-1-23	41	-	42.5	SS-24	36	78	-	66	21	8	4	1	15	NP	NP	9	A-1-a	Granular	3	N/A				35	42.0	728.9	120	140	0.045	2.71	0.409
EB SR 4	770.9	B-002-1-23	43.5	-	45	SS-25	51	89	-	-	-	-	-	-	-	-	-	8	A-1-a	Granular	3	N/A				38	44.0	726.9	130	150		2.71	0.301
EB SR 4	770.9	B-002-1-23	46	-	47.5	SS-26	38	100	-	-	-	-	-	-	-	-	-	10	A-1-a	Granular	3	N/A				35	47.0	723.9	120	140		2.71	0.409
Ramp J	768.2	B-002-2-23	11.5	-	13	SS-8	41	100	-	-	-	-	-	-	-	-	-	7	A-1-b	Granular	3	N/A				35	12.0	756.2	120	135		2.71	0.409
Ramp J	768.2	B-002-2-23	13	-	14.5	SS-9	68	89	-	-	-	-	-	-	-	-	-	5	A-1-b	Granular	3	N/A				40	14.0	754.2	125	140		2.71	0.353
Ramp J	768.2	B-002-2-23	14.5	-	16	SS-10	61	100	-	46	35	12	4	3	13	NP	NP	8	A-1-b	Granular	3	N/A				40	15.0	753.2	125	140	0.027	2.71	0.353
Ramp J	768.2	B-002-2-23	16	-	17.5	SS-11	84	100	-	-	-	-	-	-	-	-	-	7	A-1-b	Granular	3	N/A				40	17.0	751.2	125	140		2.71	0.353
Ramp J	768.2	B-002-2-23	17.5	-	19	SS-12	21/32/50/4"	100	-	-	-	-	-	-	-	-	-	7	A-1-b	Granular	3	N/A				40	18.0	750.2	125	140		2.71	0.353
Ramp J	768.2	B-002-2-23	19	-	20.5	SS-13	56	89	-	-	-	-	-	-	-	-	-	7	A-1-b	Granular	3	N/A				39	20.0	748.2	125	140		2.71	0.353
Ramp J	768.2	B-002-2-23	20.5	-	22	SS-14	61	78	-	-	-	-	-	-	-	-	-	5	A-1-b	Granular	3	N/A				40	21.0	747.2	125	140		2.71	0.353
Ramp J	768.2	B-002-2-23	22	-	23.5	SS-15	17	67	-	43	22	15	19	1	21	18	3	9	A-1-b	Granular	3	N/A				31	23.0	745.2	115	135	0.099	2.71	0.470
Ramp J	768.2	B-002-2-23	23.5	-	25	SS-16	43	89	-	-	-	-	-	-	-	-	-	6	A-1-b	Granular	3	N/A				35	24.0	744.2	120	140		2.71	0.409
Ramp J	768.2	B-002-2-23	25	-	26.5	SS-17	51	89	-	-	-	-	-	-	-	-	-	5	A-1-a	Granular	3	N/A				38	26.0	742.2	125	140		2.71	0.353
Ramp J	768.2	B-002-2-23	26.5	-	28	SS-18	71	100	-	-	-	-	-	-	-	-	-	5	A-1-a	Granular	3	N/A				40	27.0	741.2	125	140		2.71	0.353
Ramp J	768.2	B-002-2-23	28	-	29.5	SS-19	74	89	-	61	33	2	3	1	14	NP	NP	5	A-1-a	Granular	3	N/A				40	29.0	739.2	125	140	0.036	2.71	0.353
Ramp J	768.2	B-002-2-23	29.5	-	31	SS-20	65	89	-	-	-	-	-	-	-	-	-	8	A-1-a	Granular	3	N/A				40	30.0	738.2	125	140		2.71	0.353
Ramp J	768.2	B-002-2-23	32	-	33.5	SS-21	65	89	-	-	-	-	-	-	-	-	-	10	A-1-a	Granular	3	N/A				40	33.0	735.2	125	140		2.71	0.353
Ramp L	770.3	B-003-1-23	27	-	28.5	SS-18	52	78	-	-	-	-	-	-	-	-	-	5	A-1-a	Granular	3	N/A				39	28.0	742.3	125	140		2.71	0.353
Ramp L	770.3	B-003-1-23	29.5	-	31	SS-19	68	100	-	57	22	11	7	3	15	NP	NP	4	A-1-a	Granular	3	N/A				40	30.0	740.3	125	140	0.045	2.71	0.353
Ramp L	770.3	B-003-1-23	32	-	33.5	SS-20	41	89	-	-	-	-	-	-	-	-	-	15	A-1-b	Granular	3	N/A				35	33.0	737.3	120	140		2.71	0.409
Ramp L	770.3	B-003-1-23	34.5	-	36	SS-21	14	100	-	46	49	1	1	3	14	NP	NP	10	A-1-b	Granular	3	N/A				31	35.0	735.3	115	135	0.036	2.71	0.470
Ramp L	770.3	B-003-1-23	37	-	38.5	SS-22	26	89	-	-	-	-	-	-	-	-	-	10	A-1-a	Granular	3	N/A				33	38.0	732.3	115	135		2.71	0.470
Ramp L	770.3	B-003-1-23	39.5	-	41	SS-23	45	67	-	52	45	1	0	2	13	NP	NP	9	A-1-a	Granular	3	N/A				36	40.0	730.3	120	140	0.027	2.71	0.409
Ramp L	770.3	B-003-1-23	42	-	43.5	SS-24	32	89	-	-	-	-	-	-	-	-	-	12	A-1-b	Granular	3	N/A				34	43.0	727.3	120	140		2.71	0.409
Ramp L	770.3	B-003-1-23	44.5	-	46	SS-25	35	78	-	46	25	21	6	2	15	NP	NP	12	A-1-b	Granular	3	N/A				35	45.0	725.3	120	140	0.045	2.71	0.409
EB SR 4	768.1	B-004-1-23	28	-	29.5	SS-18	13	17	-	-	-	-	-	-	-	-	-	8	A-1-b	Granular	3	N/A				31	29.0	739.1	115	135		2.71	0.470
EB SR 4	768.1	B-004-1-23	29.5	-	30	SS-19	33	78	-	-	-	-	-	-	-	-	-	7	A-1-a	Granular	3	N/A				34	30.0	738.1	120	140		2.71	0.409
EB SR 4	768.1	B-004-1-23	31	-	32.5	SS-20	35	78	-	60	20	12	7	1	16	NP	NP	11	A-1-a	Granular	3	N/A				35	32.0	736.1	120	140	0.054	2.71	0.409
EB SR 4	768.1	B-004-1-23	33.5	-	35	SS-21	32	100	-	-	-	-	-	-	-	-	-	12	A-1-b	Granular	3	N/A				34	34.0	734.1	120	140		2.71	0.409
EB SR 4	768.1	B-004-1-23	36	-	37.5	SS-22	22	100	-	41	37	16	5	1	13	NP	NP	14	A-1-b	Granular	3	N/A				32	37.0	731.1	115	135	0.027	2.71	0.470
EB SR 4	768.1	B-004-1-23	38.5	-	40	SS-23	17	100	-	-	-	-	-	-	-	-	-	15	A-1-b	Granular	3	N/A				31	39.0	729.1	115	135		2.71	0.470
EB SR 4	768.1	B-004-1-23	41	-	42.5	SS-24	25	78	-	-	-	-	-	-	-	-	-	10	A-1-a	Granular	3	N/A				33	42.0	726.1	115	140		2.71	0.470
EB SR 4	768.1	B-004-1-23	43.5	-	45	SS-25	14	44	-	56	42	1	0	1	13	NP	NP	13	A-1-a	Granular	3	N/A				31	44.0	724.1	115	140	0.027	2.71	0.470
EB SR 4	768.1	B-004-1-23	46	-	47.5	SS-26	16	44	-	-	-	-	-	-	-	-	-	10	A-1-a	Granular	3	N/A				31	47.0	721.1	115	140		2.71	0.470
EB SR 4	768.1	B-004-1-23	48.5	-	50	SS-27	25	56	-	-	-	-	-	-	-	-	-	14	A-1-a	Granular	3	N/A				33	49.0	719.1	115	140		2.71	0.470
WB SR 4	771.5	B-004-2-23	26	-	27.5	SS-17	51	67	-	-	-	-	-	-	-	-	-	8	A-1-a	Granular	3	N/A				38	27.0	744.5	125	140		2.71	0.353
WB SR 4	771.5	B-004-2-23	27.5	-	29	SS-18	32	78	-	55	26	6	10	3	22	17	5	7	A-1-a	Granular	3	N/A				34	28.0	743.5	120	140	0.108	2.71	0.409
WB SR 4	771.5	B-004-2-23	30	-	30.42	SS-19	50/5"	60	-	-	-	-	-	-	-	-	-	12	A-1-a	Granular	3	N/A				40	30.0	741.5	125	140		2.71	0.353
WB SR 4	771.5	B-004-2-23	32.5	-	34	SS-20	62	89	-	-	-	-	-	-	-	-	-	8	A-1-a	Granular	3	N/A				40	33.0	738.5	125	140		2.71	0.353
WB SR 4	771.5	B-004-2-23	35	-	36.5	SS-21	67	83	-	65	15	12	6	2	13	NP	NP	10	A-1-a	Granular	3	N/A				40	36.0	735.5	125	140	0.027	2.71	0.353
WB SR 4	771.5	B-004-2-23	37.5	-	39	SS-22	68	100	-	-	-	-	-	-	-	-	-	11	A-1-a	Granular	3	N/A				40	38.0	733.5	125	140		2.71	0.353
WB SR 4	771.5	B-004-2-23	40	-	41.5	SS-23	68	100	-	53	43	2	0	2	14	11	3	10	A-1-a	Granular	3	N/A				40	41.0	730.5	130	150	0.036	2.71	0.301
WB SR 4	771.5	B-004-2-23	42.5	-	44	SS-24	80	100	-	-	-	-	-	-	-	-	-	10	A-1-a	Granular	3	N/A				41	43.0	728.5	130	150		2.71	0.301
WB SR 4	771.5	B-004-2-23	45	-	46.5	SS-25	49	100	-	-	-	-	-	-	-	-	-	10	A-1-a	Granular	3	N/A				36	46.0	725.5	120	140		2.71	0.409
WB SR 4	768.1	B-008-0-23	21.5	-	23	SS-14	33	67	-	-	-	-	-	-	-	-	-	4	A-1-a	Granular	3	N/A				34	22.0	746.1	120	140		2.71	0.409
WB SR 4	768.1	B-008-0-23	23	-	24.5	SS-15	33	89	-	70	14	6	7	3	15	18	NP	5	A-1-a	Granular	3	N/A				34	24.0	744.1	120	140	0.045	2.71	0.409
WB SR 4	768.1	B-008-0-23	25.5	-	27	SS-16	56	78	-	-	-	-	-	-	-	-	-	4	A-1-a	Granular	3	N/A				39	26.0	742.1	125	140		2.71	0.353
WB SR 4	768.1	B-008-0-23	27.5	-	29	SS-17	33	67	-	-	-	-	-	-	-	-	-	5	A-1-a	Granular	3	N/A				34	28.0	740.1	120	140		2.71	0.409
WB SR 4	768.1	B-008-0-23	30	-	31.5	SS-18	38	78	-	57	26	10	5	2	13	NP	NP	9	A-1-a	Granular	3	N/A				35	31.0	737.1	120	140	0.027	2.71	0.409
WB SR 4	768.1	B-008-0-23	32.5	-	34	SS-19	36	89	-	-	-	-	-	-	-	-	-	11	A-1-a	Granular	3	N/A				35	33.0	735.1	120	140		2.71	0.409
WB SR 4	768.1	B-008-0-23	35	-	36.5	SS-20	64	89	-	-	-	-	-	-	-	-	-	9	A-1-a	Granular	3	N/A				40	36.0	732.1	125	140		2.71	0.353
WB SR 4	768.1	B-008-0-23	37.5	-	39	SS-21	43	67	-	-	-	-	-	-	-	-	-	12	A-1-a	Granular	3	N/A				35	38.0	730.1	120	140		2.71	0.409
EB SR 4	764.2	B-009-0-23	21	-	22.5	SS-14	38	44	-	-	-	-	-	-	-	-	-	6	A-1-a	Granular	3	N/A				35	22.0	742.2	120	140		2.71	0.409
EB SR 4	764.2	B-009-0-23	23.5	-	25	SS-15	30	78	-	-	-	-	-	-	-	-	-	4	A-1-a	Granular	3	N/A				34	24.0	740.2	115	135		2.71	0.470
EB SR 4	764.2	B-009-0-23	26	-	27.5	SS-16	35	89	-	59	23	10	7	1	15	NP	NP	8	A-1-a	Granular	3	N/A				35	27.0	737.2	120	140	0.045	2.71	0.409
EB SR 4	764.2	B-009-0-23	28.5	-	30	SS-17	32	100	-	-	-	-	-	-	-	-	-	9	A-1-a	Granular	3	N/A				34	29.0	735.2	120	140		2.71	0.409
EB SR 4	764.2	B-009-0-23	31	-	32.5	SS-18	33	89	-	-	-	-	-	-	-	-	-	8	A-1-a	Granular	3	N/A				34	32.0	732.2	120	140		2.71	0.409

EB SR 4	764.2	B-009-0-23	33.5	-	35	SS-19	26	100	-	16	42	32	8	2	12	NP	NP	16	A-1-b	Granular	3	N/A	33	34.0	730.2	115	135	0.018	2.71	0.470
Ramp K	763.0	B-010-0-23	24.5	-	26	SS-14	20	44	-	-	-	-	-	-	-	-	-	10	A-2-4	Granular	3	N/A	32	25.0	738.0	115	135		2.71	0.470
Ramp K	763.0	B-010-0-23	27	-	28.5	SS-15	9	67	-	59	29	9	2	1	14	NP	NP	13	A-1-a	Granular	3	N/A	30	28.0	735.0	110	130	0.036	2.71	0.537
Ramp K	763.0	B-010-0-23	29.5	-	31	SS-16	14	78	-	-	-	-	-	-	-	-	-	11	A-1-a	Granular	3	N/A	31	30.0	733.0	115	135		2.71	0.470
Ramp K	763.0	B-010-0-23	35	-	36.5	SS-18	19	44	-	71	4	2	-	-	NP	NP	NP	9	A-1-b	Granular	3	N/A	32	36.0	727.0	115	135	N/A	2.71	0.470
Ramp K	763.0	B-010-0-23	37	-	38.5	SS-19	19	44	-	-	-	-	-	-	-	-	-	11	A-1-b	Granular	3	N/A	32	38.0	725.0	115	135		2.71	0.470
Ramp K	763.0	B-010-0-23	39.5	-	41	SS-20	10	67	-	48	48	2	1	1	14	NP	NP	9	A-1-b	Granular	3	N/A	30	40.0	723.0	110	130	0.036	2.71	0.537
Ramp K	763.0	B-010-0-23	42	-	43.5	SS-21	36	33	-	-	-	-	-	-	-	-	-	22	A-1-b	Granular	3	N/A	35	43.0	720.0	120	140		2.71	0.409
Ramp K	763.0	B-010-0-23	44.5	-	45.25	SS-22	40/50/3"	89	-	-	-	-	-	-	-	-	-	10	A-1-b	Granular	3	N/A	40	45.0	718.0	130	150		2.71	0.301
EB SR 4	761.5	B-011-0-23	13	-	14.5	SS-9	35	89	-	-	-	-	-	-	-	-	-	4	A-1-a	Granular	3	N/A	35	14.0	747.5	120	135		2.71	0.409
EB SR 4	761.5	B-011-0-23	14.5	-	16	SS-10	43	89	-	64	19	7	1	9	16	NP	NP	5	A-1-a	Granular	3	N/A	35	15.0	746.5	120	135	0.054	2.71	0.409
EB SR 4	761.5	B-011-0-23	16	-	17.5	SS-11	45	78	-	-	-	-	-	-	-	-	-	4	A-1-a	Granular	3	N/A	36	17.0	744.5	120	135		2.71	0.409
EB SR 4	761.5	B-011-0-23	17.5	-	19	SS-12	58	78	-	-	-	-	-	-	-	-	-	6	A-1-a	Granular	3	N/A	39	18.0	743.5	125	140		2.71	0.353
EB SR 4	761.5	B-011-0-23	20	-	21.5	SS-13	38	78	-	-	-	-	-	-	-	-	-	5	A-1-a	Granular	3	N/A	35	21.0	740.5	120	140		2.71	0.409
EB SR 4	761.5	B-011-0-23	22.5	-	24	SS-14	30	78	-	-	-	-	-	-	-	-	-	11	A-1-a	Granular	3	N/A	34	23.0	738.5	115	135		2.71	0.470
EB SR 4	761.5	B-011-0-23	25	-	26.5	SS-15	23	78	-	55	43	1	1	0	13	NP	NP	13	A-1-a	Granular	3	N/A	33	26.0	735.5	115	135	0.027	2.71	0.470
EB SR 4	761.5	B-011-0-23	27.5	-	29	SS-16	48	78	-	-	-	-	-	-	-	-	-	10	A-1-a	Granular	3	N/A	36	28.0	733.5	120	140		2.71	0.409
EB SR 4	761.5	B-011-0-23	30	-	31.5	SS-17	42	89	-	-	-	-	-	-	-	-	-	10	A-1-a	Granular	3	N/A	35	31.0	730.5	120	140		2.71	0.409
EB SR 4	761.5	B-011-0-23	32.5	-	34	SS-18	41	78	-	1	11	75	12	1	13	NP	NP	20	A-3a	Granular	3	N/A	35	33.0	728.5	120	140	0.027	2.65	0.378

Values for Soil Strength Correlation	
Reference	**Value**
HI PI (Sowers)	0.25
MD PI (Sowers)	0.175
LO PI (Sowers)	0.075
T&P	0.133

Layer 4

	N_{60}	% Rec	HP	% Gr	% CS	% FS	% Silt	% Clay	LL	PL	PI	% WC		Short-Term Cohesion (psf) N-values PPR	Sowers	T & P	Correlated LT Cohesion (psf) per GB-7	phi (deg)	Midpoint Sample Depth (ft.)	Midpoint Sample Elevation (ft.)	Correlated Dry Unit Wt. (pcf) per GB-7	Correlated Moist Unit Wt. (pcf) per GB-7	Correlated C_c	Assumed Specific Gravity (G_s)	Computed Void Ratio (e)
Max	67	100	4.5	30	14	21	36	23	18	12	6	12	**Max**	4500	4000	4000	250	28	42.0	727.2	135	145	0.072	2.72	0.476
Min	10	50	4.5	16	8	17	26	8	15	10	5	10	**Min**	4500	750	1330	114	23	37.0	722.2	115	130	0.045	2.72	0.257
Average	35	82	4.5	24	12	20	31	14	17	11	6	11	**Average**	4500	2308	3062	187	25	39.8	725.4	123	137	0.060	2.72	0.383
Std Dev	29	22	N/A	7	3	2	5	8	2	1	1	1	**Std Dev**	N/A	1629	1502	69	3	2.2	2.3	10	8	0.014	0.00	0.113
Avg + Std	64	104	N/A	31	15	22	36	22	18	12	6	12	**Avg + Std**	N/A	3937	4564	256	28	42.0	727.8	134	144	0.074	2.72	0.495
Avg - Std	6	60	N/A	17	8	17	26	6	15	10	5	10	**Avg - Std**	N/A	679	1560	118	23	37.5	723.1	113	129	0.046	2.72	0.270

Alignment	Surface Elevation	Exploration ID	From		To	Sample ID	N_{60}	% Rec	HP	% Gr	% CS	% FS	% Silt	% Clay	LL	PL	PI	% WC	ODOT Class.	Soil Type	Layer	Short-Term Cohesion (psf) N-values PPR	Sowers	T & P	Correlated LT Cohesion (psf) per GB-7	phi (deg)	Midpoint Sample Depth (ft.)	Midpoint Sample Elevation (ft.)	Correlated Dry Unit Wt. (pcf) per GB-7	Correlated Moist Unit Wt. (pcf) per GB-7	Correlated C_c	Assumed Specific Gravity (G_s)	Computed Void Ratio (e)
WB SR 4	768.1	B-008-0-23	40	-	41.5	SS-22	67	89	4.5	16	8	17	36	23	18	12	6	10	A-4a	Cohesive	4	4500	4000	4000	250	28	41.0	727.1	135	145	0.072	2.72	0.257
EB SR 4	764.2	B-009-0-23	36	-	37.5	SS-20	29	100	-	-	-	-	-	-	-	-	-	12	A-4a	Cohesive	4	N/A	2175	3857	197	26	37.0	727.2	120	135		2.72	0.414
EB SR 4	764.2	B-009-0-23	38.5	-	40	SS-21	10	89	-	25	14	21	32	8	17	11	6	11	A-4a	Cohesive	4	N/A	750	1330	114	23	39.0	725.2	115	130	0.063	2.72	0.476
EB SR 4	764.2	B-009-0-23	41	-	43	ST-22	ST	50	-	30	13	21	26	10	15	10	5	11	A-4a	Cohesive	4	N/A	N/A	N/A			42.0	722.2			0.045	2.72	

Soil layers Boring	GS Elev	Asphalt Thickness (in)	Reinforced Concrete Thickness (IN)	Granular Base Thickness (in)	Layer	Surficial Layer Thickness Ft.	1 Granular Embankment Bottom Depth	2 Refuse Material Bottom Depth	3 Granular Soil Bottom Depth	Bottom Elevation	Surficial Layer Thickness	1 Granular Embankment Thickness	2 Refuse Material thickness	3 Granular Soil Thickness
B-001-1-23	772.6	12			Surficial	1				771.6	1.00			
					1		16.5			756.1		15.50		
					2			22.5		750.1			6.00	
					3				42	730.6				19.50
B-002-1-23	770.9	12	9		Surficial	1.75				770.8	1.75			
					1		15.5			755.4		13.75		
					2			27.5		743.4			12.00	
					3				47.5	723.4				20.00
B-002-2-23	768.2	9			Surficial	0.75				771.8	0.75			
					1		4			764.2		3.25		
					2			11		757.2			7.00	
					3				33.5	734.7				22.50
B-003-1-23	770.3	14			Surficial	1.17				771.4	1.17			
					1		9			761.3		7.83		
					2			27.25		743.0			18.25	
					3				46	724.3				18.75
B-004-1-23	768.1	27			Surficial	2.25				770.3	2.25			
					1		12			756.1		9.75		
					2			29.5		738.6			17.50	
					3				50	718.1				20.50
B-004-2-23	771.5	10	8		Surficial	1.5				771.1	1.50			
					1		10.5			761.0		9.00		
					2			25.5		746.0			15.00	
					3				46.5	725.0				21.00
B-008-0-23	768.1	13			Surficial	1.08				771.5	1.08			
					1		9			759.1		7.92		
					2			21.5		746.6			12.50	
					3				41.5	726.6				20.00
B-009-0-23	764.2	17			Surficial	1.42				771.1	1.42			
					1		8			756.2		6.58		
					2			21		743.2			13.00	
					3				43	721.2				22.00
B-010-0-23	763	30	12		Surficial	3.5				769.1	3.50			
					1		7.5			755.5		4.00		
					2			25		738.0			17.50	
					3				45.3	717.7				20.30
B-011-0-23	761.5	12			Surficial	1				771.6	1.00			
					1		10			751.5		9.00		
					2			13.5		748.0			3.50	
					3				34	727.5				20.50

Boring	GS Elev	Asphalt	Reinforced Concrete	Granular Base		= Surficial Layer	1 Granular Embankment Bottom Depth	2 Refuse Material Bottom Depth	3 Granular Soil Bottom Depth		Surficial Layer Thickness	1 Granular Embankment Thickness	2 Refuse Material thickness	3 Granular Soil Thickness
	Max	30.0	12.0	0.0	Max	3.5	16.5	29.5	50.0	Max	3.5	15.5	18.3	22.5
	Min	9.0	8.0	0.0	Min	0.8	4.0	11.0	33.5	Min	0.8	3.3	3.5	18.8
	Average	15.6	9.7	NA	Average	1.5	10.2	22.4	42.9	Average	1.5	8.7	12.2	20.5
	Std Dev	7.2	2.1	NA	Std Dev	0.8	3.7	6.0	5.5	Std Dev	0.8	3.8	5.2	1.1
					Avg + Std Dev	2.4	13.9	28.5	48.4	Avg + Std Dev	2.4	12.5	17.4	21.6
					Avg - Std Dev	0.7	6.5	16.4	37.5	Avg - Std Dev	0.7	4.8	7.0	19.4
										Refuse Elevations				

Grain Size of Existing Embankment Material (approximately upper 6 feet)

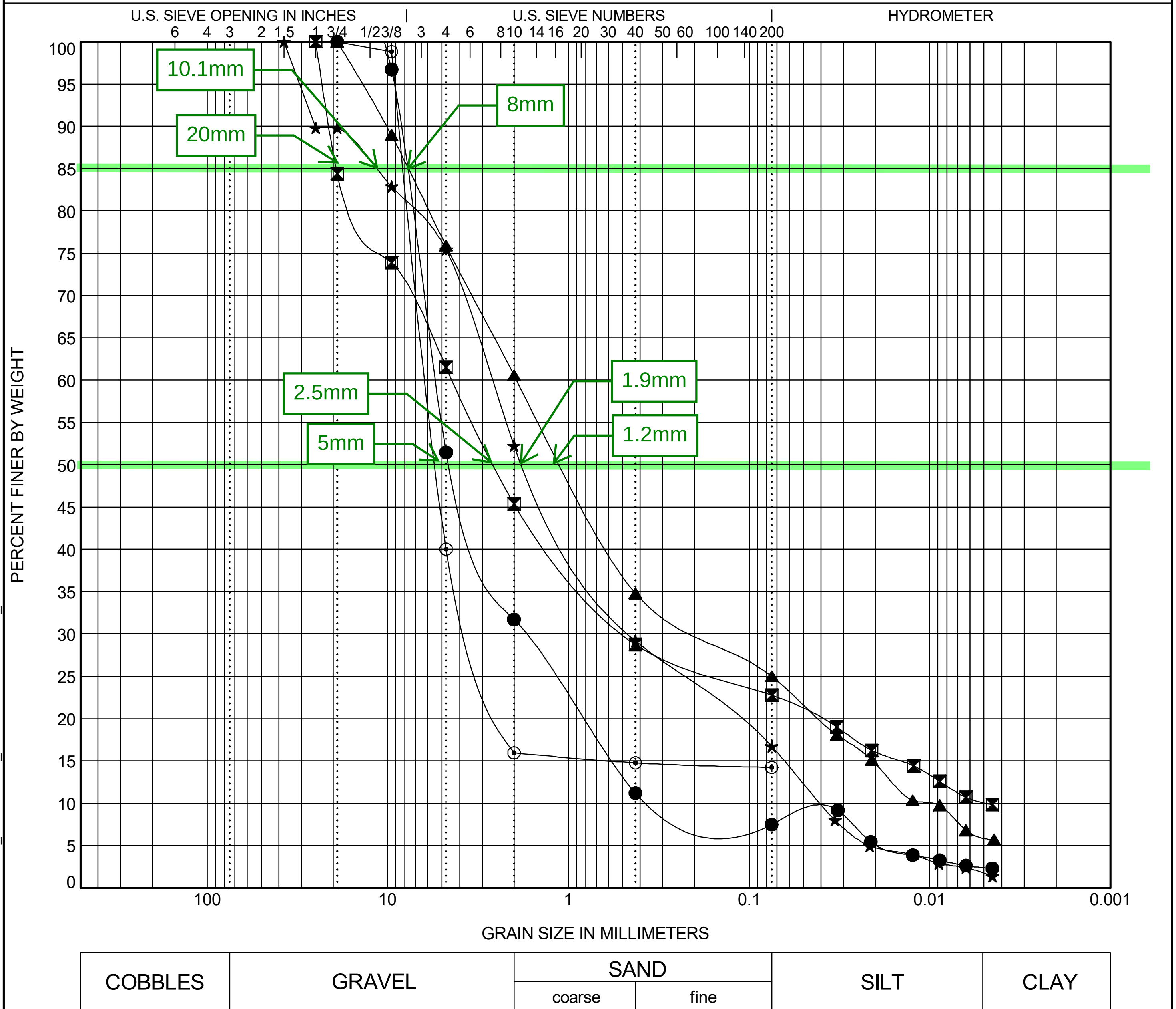
< 3 inches
<15% passing the No. 200

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTATION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT MOT-4-19.30 **PID** 117239

OGE NUMBER 111112 **PROJECT TYPE** ROADWAY



	Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	B-001-1-23	1.0	**A-1-a ~ WELL-GRADED GRAVEL with SILT and SAND(GW-GM)**	15	15	NP
⊠	B-001-1-23	4.5	**A-1-b ~ SILTY, CLAYEY SAND with GRAVEL(SC-SM)**	27	21	6
▲	B-002-1-23	5.0	**A-2-4 ~ CLAYEY SAND with GRAVEL(SC)**	21	13	8
★	B-003-1-23	1.2	**A-1-b ~ SILTY SAND with GRAVEL(SM)**	16	NP	NP
⊙	B-004-1-23	4.0	**A-1-a ~ SILTY GRAVEL with SAND(GM)**	NP	NP	NP

	Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	B-001-1-23	1.0	8.571	4.456	1.758	0.094	68	21	4	5	2	6.06	57.44
⊠	B-001-1-23	4.5	20.968	2.561	0.478	0.005	54	17	6	13	10	10.84	907.60
▲	B-002-1-23	5.0	10.148	1.059	0.182	0.01	39	26	10	19	6	1.67	189.44
★	B-003-1-23	1.2	25.178	1.719	0.449	0.04	48	23	12	15	2	1.88	66.30
⊙	B-004-1-23	4.0	8.561	5.343	3.314		84	1	1	14			

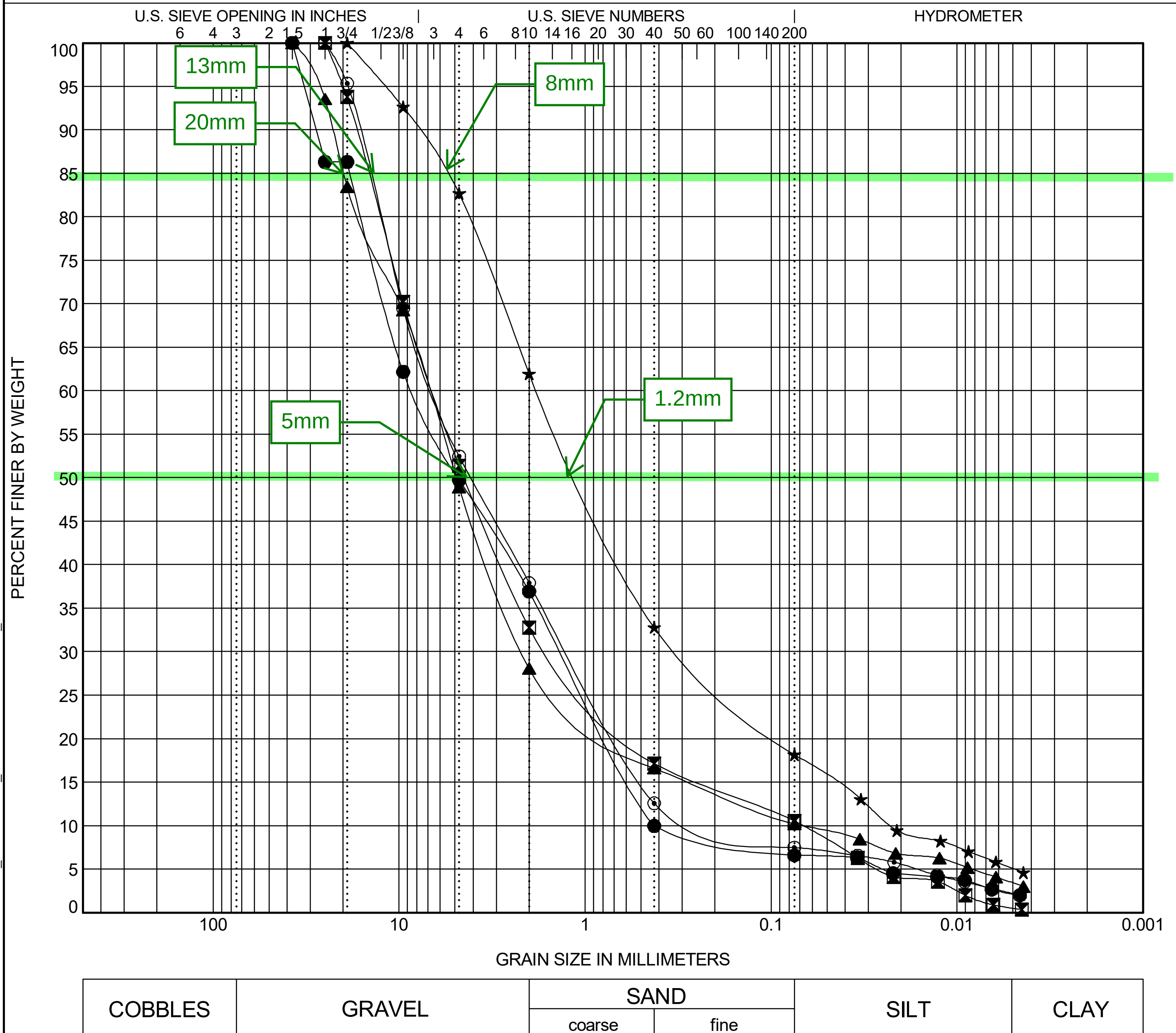
Fines < 15%, and Well-Graded (Cu>4, 1.0<Cc<3.0) Acceptable Ranges Marginal Ranges

GRAIN SIZE DISTRIBUTION

OHIO DEPARTMENT OF TRANSPORTATION
OFFICE OF GEOTECHNICAL ENGINEERING

PROJECT MOT-4-19.30 **PID** 117239

OGE NUMBER 111112 **PROJECT TYPE** ROADWAY



	Specimen Identification		ODOT (Modified AASHTO) ~ USCS Classification	LL	PL	PI
●	**B-004-2-23**	**3.0**	**A-1-a ~ POORLY GRADED GRAVEL with SILTY CLAY and SAND(GP-GC)**	**21**	**16**	**5**
⧗	**B-008-0-23**	**3.5**	**A-1-a ~ POORLY GRADED GRAVEL with SILTY CLAY and SAND(GP-GC)**	**19**	**15**	**4**
▲	**B-009-0-23**	**4.5**	**A-1-a ~ POORLY GRADED GRAVEL with SILT and SAND(GP-GM)**	**18**	**16**	**2**
★	**B-010-0-23**	**3.5**	**A-1-b ~ SILTY SAND with GRAVEL(SM)**	**12**	**NP**	**NP**
⊙	**B-011-0-23**	**2.5**	**A-1-a ~ WELL-GRADED GRAVEL with SILT and SAND(GW-GM)**	**18**	**15**	**3**

	Specimen Identification		D90	D50	D30	D10	%G	%CS	%FS	%M	%C	Cc	Cu
●	**B-004-2-23**	**3.0**	**27.891**	**4.826**	**1.344**	**0.426**	**63**	**27**	**3**	**5**	**2**	**0.50**	**19.75**
⧗	**B-008-0-23**	**3.5**	**17.009**	**4.459**	**1.526**	**0.068**	**66**	**16**	**7**	**10**	**1**	**5.22**	**95.66**
▲	**B-009-0-23**	**4.5**	**22.695**	**4.926**	**2.169**	**0.069**	**73**	**11**	**6**	**7**	**3**	**9.87**	**100.68**
★	**B-010-0-23**	**3.5**	**7.905**	**1.06**	**0.306**	**0.023**	**38**	**29**	**15**	**13**	**5**	**2.29**	**79.63**
⊙	**B-011-0-23**	**2.5**	**16.468**	**4.103**	**1.234**	**0.178**	**63**	**25**	**5**	**5**	**2**	**1.32**	**36.28**

Fines < 15%, and Well-Graded (Cu>4, 1.0<Cc<3.0) | Acceptable Ranges | Marginal Ranges

Potential Void Analysis

5 ft Void Width (Infinite Length) or Radius (Circular Void)

Reduction Values:	RFID = 1.2		RFCR = 3		RFCBD = 2			
Geogrid Layer	**Depth below top of pavement**	**Vertical Stress on the Structure or Reinforcement layer, σz**		**Required Tensile Strength, T_{reqd}**		**Geogrid**	**Allowable Tensile Strength**	**Factor of Safety**	
No.	**(ft)**	**(psf)**		**lb / ft**		**Type**	**lb / ft**	**Cumulative**	**Incremental**
		Cumulative	**Incremental[1]**	**Cumulative**	**Incremental[1]**				
1	1	144.2	-	1821	-	NX850[2]	3332	1.8	1.8
2	2	237.3	93.1	2996	1175	NX850[2]	3332	1.1	2.8
3	3	337.1	99.7	4256	1259	NX850[2]	3332	0.8	2.6

1. Incremental values based on the difference between layer cumulative value less the cumulative value of the overlying layer.

2. Ultimate Strength Based on values for Triax TX190L (NX850 not available)

3. RFID = Reduction Factor For Install Damage

4. RFCR = Reduction Factor for Avoiding Lifetime Creep

5. RFCBD = Reduction Factor for Chemical and Biological Degradation

Assumptions :

As the subgrade begins to settle, there is a loss of support beneath the geogrid (void or subsidence). It is assumed this subsidence will propagate from the lowest (3rd layer) of geogrid, to the middle (2nd layer), and finally to the top (1st layer). As such, it is assumed there is a moment where the 1st layer is acting independently to support the material above it.

In turn, there is a moment where the second layer is supporting the material above it, which includes the 1st layer and material above it. However, for simplicity's sake, it is assumed that the load carried by the 1st layer will not transfer to the 2nd layer. As such, the vertical stress (and thereby required tensile strength) of the 2nd layer is reduced by an amount equal to the vertical stress (and required tensile strength) determined for the 1st layer.

This same process is carried through to the 3rd layer where the determined vertical stress and required tensile strength for the 3rd layer is reduced by the cumulative vertical stress and required tensile strength calculated for the 2nd layer.

Project:	MOT-4-19.30	Computed: DCM	Date: 11/13/2023
Project No.:	10368622	Checked: DMV	Date: 12/6/2023
Subject:	Pavement Design - 5 ft Void	Page: 1	of 2
Task:	Reinforced Earth Mat Settlement - Upper GeoGrid Layer		

REFERENCE:

1) *Giroud, Jeanpierre & Bonaparte, Rudolph & Beech, J. & Gross, Beth. (1988). Load-carrying capacity of a soil layer supported by a geosynthetic overlying a void. Proceedings of the International Geotechnical Symposium on Theory and Practice of Earth Reinforcement. 185-190.*
2) *Giroud, Jeanpierre & Bonaparte, Rudolph & Beech, J.F. & Gross, Beth. (1990). Design of soil layer-geosynthetic systems overlying voids. Geotextiles and Geomembranes. 9. 11-50. 10.1016/0266-1144(90)90004-V.*
3) *Koerner, Robert M.. "Designing with Geosynthetics." Fifth Edition. (2005).*
4) *Sloan, Joel A., Filz, George M., Collin, Fames G., Kumar, Pawan. "Column-Supported Embankments: Full-Scale Tests and Design Recommendations." 2nd Edition. Virginia Tech Center for Geotechnical Practice Research (CGRP) Report 77. (2014).*
5) *Federal Highway Administration (FHWA), Elias, Victor et al. "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes: Design and Construction Guidelines (Updated Version)" , FHWA-NHI-00-043, 2001*

Vertical Stress on the Structure or Reinforcement layer, σz =	**144.2**	psf
Avg Unit Weight of Material Above the Settlement Area, Yavg =	130	pcf
Size of void (b,r) =	5	feet
for	*Circular Voids, use Radius, r*	
	Infinitiely long voids use Width, b	
Total Height Above Settlement Zone, H =	1	feet
Geosynthetic Deflection, y =	0.25	feet
Surcharge Pressure Placed at Ground Surface, q =	250	psf

$$\sigma_z = 2(y_{avg})(b)\left[1 - e^{-0.5H/b}\right] + qe^{-0.5\,H/b}$$ (Koerner Eq. 3.11) (Giroud Eq 4)

Simplified to: $\sigma_z = 2(y_{avg})R,\ \ for\ H \geq 6R$

Required Tensile Strength, Treqd =	**1821**	lbs/ft
Dimensionless Factor, Ω =	**2.53**	dim

$T_{reqd} = \sigma_z R\Omega$ (Koerner Eq 3.13) (see also α = pbΩ Giroud Eq. 7)

$where,\ \Omega = 0.25\left[\frac{(2y)}{B} + \frac{B}{(2y)}\right]$ (Koerner Eq. 3.14) (Giroud Eq. 6)

Allowable Tensile Strength to be used for Final Design, Tallow =	**3332**	lbs / ft	
Ultimate Tensile Strength from A Standard In-Isolation Tensile Test, Tult =	**23989**	lbs / ft	(Total of all layers)
Ultimate Tensile Strength from A Standard In-Isolation Tensile Test, Tult =	23,989	lb / ft	(per Layer)
number of geogrid layers =	1		
Reduction Factor For Install Damage, RFID =	1.2	dim	
Reduction Factor for Avoiding Lifetime Creep, RFCR =	3	dim	*Increase due to settlement potential*
Reduction Factor for Chemical and Biological Degradation, RFCBD =	2	dim	*Increase due to underlying dump*

$$T_{allow} = \frac{T_{ult}}{RF_{ID}+RF_{CR}+RF_{CBD}}$$ (Koerner Eq. 3.13)

Ultimate Strength Based on values for Triax TX190L (NX850 not available)

Typical Values		*Average Typical Values*
1.05 - 3.0	*Installation Damage Reduction Factor*	*2.03*
1.65 - 5.0	*Creep Reduction Factor*	*3.325*
1.1 - 2.0	*Durability Reduction Factor*	*1.55*

Required Factor of Safety, FS_{req} =	1.5	
Factor of Safety, FS =	**1.83**	**OK**
Tallow =	**3332**	
Treqd =	**1821**	

$$FS = \frac{T_{allow}}{T_{reqd}}$$ (3.5)

Refer to discussion on the summary page regarding assumptions for multiple layers of geogrid and factors of safety determination.

Note:

Deflection of the geosynthetic is based on the encountered pavement thicknesses compared to existing and proposed thicknesses provided on the 2003 Construction Drawings. These drawings indicated a minimum of approximately 4.5 inches of asphalt over 9 inches of portland concrete with a proposed overlay of 3.25 inches for a total assumed pavement of 16.75 inches. The difference was assumed to be overlays to accomodate for deflection between 2003 and 2023. This ranged from approximately 2.5 inches to 4.5 inches. A value of 3.0 inches was adopted as the estimated possible additional settlement over the next 20 years.

From: Designing with Geosynthetics (Koerner)

348 Designing with Geogrids Chap. 3

TABLE 3.3 RECOMMENDED REDUCTION FACTOR VALUES FOR USE IN EQUATION (3.6) FOR DETERMINING ALLOWABLE TENSILE STRENGTH OF GEOGRIDS

	Reduction Factor Values		
Application Area	RF_{ID}	RF_{CR}	RF_{CBD}
Unpaved roads	1.1–1.6	1.5–2.5	1.0–1.6
Paved roads	1.2–1.5	1.5–2.5	1.1–1.7
Embankments	1.1–1.4	2.0–3.0	1.1–1.5
Slopes	1.1–1.4	2.0–3.0	1.1–1.5
Walls	1.1–1.4	2.0–3.0	1.1–1.5
Foundations	1.2–1.5	2.0–3.0	1.1–1.6

Reduction for Installation Damage

Based on the low-end for paved roads per Koerner (1.20) which is slightly more conservative than the values in FHWA (1.05). It is assumed the construction conditions will be generally favorable based on the encountered granular embankment fill and location along an already established State highway.

Reduction for Creep

As the geogrid soil mat will extend to the embankment slopes, the upper end of the recommended value as provided by Koerner (3.0) for Embankments was adopted. This also served to select a value that was closer to the FHWA values ranging from 4 or 5.

Reduction for Chemical

As the project site is overlying a known abandoned dump comprised of various refuse and construction debris, the maximum recommended value for both Koerner and FHWA of 2.0 was adopted.

From : "Column-Supported Embankments: Full-Scale Tests and Design Recommendations."

b. The required tension shall be less than the allowable long-term geosynthetic tensile strength of the combined layers of geosynthetic reinforcement after applying appropriate reduction factors for durability, installation damage, creep, and an overall factor of safety, i.e.,

$$T_g \leq T_a = \frac{T_{ult}}{RF_D \times RF_{ID} \times RF_{CR} \times FS_{UNC}}$$

where, T_a = allowable tensile strength of geosynthetic

T_{ult} = ultimate tensile strength from single or multi-rib tensile strength tests (ASTM D 6637) for geogrids or wide width tensile strength tests (ASTM D 4595) for geotextiles.

T_g = T_{rp} if the reinforcement is design to carry vertical stresses only; if the reinforcement is also designed to carry the lateral spreading loads, then $T_g = T_{rp} + T_{ls}$ (see 8a below).

RF_D = Durability reduction factor is dependent on the susceptibility of the geosynthetic to attack by microorganisms, chemicals, thermal oxidation, hydrolysis and stress cracking. The typical range is from 1.1 to 2.0.

RF_{ID} = Installation damage reduction factor can range from 1.05 to 3.0, depending on backfill gradation and product mass per unit weight.

RF_{CR} = Creep reduction factor is the ratio of the ultimate strength (T_{ult}) to the creep limited strength obtained from laboratory creep tests for each product, and can vary typically from 1.65 to 5.0.

FS_{UNC} = Overall factor of safety or load factor reduction to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and externally applied loads. For load transfer platforms a minimum overall factor of safety of 1.5 is typical.

Guidelines for determining specific values for the reduction factors (RF_D, RF_{ID}, RF_{CR}) used in design are found in *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes* (FHWA NHI-00-043). Values for some manufacturers and products are established by the National Transportation Product Evaluation Program (NTPEP) and can be found online at: http://www.ntpep.org/Pages/GeosyntheticsReports.aspx

From: FHWA-NHI-00-043

For geosynthetic reinforcements, the design life is achieved by developing an allowable design load which considers all time dependent strength losses over the design life period as follows:

$$T_a = \frac{T_{ULT}}{RF \cdot FS} = \frac{T_{al}}{FS} \quad (12)$$

where T_a is the design long-term reinforcement tension load for the limit state, T_{ULT} the ultimate geosynthetic tensile strength and RF is the product of all applicable reduction factors and FS the overall factor of safety. T_{al} is the long-term material strength or more specifically:

$$T_{al} = \frac{T_{ULT}}{RF_{CR} \cdot RF_D \cdot RF_{ID}} \quad (13)$$

where:

T_{al} = Long-term tensile strength on a load per unit width of reinforcing basis.

T_{ULT} = Ultimate (or yield) tensile strength from wide strip test (ASTM D 4595) for geotextiles and wide strip (ASTM D 4595) or single rib test (GR1:GG1) for geogrids (note, that the same test shall be used for definition of the geogrid creep reduction factor), based on minimum average roll value (MARV) for the product.

RF_{CR} = Creep Reduction Factor is the ratio of the ultimate strength (T_{ULT}) to the creep limit strength obtained from laboratory creep tests for each product. Typical ranges of reduction factors as a function of polymer type, are indicated below:

Polymer Type	Creep Reduction Factors
Polyester	2.5 to 1.6
Polypropylene	5 to 4.0
High Density Polyethylene	5 to 2.6

RF_D = Durability reduction factor. It is dependent on the susceptibility of the geosynthetic to attack by microorganisms, chemicals, thermal oxidation, hydrolysis and stress cracking, and can vary typically from 1.1 to 2.0. The minimum reduction factor shall be 1.1.

RF_{ID} = Installation Damage reduction factor. It can range from 1.05 to 3.0, depending on backfill gradation and product mass per unit weight. The minimum reduction factor shall be 1.1 to account for testing uncertainties.

FS = Overall factor of safety to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and externally applied loads. For permanent, MSEW structures only, a minimum factor of safety of 1.5 has been typically used (thus $T_a = T_{al} / 1.5$).

For RSS structures, it is taken as 1.0, as the required factor of safety, is accounted in the stability analysis (thus $T_a = T_{al}$).

T_{al} is typically obtained directly from the manufacturer. It typically includes reduction factors but does not include a design or material factor of safety, FS. The determination of reduction factors for each geosynthetic product require extensive field and/or laboratory testing, briefly summarized as follows:

REFERENCE:

1) *Giroud, Jeanpierre & Bonaparte, Rudolph & Beech, J. & Gross, Beth. (1988). Load-carrying capacity of a soil layer supported by a geosynthetic overlying a void. Proceedings of the International Geotechnical Symposium on Theory and Practice of Earth Reinforcement. 185-190.*
2) *Giroud, Jeanpierre & Bonaparte, Rudolph & Beech, J.F. & Gross, Beth. (1990). Design of soil layer-geosynthetic systems overlying voids. Geotextiles and Geomembranes. 9. 11-50. 10.1016/0266-1144(90)90004-V.*
3) *Koerner, Robert M.. "Designing with Geosynthetics." Fifth Edition. (2005).*
4) *Sloan, Joel A., Filz, George M., Collin, Fames G., Kumar, Pawan. "Column-Supported Embankments: Full-Scale Tests and Design Recommendations." 2nd Edition. Virginia Tech Center for Geotechnical Practice Research (CGRP) Report 77. (2014).*
5) *Federal Highway Administration (FHWA), Elias, Victor et al. "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes: Design and Construction Guidelines (Updated Version)" , FHWA-NHI-00-043, 2001*

Vertical Stress on the Structure or Reinforcement layer, σz =	237.3	psf
Avg Unit Weight of Material Above the Settlement Area, Yavg =	130	pcf
Size of void (b,r) =	5	feet
for	*Circular Voids, use Radius, r*	
	Infinitiely long voids use Width, b	
Total Height Above Settlement Zone, H =	2	feet
Geosynthetic Deflection, y =	0.25	feet
Surcharge Pressure Placed at Ground Surface, q =	250	psf

$$\sigma_z = 2(y_{avg})(b)\left[1 - e^{-0.5H/b}\right] + qe^{-0.5\,H/b}$$ (Koerner Eq. 3.11) (Giroud Eq 4)

Simplified to: $\sigma_z = 2(y_{avg})R, \;\; for\ H \geq 6R$

Required Tensile Strength, Treqd =	2996	lbs/ft
Dimensionless Factor, Ω =	2.53	

$T_{reqd} = \sigma_z R\Omega$ (Koerner Eq 3.13) (see also α = pbΩ Giroud Eq. 7)

$where,\ \Omega = 0.25\left[\frac{(2y)}{B} + \frac{B}{(2y)}\right]$ (Koerner Eq. 3.14) (Giroud Eq. 6)

Allowable Tensile Strength to be used for Final Design, Tallow =	3332	lbs / ft	
Ultimate Tensile Strength from A Standard In-Isolation Tensile Test, Tult =	23989	lbs / ft	(Total of all layers)
Ultimate Tensile Strength from A Standard In-Isolation Tensile Test, Tult =	23,989	lb / ft	(per Layer)
number of geogrid layers =	1		
Reduction Factor For Install Damage, RFID =	1.2	dim	
Reduction Factor for Avoiding Lifetime Creep, RFCR =	3	dim	*Increase due to settlement potential*
Reduction Factor for Chemical and Biological Degradation, RFCBD =	2	dim	*Increase due to underlying dump*

$$T_{allow} = \frac{T_{ult}}{RF_{ID}+RF_{CR}+RF_{CBD}}$$ (Koerner Eq. 3.13)

Ultimate Strength Based on values for Triax TX190L (NX850 not available)

Typical Values		***Average Typical Values***
1.05 - 3.0	*Installation Damage Reduction Factor*	*2.03*
1.65 - 5.0	*Creep Reduction Factor*	*3.325*
1.1 - 2.0	*Durability Reduction Factor*	*1.55*

Required Factor of Safety, FS_{req} =	1.5	
Factor of Safety, FS =	1.11	NO
Tallow =	3332	
Treqd =	2996	

$$FS = \frac{T_{allow}}{T_{reqd}}$$ (3.5)

Refer to discussion on the summary page regarding assumptions for multiple layers of geogrid and factors of safety determination.

Note:

Deflection of the geosynthetic is based on the encountered pavement thicknesses compared to existing and proposed thicknesses provided on the 2003 Construction Drawings. These drawings indicated a minimum of approximately 4.5 inches of asphalt over 9 inches of portland concrete with a proposed overlay of 3.25 inches for a total assumed pavement of 16.75 inches. The difference was assumed to be overlays to accomodate for deflection between 2003 and 2023. This ranged from approximately 2.5 inches to 4.5 inches. A value of 3.0 inches was adopted as the estimated possible additional settlement over the next 20 years.

From: Designing with Geosynthetics

348 Designing with Geogrids Chap. 3

TABLE 3.3 RECOMMENDED REDUCTION FACTOR VALUES FOR USE IN EQUATION (3.6) FOR DETERMINING ALLOWABLE TENSILE STRENGTH OF GEOGRIDS

Application Area	Reduction Factor Values RF_{ID}	RF_{CR}	RF_{CBD}
Unpaved roads	1.1–1.6	1.5–2.5	1.0–1.6
Paved roads	1.2–1.5	1.5–2.5	1.1–1.7
Embankments	1.1–1.4	2.0–3.0	1.1–1.5
Slopes	1.1–1.4	2.0–3.0	1.1–1.5
Walls	1.1–1.4	2.0–3.0	1.1–1.5
Foundations	1.2–1.5	2.0–3.0	1.1–1.6

Reduction for Installation Damage

Based on the low-end for paved roads per Koerner (1.20) which is slightly more conservative than the values in FHWA (1.05). It is assumed the construction conditions will be generally favorable based on the encountered granular embankment fill and location along an already established State highway.

From : "Column-Supported Embankments: Full-Scale Tests and Design Recommendations."

b. The required tension shall be less than the allowable long-term geosynthetic tensile strength of the combined layers of geosynthetic reinforcement after applying appropriate reduction factors for durability, installation damage, creep, and an overall factor of safety, i.e.,

$$T_g \leq T_a = \frac{T_{ult}}{RF_D \times RF_{ID} \times RF_{CR} \times FS_{UNC}}$$

where, T_a = allowable tensile strength of geosynthetic

T_{ult} = ultimate tensile strength from single or multi-rib tensile strength tests (ASTM D 6637) for geogrids or wide width tensile strength tests (ASTM D 4595) for geotextiles.

T_g = T_{rp} if the reinforcement is design to carry vertical stresses only; if the reinforcement is also designed to carry the lateral spreading loads, then $T_g = T_{rp} + T_{ls}$ (see 8a below).

RF_D = Durability reduction factor is dependent on the susceptibility of the geosynthetic to attack by microorganisms, chemicals, thermal oxidation, hydrolysis and stress cracking. The typical range is from 1.1 to 2.0.

RF_{ID} = Installation damage reduction factor can range from 1.05 to 3.0, depending on backfill gradation and product mass per unit weight.

RF_{CR} = Creep reduction factor is the ratio of the ultimate strength (T_{ult}) to the creep limited strength obtained from laboratory creep tests for each product, and can vary typically from 1.65 to 5.0.

FS_{UNC} = Overall factor of safety or load factor reduction to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and externally applied loads. For load transfer platforms a minimum overall factor of safety of 1.5 is typical.

Guidelines for determining specific values for the reduction factors (RF_D, RF_{ID}, RF_{CR}) used in design are found in *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes* (FHWA NHI-00-043). Values for some manufacturers and products are established by the National Transportation Product Evaluation Program (NTPEP) and can be found online at:
http://www.ntpep.org/Pages/GeosyntheticsReports.aspx

From: FHWA-NHI-00-043

For geosynthetic reinforcements, the design life is achieved by developing an allowable design load which considers all time dependent strength losses over the design life period as follows:

$$T_a = \frac{T_{ULT}}{RF \cdot FS} = \frac{T_{al}}{FS} \quad (12)$$

where T_a is the design long-term reinforcement tension load for the limit state, T_{ULT} the ultimate geosynthetic tensile strength and RF is the product of all applicable reduction factors and FS the overall factor of safety. T_{al} is the long-term material strength or more specifically:

$$T_{al} = \frac{T_{ULT}}{RF_{CR} \cdot RF_D \cdot RF_{ID}} \quad (13)$$

where:

T_{al} = Long-term tensile strength on a load per unit width of reinforcing basis.

T_{ULT} = Ultimate (or yield) tensile strength from wide strip test (ASTM D 4595) for geotextiles and wide strip (ASTM D 4595) or single rib test (GR1:GG1) for geogrids (note, that the same test shall be used for definition of the geogrid creep reduction factor), based on minimum average roll value (MARV) for the product.

RF_{CR} = Creep Reduction Factor is the ratio of the ultimate strength (T_{ULT}) to the creep limit strength obtained from laboratory creep tests for each product. Typical ranges of reduction factors as a function of polymer type, are indicated below:

Polymer Type	Creep Reduction Factors
Polyester	2.5 to 1.6
Polypropylene	5 to 4.0
High Density Polyethylene	5 to 2.6

RF_D = Durability reduction factor. It is dependent on the susceptibility of the geosynthetic to attack by microorganisms, chemicals, thermal oxidation, hydrolysis and stress cracking, and can vary typically from 1.1 to 2.0. The minimum reduction factor shall be 1.1.

RF_{ID} = Installation Damage reduction factor. It can range from 1.05 to 3.0 depending on backfill gradation and product mass per unit weight. The minimum reduction factor shall be 1.1 to account for testing uncertainties.

FS = Overall factor of safety to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and externally applied loads. For permanent, MSEW structures only, a minimum factor of safety of 1.5 has been typically used (thus $T_a = T_{al} / 1.5$).

For RSS structures, it is taken as 1.0, as the required factor of safety, is accounted in the stability analysis (thus $T_a = T_{al}$).

T_{al} is typically obtained directly from the manufacturer. It typically includes reduction factors but does not include a design or material factor of safety, FS. The determination of reduction factors for each geosynthetic product require extensive field and/or laboratory testing, briefly summarized as follows:

Reduction for Creep

As the geogrid soil mat will extend to the embankment slopes, the upper end of the recommended value as provided by Koerner (3.0) for Embankments was adopted. This also served to select a value that was closer to the FHWA values ranging from 4 or 5.

Reduction for Chemical

As the project site is overlying a known abandoned dump comprised of various refuse and construction debris, the maximum recommended value for both Koerner and FHWA of 2.0 was adopted.

REFERENCE:

1) *Giroud, Jeanpierre & Bonaparte, Rudolph & Beech, J. & Gross, Beth. (1988). Load-carrying capacity of a soil layer supported by a geosynthetic overlying a void. Proceedings of the International Geotechnical Symposium on Theory and Practice of Earth Reinforcement. 185-190.*
2) *Giroud, Jeanpierre & Bonaparte, Rudolph & Beech, J.F. & Gross, Beth. (1990). Design of soil layer-geosynthetic systems overlying voids. Geotextiles and Geomembranes. 9. 11-50. 10.1016/0266-1144(90)90004-V.*
3) *Koerner, Robert M.. "Designing with Geosynthetics." Fifth Edition. (2005).*
4) *Sloan, Joel A., Filz, George M., Collin, Fames G., Kumar, Pawan. "Column-Supported Embankments: Full-Scale Tests and Design Recommendations." 2nd Edition. Virginia Tech Center for Geotechnical Practice Research (CGRP) Report 77. (2014).*
5) *Federal Highway Administration (FHWA), Elias, Victor et al. "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes: Design and Construction Guidelines (Updated Version)" , FHWA-NHI-00-043, 2001*

Vertical Stress on the Structure or Reinforcement layer, σz =	337.1	psf
Avg Unit Weight of Material Above the Settlement Area, Yavg =	130	pcf
Size of void (b,r) =	5	feet
for	*Circular Voids, use Radius, r*	
	Infinitiely long voids use Width, b	
Total Height Above Settlement Zone, H =	3	feet
Geosynthetic Deflection, y =	0.25	feet
Surcharge Pressure Placed at Ground Surface, q =	250	psf

$$\sigma_z = 2(y_{avg})(b)\left[1-e^{-0.5H/b}\right] + qe^{-0.5\,H/b}$$ (Koerner Eq. 3.11) (Giroud Eq 4)

Simplified to: $\sigma_z = 2(y_{avg})R,\ \ for\ H \geq 6R$

Required Tensile Strength, Treqd =	4256	lbs/ft
Dimensionless Factor, Ω =	2.53	

$T_{reqd} = \sigma_z R\Omega$ (Koerner Eq 3.13) (see also α = pbΩ Giroud Eq. 7)

$where,\ \Omega = 0.25\left[\frac{(2y)}{B} + \frac{B}{(2y)}\right]$ (Koerner Eq. 3.14) (Giroud Eq. 6)

Allowable Tensile Strength to be used for Final Design, Tallow =	3332	lbs / ft	
Ultimate Tensile Strength from A Standard In-Isolation Tensile Test, Tult =	23989	lbs / ft	(Total of all layers)
Ultimate Tensile Strength from A Standard In-Isolation Tensile Test, Tult =	23,989	lb / ft	(per Layer)
number of geogrid layers =	1		
Reduction Factor For Install Damage, RFID =	1.2	dim	
Reduction Factor for Avoiding Lifetime Creep, RFCR =	3	dim	*Increase due to settlement potential*
Reduction Factor for Chemical and Biological Degradation, RFCBD =	2	dim	*Increase due to underlying dump*

$$T_{allow} = \frac{T_{ult}}{RF_{ID}+RF_{CR}+RF_{CBD}}$$ (Koerner Eq. 3.13)

Ultimate Strength Based on values for Triax TX190L (NX850 not available)

Typical Values		*Average Typical Values*
1.05 - 3.0	Installation Damage Reduction Factor	2.03
1.65 - 5.0	Creep Reduction Factor	3.325
1.1 - 2.0	Durability Reduction Factor	1.55

Required Factor of Safety, FS_{req} =	1.5	
Factor of Safety, FS =	0.78	NO
Tallow =	3332	
Treqd =	4256	

$$FS = \frac{T_{allow}}{T_{reqd}}$$ (3.5)

Refer to discussion on the summary page regarding assumptions for multiple layers of geogrid and factors of safety determination.

Note:

Deflection of the geosynthetic is based on the encountered pavement thicknesses compared to existing and proposed thicknesses provided on the 2003 Construction Drawings. These drawings indicated a minimum of approximately 4.5 inches of asphalt over 9 inches of portland concrete with a proposed overlay of 3.25 inches for a total assumed pavement of 16.75 inches. The difference was assumed to be overlays to accomodate for deflection between 2003 and 2023. This ranged from approximately 2.5 inches to 4.5 inches. A value of 3.0 inches was adopted as the estimated possible additional settlement over the next 20 years.

From: Designing with Geosynthetics

348 Designing with Geogrids Chap. 3

TABLE 3.3 RECOMMENDED REDUCTION FACTOR VALUES FOR USE IN EQUATION (3.6) FOR DETERMINING ALLOWABLE TENSILE STRENGTH OF GEOGRIDS

	Reduction Factor Values		
Application Area	RF_{ID}	RF_{CR}	RF_{CBD}
Unpaved roads	1.1–1.6	1.5–2.5	1.0–1.6
Paved roads	1.2–1.5	1.5–2.5	1.1–1.7
Embankments	1.1–1.4	2.0–3.0	1.1–1.5
Slopes	1.1–1.4	2.0–3.0	1.1–1.5
Walls	1.1–1.4	2.0–3.0	1.1–1.5
Foundations	1.2–1.5	2.0–3.0	1.1–1.6

From : "Column-Supported Embankments: Full-Scale Tests and Design Recommendations."

b. The required tension shall be less than the allowable long-term geosynthetic tensile strength of the combined layers of geosynthetic reinforcement after applying appropriate reduction factors for durability, installation damage, creep, and an overall factor of safety, i.e.,

$$T_g \leq T_a = \frac{T_{ult}}{RF_D \ x \ RF_{ID} \ x \ RF_{CR} \ x \ FS_{UNC}}$$

where, T_a = allowable tensile strength of geosynthetic

T_{ult} = ultimate tensile strength from single or multi-rib tensile strength tests (ASTM D 6637) for geogrids or wide width tensile strength tests (ASTM D 4595) for geotextiles.

$T_g = T_{rp}$ if the reinforcement is design to carry vertical stresses only; if the reinforcement is also designed to carry the lateral spreading loads, then $T_g = T_{rp} + T_{ls}$ (see 8a below).

RF_D = Durability reduction factor is dependent on the susceptibility of the geosynthetic to attack by microorganisms, chemicals, thermal oxidation, hydrolysis and stress cracking. The typical range is from 1.1 to 2.0.

RF_{ID} = Installation damage reduction factor can range from 1.05 to 3.0, depending on backfill gradation and product mass per unit weight.

RF_{CR} = Creep reduction factor is the ratio of the ultimate strength (T_{ult}) to the creep limited strength obtained from laboratory creep tests for each product, and can vary typically from 1.65 to 5.0.

FS_{UNC} = Overall factor of safety or load factor reduction to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and externally applied loads. For load transfer platforms a minimum overall factor of safety of 1.5 is typical.

Guidelines for determining specific values for the reduction factors (RF_D, RF_{ID}, RF_{CR}) used in design are found in *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes* (FHWA NHI-00-043). Values for some manufacturers and products are established by the National Transportation Product Evaluation Program (NTPEP) and can be found online at:
http://www.ntpep.org/Pages/GeosyntheticsReports.aspx

From: FHWA-NHI-00-043

For geosynthetic reinforcements, the design life is achieved by developing an allowable design load which considers all time dependent strength losses over the design life period as follows:

$$T_a = \frac{T_{ULT}}{RF \cdot FS} = \frac{T_{al}}{FS} \qquad (12)$$

where T_a is the design long-term reinforcement tension load for the limit state, T_{ULT} the ultimate geosynthetic tensile strength and RF is the product of all applicable reduction factors and FS the overall factor of safety. T_{al} is the long-term material strength or more specifically:

$$T_{al} = \frac{T_{ULT}}{RF_{CR} \cdot RF_D \cdot RF_{ID}} \qquad (13)$$

where:

T_{al} = Long-term tensile strength on a load per unit width of reinforcing basis.

T_{ULT} = Ultimate (or yield) tensile strength from wide strip test (ASTM D 4595) for geotextiles and wide strip (ASTM D 4595) or single rib test (GR1:GG1) for geogrids (note, that the same test shall be used for definition of the geogrid creep reduction factor), based on minimum average roll value (MARV) for the product.

RF_{CR} = Creep Reduction Factor is the ratio of the ultimate strength (T_{ULT}) to the creep limit strength obtained from laboratory creep tests for each product. Typical ranges of reduction factors as a function of polymer type, are indicated below:

Polymer Type	Creep Reduction Factors
Polyester	2.5 to 1.6
Polypropylene	5 to 4.0
High Density Polyethylene	5 to 2.6

RF_D = Durability reduction factor. It is dependent on the susceptibility of the geosynthetic to attack by microorganisms, chemicals, thermal oxidation, hydrolysis and stress cracking, and can vary typically from 1.1 to 2.0. The minimum reduction factor shall be 1.1.

RF_{ID} = Installation Damage reduction factor. It can range from 1.05 to 3.0 depending on backfill gradation and product mass per unit weight. The minimum reduction factor shall be 1.1 to account for testing uncertainties.

FS = Overall factor of safety to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and externally applied loads. For permanent, MSEW structures only, a minimum factor of safety of 1.5 has been typically used (thus $T_a = T_{al} / 1.5$).

For RSS structures, it is taken as 1.0, as the required factor of safety, is accounted in the stability analysis (thus $T_a = T_{al}$).

T_{al} is typically obtained directly from the manufacturer. It typically includes reduction factors but does not include a design or material factor of safety, FS. The determination of reduction factors for each geosynthetic product require extensive field and/or laboratory testing, briefly summarized as follows:

Reduction for Installation Damage

Based on the low-end for paved roads per Koerner (1.20) which is slightly more conservative than the values in FHWA (1.05). It is assumed the construction conditions will be generally favorable based on the encountered granular embankment fill and location along an already established State highway.

Reduction for Creep

As the geogrid soil mat will extend to the embankment slopes, the upper end of the recommended value as provided by Koerner (3.0) for Embankments was adopted. This also served to select a value that was closer to the FHWA values ranging from 4 or 5.

Reduction for Chemical

As the project site is overlying a known abandoned dump comprised of various refuse and construction debris, the maximum recommended value for both Koerner and FHWA of 2.0 was adopted.

Client Chagrin Valley Engineers
Project MOT-4-19.30
Task Determination of Assumed Future Settlement for Void Calculation

2003 Plans Composite Pavement Section	
4.5	Asphalt (in.)
9	Portland Cement Concrete (in.)
1.5	Surface Course (in.)
1.75	Intermediate Course (in.)
16.75	**Total (in.)**

Exploration Number	Asphalt Thickness (in.)	Reinforced Concrete Thickness (in.)	Total Pavement Thickness (in.)	(Total 2003 Pavement) - (2023 Encountered Pavement)**	NOTES
B-001-1-23	12	--	12	0	
B-002-1-23	**12**	**9**	**21**	4.25	**Composite Pavement**
B-002-2-23	9	--	9	0	
B-003-1-23	14	--	14	0	
B-004-1-23	27	--	27	10.25	
B-004-2-23	**10**	**8**	**18**	1.25	**Composite Pavement**
B-008-0-23	13	--	13	0	
B-009-0-23	17	--	17	0.25	
B-010-0-23	30	12	42	25.25	
B-011-0-23	12	--	12	0	
Average =	12.375	(All Asphalt)		2.750	Average (considering composite pavements)
	2003 Asphalt Thickness =	7.75		**Negative values reduced to zero	
Average (-7.75 inch) =	4.625	(Encountered Asphalt Reduced by 2003 Asphalt Values)			

XYZ = omitted as outlier

Assumed overlays due to settlement ranges from approximately

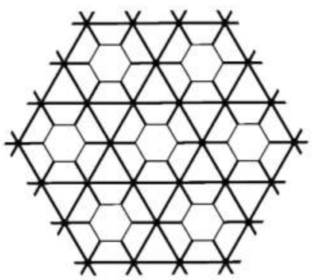
From	2.750	inches
To	4.625	inches
Adopted Value :	3	inches

Product Data Sheet
InterAx® FilterGrid™ NX850-FG™

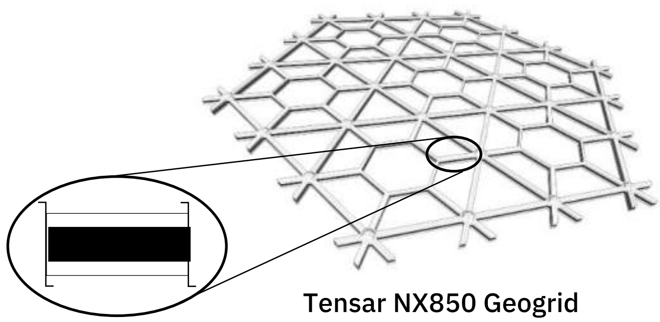
InterAx® FilterGrid™ is a composite geosynthetic consisting of InterAx geogrid bonded to a nonwoven geotextile. This product combines the most advanced InterAx geogrid technology with the added functionality of a nonwoven geotextile where site conditions require additional filtration and/or separation.

General

1. The geogrid is manufactured from a coextruded, composite polymer sheet, which is then punched and oriented. The resulting structure consists of continuous and non-continuous ribs forming three aperture geometries (hexagon, trapezoid, and triangle) and an unimpeded suspended hexagon.



Tensar NX850 Geogrid
Plan View



Tensar NX850 Geogrid
Perspective View

2. The following properties are intended for product identification:

Identification Properties[1]	General
▪ Aperture shapes	Hexagonal, Trapezoidal, & Triangular
▪ Structure	Coextruded & Integrally Formed
▪ Rib shape	Rectangular
▪ Continuous parallel rib pitch(2), mm (in)	80 (3.2)
▪ Rib aspect ratio(3)	> 1.0
▪ Node thickness(2), mm (in)	4.5 (0.18)
▪ Color identification	White / Black / White

3. The needle punched nonwoven geotextile (nominal 6 oz / sy) is thermally bonded to the geogrid and is manufactured at a NTPEP audited facility. The geotextile shall have the following properties:

Index Properties - Geotextile	**Test Method**	**English (MARV[2])**	**Metric (MARV[2])**
▪ Grab Tensile Strength	ASTM D 4632	160lbs.	0.711 kN
▪ Grab Elongation	ASTM D 4632	50%	50%
▪ Trapezoid Tear Strength	ASTM D 4533	60 lbs.	0.267 kN
▪ CBR Puncture Resistance	ASTM D 6241	410 lbs.	1.823 kN
▪ Permittivity	ASTM D 4491	1.5 sec^{-1}	1.5 sec^{-1}
▪ Water Flow	ASTM D 4491	110 gpm/ft^2	4480 l/min/m^2
▪ Apparent Opening Size (AOS)	ASTM D 4751	70 Std. U.S.	0.212 mm
▪ UV Resistance	ASTM D 4355	70%/500 hrs.	70%/500 hrs.

Dimensions and Delivery

The geogrid shall be delivered to the jobsite in roll form with each roll individually identifie. Rolls are shipped with nominal measurements.

Notes

1. Unless indicated otherwise, values shown are minimum average roll values determined in accordance with ASTM D4759-02
2. Nominal dimensions
3. Ratio of the mid-rib depth to the mid-rib width

Tensar, a Division of CMC
2500 Northwinds Pkwy., Ste. 500
Alpharetta, Georgia 30009

Phone: 800-TENSAR-1
www.tensarcorp.com

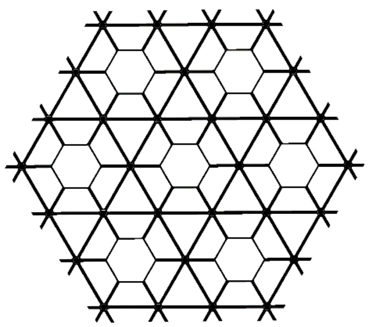
Tensar
A Division of CMC

Product Data Sheet
InterAx® NX850™ Geogrid

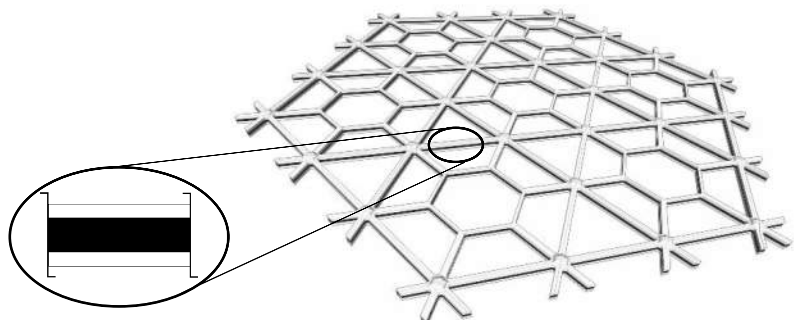
Tensar, a division of CMC, reserves the right to change its product specifications at any time. It is the responsibility of the person specifying the use of this product and of the purchaser to ensure that product specifications relied upon for design or procurement purposes are current and that the product is suitable for its intended use in each instance.

General

1. The geogrid is manufactured from a coextruded, composite polymer sheet, which is then punched and oriented. The resulting structure consists of continuous and non-continuous ribs forming three aperture geometries (hexagon, trapezoid, and triangle) and an unimpeded suspended hexagon.



Tensar NX850 Geogrid
Plan View



Tensar NX850 Geogrid
Perspective View

2. The following properties are intended for product identification:

Identification Properties[1]	General
▪ Aperture shapes	Hexagonal, Trapezoidal, & Triangular
▪ Structure	Coextruded & Integrally Formed
▪ Rib shape	Rectangular
▪ Continuous parallel rib pitch[(2)], mm (in)	80 (3.2)
▪ Rib aspect ratio[(3)]	> 1.0
▪ Node thickness[(2)], mm (in)	4.5 (0.18)
▪ Color identification	White / Black / White

Dimensions and Delivery

The geogrid shall be delivered to the jobsite in roll form with each roll individually identified. Rolls are shipped with nominal measurements.

Notes

1. Unless indicated otherwise, values shown are minimum average roll values determined in accordance with ASTM D4759-02
2. Nominal dimensions
3. Ratio of the mid-rib depth to the mid-rib width

Tensar, a Division of CMC
2500 Northwinds Pkwy., Ste. 500
Alpharetta, Georgia 30009

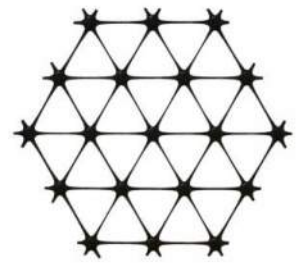
Phone: 800-TENSAR-1
www.tensarcorp.com

Tensar | THE COMPANY YOU CAN BUILD ON®

Product Specification - TriAx® TX190L Geogrid

Tensar International Corporation reserves the right to change its product specifications at any time. It is the responsibility of the person specifying the use of this product and of the purchaser to ensure that product specifications relied upon for design or procurement purposes are current and that the product is suitable for its intended use in each instance.

Tensar TriAx® Geogrid



General

1. The geogrid is manufactured from a punched polypropylene sheet, which is then oriented in three substantially equilateral directions so that the resulting ribs shall have a high degree of molecular orientation, which continues at least in part through the mass of the integral node.
2. The properties contributing to the performance of a mechanically stabilized layer include the following:

Index Properties	Longitudinal[1]	Diagonal[1]	General[1]
▪ Rib pitch[2], mm (in)	60 (2.40)	60 (2.40)	
▪ Rib shape			Rectangular
▪ Aperture shape			Triangular

Structural Integrity	
▪ Junction efficiency[3], %	93
▪ Isotropic Stiffness Ratio[4]	0.6
▪ Overall Flexural Rigidity[5], mg-cm	1,500,000
▪ Radial stiffness at low strain[6], kN/m @ 0.5% strain	350
(lb/ft @ 0.5% strain)	(23,989)

Durability	
▪ Resistance to chemical degradation[7]	100%
▪ Resistance to ultra-violet light and weathering[8]	70%

Dimensions and Delivery

The TX geogrid shall be delivered to the jobsite in roll form with each roll individually identified and nominally measuring 4.0 meters (13.1 feet) in width and 50 meters (164 feet) in length and/or 3.8 meters (12.5 feet) in width and 50 meters (164 feet) in length.

Notes

1. Unless indicated otherwise, values shown are minimum average roll values determined in accordance with ASTM D4759-02. Brief descriptions of test procedures are given in the following notes.
2. Nominal dimensions.
3. Load transfer capability determined in accordance with ASTM D6637-10 and ASTM D7737-11 and expressed as a percentage of ultimate tensile strength.
4. The ratio between the minimum and maximum observed values of radial stiffness at 0.5% strain, measured on rib and midway between rib directions.
5. Determined in accordance with ASTM D7748/D7748M-14.
6. Radial stiffness is determined from tensile stiffness measured in any in-plane axis from testing in accordance with ASTM D6637-10.
7. Resistance to loss of load capacity or structural integrity when subjected to chemically aggressive environments in accordance with EPA 9090 immersion testing.
8. Resistance to loss of load capacity or structural integrity when subjected to 500 hours of ultraviolet light and aggressive weathering in accordance with ASTM D4355-05.

Tensar International Corporation
2500 Northwinds Pkwy.
Atlanta, Georgia 30009
Phone: 800-TENSAR-1
www.tensarcorp.com

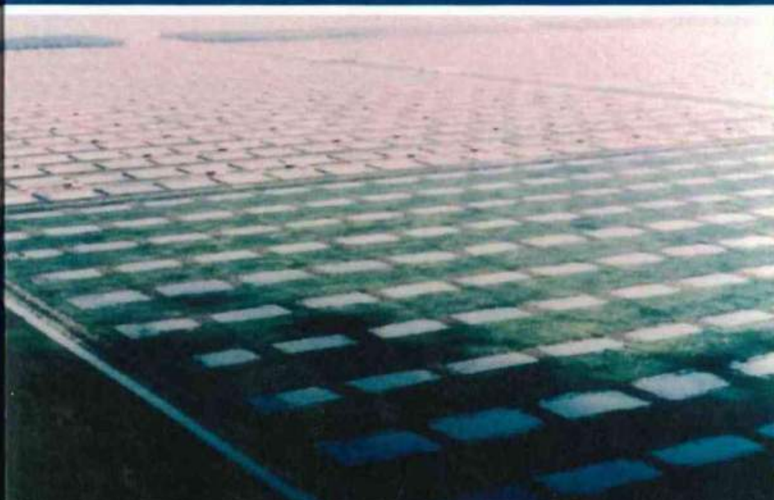
Koerner, Robert M..

“**Designing with Geosynthetics.**”

Fifth Edition. (2005).

DESIGNING WITH GEOSYNTHETICS

FIFTH EDITION




ROBERT M. KOERNER

3.1.5 Allowable Strength Considerations

The basis of the design-by-function concept is the establishment of a factor of safety. For geogrids, where reinforcement is the primary function, this factor of safety takes the following form:

$$FS = \frac{T_{\text{allow}}}{T_{\text{reqd}}} \tag{3.5}$$

where

FS = factor of safety (to accommodate unanticipated loading conditions and uncertainties in design or testing),
T_{allow} = allowable tensile strength from laboratory testing, and
T_{reqd} = required tensile strength from design of the particular field situation.

The allowable value comes from a tensile test of the type described in Section 3.1.2, where we must compare the test setup with the intended field situation. If the test method is not completely field-simulated, the laboratory value must be suitably adjusted. This will generally be the case. Thus the laboratory-generated tensile strength is usually an ultimate value, which must be reduced before being used in design:

$$T_{\text{allow}} < T_{\text{ult}}$$

One way of accomplishing this is to place reduction factors on each of the items not modeled in the laboratory test. For example, the following equation should be considered [20]:

$$T_{\text{allow}} = T_{\text{ult}}\left[\frac{1}{RF_{ID} \times RF_{CR} \times RF_{CBD}}\right] \tag{3.6}$$

where

T_{ult} = ultimate tensile strength from a standardized in-isolation tensile test,
T_{allow} = allowable tensile strength to be used in equation (3.5) for final design purposes,
RF_{ID} = reduction factor for installation damage,
RF_{CR} = reduction factor for avoiding creep over the duration of the structure's lifetime, and,
RF_{CBD} = reduction factor against chemical and biological degradation.

Note that some of these values may be 1.0 or slightly above 1.0, and may therefore be inconsequential. Still others, not specifically mentioned in equation (3.6), may be included as the situation warrants. For example, reduction factors against ultraviolet degradation, RF_{UV}, field seams, RF_{seam} or penetrations, RF_{pen}, may be included on a site-specific basis. Guidelines for the usual reduction factor values are given in Table 3.3.

TABLE 3.3 RECOMMENDED REDUCTION FACTOR VALUES FOR USE IN EQUATION (3.6) FOR DETERMINING ALLOWABLE TENSILE STRENGTH OF GEOGRIDS

	Reduction Factor Values		
Application Area	RF_{ID}	RF_{CR}	RF_{CBD}
Unpaved roads	1.1–1.6	1.5–2.5	1.0–1.6
Paved roads	1.2–1.5	1.5–2.5	1.1–1.7
Embankments	1.1–1.4	2.0–3.0	1.1–1.5
Slopes	1.1–1.4	2.0–3.0	1.1–1.5
Walls	1.1–1.4	2.0–3.0	1.1–1.5
Foundations	1.2–1.5	2.0–3.0	1.1–1.6
	INSTALL	CREEP	CHEM/BIO

Note that ranges are given rather than specific values. It is necessary to consider each item individually and make a conscious decision as to how important it is for the site-specific situation. For example, the largest is the creep reduction factor—hence its importance for proper evaluation. In Example 3.3 and 3.4 the values used are assumed on the basis of a hypothetical project and construction method.

Example 3.3

What is the allowable geogrid tensile strength to be used in the construction of an unpaved road separating stone base from subgrade soil if the ultimate strength of the geogrid is 80 kN/m?

Solution: Using estimated values from Table 3.3 in equation (3.6), the following results:

$$\begin{aligned} T_{\text{allow}} &= T_{\text{ult}}\left[\frac{1}{RF_{ID} \times RF_{CR} \times RF_{CBD}}\right] \\ &= 80\left[\frac{1}{1.3 \times 2.0 \times 1.5}\right] \\ &= 80\left[\frac{1}{3.9}\right] \\ T_{\text{allow}} &= 20.5 \text{ kN/m} \end{aligned}$$

Example 3.4

What is the allowable geogrid tensile strength to be used in the construction of a permanent wall adjacent to a major highway if the ultimate strength of the geogrid is 70 kN/m?

Solution: Using estimated values from Table 3.3 in equation (3.6) gives

$$\begin{aligned} T_{\text{allow}} &= T_{\text{ult}}\left[\frac{1}{RF_{ID} \times RF_{CR} \times RF_{CBD}}\right] \\ &= 70\left[\frac{1}{1.3 \times 2.5 \times 1.3}\right] \\ &= 70\left[\frac{1}{4.22}\right] \\ T_{\text{allow}} &= 16.6 \text{ kN/m} \end{aligned}$$

Note that these examples could just as well have been framed so as generate an ultimate strength from a given allowable value. This would be the case if we were working from an analytical method that generated a design value. This design value (as with the allowable) would have to be *increased* by reduction factors to arrive at a required (or ultimate) tensile strength.

3.2 DESIGNING FOR GEOGRID REINFORCEMENT

The primary function of geogrids is invariably reinforcement; this section will proceed from one reinforcement application area to another. The order will parallel that of Sections 2.6 and 2.7 on geotextile reinforcement, with the addition of several areas that are unique to geogrids.

3.2.1 Paved Roads—Base Courses

The use of geogrids in paved road aggregate base courses is an area where the large aperture size of geogrids provide an excellent advantage. Here the geogrids are placed within the granular base course, typically crushed stone, with the intention of providing an increased modulus, hence a lateral confinement to the system. This lateral confinement is intended to resist the tendency for the base course aggregate to *walk out* from beneath the repetitive traffic loads imposed on the concrete- or bitumen-pavement surface. The situation is applicable for the ballast beneath railroad tracks as well, and perhaps even more so due to the nature and intensity of the dynamic loads.

A number of laboratory tests have been conducted to assess the potential benefits and mechanisms involved, most significantly the work of Haas [21] and Abd El Halim [22, 23]. In a large test setup measuring 4.0 m long by 2.4 m wide by 2 m deep and using 10 kN loads applied sinusoidally at a frequency of 10 Hz on a 300 mm diameter circular plate, five test series (called *loops*) were performed. Loop 1 compared the response of nonreinforced and reinforced sections using both dry (strong) and saturated (weak) subgrade conditions. Failure appeared in the nonreinforced sections earlier than the reinforced sections under both conditions. Loop 2 provided data that show little difference in elastic deflection between the four trials. More significant was the angle of curvature and the elastic strain at the bottom of the asphalt pavement. Both indicate a 50% reduction for the reinforced sections, thereby indicating a significant load-spreading phenomenon. The permanent surface deformation of the reinforced section is substantially improved over the nonreinforced section. At a 20 mm failure assumption, the nonreinforced section carried 110,000 load repetitions, compared with 320,000 for the reinforced case. In the context of the discussion on geotextiles used in the control of reflective cracking of paved roadways, this would be called a geogrid effectiveness factor (GEF) equal to 2.9.

Loop 3 investigated the equivalent thickness that can be attributed to the reinforcement. The results indicate that the 150 mm reinforced section carried about 80,000 load cycles compared with only 34,000 load cycles for the 200 mm nonreinforced and 92,000 loads cycles for the 250 mm nonreinforced. In other words, 150 mm of reinforced asphalt nearly compared with 250 mm of nonreinforced asphalt. Loop 4

If fines (silts and/or clays) are allowed for the reinforced zone backfill soil, any possible water in front, behind, and beneath the reinforced zone must be carefully collected, transmitted, and discharged. Proper drainage control is absolutely critical in this regard. Furthermore, the top of the zone should be waterproofed—for example, by a geomembrane or a geosynthetic clay liner—to prevent water from entering the backfill zone from the surface. Surface water drainage as well as drainage from the retained earth zone is obviously of concern with respect to potential buildup of pore water pressures behind or within the reinforced soil zone. (See Koerner and Soong [46] for wall drainage system designs in this regard.)

In closing this section on geogrid reinforced walls, the current tendency to create live (or evergreen) walls with open facing should be mentioned. As we saw earlier in Figure 3.14, the sequence is a steel wire mesh (alternatively a gabion), backed by a bidirectional geogrid and then by a geosynthetic erosion control material. The reinforcing geogrids (always unidirectional types) are either attached to the steel wire mesh facing, or they are frictionally connected by sufficient overlap length. Such walls avoid masonry block durability concerns and offer a considerably less expensive wall system. Of course, the durability of the steel wire and bidirectional geogrid backup must be considered and this is a viable research topic when considering 100-year permanent wall lifetimes.

3.2.6 Foundation and Basal Reinforcement

Geogrids have been used to increase bearing capacity of poor foundation soils in different ways: as a continuous layer, as multiple closely spaced continuous layers with granular soil between layers, and as mattresses consisting of three-dimensional interconnected cells. The technical database for the single-layer continuous sheets has been reported by Jarrett [47] and by Milligan and Love [48]; in both cases large-scale laboratory tests are used. Figure 3.19 presents some of Milligan and Love's work graphed in the conventional nondimensionalized q/c_u versus ρ/B manner and also as $q/\sqrt{c_u}$ versus ρ/B where q is the bearing capacity and ρ is the settlement. The latter graph is not conventional but does sort out the data nicely. Clearly shown in both instances is the marked improvement in load-carrying capacity using geogrids at high deformation and only a nominal beneficial effect at low deformation. Beyond these observations, a precise design formulation is not currently available.

Instead of focusing on a global increase in bearing capacity, it is quite likely that single or multiple layers of geogrid (or geotextile) will aid in minimizing or eliminating differential settlement. Here localized settlements due to abruptly settling or subsiding weak zones can be spanned by the layer of reinforcement. This is known as *foundation improvement* (rather than bearing capacity via base reinforcement). Notable in this regard is a technique called *piggybacking*—the construction of new landfills above existing landfills. The approach is to use arching theory in the calculation of the vertical stress arising from localized subsidence (i.e., differential settlement) and to provide suitably strong reinforcement.

It should be recognized that arching in natural soils overlying a locally yielding foundation is well established. In the 1930s, both Karl Terzaghi in Austria (calculating

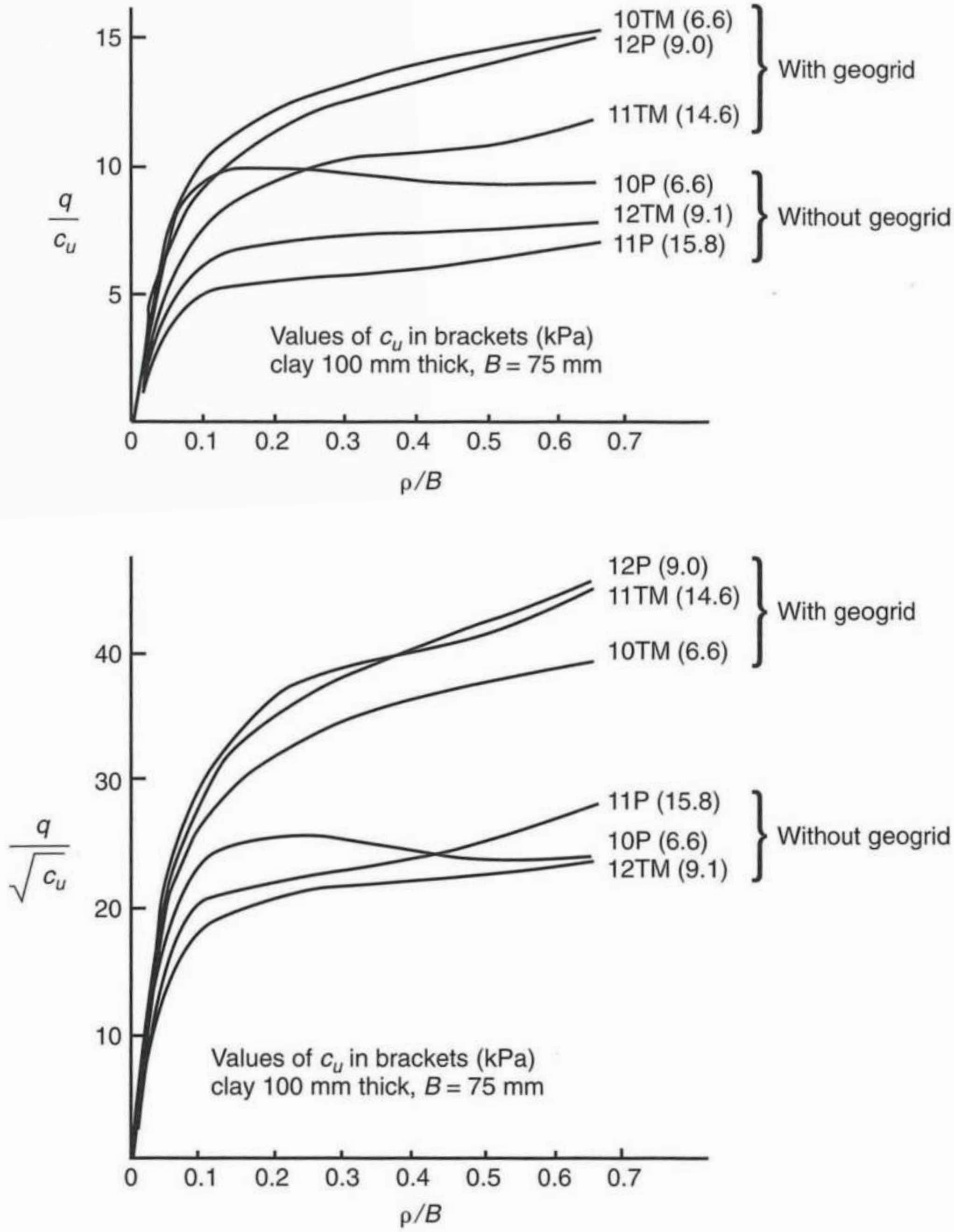


Figure 3.19 Load versus deflection curves of large laboratory tests with and without geogrid reinforcement. (After Milligan and Love [48])

stresses on deep tunnels) and Aston Marston in the United States (calculating stresses on buried pipelines) developed the analytic theory. Their work resulted in the following simplified formula for vertical stress on the surface of the particular underground structure (tunnel or pipe, respectively):

$$\sigma_z = 2\gamma_{ave}R[1 - e^{-0.5\,H/R}] + qe^{-0.5\,H/R} \tag{3.11}$$

where

σ_z = vertical stress on the structure or reinforcement layer,
γ_{ave} = average unit weight of material above the settlement area,
R = radius of differential settlement zone,
H = total height above the settlement area, and
q = surcharge pressure placed at the ground surface.

Note that for large values of H (typically $H \geq 6R$) the formula reduces to the following value of constant vertical stress:

$$\sigma_z = 2\gamma_{\text{ave}}R \qquad \text{for } H \geq 6R \tag{3.12}$$

Having a method to calculate the vertical stress, we can now use the value to calculate the stress in the reinforcement layer for a new landfill placed over an existing one. Note that the reinforcement can be either a geogrid or a geotextile. For support over a differential settlement area, the value of T_{reqd} is calculated as follows:

$$T_{\text{reqd}} = \sigma_z R \Omega \tag{3.13}$$

where

$\Omega = 0.25[(2y)/B + B/(2y)]$, where (3.14)
B = width of settlement void, and
y = depth of settlement void.

Giroud et al. [49] have combined the above equations to develop a design chart that can be used to avoid direct calculation (see Figure 3.20). Note that the chart can be used for either circular voids or long extended voids.

Once the value of T_{reqd} is determined, it must be compared to T_{allow} using equation (3.6), which includes the site-specific reduction factors. Example 3.11 illustrates the technique.

Example 3.11

Using the Terzaghi/Marston formulation for calculating vertical stress above localized subsidence, in this case differential settlement in an old landfill of radius 1 m, **(a)** calculate the required wide-width strength of a reinforcement layer if a new 30 m high landfill is to be placed upon the existing one—that is, if the new landfill is to be *piggybacked* on the existing landfill. The compacted unit weight of the waste is 12 kN/m^3. **(b)** Check your calculated value against Figure 3.20. **(c)** Calculate the factor of safety for a geogrid with ultimate wide-width tensile strength of 125 kN/m. In the calculations use cumulative reduction factors of 5.0.

Solution:

(a) The formula for vertical stresses in arching situations under a deep fill (such as the one in this example) reduces to equation (3.12). Therefore the vertical stress is calculated as

$$\sigma_z = 2\gamma_{\text{ave}}R$$
$$= 2(12)(1.0)$$
$$= 24 \text{ kPa}$$

To transfer this vertical stress into a horizontal force, we use equation (3.13)

$$T_{\text{reqd}} = \sigma_z R \Omega$$

where Ω = strain function, $f(\epsilon)$ [recall Section 2.5.2]
$\Omega = 0.97$ at 5% strain
$= 0.73$ at 10% strain

Assuming that $\Omega = 0.73$,

$$T_{\text{reqd}} = 24 \times 1.0 \times 0.73$$
$$= 17.5 \text{ kN/m}$$

(b) Check this against Figure 3.20:

$$\frac{H}{R} = \frac{30}{1} = 30$$

$$\therefore \frac{T}{\gamma R^2 \Omega} = 2.0$$

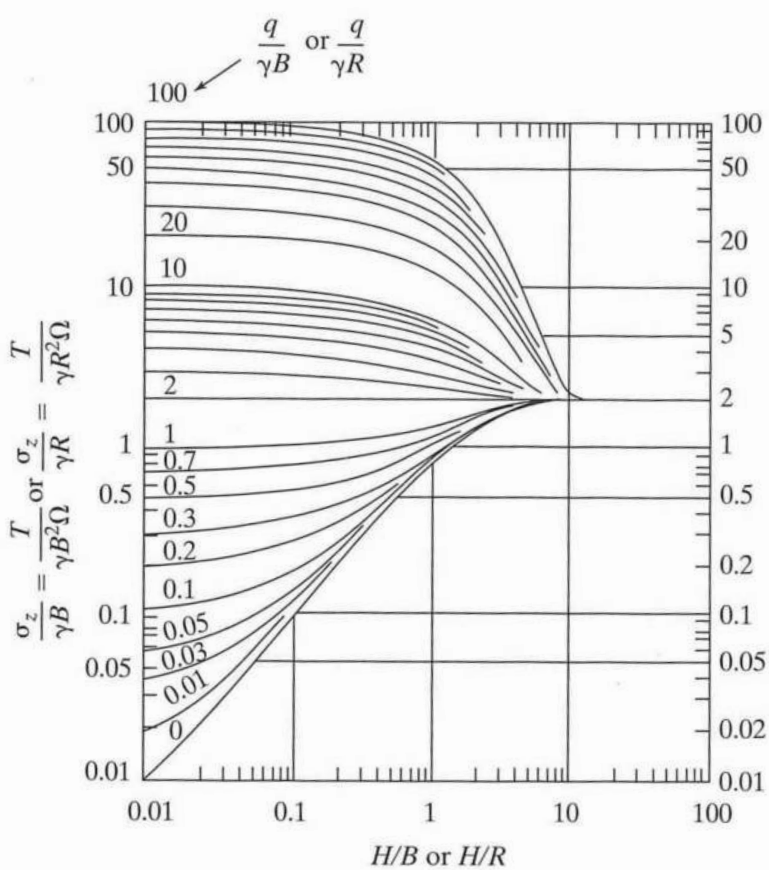


Figure 3.20 Curves of geosynthetic stress and tension that can be used for R (radius of circular void) or B (width of long voids). (After Giroud et al. [49])

$$T_{\text{reqd}} = 2.0(12)(1)^2(0.73)$$
$$= 17.5 \text{ kN/m} \qquad \text{which checks}$$

(c) The factor of safety on a geogrid with 125 kN/m ultimate strength (at 10% strain) is as follows:

$$T_{\text{allow}} = T_{\text{ult}}/\Pi\text{RF}$$
$$= \frac{125}{5}$$
$$= 25 \text{ kN/m}$$

and

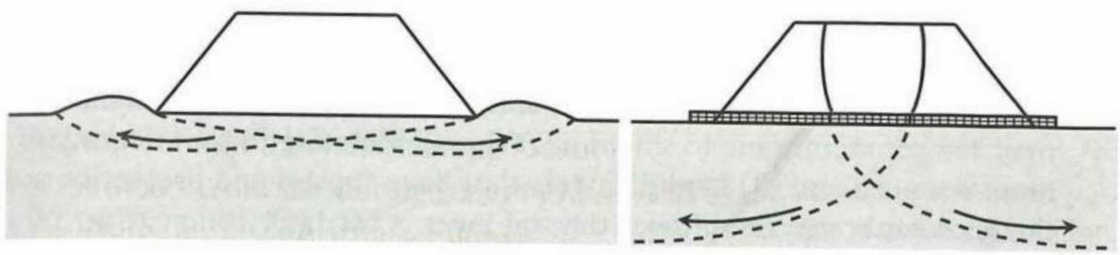
$$\text{FS} = T_{\text{allow}}/T_{\text{reqd}}$$
$$= 25/17.5$$
$$\text{FS} = 1.43 \qquad \text{which is acceptable}$$

In a somewhat different context, but still focused on foundation and basal soil improvement, Edgar [50] reports on a three-dimensional *geogrid mattress* where 1.0 m wide unitized HDPE geogrids are placed vertically and interconnected to one another. Gravel is placed within the geogrid mattress as it is constructed over soft fine-grained foundation soils. Edgar reports on a 15 m high embankment that was successfully constructed above the mattress. It was felt that the nonreinforced slip plane was forced to pass vertically through the mattress and therefore deeper into the stiffer layers of the underlying subsoils. This improved the foundation stability to the point where the mode of failure was probably changed from a circular arc to a less critical plastic failure of the soft clay. The design was considered to be a successful and economic one. Another example of a 1 m high unitized geogrid mattress was constructed to support a 30 m high landfill over extremely soft mine tailings in Hausham, Germany [51]. The mattress was filled with gravel and the liner system constructed above it. The foundation soil was so soft that a nonwoven geotextile and a bidirectional geogrid had to be initially placed to provide a stable working area for the construction of the three-dimensional mattress. Such relatively thick mattresses can also be constructed by using closely spaced layers of bidirectional geogrids separated by granular soil.

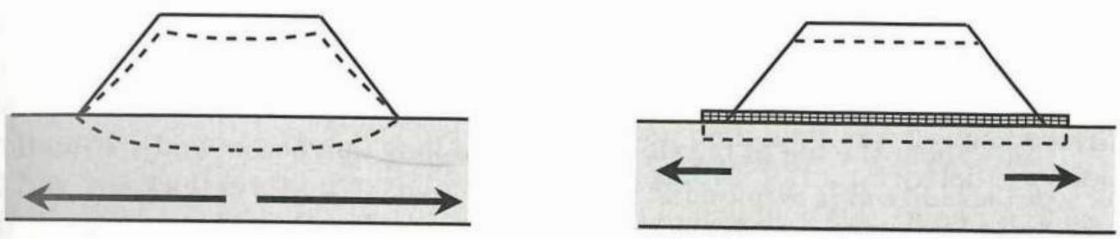
In the design of such three-dimensional geogrid mattresses, it is felt that the following phenomena are occurring, all of which improve foundation soil stability (see Figure 3.21):

- *Global slope stability:* This is improved by forcing the potential failure plane through the mattress and deeper into the foundation soil. It is also possible that the foundation soil may improve in strength characteristics at greater depths.
- *Bearing capacity:* This is improved in a similar manner to the point where it becomes a nonissue for mattresses greater than approximately 30 m in width.
- *Lateral extrusion* (or *squeeze-out*)*:* This is undoubtedly decreased because stress concentrations have been largely eliminated via a uniform pressure distribution applied through the relatively stiff geogrid mattress.

(a) Global slope stability



(b) Bearing capacity



(c) Lateral extrusion

Figure 3.21 Potential improvement of embankments on soft foundation soils via three-dimensional geogrid mattresses.

In the absence of global instability, this last item is particularly important. Squeeze-out of the foundation soil is the likely service-limiting mechanism giving rise to excessive deformations. Robertson and Gilchrist [52] and Jenner et al. [53] have used slip line fields to predict the principle stresses in the soft foundation soils. Both studies give actual case histories and the monitoring feedback as to the validity of the design assumptions.

The latest application area in the context of foundation and basal soil reinforcement is the use of geogrids to span deep foundations placed through compressible soils [54, 55]. The geogrids span from pile cap to pile cap, reducing localized settlement in the supported embankment system. From a design stand point, the situation is exactly the same as that described in Section 2.7.4 using geotextiles. High strength geotextiles and geogrids are competitive in this particular application. The technique is considered very appropriate when stone columns are used as the ground modification technique. Sometimes the stone columns are actually contained in a geogrid enclosure, which appears to be a growing application [56].

3.2.7 Veneer Cover Soils

Whenever a lined slope (geomembrane, GCL, or compacted clay) is covered with soil, a stability calculation should be made to assess the potential for sliding failure of the soil on the barrier layer. Three situations come to mind: (1) landfill liners with leachate collection sand or gravel above them until such time that the solid waste acts as a passive resistance restraint; (2) surface impoundment liners where the cover soil is placed over the geomembrane to shield it from ultraviolet light, heat degradation, and equipment damage; and (3) landfill covers that have topsoil and protection soil placed over the geomembrane. In all cases the soil layer is relatively thin (0.3 to 1.0 m), hence the sliding stability of such a veneer of cover soil is the issue.

Due to the typically low shear strength of the covering soil to the liner material, numerous stability problems have arisen. The driving forces creating the instability are gravitational forces, equipment loads, surcharge loads, seepage forces, and/or seismic forces. Each must be carefully considered in the context of the site-specific conditions.

Koerner and Soong [57] have analyzed the general situation through the use of limit equilibrium and a finite slope model, as shown in Figure 3.22. Consider a cover soil placed directly on a geomembrane (or other barrier layer) at a slope angle β. Two discrete zones can be visualized, as shown in Figure 3.22a. There is a small passive wedge near the toe of the slope resisting a long thin active wedge extending the length of the slope. It is assumed that the cover soil is of uniform thickness and constant unit weight. At the top of the slope or at an intermediate berm, we anticipate that a tension crack in the cover soil will occur, thereby breaking continuity with the remaining cover soil at the crest.

Resisting the tendency for the cover soil to slide is the interface friction and/or adhesion of the cover soil to the specific type of underlying geomembrane. The shear strength values of δ and c_a must be obtained from a laboratory direct-shear test, as described earlier. Note that the passive wedge is assumed to move on the underlying cover soil so that the shear strength parameters ϕ and c, which come from soil-to-soil friction tests, will also be required.

By taking free bodies of the passive and active wedges with the appropriate forces being applied, the formulation for the factor of safety results. The resulting equation is not an explicit solution for the FS, and it must be solved using the quadratic equation. The complete development of the equation is given in [57]. Other approaches are found in Giroud and Beech [58], Koerner and Hwu [59], and Thiel and Stewart [60].

Giroud, Jeanpierre & Bonaparte, Rudolph & Beech, J. & Gross, Beth. (1988).

Load-carrying capacity of a soil layer supported by a geosynthetic overlying a void.

Proceedings of the International Geotechnical Symposium on Theory and Practice of Earth Reinforcement. 185-190.

International Geotechnical Symposium on Theory and Practice of Earth Reinforcement / Fukuoka Japan / 5-7 October 1988
 ISBN 90 6191 820 0

Load-carrying capacity of a soil layer supported by a geosynthetic overlying a void

J.P.Giroud
GeoServices Inc. Consulting Engineers, Boynton Beach, Fla., USA

R.Bonaparte, J.F.Beech & B.A.Gross
GeoServices Inc. Consulting Engineers, Norcross, Ga., USA

ABSTRACT: This paper presents equations and charts to design soil layer-geosynthetic systems overlying voids such as cracks, sinkholes, and cavities. These equations and charts were developed by combining tensioned membrane theory (for the geosynthetic) with arching theory (for the soil layer), thereby providing a more realistic design approach than one that considers tensioned membrane theory only.

1 INTRODUCTION

1.1 Description of the problem

In many practical situations, a load is applied on a soil layer-geosynthetic system that will eventually overlie a void. (In this paper, the term "void" is used generically for cracks, cavities, depressions, etc.) Typical examples include road embankments or lining systems constructed on foundations where localized subsidence or sinkholes may develop after construction.

The design engineer has to verify that the load is adequately supported by the soil-geosynthetic system, should the subsidence or sinkhole develop.

1.2 Scope of this paper

This paper presents equations and charts for the case of a soil layer subjected to a uniformly distributed normal stress and overlying either an infinitely long void (plane-strain problem) or a circular void (axisymmetric problem). The parameters considered in this paper are (Figure 1): b - width of the infinitely long void; r - radius of the circular void; H - thickness of the soil layer; γ - unit weight of the soil; ϕ - friction angle of the soil (soil cohesion is not considered); q - uniformly distributed normal stress applied on the top of the soil layer; y - geosynthetic deflection; and α - geosynthetic tension (force per unit width) corresponding to the geosynthetic strain, ϵ.

1.3 Prior work

The use of tensioned membrane theory to evaluate the load-carrying capacity of a geosynthetic bridging a void was presented by Giroud (1981). Subsequently, Giroud (1982) developed a design chart based on tensioned membrane theory. This chart has often been used to evaluate the load-carrying capacity of a soil layer associated with a geosynthetic. By doing so, the internal shear strength of the soil layer is neglected and this can be very conservative. Therefore, Bonaparte and Berg (1987) have suggested that arching theory (for the soil layer) be combined with tensioned membrane theory (for the geosynthetic) to enable a more realistic design approach.

This paper significantly extends the earlier work of Giroud (1981 and 1982) and Bonaparte and Berg (1987) and provides the most extensive analysis yet of a soil-geosynthetic system bridging a void.

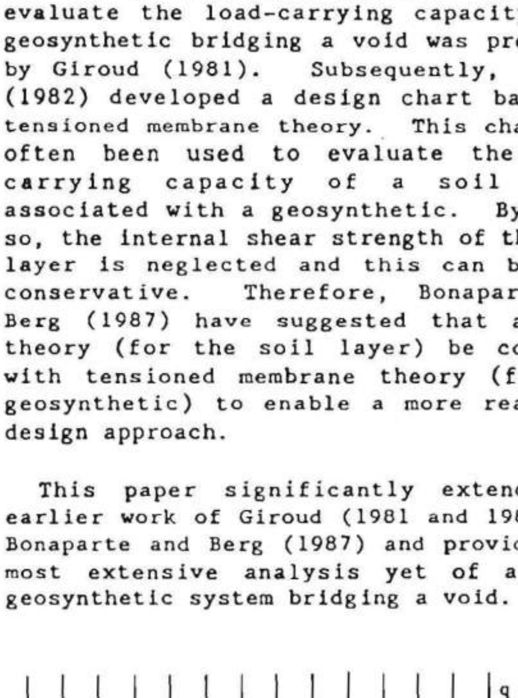


Fig. 1 Schematic cross section

1.4 Load carrying mechanism

The soil and underlying geosynthetic are assumed to initially be resting on a firm foundation. At some point in time, a void of a certain size opens below the geosynthetic. Under the weight of the soil layer and any applied loads the geosynthetic deflects. The deflection has two effects, bending of the soil layer and stretching of the geosynthetic.

The bending of the soil layer generates arching inside the soil, which transfers part of the applied load away from the void area. As a result, the stresses transmitted to the geosynthetic over the void area are smaller than the pressure due to the weight of the soil layer and applied stresses.

The stretching of the geosynthetic mobilizes a portion of the geosynthetic's strength. As a result, the geosynthetic acts as a "tensioned membrane" and can carry a load normal to its plane.

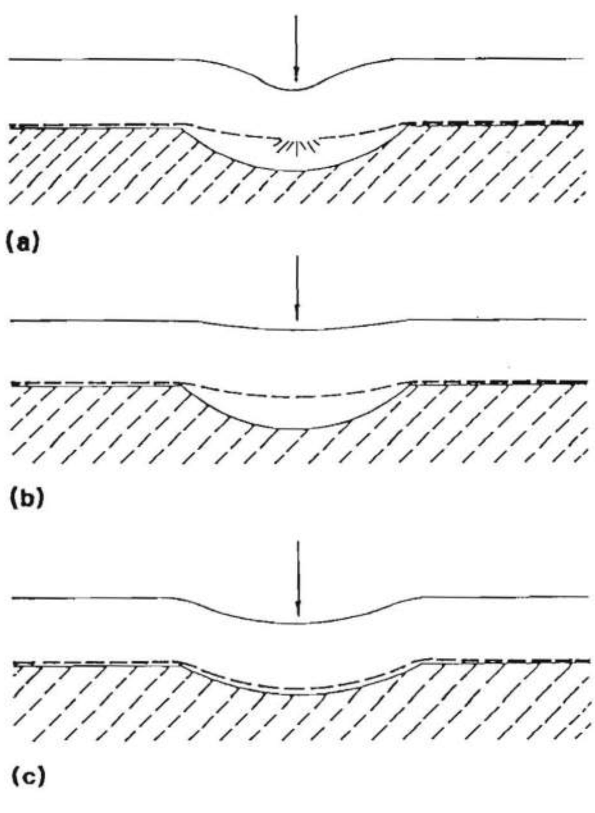


Fig. 2 Three design situations: (a) the soil-geosynthetic system fails; (b) the soil-geosynthetic system exhibits limited deflection and bridges the void; and (c) the soil-geosynthetic system deflects until the geosynthetic comes in contact with the bottom of the void

As a result of geosynthetic stretching, three cases can be considered: (i) the soil-geosynthetic system fails (Figure 2a); (ii) the soil-geosynthetic system exhibits some limited deflection and bridges the void (Figure 2b); and (iii) the soil-geosynthetic system deflects until the geosynthetic comes in contact with the bottom of the void (Figure 2c). In the latter case, the mobilized portion of the geosynthetic strength carries a portion of the load applied normal to the surface of the geosynthetic. The rest of the load is transmitted to the bottom of the void.

2 ANALYSIS

2.1 Approach

The problem under consideration involves a complex soil-geosynthetic interaction. The problem can be greatly simplified, however, if the soil response (arching) is uncoupled from the geosynthetic response (tensioned membrane). Therefore, a two-step approach is used. First, the behavior of the soil layer is analyzed using classical arching theory. This step gives the pressure at the base of the soil layer on the portion of the geosynthetic located above the void. Second, tensioned membrane theory is used to establish a relationship between the pressure on the geosynthetic, the tension and strain in the geosynthetic, and the deflection.

An inherent assumption in this uncoupled two-step approach is that the soil deformation to generate the soil arch is compatible with the tensile strain to mobilize the geosynthetic tension. This assumption has not been verified.

2.2 Arching theory

Terzaghi (1943) has established the following equation for arching in the case of an infinitely long void, assuming that the lateral load transfer is achieved through shear stresses along vertical planes located at the edges of the void:

$$p = \frac{\gamma\, b}{2\, K \tan\phi}\left[1 - e^{-2\, K \tan\phi\, H/b}\right] + q\, e^{-2\, K \tan\phi\, H/b} \qquad (1)$$

where: p = pressure on the geosynthetic over the void area; K = coefficient of lateral earth pressure; and other notations as defined in Section 1.2.

Using the same approach, Kezdi (1975) has established that Equation 1 can be used for a circular void if b is replaced by r (and not by 2r), which shows that arching is twice as significant for a circular void as compared to an infinitely long void.

Selection of a value for the coefficient of lateral earth pressure, K, is not easy since the state of stress of the soil in the zone where arching develops is not well known. Handy (1985) has proposed the following value:

$$K = 1.06\ (\cos^2\theta + K_a \sin^2\theta) \qquad (2)$$

where: $\theta = 45° + \phi/2$, and $K_a = \tan^2(45° - \phi/2)$.

Equation 2 was used previously by Bonaparte and Berg (1987). Another approach consists of using the coefficient of earth pressure at rest, expressed as follows, according to Jaky (1944):

$$K = 1 - \sin\phi \qquad (3)$$

In Equation 1, K is always multiplied by $\tan\phi$. Calculations carried out using Equations 2 and 3, show that $K\tan\phi$ does not vary significantly with ϕ, if ϕ is equal to or greater than 20°, which is the case for virtually all granular soils. The calculations show that a constant value of 0.25 can be conservatively used for $K\tan\phi$. As a result, Equation 1 becomes:

$$p = 2\ \gamma b\ (1 - e^{-0.5H/b}) + q\ e^{-0.5H/b} \qquad (4)$$

Equation 4 is also valid for the circular void if b is replaced by r, and it was used to establish the chart given in Figure 3.

2.3 Tensioned membrane theory

The tensioned membrane theory has been used by Giroud (1981, 1984) to deal with the case of a geosynthetic overlying a void and subjected to a uniformly distributed stress normal to its surface.

In the case of an infinitely long void, the deflected shape of the geosynthetic is circular, the strain uniform, and the following relationship exist if $y/b < 0.5$:

$$1 + \epsilon = 2\ \Omega \sin^{-1} [\ 1/(2\ \Omega)\ ] \qquad (5)$$

where: ϵ, y, and b are as defined in Section 1.2; and Ω = is a dimensionless factor defined by:

$$\Omega = (1/4)\ [2y/b + b/(2y)] \qquad (6)$$

As a result of Equations 5 and 6, there is a unique relationship between y/b, ϵ, and Ω, which is given in Figure 4.

Giroud (1981, 1984) has also shown that the tension in the geosynthetic, in the case of an infinitely long void, is given by:

$$\alpha = p\ b\ \Omega \qquad (7)$$

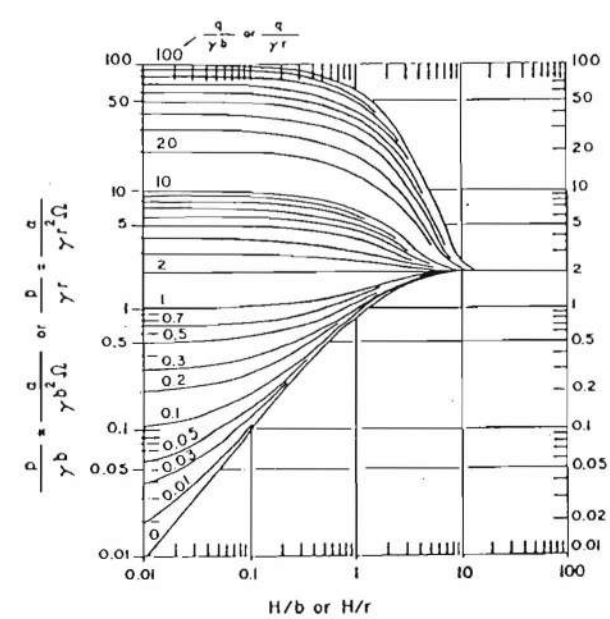


Fig. 3 Pressure, p, on the geosynthetic and geosynthetic tension, α

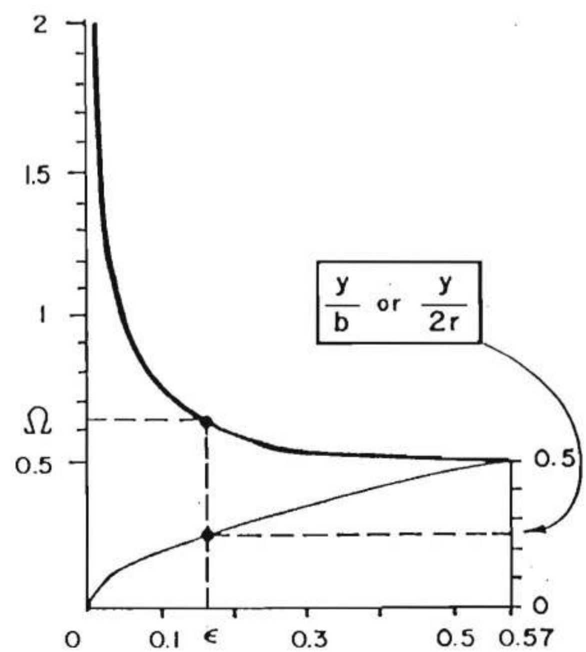


Fig. 4 Dimensionless factor Ω

As described by Giroud (1981), the deflected shape of the geosynthetic is not a sphere in the case of a circular void. As a consequence, incorporating 2r (diameter) instead of b (width) into Equations 5 and 6 gives only an approximate value of the average geosynthetic strain, ϵ.

Since the strain is not uniform, the tension, α, in the case of a circular void is not uniformly distributed in the geosynthetic and its average value is given approximately by Equation 7, with r substituted for b (Giroud, 1981, 1984).

It should be noted that Equation 7 can be used for a circular void only if the geosynthetic has isotropic tensile characteristics, i.e., the same tensile characteristics in all directions. If this is not the case, recommendations given in Section 4.1 should be followed.

3 SOLUTION OF TYPICAL DESIGN PROBLEMS

Typical design problems can be solved using the following equations which were obtained by combining Equations 4 and 7. In all the design cases considered below, the solution depends on the value of Ω which depends either on the allowable geosynthetic strain, ϵ, or on the allowable deflection, y.

In this section, the depth of the void is assumed to be such that the geosynthetic is not in contact with the void bottom. The case where the geosynthetic comes in contact with the bottom of the void is more complex and will not be addressed in this paper.

3.1 Determination of geosynthetic properties

The relevant equation for an infinitely long void is:

$$\alpha/\Omega = pb = 2\gamma b^2(1 - e^{-0.5H/b}) + q\ b\ e^{-0.5H/b} \tag{8}$$

Equation 8 can be rewritten in a dimensionless form as follows:

$$\frac{\alpha}{\gamma b^2 \Omega} = \frac{p}{\gamma b} = 2(1 - e^{-0.5H/b}) + \frac{q}{\gamma b} e^{-0.5H/b} \tag{9}$$

This equation, which is related to the infinitely long void, was used to establish the chart in Figure 3.

Equations 8 and 9 can be used for a circular void if b is replaced by r.

The above equations can be used to solve problems which consist of determining the required geosynthetic tension, α, for a given strain, ϵ, when all other parameters are given (b or r, q, H, and γ). Alternatively, the chart given in Figure 3 can be used.

3.2 Determination of soil layer thickness

The relevant equation for an infinitely long void is:

$$H = 2\ b\ \mathrm{Log} \frac{[q/(\gamma b)] - 2}{[\alpha/(\gamma b^2 \Omega)] - 2} \tag{10}$$

The same equation can be used for a circular void by substituting r for b.

The above equation can be used to solve problems which consist of determining the required soil layer thickness, H, when all other parameters are given (b or r, q, γ, α, and ϵ). Alternatively, the chart given in Figure 3 can be used.

3.3 Determination of maximum void size

There is no simple equation giving the void size (b or r) as a function of the other parameters. In order to determine the maximum void size that a given soil layer-geosynthetic system can bridge, it is necessary to solve Equation 8 by trial and error. To facilitate the process, a chart has been established (Figure 5) by rewriting the two parts of Equation 9 in a dimensionless form as follows:

$$\frac{p}{\gamma H} = \frac{2(1 - e^{-0.5H/b})}{H/b} + \frac{q}{\gamma H} e^{-0.5H/b} \tag{11}$$

$$\frac{p}{\gamma H} = \frac{\alpha}{\gamma H^2 \Omega} \frac{H}{b} \tag{12}$$

In Figure 5, Equation 11 is represented by a family of curves and Equation 12 is represented by a family of straight lines at 45°. For a given set of parameters, the abscissa of the intersection between the relevant curve and the relevant straight line gives the maximum value of the width, b, of an infinitely long void, or the radius, r, of a circular void.

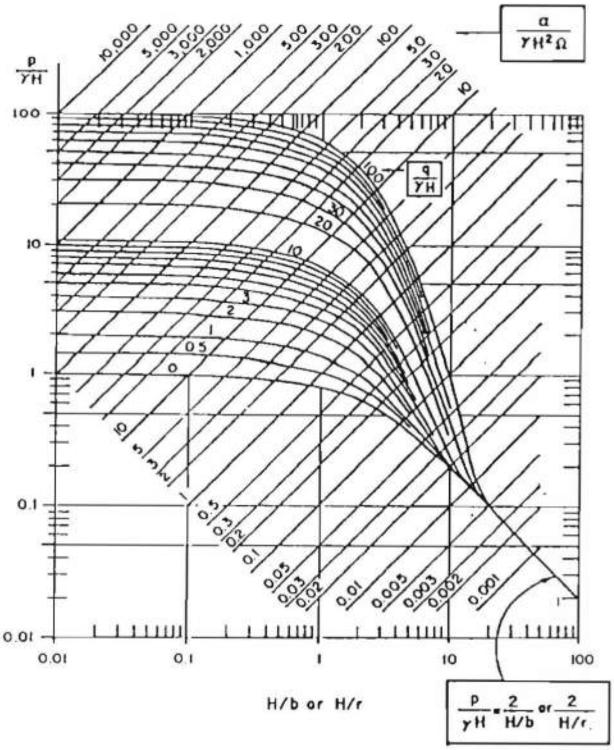


Fig. 5 Chart for maximum void size determination

3.4 Determination of the maximum load

The relevant equation for an infinitely long void is:

$$q = 2\gamma b + \left\{ \frac{[\alpha/(\gamma b^2 \Omega)] - 2}{e^{-0.5H/b}} \right\} \gamma b \qquad (13)$$

The same equation can be used for a circular void by substituting r for b.

The above equation can be used to solve problems which consist of determining the maximum uniform normal stress, q, which can be applied on the top of the soil layer, when all other parameters are given (b or r, H, γ, α, and ϵ). Alternatively, the charts given in Figure 3 or 5 can be used.

4 DISCUSSION

4.1 Anisotropic geosynthetic

Special precautions must be taken when using the equations and charts presented in this paper for anisotropic geosynthetics.

In the case of a long void, the geosynthetic should be installed with its stronger direction perpendicular to the length of the void since, theoretically, no strength is needed in the direction of the length of the void (according to the plane-strain model which corresponds to an infinitely long void). However, some strength is required lengthwise in places where the actual situation departs from a pure plane-strain situation, for instance near the end of the void.

In the case of a circular void, the tensioned membrane equation (Equation 7) is valid only if the geosynthetic has isotropic tensile characteristics. For practical purposes, Equation 7, and other equations as well as charts related to circular voids, can be used for woven geotextiles and biaxial geogrids that have similar tension-strain curves in two perpendicular directions. For woven geotextiles and biaxial geogrids that have different tensile characteristics in the two principal directions, two cases can be considered with circular voids, depending on the ratio between the geosynthetic tensions at the design strain in the weak and the strong directions: (i) if the ratio is more than 0.5, α should be taken equal to the tension in the weak direction; and (ii) if the ratio is less than 0.5, α should be taken equal to half the tension in the strong direction.

The rationale in the first case is conservativeness. The rationale in the second case is as follows. Comparison of Equation 7 written with b and the same equation written with r shows that, if the geosynthetic tension in one direction is less than half the tension in the other direction, the system placed over a circular void behaves as if it were on an infinitely long void with a width, b, equal to the diameter, 2r, of the actual void. Therefore, Equation 7 must be used, in this case, with 2r (instead of b) and α, or with r and $\alpha/2$, as recommended above.

There is another consideration when an anisotropic geosynthetic is used over a circular void. The complex pattern of strains in the geosynthetic resulting from different tensions in different directions may have a detrimental effect on the behavior of the geosynthetic. Therefore, it is recommended that, for holes which can be modeled as circular, isotropic geosynthetics (such as most nonwoven geotextiles) or "practically isotropic" geosynthetics (such as woven geotextiles or biaxial geogrids having similar tension-strain curves in the two principal directions) be used.

4.2 Influence of soil layer thickness

The influence of the thickness of the soil layer is illustrated in Figure 3. Three cases can be considered.

If the applied stress, q, is large (i.e., $q > 2\gamma b$ or $2\gamma r$), the pressure, p, on the geosynthetic, and consequently the required geosynthetic tension, α, decrease toward a limit as the soil layer thickness increases. In this case, it is beneficial to increase the thickness of the soil layer. For each particular situation, the amount by which the thickness should be increased can be determined using the chart given in Figure 3. This chart shows that it would be useless to increase the soil layer thickness beyond a limiting value of $H = 20\ b$ or $20\ r$.

If the applied stress, q, is small (i.e., $q < 2\gamma b$ or $2\gamma r$), the pressure, p, on the geosynthetic, and consequently the required geosynthetic tension, α, increase toward a limit as the soil thickness increases. In this case, it is detrimental, from the perspective of the design of the geosynthetic, to increase the thickness of the soil layer. (This is because the added load due to soil weight is not fully compensated by the effect of soil arching.)

If the applied stress, q, equals $2\gamma b$ or $2\gamma r$, the pressure, p, on the geosynthetic remains constant and equal to q, regardless of the soil layer thickness.

The limit values for p and α are independent of the applied stress, q. The limit value for p is $2\gamma b$ for an infinitely long void or $2\gamma r$ for a circular void. The limit value for α is $2\gamma b^2\Omega$ for an infinitely long void or $2\gamma r^2\Omega$ for a circular void.

5 CONCLUSION

The analysis shows that the thickness of the soil layer associated with the geosynthetic plays a significant role. In contrast, the soil mechanical properties do not. It should not be inferred, however, that any soil will provide the same degree of arching. The equations used to prepare the tables and charts assume that the friction angle of the soil is at least 20°. Granular soils virtually always meet this condition. However, they should be well compacted to ensure arching because loose granular soils tend to contract when they are sheared or vibrated, which may destroy the arch.

Further refinements of the method presented herein can be considered. For instance, it is possible that the degree of soil arching depends on the geosynthetic strain, whereas the method presented in this paper does not consider the concept of degree of soil arching. Also, the method could be expanded to include cohesive soils. (The equations and charts presented in this paper are essentially intended for granular soils; however, they can be used for saturated cohesive soils in the drained state, assuming that their cohesion is zero and provided that their drained friction angle is greater than 20°.)

In spite of its limitations, the method presented in this paper is believed to be a useful tool for engineers designing soil-geosynthetic systems resting on subgrades which may subsequently develop voids.

ACKNOWLEDGMENTS

The authors are indebted to G. Saunders, A. Mozzar, G. Kent, and A.H. Perry for assistance during the preparation of this paper.

REFERENCES

Bonaparte, R., and Berg, R.R. 1987. The use of geosynthetics to support roadways over sinkhole prone areas. Proceedings of the Second Multidisciplinary Conference on Sinkholes and the Environmental Impacts of Karst, Orlando: 437-445.

Giroud, J.P. 1981. Designing with geotextiles. Materiaux et Constructions, 14, 82: 257-272.

Giroud, J.P. 1982. Design of geotextiles associated with geomembranes. Proceedings of the Second International Conference on Geotextiles, 1, Las Vegas: 37-42.

Giroud, J.P. 1984. Geotextiles and geomembranes, definitions, properties and design. I.F.A.I. Publisher, St. Paul, MN.

Handy, R.L. 1985. The arch in soil arching. Journal of Geotechnical Engineering, ASCE, 111, 3: 302-318.

Jaky, J. 1944. The coefficient of earth pressure at rest. Journal for Society of Hungarian Architects and Engineers, Budapest: 355-358 (in Hungarian).

Kezdi, A. 1975. Lateral earth pressure. Foundation Engineering Handbook, Edited by Winterkorn, H.F. and Fang, H.Y., Van Nostrand Reinhold, New York: 197-220.

Terzaghi, K. 1943. Theoretical soil mechanics. John Wiley, New York.

Giroud, Jeanpierre & Bonaparte, Rudolph & Beech, J.F. & Gross, Beth. (1990).

Design of soil layer-geosynthetic systems overlying voids.

Geotextiles and Geomembranes. 9. 11-50. 10.1016/0266-1144(90)90004-V.

Geotextiles and Geomembranes **9** (1990) 11–50

Design of Soil Layer-Geosynthetic Systems Overlying Voids

J. P. Giroud, R. Bonaparte, J. F. Beech & B. A. Gross

GeoServices Inc. Consulting Engineers,
1200 South Federal Highway, Suite 204
Boynton Beach, Florida 33435, USA

ABSTRACT

This paper presents equations, tables, and charts to design soil layer-geosynthetic systems to span voids such as tension cracks, sinkholes, dissolution cavities, and depressions in foundation soils due to differential settlements or localized subsidence. These equations, tables, and charts were developed by combining tensioned membrane theory (for the geosynthetic) with arching theory (for the soil layer), thereby providing a more complete design approach than one that considers tensioned membrane theory only.

Design examples are presented to illustrate the solution of typical problems such as: selection of the required geosynthetic properties, determination of the maximum void size that can be bridged by a given system, and evaluation of the load-bearing capacity of a given system.

NOTATION

b	Width of the infinitely long void (m)
c	Cohesion of the soil (N/m^2)
D	Depth of the void (m)
H	Thickness of the soil layer (m)
K	Coefficient of lateral earth pressure (dimensionless)
K_a	Coefficient of active earth pressure (dimensionless)
p	Pressure on the geosynthetic (i.e. vertical stress at the bottom of the soil layer) over the void area (N/m^2)

$p_{\lim}$ Limit value for the pressure on the geosynthetic, over the void area (N/m^2)
p_b Pressure transmitted to the bottom of the void (N/m^2)
p_0 Pressure on the geosynthetic over the void area neglecting soil arching (N/m^2)
q Uniformly distributed normal stress applied on top of the soil layer (N/m^2)
r Radius of the circular void (m)
r_{max} Maximum radius of a circular void which can be bridged by a given geosynthetic (m)
s Soil shear strength (N/m^2)
y Geosynthetic deflection (m)
z Depth measured from the top of the soil layer (m)

α Geosynthetic tension (force per unit width) corresponding to the geosynthetic strain ε (N/m)
$\alpha_{\lim}$ Limit value for the required geosynthetic tension (N/m)
ε Geosynthetic strain (dimensionless)
γ Unit weight of soil (N/m^3)
Ω Factor related to y and ε (dimensionless)
ϕ Friction angle of the soil (degrees and dimensionless)
σ_H Horizontal stress at depth z (N/m^2)
σ_V Vertical stress at depth z (N/m^2)

INTRODUCTION

Description of the Problem

In many practical situations, a load is applied on a soil layer-geosynthetic system that will eventually overlie a void. (In this paper, the term 'void' is used generically for cracks, cavities, depressions, etc.) Two typical examples are a road embankment or a lining system for a reservoir constructed on a foundation where localized subsidence may develop.

The design engineer has to verify that, should subsidence develop, the geosynthetic layer can support the loads applied by the overlying soil and any other source (such as traffic on the road or the liquid in the reservoir) without failing or undergoing excessive deflection. The soil-geosynthetic system deflects over the void, and, from a design standpoint, three possibilities must be considered:

- The geosynthetic fails (Fig. 1(a)).

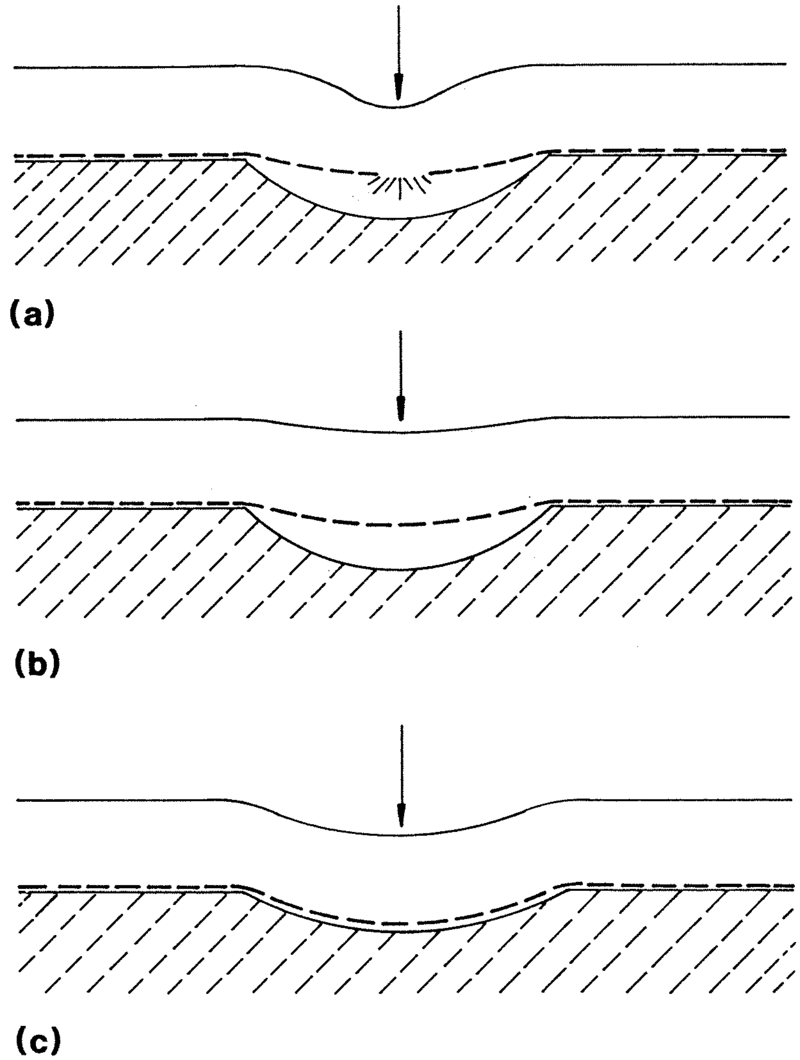


Fig. 1. Three design situations: (a) the geosynthetic fails; (b) the geosynthetic undergoes limited deflection and bridges the void; and (c) the geosynthetic deflects until it comes in contact with the bottom of the void.

- The geosynthetic undergoes limited deflection and bridges the void (Fig. 1(b)).
- The geosynthetic deflects until it comes in contact with the bottom of the void (Fig. 1(c)).

The Nature of Voids

Examples of voids that can develop under a geosynthetic are discussed below:

Tension Cracks
Such cracks can occur in non-saturated cohesive soils subjected to tensile stresses and/or differential movements caused by settlement or other

Fig. 2. Large tension crack formed under a geomembrane liner.

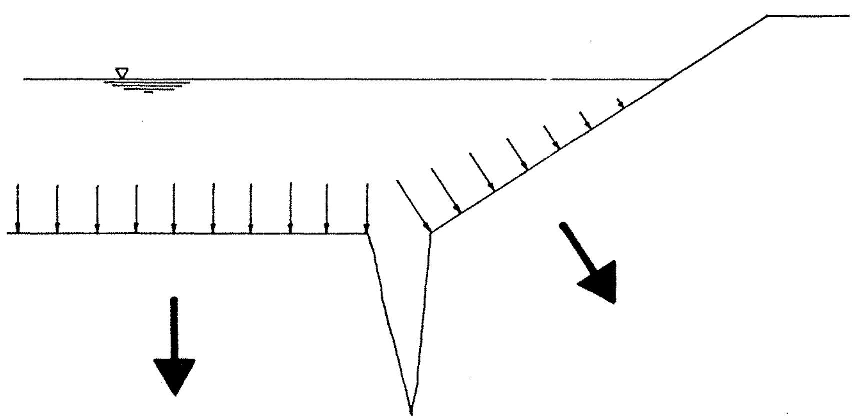


Fig. 3. Mechanism of tension crack formation at the toe of the side slope of a reservoir (not to scale). (After Loudière and Perrin.[1])

mechanisms. A case has been reported[1] where very large cracks (0·1–0·3 m wide) developed in the cohesive soil located under the geomembrane liner of a reservoir (Fig. 2). The cracks occurred near the toe of the side slopes of the reservoir. In this area, tensile stresses and differential movements resulted from the different water pressure orientations on the bottom and on the slopes, as shown in Fig. 3.

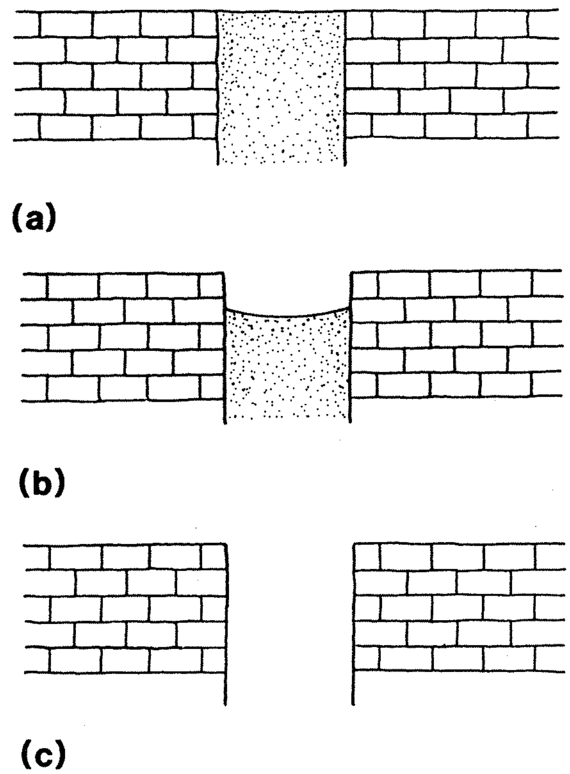


Fig. 4. Sinkhole in a karstic limestone mass: (a) before collapse; (b) after partial collapse; and (c) after complete collapse.

Fissures and Cracks in Bedrock

Soil layers or masses are sometimes constructed on a bedrock with fissures or cracks. A rare but important case is the construction of the clay core of a dam on a bedrock where cracks may develop. Some dam failures have resulted from this situation.

Sinkholes due to Karstic Collapse

Karstic limestone masses contain pockets or chimneys filled with soil. Water or other liquids seeping through a karstic limestone mass may remove soil from these pockets or chimneys, thereby creating a void which can be on the order of one to several meters in diameter (Fig. 4). These voids are usually referred to as sinkholes. The bursting of a geomembrane liner installed on a mass of karstic limestone which subsequently collapsed has been described by Giroud and Goldstein[2] and Giroud.[3] Karstic collapses can occur under other types of structures, such as road embankments, as discussed by Bonaparte and Berg.[4]

Soil Dissolution

Dissolution cavities can be caused by water in soils containing gypsum or by acid in soils containing calcium carbonate. The senior author has

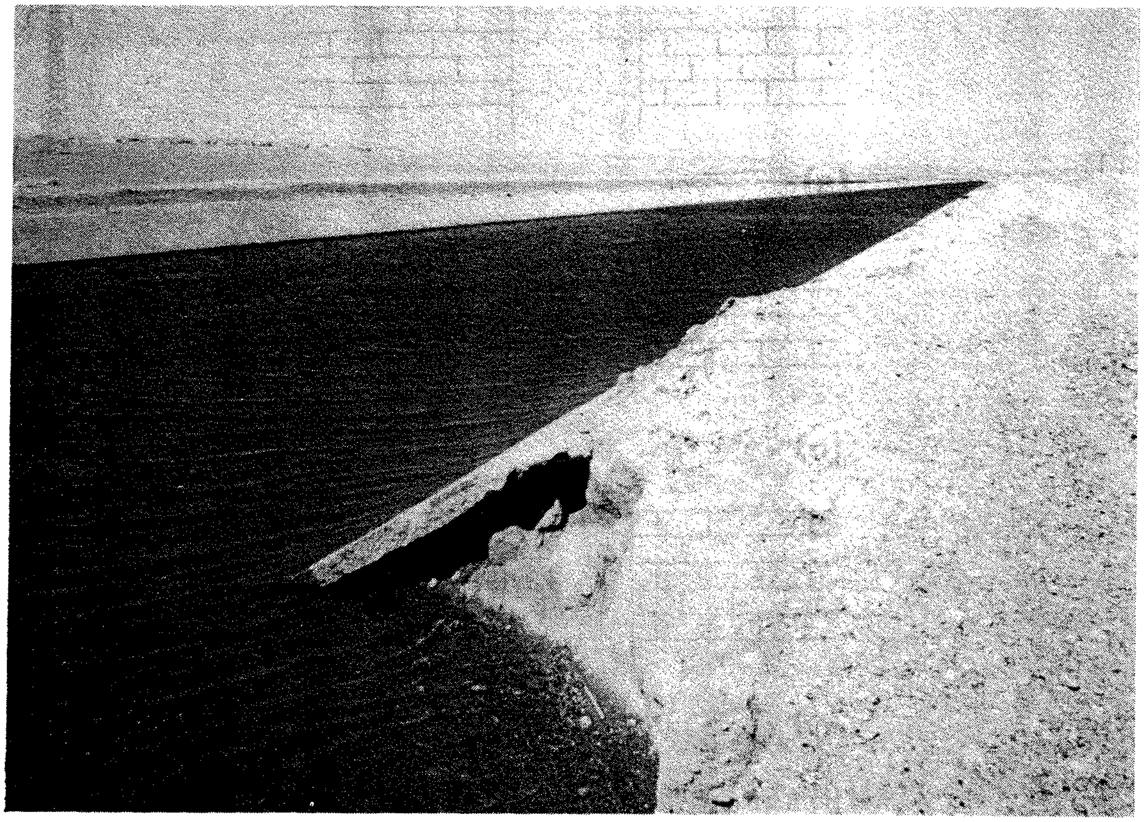


Fig. 5. Dissolution Cavity. This cavity in high gypsum content soil was caused by water leaking through a concrete canal liner.

observed cavities about one meter deep and one meter wide caused by: (i) water leaking through the concrete liner of canals constructed in soils with a high gypsum content (Fig. 5); and (ii) phosphoric acid leaking through a faulty seam of the geomembrane liner of a reservoir constructed on a high calcium-carbonate content soil (Fig. 6).[5]

Differential Settlement

Depressions in the ground surface may be formed when a localized area settles more than the rest ('differential settlement'). There are many situations where depressions result from differential settlement. These include depressions resulting from: (i) differential settlement of municipal solid waste (resulting from the heterogeneity of the waste) affecting a geosynthetic-soil cover system placed on the waste; (ii) settlement of a

Fig. 6. Dissolution Cavity. This cavity in high calcium-carbonate content soil was caused by phosphoric acid leaking through a geomembrane liner.

localized lens of compressible soil; (iii) thawing of subsurface ice lenses; and (iv) settlement of a poorly compacted trench backfill. Tisserand[6] has reported a case of geomembrane failure over the depression resulting from trench backfill settlement. Differential settlement due to lenses of compressible soils frequently occur under road embankments.

Localized Subsidence

The surface of the ground may be locally depressed as a result of the collapse of underground cavities such as: natural caves, tunnels, mine workings, pipes, and tanks. Localized subsidence may also occur at the surface of municipal solid waste as a result of the collapse of deteriorating structures such as refrigerators.

Classification of Voids

Two shapes of voids are considered in the study presented in this paper: infinitely long voids with a width b and circular voids with a diameter $2r$. The voids presented above can therefore be put into two categories:

- Cracks and depressions resulting from trench backfill settlement may be modeled approximately as an infinitely long void.
- Karstic sinkholes, dissolution cavities, municipal solid waste settlement, lens settlement, soil surface depressions and ground subsidence may be modeled approximately as a circular void.

In the case of cracks and complete karstic collapse (Fig. 4(c)), the geosynthetic deflects without reaching the bottom of the void. With the other types of voids, the geosynthetic may or may not reach the bottom of the void, depending on the geometry of the void, the modulus of the geosynthetic and the applied loads.

Load-Carrying Mechanism

The soil layer and underlying geosynthetic are assumed initially to be resting on a firm foundation. At some point in time, a void of a certain size opens below the geosynthetic. Under the weight of the soil layer and any applied loads, the geosynthetic deflects. The deflection has two effects; *bending* of the soil layer and *stretching* of the geosynthetic.

The *bending* of the soil layer generates arching inside the soil, which transfers part of the applied load away from the void area, as shown in Fig. 7. As a result, the vertical stress, σ_V, over the void area is smaller than the average vertical stress, $\gamma H + q$, due to the weight of a soil layer of thickness H and an applied uniform normal stress of magnitude q.

The *stretching* of the geosynthetic mobilizes a portion of the geosynthetic's strength. Consequently, the geosynthetic acts as a 'tensioned membrane' and can carry a load applied normally to its surface. As a result of geosynthetic stretching, two cases can be considered:

- In the first case, the stretched geosynthetic comes in contact with the bottom of the void. The mobilized portion of the geosynthetic strength carries a portion of the load applied normal to the surface of the geosynthetic. The rest of the load is transmitted to the bottom of the void.
- In the second case, the geosynthetic does not deflect enough to come in contact with the bottom of the void. In this case, either the geosynthetic is strong enough to support the entire load applied normal to its surface or it fails.

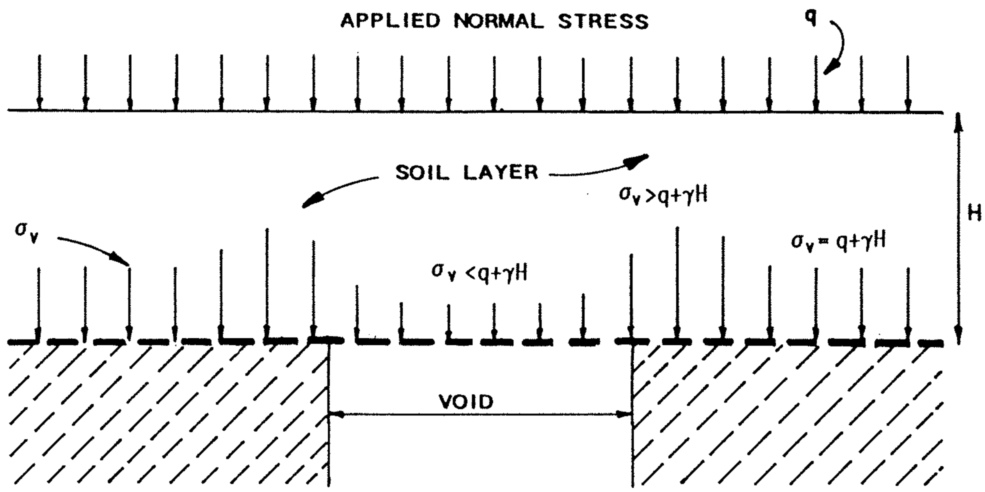


Fig. 7. Effect of soil arching on load distribution.

In summary, the soil-geosynthetic system deflects and the geosynthetic stretches until it fails (Fig. 1(a)) or until an equilibrium condition is reached (Fig. 1(b) or 1(c)).

Scope of this Paper

This paper presents the development and use of equations, tables, and charts for the case of a soil layer subjected to a uniformly distributed normal load and resting on a geosynthetic overlying a rigid foundation containing a single infinitely long void (plane-strain problem) or circular void (axisymmetric problem). The parameters considered in this paper are:

- Geometric Parameters: These include the thickness of the soil layer and the geometry of the void (width of an infinitely long void or diameter of a circular void, and depth of void) (Fig. 8).
- Mechanical Parameters: These include the soil mechanical properties and the geosynthetic tensile behavior (expressed by its tension–strain curve).
- Loading Conditions: These include the unit weight of the soil layer and the load exerted on the top of the soil layer, which is assumed to be normal and uniformly distributed.

The equations, tables, and charts make it possible to solve design problems such as:

- select the required geosynthetic mechanical properties when the geometric parameters and the loading conditions are known;

- determine the required thickness of the soil layer associated with a given geosynthetic over a given void and subjected to given loading conditions;
- determine the void size that a given geosynthetic may bridge when it is associated with a given soil layer subjected to given loading conditions; and
- determine the maximum load which can be carried by a given soil-geosynthetic system over a given void.

The solution of any of the above design problems depends on the allowable geosynthetic strain.

Originality of this Paper

The use of tensioned membrane theory to evaluate the load-carrying capacity of a geosynthetic bridging a void was presented by Giroud.[7] Subsequently, Giroud[8] developed a design chart based on tensioned membrane theory. This chart has often been used to evaluate the load-carrying capacity of a soil layer associated with a geosynthetic. By doing so, the internal shear strength of the soil layer is neglected, and this can be very conservative. Therefore, Bonaparte and Berg[4] combined arching theory (for the soil layer) with tensioned membrane theory (for the geosynthetic) to formulate a more complete design approach.

This paper significantly extends the earlier work of Giroud[7,8] and Bonaparte and Berg[4] and provides an extensive analysis of soil-geosynthetic system bridging a void.

ANALYSIS

Assumptions

The void can be either circular (diameter $2r$) or infinitely long (width b). Regarding the bottom of the void, two cases can be considered: (i) a bottomless void (Fig. 8(a)); and (ii) a bottom with a maximum depth D and a spherical shape (for the circular void) or a cylindrical shape with a circular cross section (for the infinite void) (Fig. 8(b)). From a design standpoint, both cases are identical if the deflection y of the geosynthetic is less than the depth of D of the void.

The soil layer is assumed to be horizontal and to have a uniform thickness H. The stress q applied on the soil layer is assumed to be normal and uniformly distributed.

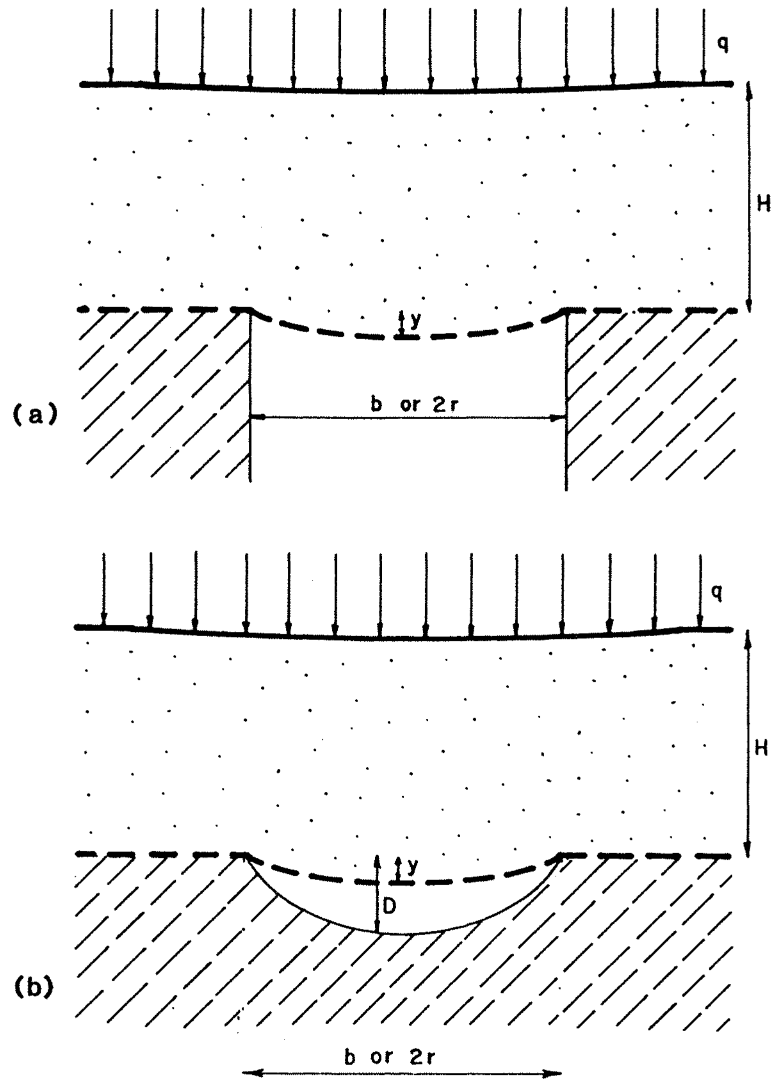


Fig. 8. Schematic cross section for theoretical analysis. Two cases can be considered: (a) the void is bottomless; and (b) the bottom of the void is assumed to have a circular cross section and the depth of the void is D. The void located under the geosynthetic is either infinitely long (with a width b), or circular (with a diameter $2r$); y is the geosynthetic deflection.

Relevant geosynthetic properties are the tension–strain curve or, at least, the tension α corresponding to the design strain ε.

Relevant soil properties are the friction angle ϕ and the cohesion c. For the analysis presented in this paper, the cohesion is neglected. In other words, the charts are established for $c = 0$ and can be conservatively used for $c > 0$. Also, it will be shown that the friction angle ϕ does not have a significant influence on the analysis results if it is equal to or greater than 20°.

Approach

The problem under consideration involves a complex soil-geosynthetic interaction. The problem can be greatly simplified, however, if the soil response (arching) is uncoupled from the geosynthetic response (tensioned membrane). Therefore, a two-step approach is used. First, the behavior of the soil layer is analyzed using classical *arching theory*. This step gives the pressure at the base of the soil layer on the portion of the geosynthetic located above the void. Second, *tensioned membrane theory* is used to establish a relationship between the pressure on the geosynthetic, the tension and strain in the geosynthetic, and the deflection of the geosynthetic. Accordingly, the following sections deal with arching theory, tensioned membrane theory, and the combination of both.

An inherent assumption in this uncoupled two-step approach is that the soil deformation required to generate the soil arch is compatible with the tensile strain required to mobilize the geosynthetic tension. This assumption has not been verified.

Arching Theory (see Fig. 9)

When the geosynthetic deflects, arching develops in the soil layer. As a result, a portion of the applied stress is transmitted laterally and, consequently, the normal stress transmitted to the portion of the geosynthetic located above the void is smaller than the average vertical stress due to the weight of the soil layer and the uniformly distributed normal stress applied on top of the soil layer (Fig. 7). The procedures for calculating the reduced

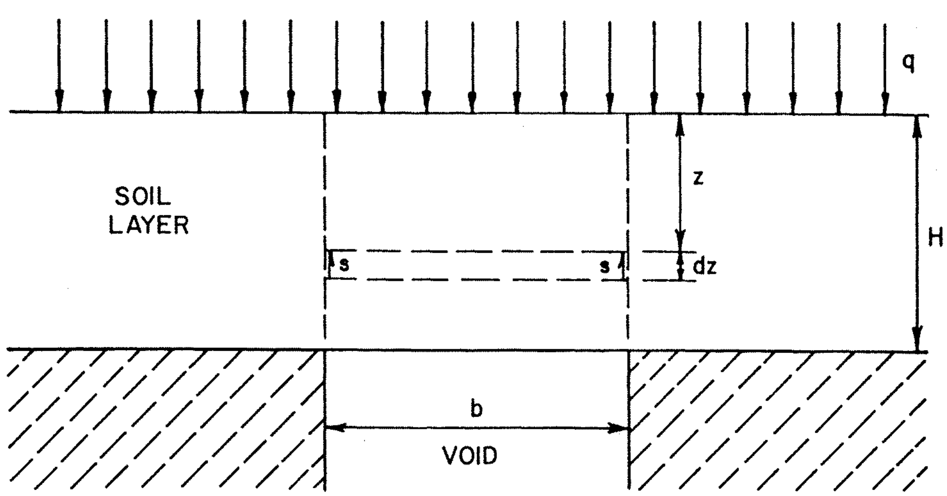


Fig. 9. Derivation of arching equation.

stress transmitted to the portion of the geosynthetic located above the void are presented below for an infinitely long void and a circular void.

Infinitely Long Void

Terzaghi[9] has established equations for soil arching over an infinitely long void assuming that the lateral load transfer is achieved through shear stresses along vertical planes located at the edges of the void (Fig. 9). As a result of this assumption, the incremental change in vertical stress, $d\sigma_V$, due to an incremental change in depth, dz, is given by

$$d\sigma_V = [\gamma - 2(s/b)]\,dz \qquad (1)$$

where: b = width of the infinitely long void; σ_V = vertical stress at depth z; γ = unit weight of soil; z = depth measured from the top of the soil layer; and s = soil shear strength. Basic SI units are: b(m), σ_V (N/m^2), γ (N/m^3), z (m), and s (N/m^2).

The soil shear strength along a vertical plane is expressed by

$$s = c + \sigma_H \tan\phi \qquad (2)$$

where: c = cohesion of the soil; σ_H = horizontal stress at depth z; and ϕ = friction angle of the soil. Basic SI units are: s (N/m^2), c (N/m^2), σ_H (N/m^2), and ϕ (degrees); ϕ is dimensionless.

The relationship between the horizontal stress and the vertical stress is given by the following classical relationship

$$\sigma_H = K\sigma_V \qquad (3)$$

where: K = coefficient of lateral earth pressure (dimensionless).

It should be noted that many of the relationships presented in this paper are valid for both effective and total stress conditions; however, eqn (3) is valid only for effective stress conditions.

Combining eqns (1), (2) and (3) and solving the differential equation for the boundary condition $\sigma_V = q$ for $z = 0$ gives

$$\sigma_V = \frac{b(\gamma - 2c/b)}{2K\tan\phi}\left[1 - e^{-K\tan\phi(2z/b)}\right] + q\,e^{-K\tan\phi(2z/b)} \qquad (4)$$

where: q = uniformly distributed normal stress applied on the top of the soil layer (basic SI unit: N/m^2); all other notations as defined above and in the Notations section.

The pressure on top of the geosynthetic, over the void area, p, is the value of σ_V for $z = H$ in eqn (4). If the soil cohesion, c, is assumed to equal zero, the value of p is

$$p = \frac{\gamma b}{2K \tan \phi}[1 - e^{-2K \tan \phi H/b}] + q\, e^{-2K \tan \phi H/b} \tag{5}$$

where: p = pressure on top of the geosynthetic (i.e. vertical stress at the bottom of the soil layer), over the void area (basic SI unit: N/m^2); and other notations as defined above and in the Notations section.

Circular Void

Using the same approach, Kezdi[10] has established that eqn (5) can be used for a circular void if b is replaced by r (and not by $2r$), which shows that arching is twice as significant for a circular void compared to an infinitely long void.

Practical Approximate Equations

Selection of the value of the coefficient of lateral earth pressure is not easy since the state of stress of the soil in the zone where arching develops is not fully understood. Handy[11] has made a thorough analysis of soil arching and proposed the following value

$$K = 1{\cdot}06(\cos^2 \theta + K_a \sin^2 \theta) \tag{6}$$

with

$$\theta = 45° + \phi/2 \tag{7}$$

and

$$K_a = \tan^2(45° - \phi/2) \tag{8}$$

where: K_a = coefficient of active earth pressure (dimensionless); and other notations as defined above and in the Notations section.

Another approach would consist of using the coefficient of earth pressure at rest, expressed as follows, according to Jaky[12]

$$K = 1 - \sin \phi \tag{9}$$

In eqn (5), K is multiplied by $\tan\phi$. Values of $K\tan\phi$, calculated using eqns (6) and (9), are given in Table 1. It appears that $K\tan\phi$ does not vary significantly with ϕ, if ϕ is equal to or greater than 20°, which is the case for virtually all granular soils and for many fine-grained soils under drained conditions. Therefore, a constant value of 0·25 can be used for $K\tan\phi$ when ϕ is equal to or greater than 20°. As a result, eqn (5) becomes

$$p = 2\gamma b(1 - e^{-0{\cdot}5H/b}) + q\, e^{-0{\cdot}5H/b} \tag{10}$$

TABLE 1
Values of $K \tan \phi$

Soil friction angle (ϕ, degrees)	*Values of $K \tan \phi$*	
	Using K from Handy (eqn (6))	*Using K from Jaky (eqn (9))*
0	0	0
5	0·08	0·08
10	0·15	0·15
15	0·21	0·20
20	0·25	0·24
25	0·29	0·27
30	0·31	0·29
35	0·32	0·30
40	0·32	0·30
45	0·31	0·29
50	0·30	0·28
55	0·27	0·26

Two values of K, the coefficient of lateral earth pressure, are considered: the value proposed by Handy[11] for arching and the value proposed by Jaky[12] for the 'at rest' state of stress.

Like eqn (5), eqn (10) is also valid for the circular void if b is replaced by r.

Equation 10 was used to establish Tables 3 and 4, and the charts given in Figs 11 and 14.

Comment on the Validity of Arching Theory

The analysis presented above is the classical analysis by Terzaghi.[9] This analysis does not consider soil dilatancy, which can increase the horizontal stress in the soil, thereby increasing the ability of the soil to arch. Therefore, the analysis presented in this paper can be considered conservative from this viewpoint. On the other hand, the analysis may not be conservative for loose soils that tend to contract when sheared.

Tensioned Membrane Theory

The tensioned membrane theory has been used by Giroud[3,7] to deal with the case of a geosynthetic overlying a void and subjected to a uniformly distributed stress normal to its surface.

The equations given below have been established with the following assumptions: (i) the strain in the portion of the geosynthetic overlying the void (i.e. the deflected portion of the geosynthetic) is uniformly distributed; and (ii) the strain in the portion of the geosynthetic outside the void area is zero and, therefore, that portion of the geosynthetic does not move (i.e. the geosynthetic does not slide toward the void). These two assumptions greatly simplify the analysis, but no attempt has been made to evaluate their range of validity.

Infinitely Long Void

In the case of an infinitely long void, the deflected shape of the geosynthetic across the width of the void is cylindrical with a circular cross section, the strain is uniform, and the following relationships exist

$$1+\varepsilon = 2\Omega \sin^{-1}[1/(2\Omega)] \qquad (\text{valid if } y/b \leqq 0{\cdot}5) \tag{11}$$

$$1+\varepsilon = 2\Omega\{\pi - \sin^{-1}[1/(2\Omega)]\} \qquad (\text{valid if } y/b \geqq 0{\cdot}5) \tag{12}$$

where: ε = geosynthetic strain; y = geosynthetic deflection; b = width of the infinitely long void; and Ω= dimensionless factor. Basic SI units are: y(m) and b(m); ε and Ω are dimensionless.

The dimensionless factor Ω is defined by

$$\Omega = (1/4)[2y/b + b/(2y)] \tag{13}$$

As a result of eqns (11), (12) and (13), there is a unique relationship between y/b, ε and Ω, which is given in Table 2 and shown in Fig. 10.

It is interesting to note that as ε tends towards zero eqn (11) tends toward

$$\Omega = 1/\sqrt{24\varepsilon} \tag{14}$$

This equation gives a good approximation of Ω when ε is less than 1% (see Fig. 10).

Giroud[3,7] has also shown that the tension in the geosynthetic, in the case of an infinitely long void, is given by

$$\alpha = pb\Omega \tag{15}$$

where: α = geosynthetic tension; p = pressure on the geosynthetic over the void area (i.e. vertical stress at the bottom of the soil layer over the void area); b = width of the infintely long void; Ω = dimensionless factor

TABLE 2
Values of Ω as a Function of Deflection or Strain

y/b or $y/(2r)$	$\varepsilon(\%)$	Ω	y/b or $y/(2r)$	$\varepsilon(\%)$	Ω
0·000	0·000	∞	0·242	15·00	0·64
0·010	0·027	12·51	0·250	15·91	0·62
0·020	0·107	6·26	0·260	17·15	0·61
0·030	0·240	4·18	0·270	18·43	0·60
0·040	0·425	3·15	0·280	19·75	0·59
0·050	0·663	2·53	0·282	20·00	0·58
0·060	0·960	2·11	0·290	21·10	0·58
0·061	1·000	2·07	0·300	22·50	0·57
0·070	1·30	1·82	0·310	23·93	0·56
0·080	1·70	1·60	0·317	25·00	0·55
0·087	2·00	1·47	0·320	25·39	0·55
0·090	2·15	1·43	0·330	26·89	0·54
0·100	2·65	1·30	0·340	28·43	0·54
0·107	3·00	1·23	0·350	30·00	0·53
0·110	3·20	1·19	0·360	31·60	0·53
0·120	3·80	1·10	0·370	33·23	0·52
0·123	4·00	1·08	0·380	34·90	0·52
0·130	4·45	1·03	0·381	35·00	0·52
0·138	5·00	0·97	0·390	36·60	0·52
0·140	5·15	0·96	0·400	38·32	0·51
0·150	5·90	0·91	0·410	40·00	0·52
0·151	6·00	0·90	0·420	41·86	0·51
0·160	6·69	0·86	0·430	43·67	0·51
0·164	7·00	0·84	0·437	45·00	0·50
0·170	7·54	0·82	0·440	45·51	0·50
0·175	8·00	0·80	0·450	47·38	0·50
0·180	8·43	0·78	0·460	49·27	0·50
0·186	9·00	0·76	0·464	50·00	0·50
0·190	9·36	0·75	0·470	51·18	0·50
0·197	10·00	0·73	0·480	53·13	0·50
0·200	10·35	0·72	0·490	55·00	0·50
0·210	11·37	0·70	0·500	57·08	0·50
0·216	12·00	0·69	0·562	70·00	0·50
0·220	12·44	0·68	0·631	85·00	0·51
0·230	13·56	0·66	0·696	100·00	0·53
0·240	14·71	0·64	0·819	130·00	0·56

This table also gives values of the strain as a function of the deflection, and vice versa. (See also Fig. 10.) Notations: Ω = dimensionless factor used for the calculation of the tension in the geosynthetic; y = geosynthetic deflection; b = width of the infinitely long void; $2r$ = diameter of the circular void; and ε = geosynthetic strain. (Note: in the case of a circular void, the values of ε and Ω given in this table are approximate.)

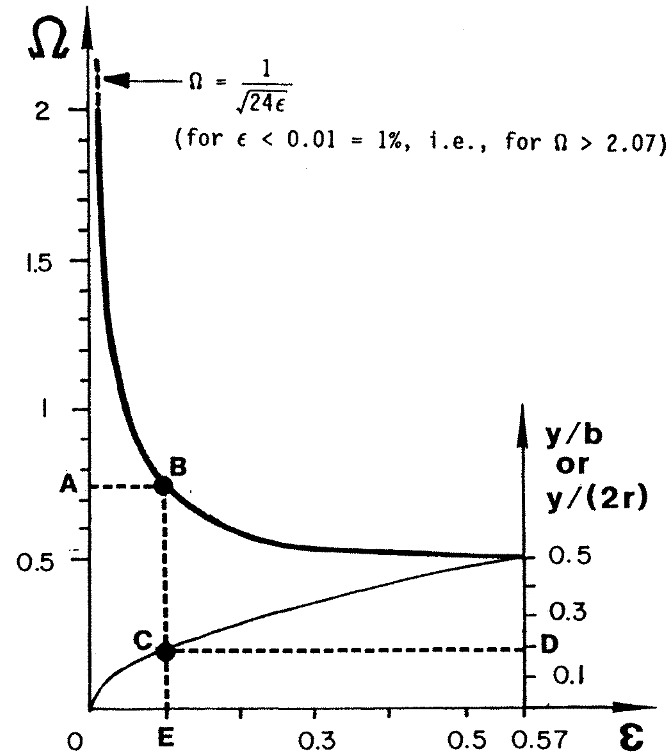


Fig. 10. Dimensionless factor Ω. (See also Table 2.) Notations: b = width of the infinitely long void; $2r$ = diameter of the circular void; y = geosynthetic deflection; ε = geosynthetic strain; and Ω = dimensionless factor. (Note that, in the case of a circular void, y is divided by $2r$, not by r.) This chart can be used as follows: (i) entering a known value of the geosynthetic strain, ε, in E and following EBA gives the value of Ω in A; (ii) entering a known value of the relative deflection, y/b *or* $y/(2r)$, in D and following DCBA gives the value of Ω in A; (iii) entering a known value of the relative deflection, y/b or $y/(2r)$, in D and following DCE gives the value of ε in E; and (iv) vice versa. (For example, $\varepsilon = 0{\cdot}1$ (10%), $\Omega = 0{\cdot}73$, and $y/b = 0{\cdot}197$ are related.)

given in Table 2 and Fig. 10 as a function of ε or y/b; ε = geosynthetic strain; and y = geosynthetic deflection. Basic SI units are: α (N/m), p (N/m^2), b (m), and y (m); Ω and ε are dimensionless.

Circular Void

As described by Giroud,[7] the deflected shape of the geosynthetic is not a sphere in the case of a circular void. As a consequence, incorporating $2r$ (diameter) instead of b (width) into eqns (11), (12) and (13), gives only an *approximate* value of the *average* geosynthetic strain, ε.

Since the strain is not uniform, the tension, α, in the case of a circular void is not uniformly distributed in the geosynthetic and its *average* value is given *approximately* by eqn (15) with r substituted for b.[3,7] It should be noted that, for a circular void, r is substituted for b in eqn (15) whereas $2r$ is used to determine Ω, as indicated in Table 2 and Fig. 10.

Equation (15) can be used for a circular void only if the geosynthetic has isotropic tensile characteristics, i.e. the same tensile characteristics in all directions. If this is not the case, recommendations given in the section 'Discussion of Special Problems' should be followed.

Applications of Tensioned Membrane Theory

Tensioned membrane theory can be used alone (i.e. not combined with arching theory) to solve design problems relating to the case of a geosynthetic acting alone and subjected to a uniformly distributed pressure. This typically occurs in the case of geomembranes directly overlying a void and subjected to pressure from a liquid. Typical design problems are as follows:

- Determine the maximum pressure that a geomembrane can withstand over a void of a given size.
- Select the required geomembrane properties for a geomembrane to bridge a given void when it is subjected to a given pressure.
- Determine the void size that a given geomembrane may bridge when it is subjected to a given pressure.
- Determine the deflection of a geomembrane subjected to a given pressure on a given void, and determine if the deflected geomembrane will come in contact with the bottom of the void.

A chart has been published[2] to help solve these problems. It is also possible to use Table 3 with $H = 0$.

Combination of Arching and Tensioned Membrane Theories

The problem of a bottomless void is entirely solved by using eqns (10) and (15). The case when the geosynthetic comes in contact with the bottom of the void is more complex and will be discussed later in this paper, in the section 'Discussion of Special Problems'.

Equation (10) gives a relationship between the applied stress, the soil layer thickness, the void size, and the pressure on the geosynthetic. This equation was established using *arching theory*.

Equation (15) gives a relationship between the pressure on the geosynthetic, the void size, and the geosynthetic tensile characteristics (tension and strain). This equation was established using *tensioned membrane theory*.

The solution of typical design problems using the equations mentioned above is discussed in the next section.

SOLUTION OF TYPICAL DESIGN PROBLEMS

Overview of the Methods Used

In the presentation of the scope of this paper, a list of typical design problems was given. Solutions to these problems are presented below for the case when the geosynthetic does not come in contact with the bottom of the void. Solutions for the case where the geosynthetic comes in contact with the bottom of the void are presented in the section 'Discussion of Speecial Problems'.

Allowable Strain and Deflection
In all of the design cases considered below, the solution depends on the value of Ω, which depends either on the allowable geosynthetic strain, ε, or the allowable geosynthetic deflection, y. The *allowable geosynthetic strain* is the lesser of the maximum design strain for the considered geosynthetic and the strain beyond which the soil layer would be unacceptably deformed or cracked. The *allowable geosynthetic deflection* is considered when excessive deflection of the soil surface impairs the serviceability of the system. No method is proposed in this paper to evaluate the deflection of the soil surface; however, in the case of relatively thin soil layers, the soil surface deflection can be assumed to be on the same order as the geosynthetic deflection. In some instances, both the allowable geosynthetic strain and the allowable geosynthetic deflection may need to be considered.

Equations and Notations
All equations presented below were obtained by combining eqns (10) and (15). Notations for all subsequent equations are: b = width of the infinitely long void; r = radius of the circular void; Ω = dimensionless factor given in Table 2 as a function of ε or y; H = soil layer thickness; p = normal stress applied on the portion of the geosynthetic located over the void ('pressure on the geosynthetic'); q = uniformly distributed normal stress applied on the top of the soil layer; y = geosynthetic deflection; α = geosynthetic tension; γ = unit weight of soil; and ε = geosynthetic strain. Basis SI units are: b (m), r (m), H (m), p (N/m^2), q (N/m^2), y (m), α (N/m), and γ (N/m^3); Ω and ε are dimensionless.

Factor of Safety
In the following sections, each design problem is illustrated by an example. For the sake of simplicity, no factor of safety is used in the design examples. Engineers using the equations, tables, and charts presented in

this paper should use appropriate factors of safety. The factor of safety can be applied to the geosynthetic tension or the applied loads, with application to the geosynthetic tension being more common. The factor of safety should not be applied to the soil shear strength (as is commonly the case in geotechnical problems) due to the insensitivity of the arching theory results (eqn (5)) to the soil shear strength.

Determination of Required Geosynthetic Properties

The relevant equation for an infinitely long void is

$$\alpha/\Omega = pb = 2\gamma b^2(1 - e^{-0{\cdot}5H/b}) + qb\, e^{-0{\cdot}5H/b} \tag{16}$$

Equation (16) can be rewritten in a dimensionless form as follows:

$$\frac{\alpha}{\gamma b^2 \Omega} = \frac{p}{\gamma b} = 2(1 - e^{-0{\cdot}5H/b}) + \frac{q}{\gamma b} e^{-0{\cdot}5H/b} \tag{17}$$

Equations (16) and (17) can be used for a circular void if b is replaced by r. Equation (17) was used to establish the chart in Fig. 11.

The above equations can be used to solve problems that consist of determining the required geosynthetic tension, α, for a given strain, ε, when all other parameters are given (b or r, q, H, and γ). Alternatively, the chart given in Fig. 11 and the corresponding Table 3 can be used.

Example 1. The bedding soil supporting a geomembrane liner is placed on a geosynthetic reinforcement resting on a soil where karstic sinkholes may develop (Fig. 12). The function of the geosynthetic reinforcement is to support the bedding soil and the geomembrane liner should a sinkhole develop. The thickness of the bedding soil layer is 0·45 m and the depth of water on the geomembrane when the reservoir is full is 9 m. The unit weight of the bedding soil is 19,600 N/m^3. A deep sinkhole with a radius of 0·75 m is assumed for design purposes. Since the function of the geosynthetic reinforcement is only to act as a ‘safety net’, a rather large geosynthetic reinforcement strain is acceptable: $\varepsilon = 10\%$. What is the required geosynthetic reinforcement tensile strength?

First, the applied stress, q, is calculated

$$q = 1000 \times 9{\cdot}81 \times 9 = 88\,290 \text{ N/m}^2$$

Then, eqn (16) is used as follows, with $H/r = 0{\cdot}45/0{\cdot}75 = 0{\cdot}6$

$$\alpha/\Omega = 2 \times 19\,600 \times (0{\cdot}75)^2(1 - e^{-0{\cdot}3}) + 88\,290 \times 0{\cdot}75\, e^{-0{\cdot}3}$$

$$\alpha/\Omega = 54\,395 \text{ N/m}$$

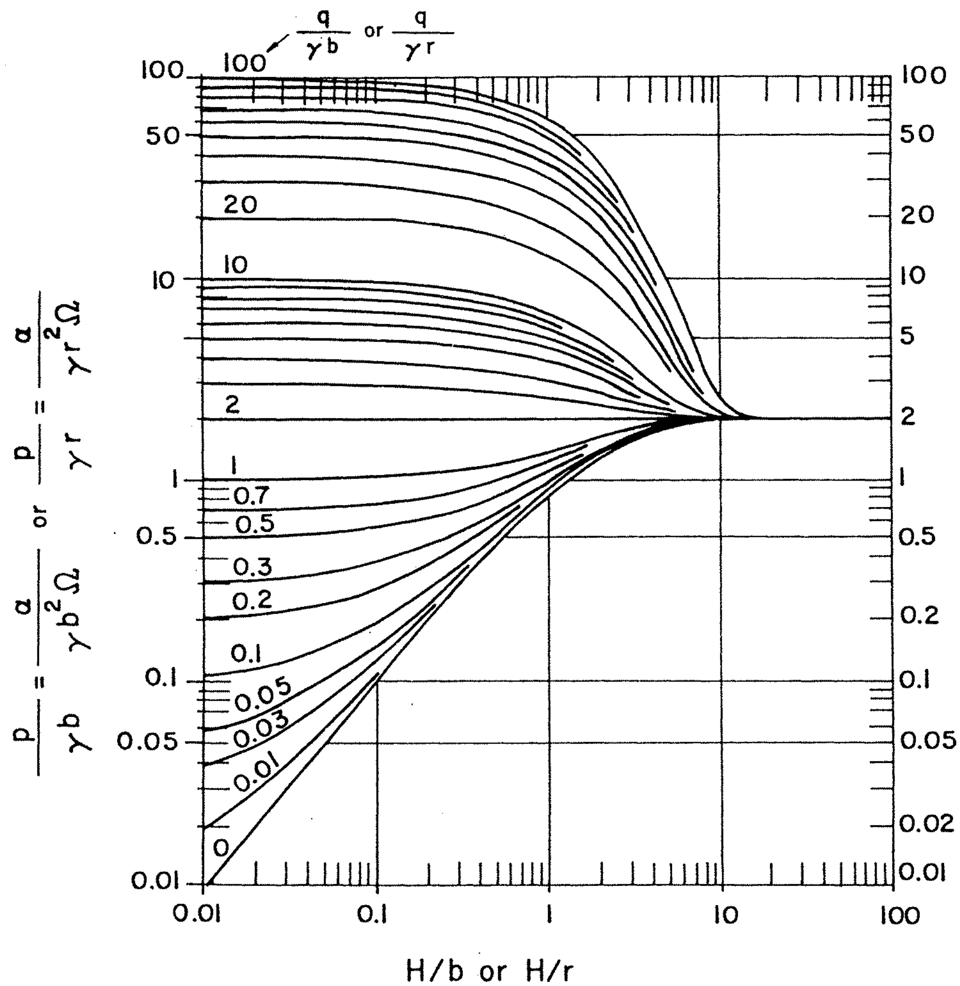


Fig. 11. Pressure on and tension in the geosynthetic. An example of use of this chart is given in Fig. 13. Notations: p = pressure on the geosynthetic over the void area; q = uniformly distributed normal stress applied on the top of the soil layer; H = thickness of the soil layer; γ = unit weight of soil; b = width of the infinitely long void; r = radius of the circular void; α = geosynthetic tension; and Ω = dimensionless factor given in Table 2 and Fig. 10. (Values of $p/(\gamma b)$ or $p/(\gamma r)$ used to draw the curves in this figure can be found in Table 3.)

Finally, according to Table 2 or Fig. 10, $\Omega = 0{\cdot}73$ for $\varepsilon = 10\%$. Therefore, the required value of the geosynthetic tension at a 10% strain is:

$$\alpha = 0{\cdot}73 \times 54\,395 = 39\,708 \text{ N/m} = 40 \text{ kN/m}$$

The same problem can be solved using the tables and charts with

$H/r = 0{\cdot}45/0{\cdot}75 = 0{\cdot}6$ and

$q/(\gamma r) = 88\,290/(19\,600 \times 0{\cdot}75) = 6{\cdot}0$

Table 3 or the chart given in Fig. 11 (see also Fig. 13) gives:

$\alpha/(\gamma r^2 \Omega) = 4{\cdot}963$ hence

$\alpha = (4{\cdot}963) \times 19\,600 \times (0{\cdot}75)^2 \times 0{\cdot}73 = 39\,943$ N/m $= 40$ kN/m

It is interesting to compare the required geosynthetic reinforcement tension calculated above to that required if the bedding soil is a layer of compacted clay associated with the geomembrane to form a composite liner. In this case, it is important that the integrity of the clay layer be maintained. Therefore, the geosynthetic reinforcement strain must be small enough to prevent the development of tension cracks in the clay layer. Calculations similar to the above, with $\varepsilon = 1\%$ instead of 10%, give a required geosynthetic reinforcement tension of 113 kN/m, which is about three times greater than 40 kN/m. Therefore, the geosynthetic reinforcement required in the case of a 1% allowable strain has a tension about three times greater, and consequently a modulus about 30 times greater, than in the case of a 10% allowable strain. (Several layers of a very high-modulus geotextile would probably be needed.)

Determination of Required Soil Layer Thickness

The relevant equation for an infinitely long void is

$$H = 2b \ln \frac{[q/(\gamma b)] - 2}{[\alpha/(\gamma b^2 \Omega)] - 2} \tag{18}$$

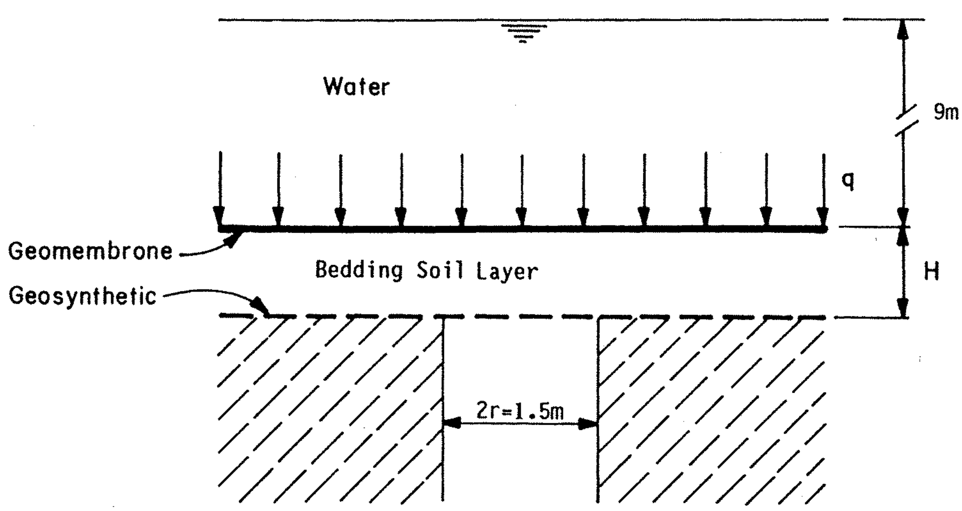


Fig. 12. Cross section for design examples.

TABLE 3
Pressure on the Geosynthetic

$q/(\gamma b)$ or $q/(\gamma r)$	*H/b or H/r*													
	0	*0·01*	*0·03*	*0·1*	*0·3*	*0·5*	*0·6*	*1·0*	*3·0*	*5·0*	*7·0*	*10·0*	*20·0*	∞
	(Values of $p/(\gamma/b) = \alpha/(\gamma b^2 \Omega)$ or $p/(\gamma r) = \alpha/(\gamma r^2 \Omega)$)													
0·0	0	0·010	0·030	0·098	0·279	0·442	0·518	0·787	1·554	1·836	1·940	1·987	2·000	2·000
0·01	0·010	0·020	0·040	0·107	0·287	0·450	0·526	0·793	1·556	1·837	1·940	1·987	2·000	2·000
0·03	0·030	0·040	0·059	0·126	0·304	0·466	0·541	0·805	1·560	1·838	1·941	1·987	2·000	2·000
0·05	0·050	0·060	0·079	0·145	0·322	0·481	0·555	0·817	1·565	1·840	1·941	1·987	2·000	2·000
0·1	0·100	0·109	0·128	0·193	0·365	0·520	0·592	0·848	1·576	1·844	1·943	1·987	2·000	2·000
0·2	0·200	0·209	0·227	0·288	0·451	0·598	0·667	0·908	1·598	1·852	1·946	1·988	2·000	2·000
0·3	0·300	0·308	0·325	0·383	0·537	0·676	0·741	0·969	1·621	1·860	1·949	1·989	2·000	2·000
0·5	0·500	0·507	0·522	0·573	0·709	0·832	0·889	1·090	1·665	1·877	1·955	1·990	2·000	2·000
0·7	0·700	0·706	0·719	0·763	0·881	0·988	1·037	1·212	1·710	1·893	1·961	1·991	2·000	2·000
1·0	1·000	1·005	1·015	1·049	1·139	1·221	1·259	1·393	1·777	1·918	1·970	1·993	2·000	2·000
1·5	1·500	1·502	1·507	1·524	1·570	1·611	1·630	1·697	1·888	1·959	1·985	1·997	2·000	2·000
2·0	2·000	2·000	2·000	2·000	2·000	2·000	2·000	2·000	2·000	2·000	2·000	2·000	2·000	2·000
2·5	2·500	2·498	2·493	2·476	2·430	2·389	2·370	2·303	2·112	2·041	2·015	2·003	2·000	2·000
3·0	3·000	2·995	2·985	2·951	2·861	2·779	2·741	2·607	2·223	2·082	2·030	2·007	2·000	2·000
4·0	4·000	3·990	3·970	3·902	3·721	3·558	3·482	3·213	2·446	2·164	2·060	2·013	2·000	2·000
5·0	5·000	4·985	4·955	4·854	4·582	4·336	4·222	3·820	2·669	2·246	2·091	2·020	2·000	2·000

6·0	6·000	5·980	5·940	5·805	5·443	5·115	4·963	4·426	2·893	2·328	2·121	2·027	2·000	2·000
7·0	7·000	6·975	6·926	6·756	6·304	5·894	5·704	5·033	3·116	2·410	2·151	2·034	2·000	2·000
8·0	8·000	7·970	7·911	7·707	7·164	6·673	6·445	5·639	3·339	2·493	2·181	2·040	2·000	2·000
9·0	9·000	8·965	8·896	8·659	8·025	7·452	7·186	6·246	3·562	2·575	2·211	2·047	2·000	2·000
10	10·000	9·960	9·881	9·610	8·886	8·230	7·927	6·852	3·785	2·657	2·242	2·054	2·000	2·000
15	15·000	14·935	14·806	14·366	13·189	12·124	11·631	9·885	4·901	3·067	2·393	2·088	2·000	2·000
20	20·000	19·910	19·732	19·122	17·493	16·018	15·335	12·918	6·016	3·478	2·544	2·121	2·000	2·000
25	25·000	24·885	24·658	23·878	21·796	19·912	19·039	15·950	7·132	3·888	2·695	2·155	2·000	2·000
30	30·000	29·860	29·583	28·634	26·100	23·806	22·743	18·983	8·248	4·298	2·846	2·189	2·000	2·000
40	40·000	39·810	38·434	38·147	34·707	31·594	30·151	25·048	10·479	5·119	3·148	2·256	2·000	2·000
50	50·000	49·761	49·285	47·659	43·314	39·382	37·559	31·113	12·710	5·940	3·449	2·323	2·000	2·000
60	60·000	59·711	59·136	57·171	51·921	47·170	44·967	37·179	14·942	6·761	3·751	2·391	2·000	2·000
70	70·000	69·661	68·988	66·684	60·528	54·958	52·376	43·244	17·173	7·582	4·053	2·458	2·000	2·000
80	80·000	79·611	78·839	76·196	69·135	62·746	59·784	49·309	19·404	8·403	4·355	2·526	2·000	2·000
90	90·000	89·561	88·690	85·708	77·742	70·534	67·192	55·375	21·635	9·223	4·657	2·593	2·000	2·000
100	100·000	99·511	98·541	95·220	86·349	78·322	74·600	61·440	23·867	10·044	4·959	2·660	2·000	2·000

This table gives $p/(\gamma b)$ or $p/(\gamma r)$ and the geosynthetic tension as a function of the other parameters involved. Notation: p = pressure on the geosynthetic over the void area; q = uniformly distributed normal stress applied on the top of the soil layer; H = thickness of the soil layer; γ = unit weight of the soil in the soil layer; b = width of the infinitely long void; r = radius of the circular void; α = geosynthetic tension; and Ω = dimensionless factor given in Table 2 as a function of the geosynthetic strain, ε. Note that: values of $p/(\gamma b)$ or $p/(\gamma r)$ for $H/b = 0$ are identical to values of $q/(\gamma b)$ or $q/(\gamma r)$; and $p = 2\gamma b$ if H is greater than approximately $20b$ and $p = 2\gamma r$ if H is greater than approximately $20r$. (See the chart given in Fig. 11.)

The same equation can be used for a circular void by substituting r for b.

The above equations can be used to solve problems that consist of determining the required soil layer thickness, H, when all other parameters are given (b or r, q, γ, α, and ε). Alternatively, the charts given in Fig. 11, and the corresponding Table 3, can be used.

Example 2. This example is identical to Example 1, except that the soil layer thickness, H, is unknown, and the geosynthetic tension at a strain $\varepsilon = 10\%$ is known and is equal to 40 kN/m. What is the required soil layer thickness?

From Example 1, the relevant parameters are: $q = 88\,290$ N/m^2; $\gamma = 19\,600$ N/m^3; and $r = 0{\cdot}75$ m.

In order to use eqn (18), the following values must be calculated

$$q/(\gamma r) = 6{\cdot}0 \text{ (from Example 1)}$$
$$\alpha/(\gamma r^2 \Omega) = 40\,000/(19\,600 \times (0{\cdot}75)^2 \times 0{\cdot}73) = 4{\cdot}97$$

Hence, using eqn 18

$$H = 2 \times 0{\cdot}75 \times \ln \frac{6{\cdot}0 - 2}{4{\cdot}97 - 2} = 0{\cdot}44 \text{ m}$$

It is also possible to solve this problem using Table 3 or Fig. 11 which gives $H/r = 0{\cdot}6$ for $q/(\gamma r) = 6{\cdot}0$ and $\alpha/(\gamma r^2 \Omega) = 4{\cdot}97$ (see Fig. 13). Hence, $H = 0{\cdot}6 \times 0{\cdot}75 = 0{\cdot}45$ m.

Determination of Maximum Void Size

There is no simple equation giving the void size (b or r) as a function of the other parameters. In order to determine the maximum void size that a given soil layer-geosynthetic system can bridge, it is necessary to solve eqn (16) by trial and error. To facilitate the process, a chart has been established (Fig. 14) by rewriting the two parts of eqn (17) in a dimensionless form as follows:

$$\frac{p}{\gamma H} = \frac{2(1 - e^{-0{\cdot}5H/b})}{H/b} + \frac{q}{\gamma H} e^{-0{\cdot}5H/b} \tag{19}$$

$$\frac{p}{\gamma H} = \frac{\alpha}{\gamma H^2 \Omega} \frac{H}{b} \tag{20}$$

In Fig. 14, eqn (19) is represented by a family of curves and eqn (20) is represented by a family of straight lines at 45°. For a given set of parameters, the abscissa of the intersection between the relevant curve

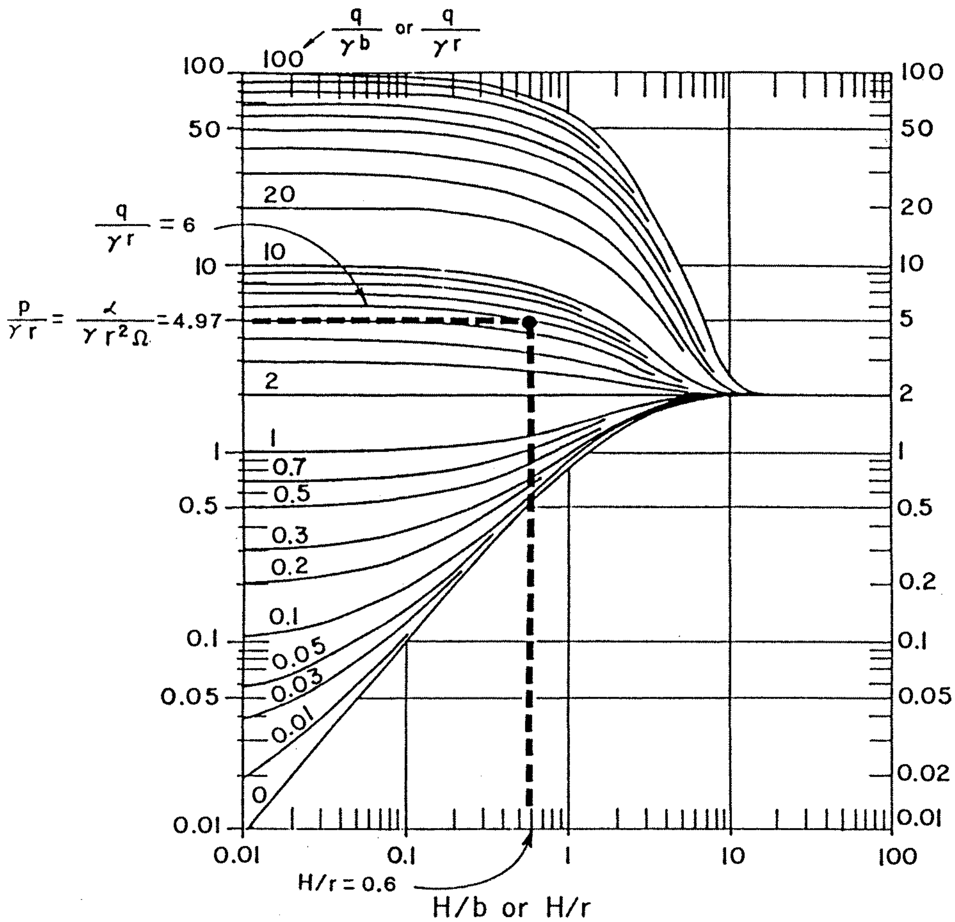


Fig. 13. Example of use of the chart given in Fig. 11.

and the relevant straight line gives the maximum value of the width, b, of an infinitely long void or the radius, r, of a circular void.

Example 3. This example is identical to Example 1, except that the radius of the void, r, is unknown, and the geosynthetic tension at a strain $\varepsilon = 10\%$ is known and is equal to 40 kN/m. What maximum void radius can be bridged by the considered soil-geosynthetic system?
From Example 1, the relevant parameters are: $q = 88\,290\,\mathrm{N/m^2}$; $\gamma = 19\,600\,\mathrm{N/m^3}$; and $H = 0{\cdot}45$ m.

In order to use the chart given in Fig. 14, the following must be calculated

$$q/(\gamma H) = 88\,290/(19\,600 \times 0{\cdot}45) = 10{\cdot}0$$

$$\alpha/(\gamma H^2 \Omega) = 40\,000/(19\,600 \times (0{\cdot}45)^2 \times 0{\cdot}73) = 13{\cdot}8$$

(Note: $\Omega = 0{\cdot}73$ is obtained from Table 2 with $\varepsilon = 10\%$)

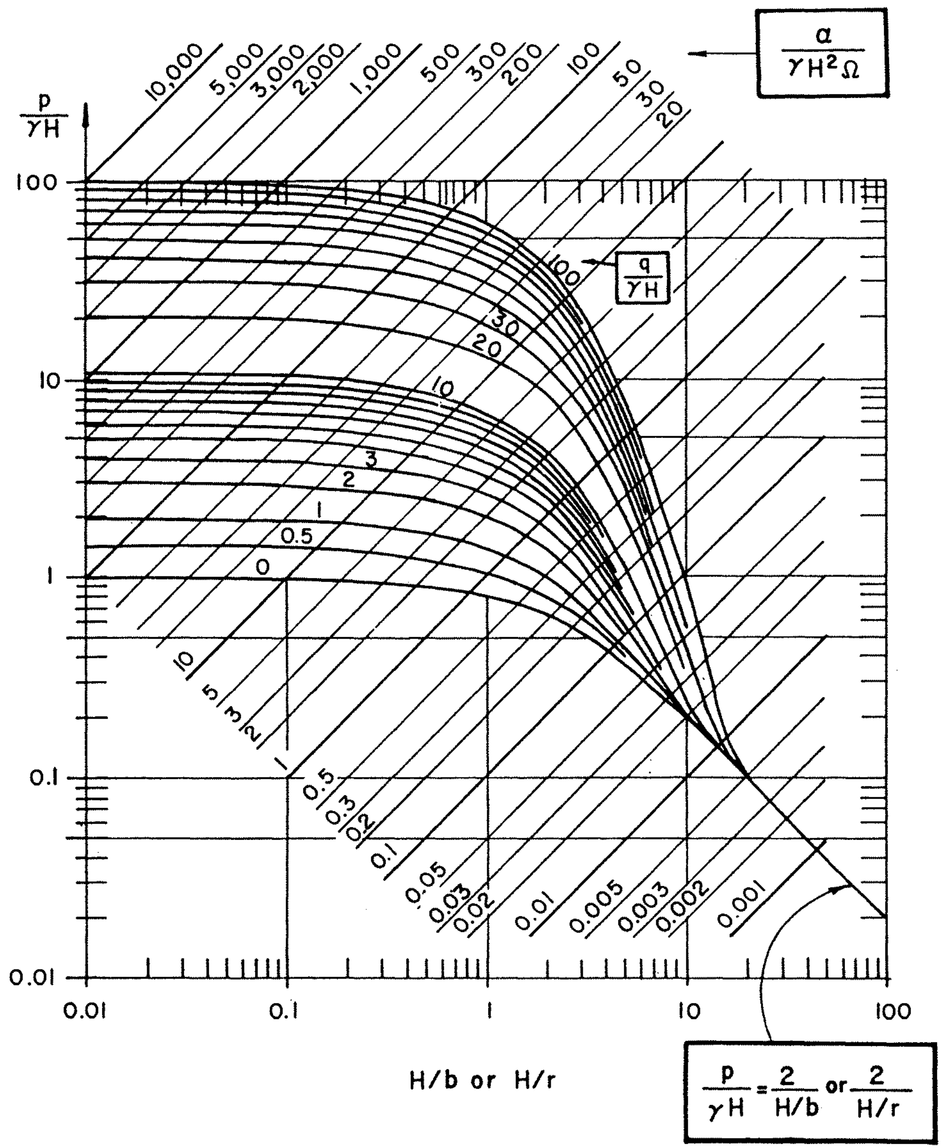


Fig. 14. Pressure on and tension in the geosynthetic. An example of use of this chart is given in Fig. 15. Notations: p = pressure on the geosynthetic over the void area; q = uniformly distributed normal stress applied on the top of the soil layer; H = thickness of the soil layer; γ = unit weight of soil; b = width of the infinitely long void; r = radius of the circular void; α = geosynthetic tension; and Ω = dimensionless factor given in Table 2 and Fig. 10. (Values of $p/(\gamma H)$ which were used to draw the curves in this figure can be found in Table 4.)

In Fig. 14, the curve related to $q/(\gamma H) = 10$ and the straight line at 45° related to $\alpha/(\gamma H^2\Omega) = 13{\cdot}8$ intersect at a point the abscissa of which is $H/r = 0{\cdot}6$ (see Fig. 15). Hence

$$r_{max} = 0{\cdot}45/0{\cdot}6 = 0{\cdot}75 \text{ m}$$

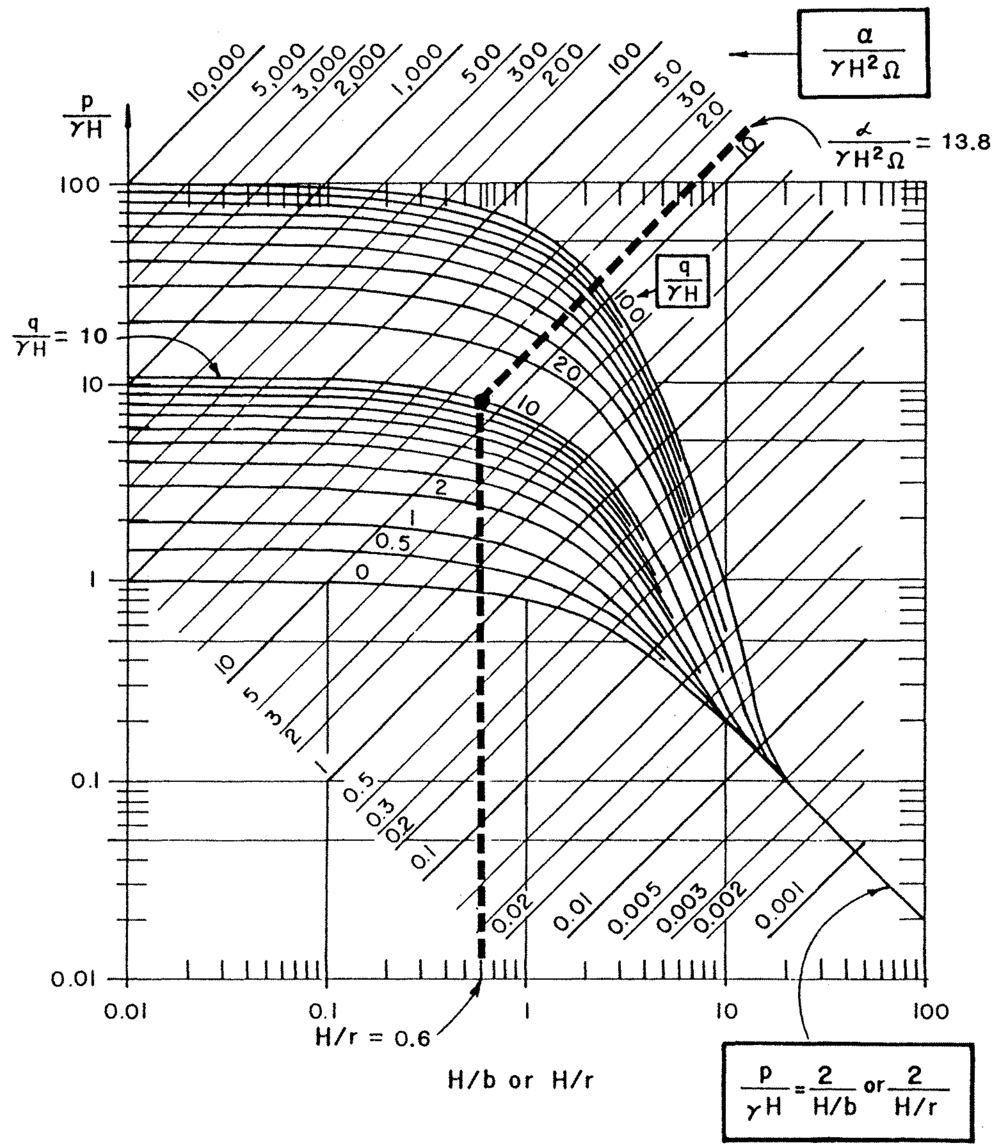


Fig. 15. Example of use of the chart given in Fig. 14.

Determination of the Maximum Load

The relevant equation for an infinitely long void is

$$q = 2\gamma b + \left\{ \frac{[\alpha/(\gamma b^2 \Omega)] - 2}{e^{-0{\cdot}5H/b}} \right\} \gamma b \qquad (21)$$

The same equation can be used for a circular void by substituting r for b.

The above equation can be used to solve problems that consist of determining the maximum uniform normal stress, q, which can be applied on the top of the soil layer, when all other parameters are given (b or r, γ, H, α, and ε). Alternatively, the charts given in Fig. 11 or 14 can be used, as well as Table 3 or 4.

TABLE 4
Pressure on the Geosynthetic

$q/(\gamma H)$	*H/b or H/r*												
	0	*0·01*	*0·1*	*0·3*	*0·5*	*0·7*	*1·0*	*3·0*	*5·0*	*7·0*	*10·0*	*20·0*	∞
	(Values of $p/(\gamma H)$)												
0·0	∞	0·998	0·975	0·929	0·885	0·844	0·787	0·518	0·367	0·277	0·199	0·100	0
0·5	∞	1·495	1·451	1·359	1·274	1·196	1·090	0·629	0·408	0·292	0·202	0·100	0
1·0	∞	1·993	1·927	1·789	1·664	1·548	1·393	0·741	0·449	0·307	0·205	0·100	0
2·0	∞	2·988	2·878	2·650	2·442	2·253	2·000	0·964	0·531	0·337	0·212	0·100	0
3·0	∞	3·983	3·829	3·511	3·221	2·958	2·607	1·187	0·613	0·368	0·219	0·100	0
4·0	∞	4·978	4·780	4·371	4·000	3·663	3·213	1·410	0·696	0·398	0·226	0·100	0
5·0	∞	5·973	5·732	5·232	4·779	4·367	3·820	1·634	0·778	0·428	0·232	0·100	0
6·0	∞	6·968	6·683	6·093	5·558	5·072	4·426	1·857	0·860	0·458	0·239	0·100	0
7·0	∞	7·963	7·634	6·954	6·336	5·777	5·033	2·080	0·942	0·488	0·246	0·100	0
8·0	∞	8·958	8·585	7·814	7·115	6·481	5·639	2·303	1·024	0·519	0·253	0·100	0
9·0	∞	9·953	9·536	8·675	7·894	7·186	6·246	2·526	1·106	0·549	0·259	0·100	0
10	∞	10·948	10·488	9·536	8·673	7·891	6·852	2·749	1·188	0·579	0·266	0·100	0
15	∞	15·923	15·244	13·839	12·567	11·414	9·885	3·865	1·598	0·730	0·300	0·101	0
20	∞	20·898	20·000	18·143	16·461	14·938	12·918	4·981	2·009	0·881	0·333	0·101	0
25	∞	25·873	24·756	22·446	20·355	18·461	15·950	6·096	2·419	1·032	0·367	0·101	0
30	∞	30·848	29·512	26·750	24·249	21·984	18·983	7·212	2·830	1·183	0·401	0·101	0
40	∞	40·798	39·025	35·357	32·037	29·031	15·048	9·443	3·651	1·485	0·468	0·102	0
50	∞	50·748	48·537	43·964	39·825	36·078	31·113	11·674	4·471	1·787	0·536	0·102	0
60	∞	60·698	58·049	52·571	47·613	43·125	37·179	13·906	5·292	2·089	0·603	0·103	0
70	∞	70·648	67·561	61·178	55·401	50·172	43·244	16·137	6·113	2·391	0·670	0·103	0
80	∞	80·599	77·074	69·786	63·189	57·219	49·309	18·368	6·934	2·693	0·738	0·104	0
90	∞	90·549	86·586	78·392	70·977	64·266	55·375	20·600	7·755	2·995	0·805	0·104	0
100	∞	100·499	96·098	86·999	78·765	71·313	61·440	22·831	8·576	3·297	0·872	0·105	0

This table gives $p/(\gamma H)$. Notation: p = pressure on the geosynthetic over the void area; q = uniformly distributed normal stress applied on the top of the soil layer; H = thickness of the soil layer; γ = unit weight of the soil in the soil layer; b = width of the infinitely long void; and r = radius of the circular void. Note that: values of $p/(\gamma H)$ are equal to: $1 + q/(\gamma H)$ if $H/b = 0$ or $H/r = 0$; and $2b/H$ or $2r/H$ if $H/b > 20$ or $H/r > 20$. (See the chart given in Fig. 14.)

Example 4. This example is identical to Example 1, except that the stress on top of the soil layer, q, is unknown, and the geosynthetic tension at strain $\varepsilon = 10\%$ is known and is equal to 40 kN/m. What maximum stress on top of the soil layer can be supported by the soil-geosynthetic system? From Example 1, the relevant parameters are: $H = 0{\cdot}45$ m; $r = 0{\cdot}75$ m; and $\gamma = 19\,600$ N/m^3.
In order to use eqn (21), the value of Ω must be obtained first from Table 2

$$\Omega = 0{\cdot}73 \quad \text{for } \varepsilon = 10\%.$$

Then, eqn (21) is used as follows

$$q = 2 \times 19\,600 \times 0{\cdot}75 + \left\{ \frac{[40\,000/(19\,600 \times (0{\cdot}75)^2 \times 0{\cdot}73)] - 2}{\mathrm{e}^{-0{\cdot}5 \times 0{\cdot}45/0{\cdot}75}} \right\} 19\,600 \times 0{\cdot}75$$

$$= 88\,334 \text{ N/m}^2$$

The problem can also be solved using charts and tables. To use Table 3 or the chart given in Fig. 11, the following must be calculated:

$$H/r = 0{\cdot}45/0{\cdot}75 = 0{\cdot}6$$

$$\alpha/(\gamma r^2 \Omega) = 40\,000/(19\,600 \times (0{\cdot}75)^2 \times 0{\cdot}73) = 4{\cdot}97$$

With $H/r = 0{\cdot}6$ and $\alpha/(\gamma r^2 \Omega) = 4{\cdot}97$, Table 3 or the chart given in Fig. 11 show that $q/(\gamma r) = 6$ (see Fig. 13). Therefore

$$q = 6 \times 19\,600 \times 0{\cdot}75 = 88\,200 \text{ N/m}^2 = 88 \text{ kN/m}^2$$

To use the chart given in Fig. 14, the following must be calculated

$$\alpha/(\gamma H^2 \Omega) = 40\,000/(19\,600 \times (0{\cdot}45)^2 \times 0{\cdot}73) = 13{\cdot}8$$

With $H/r = 0{\cdot}6$ and $\alpha/(\gamma H^2 \Omega) = 13{\cdot}8$, the chart given in Fig. 14 shows that $q/(\gamma H) = 10$ (see Fig. 15). Therefore

$$q = 10 \times 19\,600 \times 0{\cdot}45 = 88\,200 \text{ N/m}^2 = 88 \text{ kN/m}^2$$

DISCUSSION OF SPECIAL PROBLEMS

Anisotropic Geosynthetic

A geosynthetic is isotropic regarding a given characteristic when this characteristic has the same value in all directions. In this paper, a geosynthetic will be considered isotropic when it has the same tension–strain curve in all directions. This requirement is fulfilled by some nonwoven geotextiles. Woven geotextiles and biaxial geogrids are stronger in two directions ('principal directions') than in the others and, therefore, they are anisotropic. However, we assume that the design method presented in this paper can be used with woven geotextiles and biaxial geogrids that have the same tensile characteristics in the two principal directions (i.e. in the design, these materials are considered isotropic).

Special precautions must be taken when using the design method presented in this paper for geosynthetics that cannot be considered isotropic, as discussed below.

Infinitely Long Void

In the case of an infinitely long void, no geosynthetic tension is required in the direction of the length of the void (according to the plane-strain model which corresponds to an infinitely long void). Therefore, the value of α to be used in the equations, tables, and charts related to the infinitely long void is the geosynthetic tension in the direction of the width of the void for the considered design strain. However, some strength is required lengthwise in places where the actual situation departs from a pure plane-strain situation (for instance near the end of the void).

Circular Void

In the case of a circular void, the tensioned membrane equation (eqn (15)) is valid only if the geosynthetic has isotropic tensile characteristics. For practical purposes, eqn (15), and other equations as well as tables and charts related to circular voids, can be used for woven geotextiles and biaxial geogrids that have the same tension–strain curve *in the two principal directions* (instead of *in all directions* for a truly isotropic material). For woven geotextiles and biaxial geogrids that have different tensile characteristics in the two principal directions, two cases can be considered, depending on the ratio between the geosynthetic tensions at the design strain in the weak and the strong directions: (i) if the ratio is more than 0·5, α should be taken equal to the tension in the weak direction; and (ii) if the ratio is less than 0·5, α should be taken equal to half the tension in the strong direction.

The rationale for the above recommendation is as follows. There are two conservative approaches and the less conservative, which is closer to reality, should be selected.

The first conservative approach consists of designing with an isotropic geosynthetic weaker than the considered anisotropic geosynthetic. This is achieved by taking the geosynthetic strength in all directions equal to the strength in the weak direction, α_{weak}. Equation (15) thus gives for the pressure which can be carried by the geosynthetic

$$p_1 = \frac{\alpha_{weak}}{r\Omega} \tag{22}$$

The second conservative approach consists of designing with: (i) a void larger than the circular void by replacing the circular void by an infinitely long void with a width, b, equal to the diameter, $2\,r$, of the circular void; and (ii) a geosynthetic weaker than the considered anisotropic geosynthetic by neglecting the tensile strength in the weak direction ($\alpha_{weak} = 0$). Equation (15) thus gives for the pressure which can be carried by the geosynthetic

$$p_2 = \frac{\alpha_{strong}}{b\Omega} = \frac{\alpha_{strong}}{2r\Omega} \tag{23}$$

To compare p_1 and p_2, it is important to note that the values of Ω in eqns (22) and (23) are identical because they are both determined for $y/(2r)$, according to Table 2. Therefore, the comparison between p_1 and p_2 boils down to a comparison between α_{weak} and $0{\cdot}5\ \alpha_{strong}$.

It appears that

$$p_1 > p_2 \quad \text{if} \quad \alpha_{weak} > 0{\cdot}5\alpha_{strong}$$

$$p_1 < p_2 \quad \text{if} \quad \alpha_{weak} < 0{\cdot}5\alpha_{strong}$$

hence the above recommendation.

There is another consideration when an anisotropic geosynthetic is used over a circular void. The complex pattern of strains in the geosynthetic resulting from different tensions in different directions may have a detrimental effect on the behavior of the geosynthetic. Therefore, it is recommended that for holes which can be modeled as circular, one of the following solutions be adopted: (i) an isotropic geosynthetic (only some nonwoven geotextiles are isotropic but usually they do not have adequate tensile characteristics for this application); or (ii) a 'practically isotropic' geosynthetic (such as a woven geotextile or a biaxial geogrid having similar tension–strain curves in the two principal directions); or (iii) two perpendicularly orientated layers of the same anisotropic geosynthetic.

Geosynthetic in Contact with Void Bottom

In some cases, the geosynthetic elongates to the point that it comes in contact with the bottom of the void (Fig. 1(c)); the geosynthetic deflection is then equal to the void depth ($y = D$). In design, these cases correspond to a calculated geosynthetic deflection greater than or equal to the void depth ($y \geqq D$). Usually, the design is complete when it is found that $y \geqq D$. However, it may be of interest to *determine the pressure actually transmitted to the bottom of the void.* This pressure is obtained by subtracting the pressure inducing geosynthetic tension (which results from the tensioned membrane effect) from the pressure exerted by the soil layer on the geosynthetic.

In the case of an infinitely long void, the following equation can be obtained by subtracting the pressure given by eqn (15) from the pressure given by eqn (10)

$$p_b = 2\gamma b(1 - e^{-0 \cdot 5H/b}) + q\, e^{-0 \cdot 5H/b} - \frac{\alpha}{b\Omega} \tag{24}$$

where: p_b = pressure transmitted to the bottom of the void; γ = unit weight of the soil (in the soil layer above the geosynthetic); b = width of the infinitely long void; H = soil layer thickness; q = uniformly distributed normal stress applied on the top of the soil layer; α = geosynthetic tension corresponding to the geosynthetic strain, ε, when the geosynthetic is in contact with the bottom of the void (i.e., ε corresponding to a deflection $y = D$ in Table 2); Ω = dimensionless factor given in Table 2 as a function of ε or y; and y = geosynthetic deflection, which, in this case, is equal to D; and D = depth of the void. Basic SI units are: p_b (N/m^2), γ (N/m^3), b (m), H (m), q (N/m^2), α (N/m), y (m), and D (m); Ω is dimensionless. Note that eqn (24) assumes that the shape of the bottom of the void is approximately cylindrical with a circular cross section, so the geosynthetic will come in contact with all points on the surface of the void at the same time. If this were not the case, portions of the geosynthetic which come in contact with the bottom of the void last would elongate more than the others.

The same equation can be used for a circular void by substituting r for b, with r = radius of the circular void.

If a negative value were obtained for p_b when using the above equation, it would mean that the load on the geosynthetic is not large enough to force the geosynthetic to come in contact with the bottom of the void.

Example 5. This example is identical to Example 1 except that: (i) the void is not bottomless but has a depth $D = 0 \cdot 2$ m; and (ii) the geosynthetic tension–strain curve is assumed to be a straight line between the origin and a tension $\alpha = 40$ kN/m for a strain $\varepsilon = 10\%$. What is the stress transmitted to the bottom of the hole?

From Example 1, the relevant parameters are: $r = 0{\cdot}75$ m; $H = 0{\cdot}45$ m; $\gamma = 19\,600\,\mathrm{N/m^3}$; and $q = 88\,290\,\mathrm{N/m^2}$.

First, the approximate value of the average strain of the geosynthetic when it is in contact with the bottom of the void (assumed spherical) must be determined using Table 2 with y (geosynthetic deflection) $= D$ (void depth)

$$y/2r = D/2r = 0{\cdot}2/(2 \times 0{\cdot}75) = 0{\cdot}133$$

Hence, interpolating in Table 2, $\varepsilon = 4{\cdot}65\%$ and $\Omega = 1{\cdot}01$.

Then, the geosynthetic tension corresponding to a 4·65% geosynthetic strain can be calculated as follows:

$$\alpha = 40\,000 \times 4{\cdot}65/10 = 18\,600\ \mathrm{N/m}$$

Finally, eqn (24) can be used with the values $H/r = 0{\cdot}6$ and $q = 88\,290\,\mathrm{N/m^2}$ determined in Example 1. This equation gives the stress transmitted to the bottom of the void as follows

$$p_{\mathrm{b}} = 2 \times 19\,600 \times 0{\cdot}75(1 - \mathrm{e}^{-0{\cdot}3}) + 88\,290\,\mathrm{e}^{-0{\cdot}3} - \frac{18\,600}{0{\cdot}75 \times 1{\cdot}01}$$

$$= 73\,029 - 24\,554 = 48\,475\ \mathrm{N/m^2} = 48{\cdot}5\ \mathrm{kN/m^2}$$

Therefore, this design example can be summarized as follows:

- A stress of $88{\cdot}3\,\mathrm{kN/m^2}$ is applied on top of the soil layer.
- As a result of soil arching, the soil layer transmits only a stress of $73\,\mathrm{kN/m^2}$ to the top of the geosynthetic.
- As a result of the tensioned membrane effect, the geosynthetic supports $24{\cdot}5\,\mathrm{kN/m^2}$.
- The remainder, $48{\cdot}5\,\mathrm{kN/m^2}$, is transmitted to the bottom of the void.

It should be noted that, if the depth of the void had been $D = 0{\cdot}3$ m, the strain of the geosynthetic would have been 10% and the last term of the above equation would have been

$$\frac{\alpha}{r\Omega} = \frac{40\,000}{0{\cdot}75 \times 0{\cdot}73} = 73\,059\ \mathrm{N/m^2}$$

Hence, $p_{\mathrm{b}} = 0$. In this case, Example 5 becomes identical to Example 1.

Influence of Soil Layer Thickness

The influence of the thickness of the soil layer is illustrated in Fig. 11. Three cases can be considered:

(1) *Large Applied Stress*. If the applied stress, q, is large (i.e. $q > 2\gamma b$ or $2\gamma r$), the pressure, p, on the geosynthetic and consequently the required geosynthetic tension, α, decrease towards a limit when the soil layer thickness increases. In this case, *it is beneficial to increase the thickness of the soil layer.* For each particular situation, the amount by which the thickness should be increased can be determined using the chart given in Fig. 11 or Table 3. The chart and table show that it would be useless to increase the soil layer thickness beyond a limiting value of $H = 20b$ or $20r$.

(2) *Small Applied Stress*. If the applied stress, q, is small (i.e. $q < 2\gamma b$ or $2\gamma r$), the pressure, p, on the geosynthetic and consequently the required geosynthetic tension, α, increase toward a limit when the soil thickness increases. In this case, from the perspective of the design of the geosynthetic, *it is detrimental to increase the thickness of the soil layer.* (This is because the added load due to soil weight is not fully compensated by the effect of soil arching.)

(3) *Limit Applied Stress*. If the applied stress, q, equals the limit (i.e. $q = 2\gamma b$ or $2\gamma r$), the pressure, p, on the geosynthetic remains constant and equal to q, regardless of the soil layer thickness.

The limit values for p and α are independent of the applied stress, q. The limit value for the pressure on the geosynthetic is

$$p_{\text{lim}} = 2\gamma b \quad \text{for an infinitely long void} \tag{25}$$

The limit value for the required geosynthetic tension is

$$\alpha_{\text{lim}} = 2\gamma b^2 \Omega \quad \text{for an infinitely long void} \tag{26}$$

Equations (25) and (26) can be used for a circular void by substituting r for b.

Comparison with Tensioned Membrane Theory

In the past, the tensioned membrane theory has been used alone to evaluate the required tensile characteristics of a geosynthetic located beneath a soil layer and bridging a void. This method neglects arching in the soil layer and is, therefore, conservative. This conservativeness can be evaluated by comparing the pressure on the geosynthetic over the void area, p, calculated taking soil arching into account to the following value obtained by neglecting soil arching

$$p_0 = \gamma H + q \tag{27}$$

where: p_0 = pressure on the geosynthetic over the void area neglecting soil arching; γ = unit weight of the soil in the soil layer; H = thickness of the soil layer; and q = uniformly distributed normal stress applied on the top of the soil layer. Basic SI units are: p_0 (N/m^2), γ (N/m^3), H (m), and q (N/m^2).

The pressure, p, obtained taking soil arching into account is given by eqn (10).

Values of p/p_0 are given in Table 5 and Fig. 16. It appears that neglecting soil arching is conservative. However, when the soil thickness, H, is large

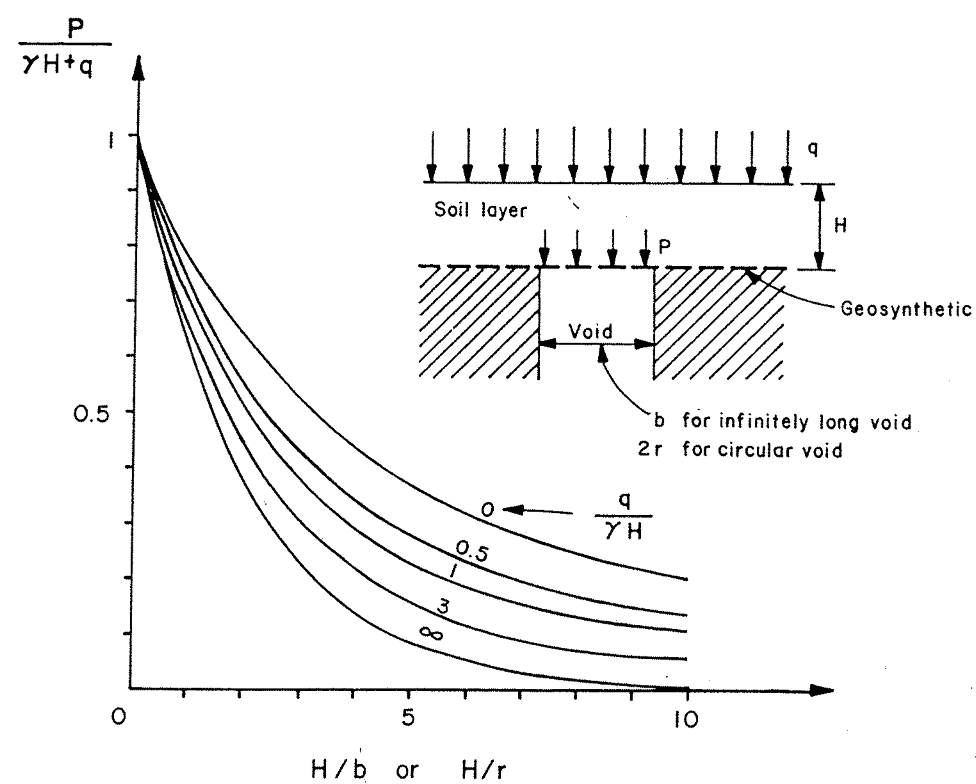


Fig. 16. Effectiveness of soil arching. The curves give the ratio between the pressure, p, on the geosynthetic over the void area, calculated taking soil arching into account, and the pressure $p_0 = \gamma H + q$ obtained by neglecting soil arching. The values of p/p_0 used to plot the curve can be found in Table 5.

TABLE 5
Effectiveness of Soil Arching

$q/(\gamma H)$	*H/b or H/r*												
	0	0·3	0·6	1	1·5	2	2·3	3	4	5	10	20	∞
	(Values of p/p_0)												
0	1	0·929	0·864	0·787	0·704	0·632	0·571	0·518	0·432	0·367	0·199	0·100	0
0·5	1	0·906	0·823	0·727	0·626	0·544	0·476	0·420	0·333	0·272	0·135	0·067	0
1	1	0·895	0·802	0·697	0·588	0·500	0·429	0·371	0·284	0·225	0·103	0·050	0
2	1	0·883	0·782	0·667	0·549	0·456	0·381	0·321	0·234	0·177	0·071	0·033	0
3	1	0·878	0·772	0·652	0·530	0·434	0·358	0·297	0·210	0·153	0·055	0·025	0
5	1	0·872	0·761	0·637	0·511	0·412	0·334	0·272	0·185	0·130	0·039	0·017	0
10	1	0·867	0·752	0·623	0·493	0·392	0·312	0·250	0·162	0·108	0·024	0·009	0
20	1	0·864	0·747	0·615	0·483	0·380	0·300	0·237	0·149	0·096	0·016	0·005	0
∞	1	0·861	0·741	0·607	0·472	0·368	0·287	0·223	0·135	0·082	0·007	0·000	0

This table gives the ratio of the pressure on the geosynthetic over the void area calculated taking arching into account (p) or neglecting arching (p_0). The value of p is given by eqn (10). The value of p_0 is given by eqn (27). Notation: q = uniformly distributed stress applied on the top of the soil layer; γ = unit weight of the soil in the soil layer; H = soil layer thickness; b = width of an infinitely long void; and r = radius of circular void. (See also Fig. 16.)

compared to the width or radius of the void, neglecting soil arching is over-conservative.

CONCLUSION

This paper has presented an approach to the design of soil layer-geosynthetic systems overlying voids. The design approach superimposes arching theory for the soil layer with tensioned membrane theory for the geosynthetic. The analysis presented in this paper shows that neglecting soil arching would be over-conservative in many instances. The paper presents equations, tables, and charts that make it easy to perform design analyses for a range of possible field situations.

The analysis shows that the thickness of the soil layer associated with the geosynthetic plays a significant role. In contrast, the soil mechanical properties do not. It should not be inferred, however, that any soil will provide the same degree of arching. The equations used to prepare the tables and charts assume that the friction angle of the soil is at least 20°. Granular soils virtually always meet this condition. However, they should be well compacted to ensure arching because loose granular soils tend to contract when they are sheared or vibrated, which may destroy the arch.

Further refinements of the method presented herein can be considered. For instance, it is possible that the degree of soil arching (i.e. the amount of soil shear strength mobilized) depends on the geosynthetic strain, whereas the method presented in this paper does not consider the concept of degree of soil arching. Also, the method could be expanded to include cohesive soils, and could be refined to take into account elongation of the geosynthetic in the anchorage zone. Lastly, the method could be expanded to consider a system of regularly spaced voids.

In spite of its limitations, the method presented in this paper is believed to be a useful tool for engineers designing soil-geosynthetic systems resting on subgrades where voids may develop.

ACKNOWLEDGMENTS

This paper is an expanded version of the paper presented at the Kyushu International Geotechnical Symposium on Theory and Practice of Earth Reinforcement.[13] The authors are grateful to Professors Yamanouchi and Miura, organizers of the Kyushu Symposium, for authorizing the publication of this expanded version.

The authors are indebted to G. Saunders, A. Mozzar, G. Kent, S. Berdy, and A. H. Perry for assistance during the preparation of this paper

REFERENCES

1. Loudière, D. & Perrin, J., Désordres dans les revêtements minces de bassin. In *Colloque sur l'étanchéité superficielle des bassins, barrages, et canaux*, Vol. 1, Paris, Feb. 1983. CEMAGREF, Antony, France, pp. 147–52.
2. Giroud, J. P., & Goldstein, J. S., Geomembrane liner design. *Waste Age* (Sept. 1982) 27–30.
3. Giroud, J. P., *Geotextiles and Geomembranes, Definitions, Properties and Design*. I.F.A.I. Publisher, St Paul, MN, 1984, 325 pp.
4. Bonaparte, R. & Berg, R. R., The use of geosynthetics to support roadways over sinkhole prone areas. In *Proceedings of the Second Multidisciplinary Conference on Sinkholes and the Environmental Impacts of Karst*, Orlando, FL, Feb. 1987. Balkema, Rotterdam, pp. 437–45.
5. Giroud, J. P. & Fluet, J. E., Jr, Quality assurance of geosynthetic lining systems. *Geotextiles and Geomembranes*, **3** (1986) 249–87.
6. Tisserand, C., Le réservoir de Guazza: un grand bassin à membrane non protégée. *Colloque sur l'étanchéité superficielle des bassins, barrages et canaux*, Vol. 1, Paris, Feb. 1983. CEMAGREF, Antony, France, pp. 87–9.
7. Giroud, J. P., Designing with geotextiles. *Matériaux et Constructions*, **14** (1981), 257–72.
8. Giroud, J. P., Design of geotextiles associated with geomembranes. In *Proceedings of the Second International Conference on Geotextiles*, Vol. 1, Las Vegas, USA, Aug. 1982, pp. 37–42.
9. Terzaghi, K., *Theoretical Soil Mechanics*. John Wiley and Sons, Inc., New York, 1943, 510 pp.
10. Kezdi, A., Lateral earth pressure. In *Foundation Engineering Handbook*, ed. H. F. Winterkorn & H. Y. Fang. Van Nostrand Reinhold, New York, 1975, pp. 197–220.
11. Handy, R. L., The arch in soil arching. *Journal of Geotechnical Engineering*, ASCE, **111** (1985) 302–18.
12. Jaky, J., The coefficient of earth pressure at rest. *Journal for Society of Hungarian Architects and Engineers*, Budapest, Oct. 1944, pp. 355–8 (in Hungarian).
13. Giroud, J. P., Bonaparte, R., Beech, J. F. & Gross, B. A., Load carrying capacity of a soil layer supported by a geosynthetic located on a void. In *Proceedings of the International Geotechnical Symposium on Theory and Practice of Earth Reinforcement*, Fukuoka/Kyushu, Japan, Oct. 1988. Balkema, Rotterdam, pp. 185–90.

CBR Determination

Client:	Chargrin Valley Engineering
Project:	MOT-4-19.30
Task:	Correlated CBR from DCP Results

Calculated: DMV
Checked: DCM

2016 Advanced Materials DCP Results

ID	Blows	in/blows	mm / blow	CBR	CBR_{Low}
1	3.2	1.2303	31.2499	6.182	
	7	0.5624	14.2857	14.856	
	7	0.5624	14.2857	14.856	
	2	1.9685	49.9999	3.652	
	7	0.5624	14.2857	14.856	
	22	0.1790	4.5454	53.567	
	31	0.1270	3.2258	78.652	4
2	6	0.6562	16.6666	12.500	
	10	0.3937	10.0000	22.151	
	7	0.5624	14.2857	14.856	
	5	0.7874	20.0000	10.191	
	8	0.4921	12.5000	17.252	
	19	0.2072	5.2631	45.456	
	26	0.1514	3.8461	64.588	10
3	17	0.2316	5.8823	40.132	
	34	0.1158	2.9412	87.225	
	55	0.0716	1.8182	149.482	
	89	0.0442	1.1236	256.272	
	77	0.0511	1.2987	217.898	
	59	0.0667	1.6949	161.710	40
4	16	0.2461	6.2500	37.497	
	21	0.1875	4.7619	50.847	
	38	0.1036	2.6316	98.796	
	78	0.0505	1.2820	221.070	
	28	0.1406	3.5714	70.178	
	92	0.0428	1.0870	265.966	37

ID	Blows	in/blows	mm / blow	CBR	CBR_{Low}
5	25	0.1575	4.0000	61.813	
	44	0.0895	2.2727	116.426	
	51	0.0772	1.9608	137.361	
	21	0.1875	4.7619	50.847	
	14	0.2812	7.1428	32.288	
	38	0.1036	2.6316	98.796	
	59	0.0667	1.6949	161.710	32
6	38	0.1036	2.6316	98.796	
	140	0.0281	0.7143	425.645	
	28	0.1406	3.5714	70.178	
	41	0.0960	2.4390	107.573	
	36	0.1094	2.7778	92.991	
	109	0.0361	0.9174	321.589	70
7	80	0.0492	1.2500	227.428	
	105	0.0375	0.9524	308.401	
	46	0.0856	2.1739	122.369	122
8	53	0.0743	1.8868	143.408	
	60	0.0656	1.6667	164.783	
	72	0.0547	1.3889	202.114	
	29	0.1358	3.4483	72.991	73
9	20	0.1969	5.0000	48.143	
	48	0.0820	2.0833	128.344	
	50	0.0787	2.0000	134.348	
	63	0.0625	1.5873	174.038	48

Notes:

1. *$CBR = 292/(DCP \times 25.4)^{1.12}$ for DCP in inches/blow (ODOT Geotechnical Design Manual)*
2. *Does not include preblows of 0.2*
3. *Does not include blows below 3.28 feet (100 cm)*
4. *Omitted*

From ODOT 2020 DCP results
(use lowest value over the depth of the exploration)

ID	CBR_{Low}
D-001	15
D-002	26
D-003	19
D-004	9.5
D-005	25
D-006	30
D-007	21

Average of the 2016 and 2020 DCP Results

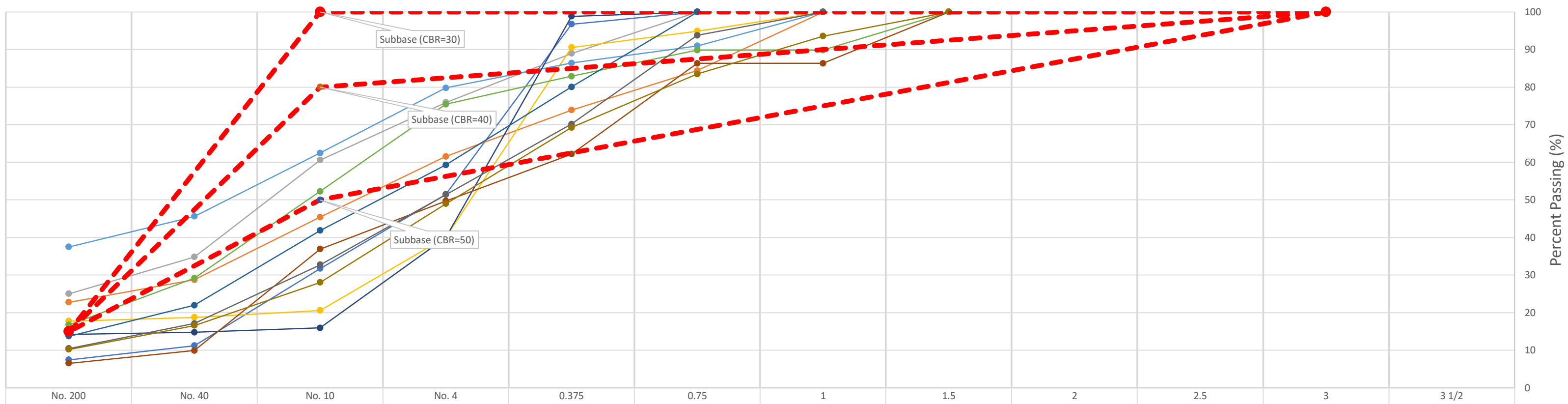
$CBR_{Avg\ Low}$ 26.04167

Say 25

CBR Grain-Size Correlation

in	mm DEPTH	B-001 1 ft	B-001 4.5 ft	B-002-1 5 ft	B-002-2 2.5 ft	B-002-2 5.5 ft	B-003-1 1.2 ft	B-004-1 4 ft	B-004-2 3 ft	B-008-0 3.5 ft	B-009-0 4.5 ft	B-010-0 3.5 ft	B-010-0 5 ft	B-011-0 2.5 ft
3 1/2	89													
3	76													
2.5	64													
2	51													
1.5	38						100		100		100			
1	25		100		100	100	89.82		86.3	100	93.55			100
0.75	19	100	84.4	100	94.91	90.97	89.82	100	86.3	93.77	83.48	100	100	95.34
0.375	10	96.72	73.89	88.95	90.54	86.41	82.89	98.83	62.17	70.18	69.26	92.63	80.04	69.45
No. 4	4.76	51.46	61.56	75.91	40.09	79.78	75.45	40.02	49.72	51.36	48.93	82.7	59.29	52.47
No. 10	2	31.71	45.38	60.57	20.6	62.45	52.25	15.94	36.92	32.73	28.04	61.96	41.86	37.89
No. 40	0.42	11.19	28.74	34.81	18.74	45.61	29.18	14.76	9.9	17.11	16.57	32.78	21.96	12.56
No. 200	0.074	7.48	22.76	25	17.76	37.49	16.72	14.22	6.57	10.51	10.19	18.15	13.71	7.45
LL		15	27	21	0	28	16	0	21	19	18	12	21	18
PL		15	21	13	0	18	0	0	16	15	16	0	19	15
PI		0	6	8	0	10	16	0	5	4	2		2	3
W														
USCS														
CBR	100%													
Y max	pci													
CBR	95%													
Y 95%	psi													
Swell	(%)													
k (psi/in)	UFC 3-260-02: Table 6-1													
wopt	%													
CBR (max)														
% Fine														
Sand (No. 4 - No.200														

Gradation Requirement for Subbase material
(MOT-4-10.30)



	No. 200	No. 40	No. 10	No. 4	0.375	0.75	1	1.5	2	2.5	3	3 1/2
B-001	7.48	11.19	31.71	51.46	96.72	100						
B-001	22.76	28.74	45.38	61.56	73.89	84.4	100					
B-002-1	25	34.81	60.57	75.91	88.95	100						
B-002-2	17.76	18.74	20.6	40.09	90.54	94.91	100					
B-002-2	37.49	45.61	62.45	79.78	86.41	90.97	100					
B-003-1	16.72	29.18	52.25	75.45	82.89	89.82	89.82	100				
B-004-1	14.22	14.76	15.94	40.02	98.83	100						
B-004-2	6.57	9.9	36.92	49.72	62.17	86.3	86.3	100				
B-008-0	10.51	17.11	32.73	51.36	70.18	93.77	100					
B-009-0	10.19	16.57	28.04	48.93	69.26	83.48	93.55	100				
B-010-0	13.71	21.96	41.86	59.29	80.04	100						
Subbase (CBR=50)	15		50								100	
Subbase (CBR=40)	15		80								100	
Subbase (CBR=30)	15		100								100	

Sieve Size (in)

3. SUBGRADE CBR. The strength of the subgrade may be expressed in terms of the CBR for flexible pavement design. The CBR test is described in CRD-C 654. It includes procedures for making tests on samples compacted to the design density in test molds and is soaked 4 days for making in-place CBR tests and for making tests on undisturbed samples. These tests are used to estimate the CBR that will develop in the pavement structure. However, a subgrade design CBR value above 20 is not permitted unless the subgrade meets the requirements for subbases. The CBR selected for the subgrade will be based on the predominant moisture conditions occurring during the life of the pavement. This moisture situation can be obtained from pavement evaluation reports and from soil tests under existing pavements. Where long duration soil moisture conditions cannot be determined with confidence, the soaked laboratory CBR will be selected for the subgrade soil.

From: UFC-3-260-02 (30 June 2001)

6-1

From: UFC-3-260-02 (30-June-2001)

Table 3-3
Maximum Permissible Values for CBR and Gradation Requirements

Material	Maximum CBR	Maximum Size	Maximum % Passing #10	Maximum % Passing #200	Maximum Liquid Limit*	Maximum Plasticity Index*
Subbase	50	50 mm (2")	50	15	25	5
Subbase	40	50 mm (2")	80	15	25	5
Subbase	30	50 mm (2")	100	15	25	5
Select Material	20	75 mm (3")	--	--	35	12

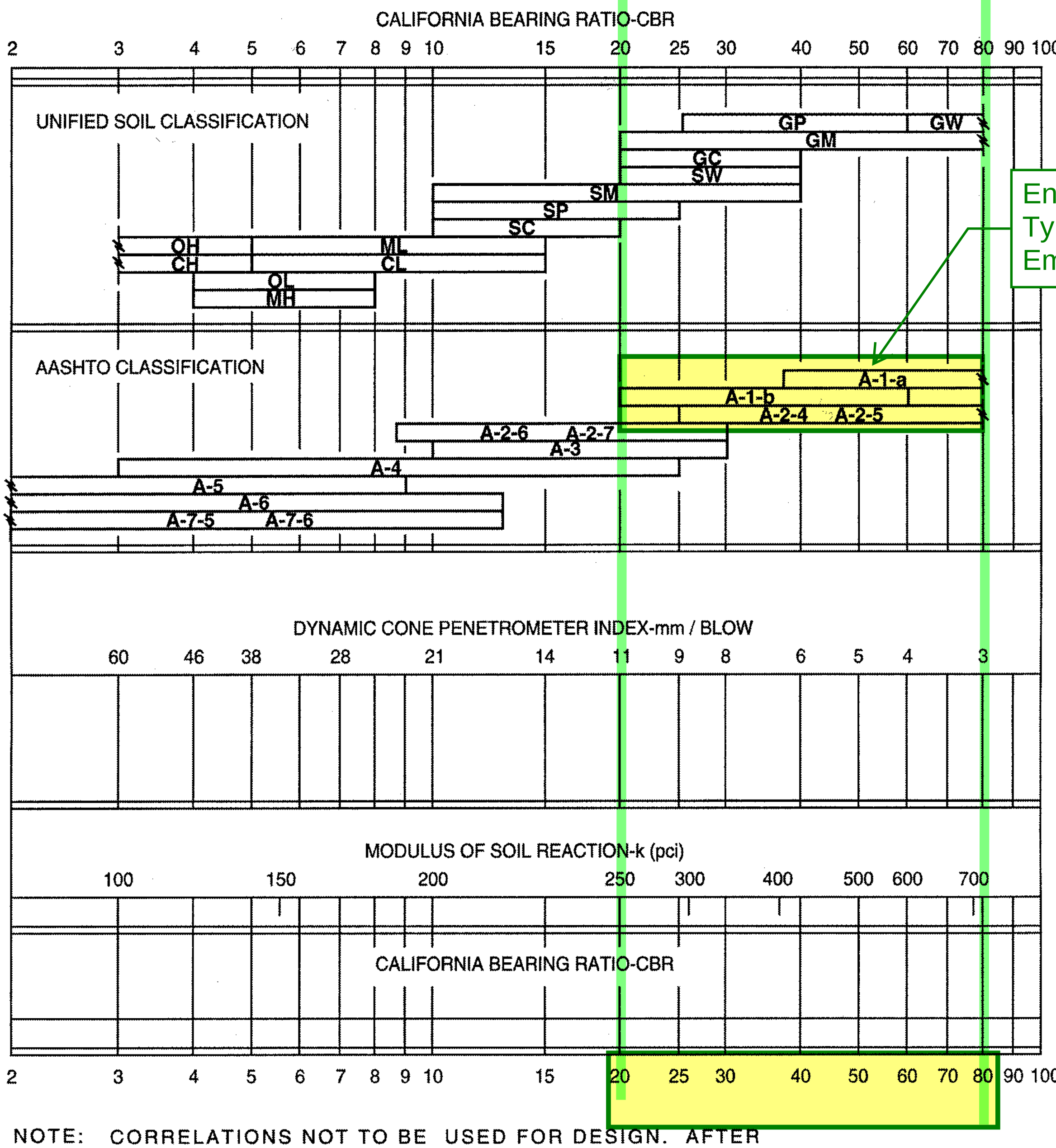
* ASTM D 4318.

The graphs above provide the grain-size distributions for tested project samples as they relate to grain-size requirements for correlated subbase CBR values as provided in the UFC-3-260-02 document.

As shown, the tested project samples do not meet the listed grain-size criteria to be considered subbase material. As such, UFC-3-260-02 recommends a maximum CBR of 20 to be designated for the subgrade.

Granular Embankment :
A-1-a
A-1-b
A-2-4

Encountered Soil Types Within Embankment Fill



NOTE: CORRELATIONS NOT TO BE USED FOR DESIGN. AFTER PORTLAND CEMENT ASSOCIATION "THICKNESS DESIGN FOR CONCRETE HIGHWAY AND STREET PAVEMENTS" PACKARD, ROBERT G., 1984. DESIGN LIMIT FOR k IS 500.

Figure 5-3. Approximate relationships of soil classification and soil strength

CHAPTER 6

SUBGRADE

1. SUITABILITY OF SUBGRADE. The information obtained from the explorations and tests previously described should be adequate to enable full consideration of all factors affecting the suitability of the subgrade and subsoil. The primary factors are as follows:

a. The general characteristics of the subgrade soils.

b. Depth to bedrock.

c. Depth to water table (including perched water table).

d. The compaction that can be attained in the subgrade and the adequacy of the existing density in the layers below the zone of compaction requirements.

e. The strength that the compacted subgrade, uncompacted subgrade, and subsoil will have under local environmental conditions.

f. The presence of weak or soft layers in the subsoil.

g. Susceptibility to detrimental frost action.

h. Settlement potential.

I. Expansion potential.

j. Drainage characteristics.

2. GRADE LINE. The soil type together with information on the drainage requirements, balancing cut and fill, flooding potential, depth to water table, depth to bedrock, and the compaction and strength characteristics should be considered in locating the grade line of the top of the subgrade. Generally, this grade line should be established to obtain the best possible subgrade material consistent with the proper utilization of available materials; however, economics of plans for construction must be given prime consideration.

3. SUBGRADE CBR. The strength of the subgrade may be expressed in terms of the CBR for flexible pavement design. The CBR test is described in CRD-C 654. It includes procedures for making tests on samples compacted to the design density in test molds and is soaked 4 days for making in-place CBR tests and for making tests on undisturbed samples. These tests are used to estimate the CBR that will develop in the pavement structure. However, a subgrade design CBR value above 20 is not permitted unless the subgrade meets the requirements for subbases. The CBR selected for the subgrade will be based on the predominant moisture conditions occurring during the life of the pavement. This moisture situation can be obtained from pavement evaluation reports and from soil tests under existing pavements. Where long duration soil moisture conditions cannot be determined with confidence, the soaked laboratory CBR will be selected for the subgrade soil.

Appendix G. Plan Notes

ITEM 204 – GEOGRID (AS PER PLAN)

All requirements of CMS Item 204 Geogrid shall be met in addition to the following:

1. HIGH PERFORMANCE MULTI-AXIAL, MULTI-APERATURE GEOGRID (NX850 AND NX850-FG)

A. Furnish multi-axial, multi-aperture geogrid that is integrally formed with hexagonal, trapezoidal and triangular apertures and high-profile ribs exhibiting significant dimensional stability through all ribs and junctions of the geogrid structure. The furnished geogrid shall maintain its stabilization and aggregate confinement capabilities under repeated dynamic loads while in service, and shall also be resistant to ultraviolet degradation, damage under normal construction practices and all forms of biological and chemical degradation encountered in the soil on which it is placed.

B. The geogrid shall be manufactured from a coextruded, composite polymer sheet, which is then punched and oriented. The resulting structure shall consist of continuous and non-continuous ribs forming three distinct aperture shapes (hexagon, trapezoid, and triangle) and an unimpeded suspended hexagon.

C. The geogrid structure shall have ribs with depth-to-width aspect ratios greater than 1.0.

D. The geogrid shall conform to the properties presented in the following table:

Required Geogrid Properties[1]

Name	Value (NX850)	Value (NX850-FG)	Unit
Aperture Shape	Hexagonal, Trapezoidal, & Triangular	Hexagonal, Trapezoidal, & Triangular	
Structure	Coextruded & Integrally Formed	Coextruded & Integrally Formed	
Rib Shape	Rectangular	Rectangular	
Rib Aspect Ratio[3]	> 1.0	> 1.0	
Node Thickness	0.18	0.18	in.
Continuous Parallel Rib Pitch	3.2	3.2	in.
Specific Dimension of the Finished Rolls[2] (Width x Length)	12.5 x 197	12.5 x 197	ft.
Grab Tensile Strength	-	160	lbs.
Grab Elongation	-	50	%
Trapezoid Tear Strength	-	160	lbs.
CBR Puncture Resistance	-	410	lbs.
Pemittivity	-	1.5	sec^{-1}
Water Flow	-	110	gpm/ft^2
Apparent Opening Size	-	70	Std. US
UV Resistance	-	70	% 500 hours
Specific Dimensions[2]	-	12.5 x 197	ft.
1. Unless indicated otherwise, values shown are minimum average roll values determined in accordance with ASTM D4759-02.			

2. Nominal dimensions.

3. Ratio of the mid-rib depth to the mid-rib width

E. Submit geogrid product data sheet and certification from the Manufacturer that the geogrid product supplied meets the requirements listed. A minimum of one material sample may be selected at random by the Engineer from the material delivered and tested for compliance with these requirements. Each sample size required shall be a minimum of three (3) feet wide with a one (1) square yard minimum area.

F. The Contractor shall check the geogrid upon delivery to verify the proper material has been received. The Contractor shall also inspect the geogrid to determine that it is free of flaws or damage that may have occurred during manufacturing, shipping, or handling.

G. Storage and Protection

(1) Follow ASTM D 4873 for geogrid labeling, shipment, and storage. Furnish product labels that clearly show the manufacturer's or supplier's name, product type, lot number, roll number, manufactured date, and roll dimension. Furnish a notation for each shipping document certifying that the material is in accordance with the manufacturer's certificate.

(2) Prevent excessive mud, wet concrete, epoxy, or other deleterious materials from coming in contact with and affixing to the geogrid materials.

(3) During shipping and storage, protect geogrid from direct sunlight, UV deterioration and temperatures greater than 160 degrees F (71 degrees C) or less than -20 degrees F (-29 degrees C).

(4) Rolled materials shall be laid flat or stood on end. Keep the geogrid dry and do not store directly on the ground.

(5) Geogrid materials should not be left directly exposed to sunlight for more than 6 weeks.

H. A minimum loose thickness of 6 inches of granular embankment material is required prior to operation of tracked vehicles over the geogrid.

ITEM 204 GRANULAR EMBANKMENT MATERIAL, AS PER PLAN

All requirements of CMS Item 204 granular embankment material shall be met, except the granular embankment to be utilized within the geogrid-reinforced soil mat shall consist of the existing on-site embankment materials meeting the Department Group Classifications A-1-a and A-1-b. The maximum grain size shall be less than 3 inches and the fines contents (passing the No. 200 sieve) no more than 15%. If additional granular embankment material is necessary to meet the planned fill thickness, materials conforming to ODOT Item 703.16 Type B Granular material is acceptable provided the fines contents does not exceed 15%.