SUBMITTED TO:
HDR Engineering, Inc.
1670 Broadway, Ste. 3400
Denver, Colorado 80202

BY:
Shannon & Wilson
1321 Bannock St., Suite 200
Denver, Colorado 80204

(303)825-3800
www.shannonwilson.com

PAVEMENT DESIGN REPORT

KS FLAP KIN 50(1) Cheney Reservoir Access

KINGMAN COUNTY, KANSAS

SHANNON & WILSON

June 2022

Shannon & Wilson No: 105834-001

Submitted To: HDR Engineering, Inc.
1670 Broadway, Ste. 3400
Denver, Colorado 80202
Attn: Mr. Mike Flick, PE

Subject: PAVEMENT DESIGN REPORT, KS FLAP KIN 50(1) CHENEY RESERVOIR ACCESS, KINGMAN COUNTY, KANSAS

We are pleased to submit this report for the above-referenced project. This report presents our pavement design recommendations and was prepared by the undersigned. Our scope of services was specified in Task Order 025/1000100061617 of Contract Number 100010021240 with HDR dated May 27, 2021.

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

Sincerely,

SHANNON & WILSON

Joseph C. Goode III
Senior Geotechnical Staff

GREGORY R. FISCHER
LICENSED
19457
KANSAS
PROFESSIONAL ENGINEER
6/8/22

Gregory R. Fischer, PhD, PE
President

JCG:DAA:GRF/mzc

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1 INTRODUCTION

This report summarizes the results of our subsurface exploration and laboratory testing program and presents pavement design recommendations and construction considerations for the proposed improvements to NE 50 Street as part of the Central Federal Lands Highway Division (CFLHD) project KS FLAP KIN 60(1) Cheney Reservoir Access (the Project) located in Kingman County, Kansas, south of Cheney Reservoir. Our scope of services was specified in Task Order 025/1000100061617 of Contract Number 100010021240 with HDR Engineering, Inc. (HDR) dated May 27, 2021. Our conclusions and recommendations in this report are based on:

- The limitations of our approved scope, schedule, and budget described in our contract;
- Our understanding of the Project and information provided by HDR;
- Subsurface conditions observed in the borings at the time our explorations were completed; and
- The results of testing performed on samples collected from the explorations.

The objective of our geotechnical studies was to provide recommendations and construction considerations, as presented herein, for the proposed roadway reconstruction. The authorized scope of services was based on this objective and this report should not be used for other purposes without Shannon & Wilson's review. If a service is not specifically indicated in this report, do not assume that it was performed.

2 PROJECT AND SITE DESCRIPTION

The Project is located in Kingman County, Kansas, approximately 20 miles west of Wichita, Kansas (Figure 1). The proposed improvements consist of a 2.1-mile segment of NE 50 Street (RS 607) located between NE 150 Avenue and NE 170 Avenue (located at the eastern extent of Kingman County). NE 50 Street provides access around the southern extent of Cheney Reservoir and serves at the primary access route from Cheney State Park and the Cheney State Park Marina Campground. The alignment of the eastern-most 1-mile segment of NE 50 Street is located directly south of the Cheney Reservoir Dam. NE 50 Street is an approximately 25-foot wide two-lane (a single eastbound and westbound lane) road paved with asphalt concrete pavement (ACP). We understand that the proposed improvements include rehabilitation or reconstruction of the existing road. Based on discussions with HDR and CFLHD, we understand that the preference is to reconstruct the road with full depth reclamation (FDR) and the proposed roadway reconstruction can accommodate a pavement grade raise. Based on discussions with HDR, we understand NE 50 Street will generally maintain the existing paved roadway footprint with portions of the alignment to be widened up to 2 feet.

3 SUBSURFACE INVESTIGATION

Shannon & Wilson conducted a field exploration program on July 15, 2021, to explore subsurface conditions along the proposed roadway alignment. The subsurface exploration program consisted of drilling and sampling five borings along NE 50 Street designated as borings SW-01 through SW-05. Refer to Figure 2 for the approximate boring locations. The borings were advanced to an approximate depth of 5.5 feet below the existing roadway grade.

Appendix A presents a discussion of the drilling and sampling procedures used to complete the borings. Appendix A also presents the individual exploration logs and an explanation of the symbols and terminology used.

4 GEOTECHNICAL LABORATORY TESTING

We completed geotechnical laboratory tests on selected samples retrieved from the borings to estimate index and engineering properties. The index tests included natural water content, grain size analysis, and Atterberg limits. The engineering property tests included corrosion and Hveem stabilometer (R-value). The laboratory test results as well as a discussion of the testing procedures are included in Appendix B. The natural water content, fines content, and Atterberg limits are also shown on the individual boring logs included in Appendix A.

5 EXISTING PAVEMENTS

5.1 Pavement Assessment

To evaluate the condition of the existing roadway pavement for potential rehabilitation options, we performed a pavement survey on July 14, 2021 to assess the overall pavement condition and to identify areas of distress and typical crack patterns using the Distress Identification Manual for the Long-Term Pavement Performance Project, (FHWA, revised 2014).

Exhibit 5-1 summarizes our pavement assessment and Exhibit 5-2 provide typical pavement condition photographs observed along the roadway. Generally, the observed transverse, longitudinal, and fatigue cracking was of low to moderate severity. Asphalt patches were common throughout the existing roadway and were generally considered to be of low to moderate severity. In our opinion, the general pavement condition was fair to poor.

Exhibit 5-1: Pavement Assessment

Roadway Range	Approximate Paving Area Effected	Notes
NE 150th to SW-01	15-25%	▪ Moderate severity and extent fatigue cracking. ▪ Low severity transverse cracking. ▪ Low severity asphalt patches.
SW-01 to SW-02	15-25%	▪ Low to moderate severity fatigue cracking. ▪ Low to moderate severity transverse cracking. ▪ Low to moderate severity asphalt patches.
SW-02 to SW-03	15-25%	▪ Moderate severity fatigue cracking. ▪ Moderate to high severity longitudinal and transverse cracking. ▪ Low to moderate severity asphalt patches.
SW-03 to SW-04	15-25%	▪ Moderate severity fatigue cracking. ▪ Moderate to high severity longitudinal cracking. ▪ Moderate severity transverse cracking. ▪ Low to moderate severity asphalt patches.
SW-04 to NE 170th	10-20%	▪ Moderate severity fatigue cracking. ▪ Low severity longitudinal cracking. ▪ Low to moderate severity transverse cracking. ▪ Low to moderate severity asphalt patches.

Exhibit 5-2: Typical Pavement Distress Photographs

Left – Low to moderate severity fatigue cracking. Right – Typical roadway distress photograph, low severity transverse and longitudinal cracking severity, and low severity patch deterioration.

5.2 Pavement Cores

Cores of the existing pavement were taken at each boring location. Existing asphalt thicknesses are summarized in Exhibit 5-3, and photographs of the pavement cores are provided in Appendix A. The observed pavements consisted of full depth ACP (paved directly on the subgrade) of varying thicknesses along NE 50 Street. We did note stripping and numerous delaminations from the cores.

Exhibit 5-3: Summary of Existing Pavement Sections

Boring	Lane	Existing Pavement Section	Notes
SW-01	EB	5-1/2 in. Full Depth ACP	Stripping observed from 2.5 to 5.5 in
SW-02	WB	6 in. Full Depth ACP	
SW-03	EB	9 in. Full Depth ACP	Delaminations at 1.5, 3.5, & 6.5 in; Stripping from 0 to 9 in.; Completely Stripped ACP 3.5 to 6.5 in.
SW-04	WB	5 in. Full Depth ACP	Delaminations at 1 & 2 in.; Stripping from 0 to 3 in.
SW-05	EB	8 in. Full Depth ACP	Delaminations at 1 & 3.5 in.; Stripping 0 to 8 in.

NOTE:
ACP = Asphalt Concrete Pavement; EB = Eastbound; in. = inches; WB = Westbound

6 SUBSURFACE CONDITIONS

6.1 Pavement Subgrade

Individual boring logs of our exploration locations are presented in Appendix A. The subgrade along the NE 50 Street varied. Observed subsurface conditions in borings SW-03 and SW-04 consisted of medium stiff to very stiff, fat clay with sand (Association of State Highway Officials [AASHTO] soil classification A-7-6) and stiff, sandy lean clay (AASHTO A-6 soil), respectively. All other borings encountered loose to medium dense, clayey sand and silty, clayey sand (AAASHTO A-2-6, A-4, and A-6 soils).

Our observations are specific to the locations, depths, and dates noted on the exploration logs in Appendix A and may not be applicable to all areas of the site. There is no amount of explorations or laboratory testing that can precisely predict the characteristics, quality, or distribution of subsurface conditions at every location throughout the site. Variations in the subsurface conditions may occur between and below the borings. Also, the passage of time or intervening causes (natural and manmade) may result in changes to the conditions of the site and subsurface conditions.

6.2 Groundwater

Groundwater was not encountered in any of our borings; however, fluctuations of groundwater levels at the site are possible and will depend on many factors, including seasonal variations, local precipitation, and flood events. Nevertheless, groundwater is not anticipated to affect the design or construction of the Project.

7 PAVEMENT DESIGN RECOMMENDATIONS

Performance of a pavement system depends on the pavement material and thicknesses, subgrade strength, traffic loads and repetitions, and design life. The following sections discuss each of these aspects as they relate to the project.

Based on discussions with CFLHD and HDR, we understand either FDR or FDR with cement are the preferred paving solutions for NE 50 Street. Although, other rehabilitation or reconstruction alternative such as mill and structural overlay, full reconstruction with either a composite pavement section (ACP over crushed aggregate base [CAB]) or a full depth ACP pavement section are feasible, only FDR pavement section is discussed.

7.1 Design Methodology

Our pavement design and results are based on the design procedures presented in the AASHTO Guide for the Design of Pavement Structures (1993) with guidance from the U.S. Department of Transportation (USDOT) and Federal Highway Administration (FHWA) Federal Lands Highway Project Development and Design Manual (PDDM) (2008); and the Kansas Department of Transportation (KDOT) geotechnical/pavement design manual (2007). Pavement design inputs and calculations are presented in Appendix C.

7.2 Full Depth Reclamation (FDR)

In our opinion, NE 50 Street is suitable for FDR. FDR consists of in-place pulverization that becomes the base of a new ACP paving subgrade. The surface of the FDR can then be adjusted to accommodate minor grading adjustments, provided a minimum FDR thickness is maintained. A new ACP pavement surface layer is then paved over the FDR prepared roadway typically resulting in an overall raise to the roadway grades. Rehabilitating the existing roadway with FDR effectively provides a new pavement and eliminates the potential for reflective cracking from the existing pavement. In addition, cement or a bituminous asphalt cement can be added to chemically stabilize the FDR. Based on discussions with CFLHD and HDR, cement stabilized FDR is preferred over bituminous treated FDR.

As indicated in Exhibit 5-3, the existing ACP thicknesses are variable. For non-chemically stabilized FDR, we recommend an FDR thickness of 6 inches to limit the mixing of subgrade fines into the FDR layer. For FDR chemically stabilized with cement, 8 inches of FDR is recommended. In locations where the NE 50 Street ACP thickness is less than 8 inches, the cement treatment will treat and stabilize subgrade that is mixed into the FDR layer. In widening areas where there is insufficient FDR material, we recommend substituting the FDR with CAB. If FDR with cement is utilized, we recommend treating any substituted CAB material for minor widenings with cement.

7.3 Design Subgrade R-value

The subsurface explorations completed along the roadway primarily consisted of clayey sand and clay subgrade. Due to the variability of subgrade soils along the NE 50 Street and limitations in our scope, we performed a single R-value test on the poorest quality subgrade (boring SW-03 consisting of AASHTO A-7-6 material). The R-value test result indicated a value of 4.1. As FDR is proposed for the site, the existing subgrade is anticipated to be left in place.

For our pavement analysis, we used an AASHTO correlation between R-value and resilient modulus, which calculates a resilient modulus value of approximately 3,200 pounds per square inch (psi) for the R-value of 4.1.

7.4 Traffic Loading

For the NE 50 Street traffic loading, HDR provided an average daily traffic loading of 650 vehicles per day from 2021. The traffic volume was anticipated to be composed of mainly local passenger car traffic with a portion of the traffic composed of recreational vehicles and pickup-trucks hauling boats to the marina. Based on discussions with HDR, we assumed the following traffic distribution:

- 80% passenger vehicles,
- 17.5% pick-up trucks hauling boats
- 1% delivery vehicles,
- 1.25% large RV, and
- 0.25% trash trucks.

Refer to Appendix C for the flexible pavement equivalency factors used in 18-kip equivalent single axle loading (ESAL) calculation of NE 50 Street for each of the above design vehicles. For a 20-year design life and an annually compounded growth rate of 2%, we estimate a design traffic loading for NE 50 Street of 80,000 ESALs.

7.5 Recommended Pavement Sections

Using the AASHTO procedures and the parameters outlined in Appendix C, we recommend the following pavement sections:

Exhibit 7-1: Recommended Pavement Sections

Location	Paving Alternative	Recommended Pavement Section
NE 50 Street	Alt. 1	5.5 in. ACP over 6 in. FDR
	Alt. 2	4 in. ACP over 8 in. FDR with Cement

NOTE:

1 If there are insufficient quantities of FDR material, we recommend substituting CAB for FDR. For FDR with cement alternative, the CAB material should also be treated with cement.

ACP = Asphalt Concrete Pavement; in.= inches; FDR = Full Depth Reclamation

8 ADDITIONAL CONSIDERATIONS

8.1 Expansive Subgrade Potential

High plasticity soils in Kansas are susceptible to volume change by swelling/shrinking. This geologic phenomenon has the potential to cause substantial damage to lightly loaded structures, such as pavements, when exposed to water. To provide an indication of the swell potential of near surface soils at the site, we performed Atterberg limits, grain size distributions, and moisture contents on soil samples encountered in our explorations; summarized in Exhibit 7-1.

Exhibit 8-1: Summary of Subgrade Index Testing

Boring Location	Fines Content (%)	Natural Moisture Content (%)	Plastic Limit	Liquid Limit	Plasticity Index	USCS	AASHTO Soil Class
SW-01	24	9.6	13	24	11	SC	A-2-6 (0)
SW-02	36	13.7	12	24	12	SC	A-6 (1)
SW-03	78	27.4	17	60	43	CH	A-7-6 (34)
SW-04	62	13.0	13	28	15	CL	A-6 (6)
SW-05	42	12.2	14	19	5	SC	A-4 (0)

NOTE:
USCS = Unified Soil Classification System

The Kansas Department of Transportation (KDOT) geotechnical design manual (2007) states: "*Swelling soils are a significant cause of pavement distress and failure. One of the best* *indicators for determining the potential for a soil to swell is the plasticity index (PI) and the liquid limit. High PI's (over 25) and liquid limits (over 50) are a strong indicator that the soil will swell."*

Although the PI is a good indication of potential volume change, it does not consider the current moisture content regime for pavement that have been in place for a number of years. Typically, pavements block evapotranspiration from occurring and the subgrade below the pavement have an overall elevated moisture content after paving. In our experience, one indication of swell potential is to compare the in-situ moisture content of the subgrade to the plastic limit (PL) in each test. Generally, for clayey soils (AASHTO A-6 and A-7-6 soils), if the in-situ moisture content is near or greater than the PL, the subgrade likely has a low swell susceptibility. For example, although the PI value in boring SW-03 does exceed 25, the in-situ moisture content is 10% above the PL and the subgrade is likely in a post-swelling condition. If the subgrade is left in place and not allowed to dry during construction or rehabilitation, the subgrade is likely to have a low volume change. In our opinion, borings SW-02 and SW-04 also have in-situ moisture contents at or above the PL and in our opinion a low swell potential.

For boring SW-01, the in-situ moisture content is 3% below the PL, but the subgrade sample primary consists of non-swell susceptible material (up to 76% sand and gravel). Similar, in boring SW-05 the in-situ moisture content is 2% below the PL, but the overall PI of the sample is 5 and is considered a low swell potential.

In summary, based on the borings and laboratory tests, we do not anticipate significant swell-related damage or heaving of the proposed rehabilitated pavement.

8.2 PDDM Subexcavation Requirements

To mitigate against potential constructability issues (pumping subgrade and or achieving compaction of subgrade), as well as reducing long-term swell potential, guidance from Section 11.3.2.1.3 of the PDDM indicates:

- 2 feet of subexcavation (overexcavation and removal of excavated material from the site) for subgrades with a PI ranging from 15 to 25,
- 2 to 4 feet of subexcavation for subgrades with a PI ranging from 25 to 35 or a LL ranging from 50 to 60, and
- 4 to 6 feet of subexcavation for subgrades with a PI greater than 35 or a LL greater than 60.

Based on the data provided in Exhibit 7-1, boring SW-04 would require a subexcavation depth of 2 feet and boring SW-03 would require 4 to 6 feet of subexcavation. The PDDM does note that the subexcavation requirements should account for the traffic volume and project significance when selecting a subexcavation depth.

Subgrade conditions in and near boring SW-03 likely will exhibit pumping and difficulty in compaction. In our opinion, the subexcavation depth is likely excessive for a rehabilitation project with only minor widening. Additional discussion is provided in Section 9.2.

8.3 Frost Damage

Frost susceptible soils can lead to pavement performance issues due to heaving or deformation from ice lenses in the underlying soil and pavement fatigue damage due to thaw-weakened subgrade of the springtime freeze-thaw cycle (USDOT, FHWA, 2008). Frost susceptible soils typically include fine grained soils such as silts and clays. In accordance with the PDDM, typical treatment for frost susceptible soils consists of assuring there is an adequate pavement layer structure to account for the loss of bearing capacity during the spring thaw and removing or replacing highly frost susceptible soil for a portion of the expected frost depth. As FDR is proposed, and based on our experience on past CFLHD projects, we understand it is cost prohibitive to provide protection against frost heave on such projects. If this is not the case, we recommend partial removal and replacement (up to about 70% of the frost depth of 24 inches) and we should be contacted to provided alternative pavement sections.

8.4 Corrosion Testing

The soil encountered at the project site can be corrosive to substructure elements. To assist in estimating the corrosion potential at the site, a clay sample was tested for pH, resistivity, water soluble sulfates, and chlorides from boring SW-03. The results are presented in Table B-1 in Appendix B and discussed in further details below.

The resistivity measured in the sample was 820 ohm-centimeters. Based on correlations developed by Roberge (2012), these values suggest extremely corrosive subsurface conditions for metal in contact with subsurface materials across the site.

The concentration of water-soluble sulfates measured in the samples were 0.026 percent by weight. Based on classifications as defined by ACI-318-19 (ACI, 2019), these test results suggest an exposure class S0 on concrete exposed to site soils.

The test results and the above discussion are provided to assist the designer in the selection of project materials, concrete type, or other features with respect to corrosion.

9 CONSTRUCTION AND MATERIAL SPECIFICATIONS

The applicability of the design parameters in Section 7 is contingent on good construction practice. Poor construction techniques may alter conditions from those upon which our recommendations are based, and therefore result in poor performance. Our analyses assumed that this project is constructed according to FP-14 U.S. Customary Units (USDOT and FHWA, 2014) construction standards. The following sections provide additional construction considerations for this project.

9.1 Site Preparation

All surface and subsurface structures associated with current development of the site, including utility poles, fence poles, underground utilities and other deleterious material, should be removed. Any existing surficial topsoil and soil containing visible organics should be stripped and removed from all areas.

9.2 Earthwork

9.2.1 General

Earthwork, including placement of fill and subgrade preparation, should conform to the requirements provided in the FP-14 U.S. Customary Units, (USDOT and FHWA, 2014) and the recommendations provided in the following sections.

9.2.2 Subgrade Preparation and Fill Placement

Proper subgrade preparation is required for adequate pavement performance. Areas of exposed subgrade should be prepared in accordance with FP-14 Section 204.11. All subgrade material and fill should be compacted to a dense/firm and unyielding condition. On-site subgrade and fill materials should be compacted to at least 95 percent maximum density, as determined by AASHTO T 180 or T 99, as presented in FP-14 Section 204. Fill should be placed in uniform, horizontal layers not exceeding 8 inches in loose thickness for heavy, self-propelled compactors, or 4 inches for hand-operated mechanical compactors. The appropriate lift thickness will depend on the Contractor’s equipment as well as the moisture content and quality of the fill material.

9.2.3 Proof-Rolling

We recommend proof-rolling or probing excavation areas in widenings to determine suitability of the subgrade. In areas that are identified as being loose, soft, or yielding during proof-rolling or probing, we recommend:

1. Overexcavating the pavement section a depth of 24 inches below the pavement section.
2. Backfilling and moisture treating 24 inches of either (a) ¾-inch minus crushed aggregate consisting of well graded sand with silt and gravel or (b) a CAB conforming to FP-14 U.S. Customary Units (USDOT and FHWA, 2014) construction standards.

9.3 Paving Materials

The following exhibit summarizes our recommendations for pavement material selection using the FP-14 specifications. Based on the project estimated paving quantities, we understand that the ACP material will utilize FP-14 Section 403, which uses the local department of transportation mix design. Exhibit 7-1 below reflects a KDOT (2015) mix design for the ACP.

Exhibit 9-1: Recommended Materials for Pavements

Material	FP-14 Specification	Additional Requirements/Comments
CAB	Section 302	Use Gradation C, D, or E
FDR	Section 304	-
FDR with Cement	Section 305	-
ACP	Section 403 (FP-14) Using KDOT Section 611 (2015)	Aggregate Gradation: Grade A, 1/2-inch NMAS PG Binder: PG 64-22 Gyratory Number (N): 75
Stabilization Geogrid	714.03	Tensar BX-1200 or equivalent product

NOTE:

ACP = Asphalt Concrete Pavement; CAB = Crushed Aggregate Base; FDR = Full Depth Reclamation; KDOT = Kansas Department of Transportation; NMAS = Nominal Maximum Aggregate Size; PG = Performance Grade;

To determine an appropriate HMA binder for the site, we used software developed by the Federal Highway Administration (FHWA) Long Term Pavement Performance (LTPP) Bind (2021). The LTPP Bind software indicated a performance grade (PG) of PG 70-22 for a 98% reliability for the top lift of ACP for the site and anticipated traffic loading. However, a PG 70-22 binder is not available in Kansas. For both the top and bottom lifts, we recommend using a PG 64-22 binder which result in an approximate reliability of 85% for the top lift. We recommend a gyratory number of 75 be used for the mix design. In addition, we recommend using a 0.5 inch nominal maximum aggregate size for the ACP mix. A tack coat should be placed between subsequent lifts if the underlying lift will be used for traffic or left uncovered for a 24-hour period.

10 CLOSURE

This report has been prepared for the exclusive use of HDR and the Central Federal Lands Highway Division for the purpose of providing pavement recommendations for the Cheney Reservoir Access improvements project. This pavement design report should not be used without our approval if any of the following occurs:

- Assumptions stated in this report have changed.
- Project details change or new information becomes available such that our analyses and recommendations may be affected.
- A substantial period of time has passed since the date of this report.

If any of these occur, we should be retained to review the applicability of our analyses and recommendations.

Within the limitations of scope, schedule and budget, the analyses, conclusions and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical and geological principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied.

Shannon & Wilson has prepared "Important Information about Your Geotechnical Report," to assist you and others in understanding the use and limitations of our reports.

11 REFERENCES

American Concrete Institute (ACI), 2019, Building code requirements for structural concrete and commentary, Farmington Hills, Mich., ACI 318-19.

American Association of State Highway and Transportation Officials (AASHTO), 1993, AASHTO guide for design of pavement structures: Washington, D.C., AASHTO, 2 v.

Federal Highway Administration (FHWA), 2021, LTPP InfoPave: Tools: LTPPBind Online, accessed August 2021; available: https://infopave.fhwa.dot.gov/Tools/LTPPBindOnline

Kansas Department of Transportation (KDOT), 2015, Standard Specifications at for State Road & Bridge Construction' available: https://www.ksdot.org/bureaus/burConsMain/specprov/2015specprov.asp

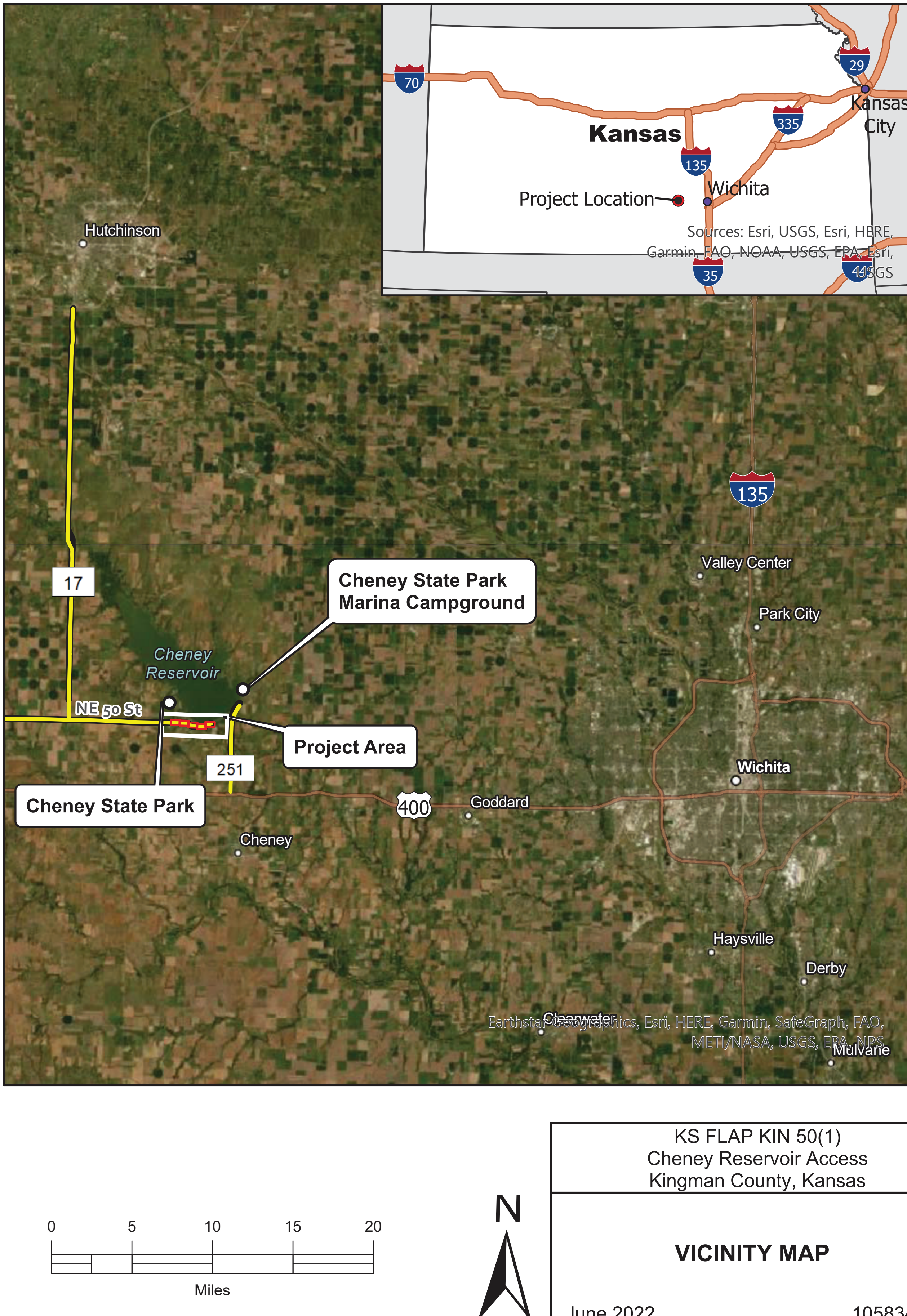
KDOT, 2007, Geotechnical Manual, Vol. 1; available: https://kart.ksdot.org/Download/DownloadDetail.aspx?FileID=470

Roberge, P.R., 2012, Handbook of corrosion engineering, Second Edition: McGraw-Hill, New York, New York.

U.S. Department of Transportation (USDOT), Federal Highway Administration (FHWA), 2014, Distress Identification Manual for the Long-Term Pavement Performance Program, Publication No. FHWA-HRT-13-092 Revised May 2014.

USDOT, FHWA, 2014, Federal standard specifications for construction of roads and bridges on Federal Highway projects, FP-14 English Units: U.S. Department of Transportation and Federal Highway Administration, available: https://highways.dot.gov/federal-lands/specs

USDOT, FHWA, 2008, Federal Lands Highway project development and design manual (PDDM): U.S. Department of Transportation and Federal Highway Administration; available: https://highways.dot.gov/federal-lands/pddm



KS FLAP KIN 50(1)
Cheney Reservoir Access
Kingman County, Kansas

VICINITY MAP

June 2022 105834-001

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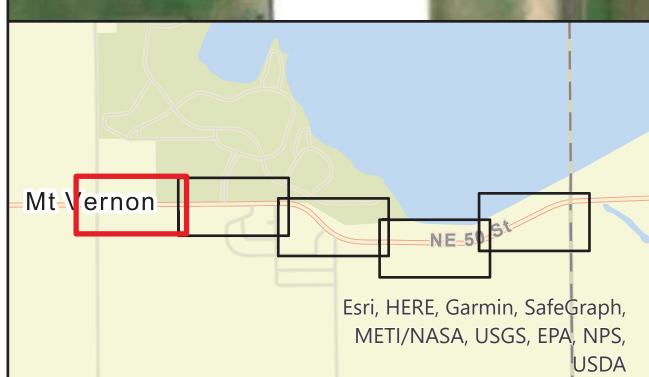
FIG. 1



LEGEND

Boring Designation and Approximate Location

0 200 400 600 800

Approximate Scale In Feet

N

KS FLAP KIN 50(1)
Cheney Reservoir Access
Kingman County, Kansas

SITE AND EXPLORATION PLAN

June 2022 105834-001

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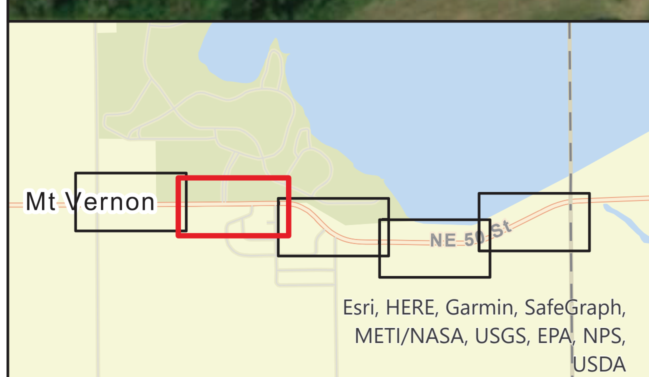
FIG. 2
Sheet 1 of 5



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Boring Designation and Approximate Location

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Approximate Scale In Feet

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Cheney Reservoir Access
Kingman County, Kansas

SITE AND EXPLORATION PLAN

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FIG. 2
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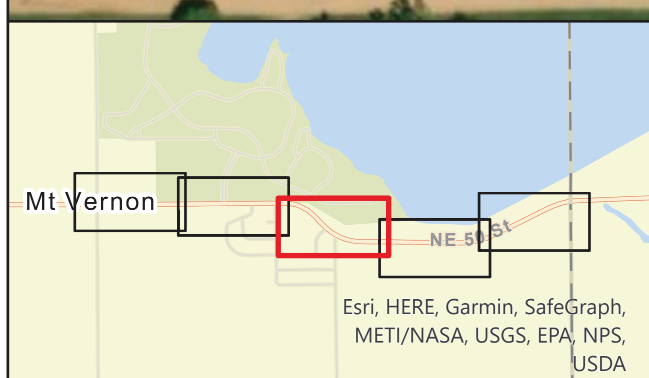

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Boring Designation and Approximate Location

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Approximate Scale In Feet

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Cheney Reservoir Access
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SITE AND EXPLORATION PLAN

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FIG. 2
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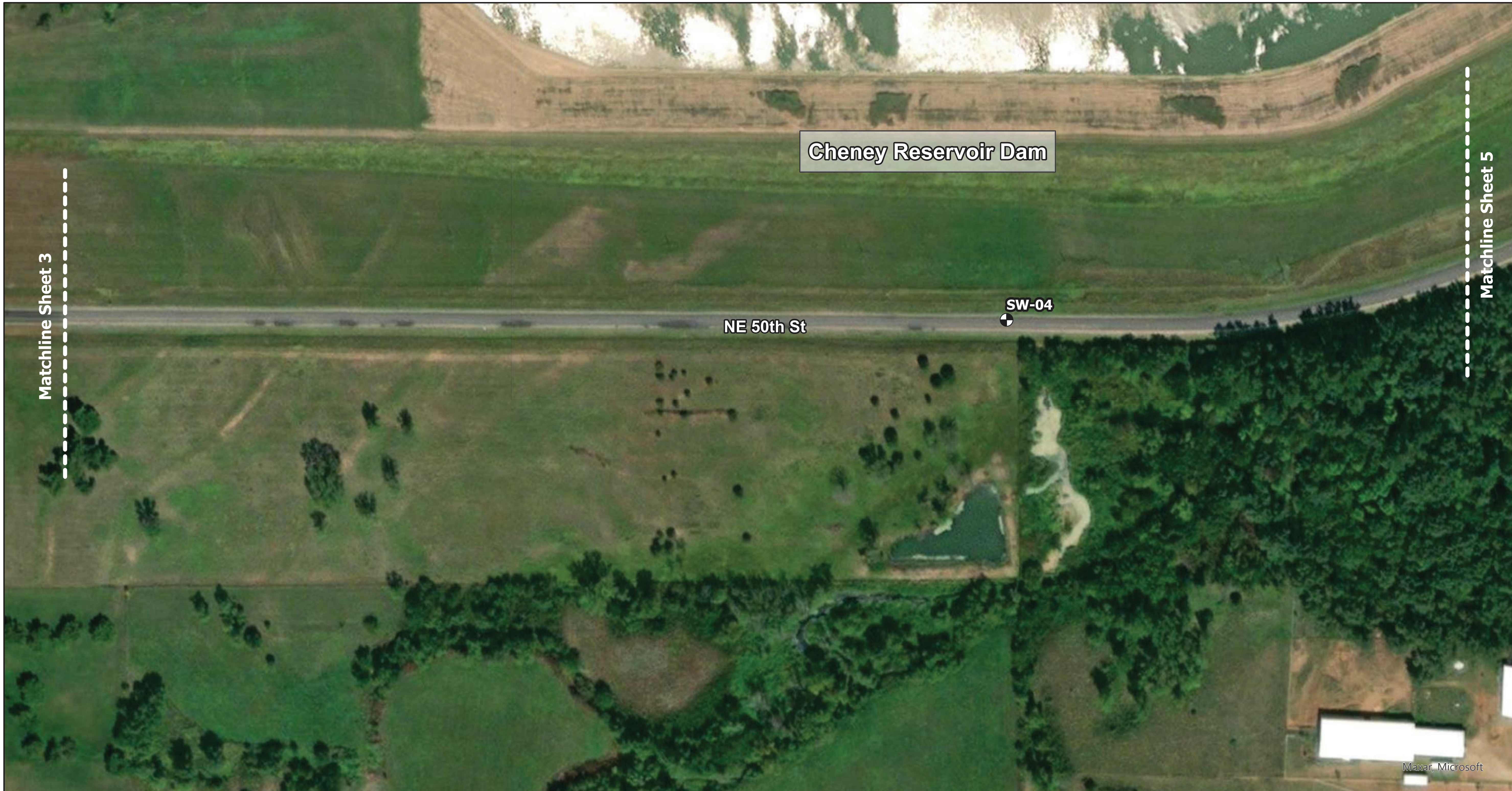

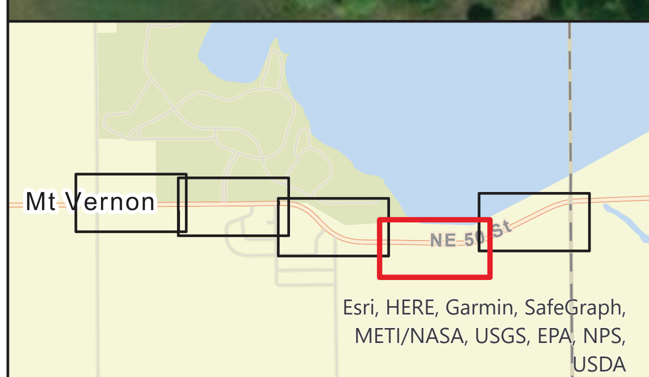

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Boring Designation and Approximate Location

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Approximate Scale In Feet

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SITE AND EXPLORATION PLAN

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FIG. 2
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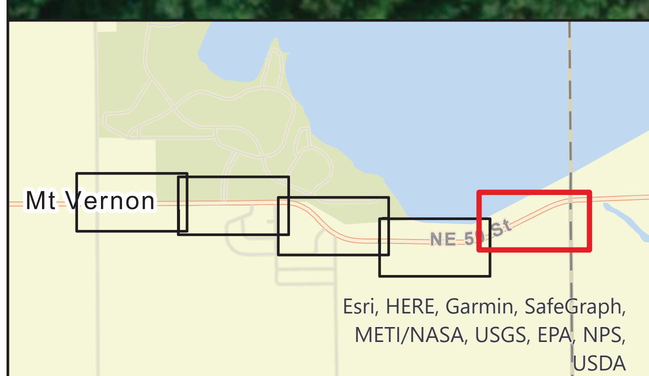

LEGEND

Boring Designation and Approximate Location

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Approximate Scale In Feet

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KS FLAP KIN 50(1)
Cheney Reservoir Access
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SITE AND EXPLORATION PLAN

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FIG. 2
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APPENDIX A: SUBSURFACE EXPLORATIONS

Appendix A
Subsurface Explorations

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Figures

APPENDIX A: SUBSURFACE EXPLORATIONS

A.1 INTRODUCTION

Shannon & Wilson's field exploration program was conducted on July 15, 2021 and consisted of drilling 5 borings at the locations shown on Figure 2. The methods used to conduct the field exploration program are described below. Laboratory testing procedures and results are presented in Appendix B.

A.2 EXPLORATIONS

The borings were coordinated (including subcontractor coordination, utility locates, permitting, and traffic control) and observed by Shannon & Wilson. Individual boring logs are presented in Figures A-2 to A-6. The exploration logs represent our interpretation of the contents of the field log and select results of laboratory testing.

Boring locations were pre-marked by Woolpert, Inc. prior to mobilization to the site. With the exception of boring SW-01, the borings were generally drilled within 5 feet of the pre-marks located approximately in the center of either the eastbound or westbound lane. Boring SW-01 was located approximately 7 feet east of the pre-mark. Refer to Figures A-8 through A-12 for the locations of the borings relative to the pre-marks. The borings were drilled by GSI Engineering of Wichita, Kansas (under subcontract to Shannon & Wilson) using a CME 45 truck mounted drill rig. Prior to drilling, pavement cores were obtained (Section A.3). The borings were advanced to depths of approximately 5.5 feet using 6-inch diameter solid-stem auger. Drill cuttings were used to backfill the bore holes up to the base of the existing pavement. The roadway surface was then repaired using cold-patch asphalt.

A.2.1 Soil Classification System

During exploration, our representative collected samples and prepared field logs of the explorations. Soil classification for this project was based on ASTM International (ASTM) Designation: D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), and ASTM Designation: D2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). The Unified Soil Classification System (USCS) is summarized in Figure A-1.

A.2.2 Standard Penetration Test (SPT)

Disturbed samples were obtained in general accordance with the Standard Penetration Test (SPT) (ASTM Designation: D1586). The SPT consists of driving a 2-inch outside diameter (O.D.), 1.375-inch inside diameter split-spoon sampler a distance of 18 inches with a 140-

pound hammer free-falling a distance of 30 inches. An automatic hammer system was used to advance the samplers. During sampling, the Shannon & Wilson field representative recorded the number of blows for each 6-inch increment of penetration and summed the blow counts for the last two 6-inch increments. This sum is recorded as the penetration resistance number, or N-value. If high penetration resistance prevented driving the total length of the sampler, the Shannon & Wilson field representative recorded the partial penetration depth and blow count. The N-values provide a means for evaluating the relative density or compactness of cohesionless (granular) soils and consistency or stiffness of cohesive (fine-grained) soils (see Figure A-1). The N-values are shown in the individual boring logs. Representative portions of the split-spoon sample obtained in conjunction with the SPT were placed in a screw-top plastic jar and transported to our laboratory.

A.2.3 Bulk Sampling

Bulk soil samples were obtained by collecting the drill cuttings from select borings. Approximately 20 to 30 pounds of cuttings from each location were placed in a plastic bag and transported to our laboratory for further evaluation and testing. The bulk samples are composite samples sometimes spanning over several soil layers. The UCSC classification of the composite bulk samples has not been incorporated into the boring logs for this reason.

A.3 PAVEMENT CORES

A portable electric-powered coring machine fitted with a 4-inch-diameter core barrel was used to obtain pavement cores of the existing pavement. The cores were brought to the Shannon & Wilson laboratory where they were measured and then photographed. A summary of the individual core thicknesses and photographs of the cores are provided in Figure A-7.

Shannon & Wilson, Inc. (S&W), uses a soil identification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following pages. Soil descriptions are based on visual-manual procedures (ASTM D2488) and laboratory testing procedures (ASTM D2487), if performed.

S&W INORGANIC SOIL CONSTITUENT DEFINITIONS

CONSTITUENT[2]	FINE-GRAINED SOILS (50% or more fines)[1]	COARSE-GRAINED SOILS (less than 50% fines)[1]
Major	***Silt, Lean Clay, Elastic Silt,*** or ***Fat Clay***[3]	***Sand*** or ***Gravel***[4]
Modifying (Secondary) Precedes major constituent	30% or more coarse-grained: ***Sandy*** or ***Gravelly***[4]	More than 12% fine-grained: ***Silty*** or ***Clayey***[3]
Minor Follows major constituent	15% to 30% coarse-grained: ***with Sand*** or ***with Gravel***[4]	5% to 12% fine-grained: ***with Silt*** or ***with Clay***[3]
	30% or more total coarse-grained *and* lesser coarse-grained constituent is 15% or more: ***with Sand*** or ***with Gravel***[5]	15% or more of a second coarse-grained constituent: ***with Sand*** or ***with Gravel***[5]

[1]All percentages are by weight of total specimen passing a 3-inch sieve.
[2]The order of terms is: *Modifying Major with Minor*.
[3]Determined based on behavior.
[4]Determined based on which constituent comprises a larger percentage.
[5]Whichever is the lesser constituent.

MOISTURE CONTENT TERMS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

STANDARD PENETRATION TEST (SPT) SPECIFICATIONS

Hammer:	140 pounds with a 30-inch free fall. Rope on 6- to 10-inch-diam. cathead 2-1/4 rope turns, > 100 rpm NOTE: If automatic hammers are used, blow counts shown on boring logs should be adjusted to account for efficiency of hammer.
Sampler:	10 to 30 inches long Shoe I.D. = 1.375 inches Barrel I.D. = 1.5 inches Barrel O.D. = 2 inches
N-Value:	Sum blow counts for second and third 6-inch increments. Refusal: 50 blows for 6 inches or less; 10 blows for 0 inches.

NOTE: Penetration resistances (N-values) shown on boring logs are as recorded in the field and have not been corrected for hammer efficiency, overburden, or other factors.

PARTICLE SIZE DEFINITIONS

DESCRIPTION	SIEVE NUMBER AND/OR APPROXIMATE SIZE
FINES	< #200 (0.075 mm = 0.003 in.)
SAND	
Fine	#200 to #40 (0.075 to 0.4 mm; 0.003 to 0.02 in.)
Medium	#40 to #10 (0.4 to 2 mm; 0.02 to 0.08 in.)
Coarse	#10 to #4 (2 to 4.75 mm; 0.08 to 0.187 in.)
GRAVEL	
Fine	#4 to 3/4 in. (4.75 to 19 mm; 0.187 to 0.75 in.)
Coarse	3/4 to 3 in. (19 to 76 mm)
COBBLES	3 to 12 in. (76 to 305 mm)
BOULDERS	> 12 in. (305 mm)

RELATIVE DENSITY / CONSISTENCY

COHESIONLESS SOILS		COHESIVE SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
< 4	Very loose	< 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
> 50	Very dense	15 - 30	Very stiff
		> 30	Hard

WELL AND BACKFILL SYMBOLS

Bentonite Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Inclinometer or Non-perforated Casing
Perforated or Screened Casing	Vibrating Wire Piezometer

PERCENTAGES TERMS [1, 2]

Trace	< 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

[1]Gravel, sand, and fines estimated by mass. Other constituents, such as organics, cobbles, and boulders, estimated by volume.

[2]Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

KS FLAP KIN 50(1)
Cheney Reservoir Access
Kingman County, Kansas

SOIL CLASSIFICATION AND LOG KEY

June 2022 105834-001

SHANNON & WILSON, INC.
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FIG. A-1
Sheet 1 of 3

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(Modified From USACE Tech Memo 3-357, ASTM D2487, and ASTM D2488)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL	TYPICAL IDENTIFICATIONS
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Gravel *(less than 5% fines)*	GW	Well-Graded Gravel; Well-Graded Gravel with Sand
			GP	Poorly Graded Gravel; Poorly Graded Gravel with Sand
		Silty or Clayey Gravel *(more than 12% fines)*	GM	Silty Gravel; Silty Gravel with Sand
			GC	Clayey Gravel; Clayey Gravel with Sand
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Sand *(less than 5% fines)*	SW	Well-Graded Sand; Well-Graded Sand with Gravel
			SP	Poorly Graded Sand; Poorly Graded Sand with Gravel
		Silty or Clayey Sand *(more than 12% fines)*	SM	Silty Sand; Silty Sand with Gravel
			SC	Clayey Sand; Clayey Sand with Gravel
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML	Silt; Silt with Sand or Gravel; Sandy or Gravelly Silt
			CL	Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
		Organic	OL	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH	Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
			CH	Fat Clay; Fat Clay with Sand or Gravel; Sandy or Gravelly Fat Clay
		Organic	OH	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat or other highly organic soils (see ASTM D4427)

NOTE: No. 4 size = 4.75 mm = 0.187 in.; No. 200 size = 0.075 mm = 0.003 in.

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, Sand with Silt)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart. Graphics shown on the logs for these soil types are a combination of the two graphic symbols (e.g., SP and SM).

2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, Lean Clay to Silt; SP-SM/SM, Sand with Silt to Silty Sand)* indicate that the soil properties are close to the defining boundary between two groups.

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SOIL CLASSIFICATION AND LOG KEY

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FIG. A-1
Sheet 2 of 3

2013_BORING_CLASS2 105834_CFL KS CHENEY RESERVOIR.GPJ SWNEW.GDT 8/25/21

GRADATION TERMS

Poorly Graded	Narrow range of grain sizes present or, within the range of grain sizes present, one or more sizes are missing (Gap Graded). Meets criteria in ASTM D2487, if tested.
Well-Graded	Full range and even distribution of grain sizes present. Meets criteria in ASTM D2487, if tested.

CEMENTATION TERMS[1]

Weak	Crumbles or breaks with handling or slight finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

PLASTICITY[2]

DESCRIPTION	VISUAL-MANUAL CRITERIA	APPROX. PLASITICTY INDEX RANGE
Nonplastic	A 1/8-in. thread cannot be rolled at any water content.	< 4
Low	A thread can barely be rolled and a lump cannot be formed when drier than the plastic limit.	4 to 10
Medium	A thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. A lump crumbles when drier than the plastic limit.	10 to 20
High	It take considerable time rolling and kneading to reach the plastic limit. A thread can be rerolled several times after reaching the plastic limit. A lump can be formed without crumbling when drier than the plastic limit.	> 20

ADDITIONAL TERMS

Mottled	Irregular patches of different colors.
Bioturbated	Soil disturbance or mixing by plants or animals.
Diamict	Nonsorted sediment; sand and gravel in silt and/or clay matrix.
Cuttings	Material brought to surface by drilling.
Slough	Material that caved from sides of borehole.
Sheared	Disturbed texture, mix of strengths.

PARTICLE ANGULARITY AND SHAPE TERMS[1]

Angular	Sharp edges and unpolished planar surfaces.
Subangular	Similar to angular, but with rounded edges.
Subrounded	Nearly planar sides with well-rounded edges.
Rounded	Smoothly curved sides with no edges.
Flat	Width/thickness ratio > 3.
Elongated	Length/width ratio > 3.

[1]Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.
[2]Adapted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

ACRONYMS AND ABBREVIATIONS

ATD	At Time of Drilling
Diam.	Diameter
Elev.	Elevation
ft.	Feet
FeO	Iron Oxide
gal.	Gallons
Horiz.	Horizontal
HSA	Hollow Stem Auger
I.D.	Inside Diameter
in.	Inches
lbs.	Pounds
MgO	Magnesium Oxide
mm	Millimeter
MnO	Manganese Oxide
NA	Not Applicable or Not Available
NP	Nonplastic
O.D.	Outside Diameter
OW	Observation Well
pcf	Pounds per Cubic Foot
PID	Photo-Ionization Detector
PMT	Pressuremeter Test
ppm	Parts per Million
psi	Pounds per Square Inch
PVC	Polyvinyl Chloride
rpm	Rotations per Minute
SPT	Standard Penetration Test
USCS	Unified Soil Classification System
q_u	Unconfined Compressive Strength
VWP	Vibrating Wire Piezometer
Vert.	Vertical
WOH	Weight of Hammer
WOR	Weight of Rods
Wt.	Weight

STRUCTURE TERMS[1]

Interbedded	Alternating layers of varying material or color with layers at least 1/4-inch thick; singular: bed.
Laminated	Alternating layers of varying material or color with layers less than 1/4-inch thick; singular: lamination.
Fissured	Breaks along definite planes or fractures with little resistance.
Slickensided	Fracture planes appear polished or glossy; sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps that resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay.
Homogeneous	Same color and appearance throughout.

KS FLAP KIN 50(1)
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Kingman County, Kansas

SOIL CLASSIFICATION AND LOG KEY

June 2022 105834-001

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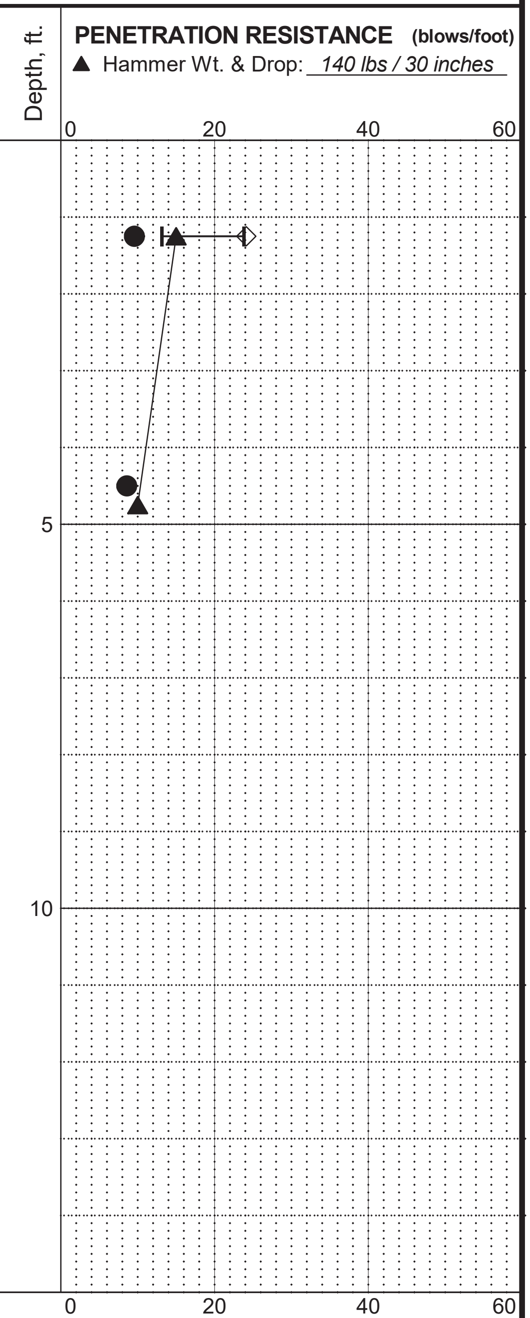
FIG. A-1
Sheet 3 of 3

2013 BORING CLASS3 105834 CFL KS CHENEY RESERVOIR.GPJ SWNEW.GDT 8/25/21

Total Depth:	5.5 ft.	Latitude:	~	Drilling Method:	Solid-Stem Auger
Top Elevation:	~	Longitude:	~	Drilling Company:	GSI
Vert. Datum:		Station:	~	Drill Rig Equipment:	CME 45 Truck
Horiz. Datum:		Offset:	~	Other Comments:	Boring located in EB lane.

Hole Diam.:	6 in.
Rod Type.:	AWJ
Hammer Type:	Automatic

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.*	Depth, ft.	Symbol	Samples	Ground Water
5.5 inches of asphalt pavement.				
Loose to medium dense, brown to red-brown, *Clayey Sandy (SC);* moist; few gravel. [A-2-6]	0.5		B-1 S-1 S-2	Groundwater not encountered during drilling.
BOTTOM OF BORING COMPLETED ON 07/15/2021	5.5			



LEGEND

- * Sample Not Recovered
- G Grab Sample
- Standard Penetration Test

NOTES

1. Refer to Figures A-1 for explanation of symbols, codes, abbreviations, and definitions.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
3. Groundwater level, if indicated above, is for the date specified and may vary.
4. USCS designation is based on visual-manual classification and selected lab testing.

KS FLAP KIN 50(1)
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LOG OF BORING SW-01

June 2022 105834-001

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FIG. A-2

MASTER_LOG_E_POCKETPEN_LAT&LONG 105834 CFL KS CHENEY RESERVOIR.GPJ 8/31/21

QCL 3

Total Depth:	*5.5 ft.*	Latitude:	~	Drilling Method:	*Solid-Stem Auger*	Hole Diam.:	*6 in.*
Top Elevation:	~	Longitude:	~	Drilling Company:	*GSI*	Rod Type.:	*AWJ*
Vert. Datum:		Station:	~	Drill Rig Equipment:	*CME 45 Truck*	Hammer Type:	*Automatic*
Horiz. Datum:		Offset:	~	Other Comments:	*Boring located in WB lane.*		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

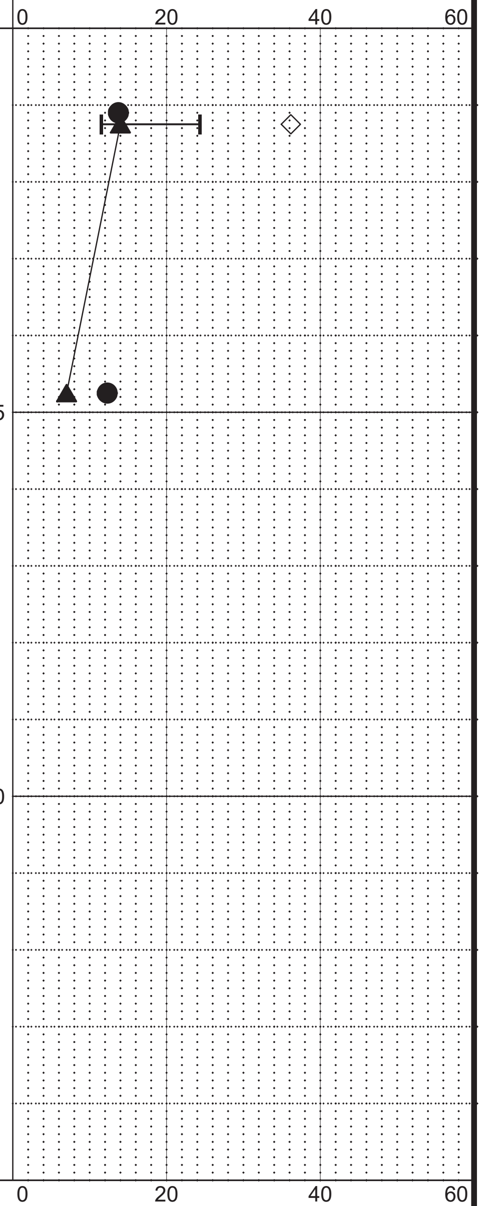
Depth, ft. | Symbol | Samples | Ground Water | Depth, ft.

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*

Soil Description	Depth, ft.	Samples
6.0 inches of asphalt pavement.	0.5	
Loose to medium dense, gray, *Clayey Sand (SC);* moist. [A-6]	5.5	S-1, S-2
BOTTOM OF BORING COMPLETED ON 07/15/2021		

Groundwater not encountered during drilling.



LEGEND

* Sample Not Recovered

Standard Penetration Test

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

KS FLAP KIN 50(1)
Cheney Reservoir Access
Kingman County, Kansas

NOTES

1. Refer to Figures A-1 for explanation of symbols, codes, abbreviations, and definitions.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
3. Groundwater level, if indicated above, is for the date specified and may vary.
4. USCS designation is based on visual-manual classification and selected lab testing.

LOG OF BORING SW-02

June 2022 105834-001

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FIG. A-3

MASTER_LOG_E_POCKETPEN_LAT&LONG 105834_CFL KS CHENEY RESERVOIR.GPJ 8/31/21

Total Depth:	5.5 ft.	Latitude:	~	Drilling Method:	*Solid-Stem Auger*	Hole Diam.:	*6 in.*
Top Elevation:	~	Longitude:	~	Drilling Company:	*GSI*	Rod Type.:	*AWJ*
Vert. Datum:		Station:	~	Drill Rig Equipment:	*CME 45 Truck*	Hammer Type:	*Automatic*
Horiz. Datum:		Offset:	~	Other Comments:	*Boring located in EB lane.*		

SOIL DESCRIPTION

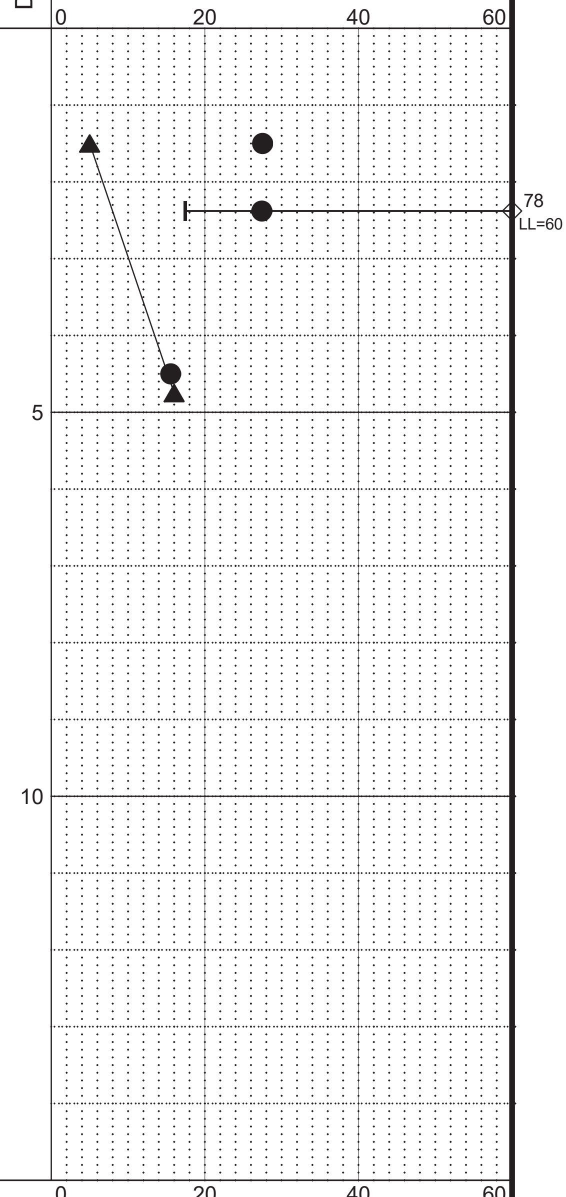
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.	Samples
9.0 inches of asphalt pavement.		
Medium stiff to very stiff, tan, *Fat Clay with Sand (CH);* moist; trace gravel. [A-7-6] -Occasional calcium carbonate nodules. -Tan, *Clayey Sand (SC)* interbeds below 4 feet.	0.8	S-1 B-1 S-2
BOTTOM OF BORING COMPLETED ON 07/15/2021	5.5	

Ground Water: Groundwater not encountered during drilling.

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

LEGEND

* Sample Not Recovered

G Grab Sample

I Standard Penetration Test

NOTES

1. Refer to Figures A-1 for explanation of symbols, codes, abbreviations, and definitions.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
3. Groundwater level, if indicated above, is for the date specified and may vary.
4. USCS designation is based on visual-manual classification and selected lab testing.

KS FLAP KIN 50(1)
Cheney Reservoir Access
Kingman County, Kansas

LOG OF BORING SW-03

June 2022 105834-001

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FIG. A-4

QCL 3

MASTER_LOG_E_POCKETPEN_LAT&LONG 105834_CFL KS CHENEY RESERVOIR.GPJ 8/31/21

Total Depth:	5.5 ft.	Latitude:	~	Drilling Method:	Solid-Stem Auger	Hole Diam.:	6 in.
Top Elevation:	~	Longitude:	~	Drilling Company:	GSI	Rod Type.:	AWJ
Vert. Datum:		Station:	~	Drill Rig Equipment:	CME 45 Truck	Hammer Type:	Automatic
Horiz. Datum:		Offset:	~	Other Comments:	Boring located in WB lane.		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.	Samples
5.0 inches of asphalt pavement.	0.4	
Stiff, brown, *Sandy Lean Clay (CL);* moist. [A-6] -Occasional 2- to 3-inch thick, blue-gray, fat clay seams.	3.0	S-1, B-1
Loose, red-brown, *Clayey Sand (SC);* moist. [A-2-6]	5.5	S-2
BOTTOM OF BORING COMPLETED ON 07/15/2021		

Symbol

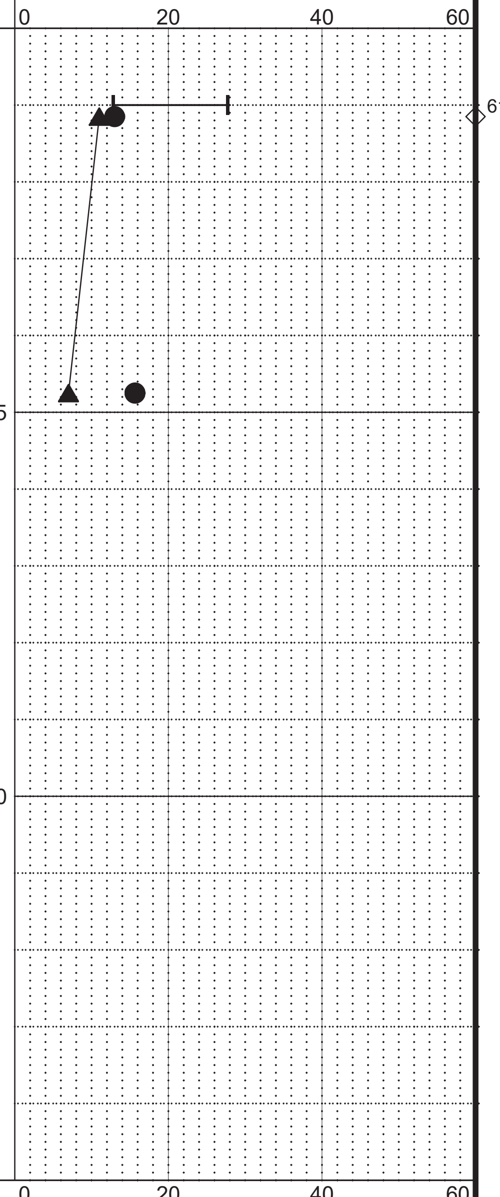
Ground Water: Groundwater not encountered during drilling.

Depth, ft.: 5, 10

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: lbs / inches

140 lbs / 30 inches



LEGEND

* Sample Not Recovered

G Grab Sample

Standard Penetration Test

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to Figures A-1 for explanation of symbols, codes, abbreviations, and definitions.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
3. Groundwater level, if indicated above, is for the date specified and may vary.
4. USCS designation is based on visual-manual classification and selected lab testing.

KS FLAP KIN 50(1)
Cheney Reservoir Access
Kingman County, Kansas

LOG OF BORING SW-04

June 2022 105834-001

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FIG. A-5

QCL 3

Total Depth:	5.5 ft.	Latitude:	~	Drilling Method:	Solid-Stem Auger
Top Elevation:	~	Longitude:	~	Drilling Company:	GSI
Vert. Datum:		Station:	~	Drill Rig Equipment:	CME 45 Truck
Horiz. Datum:		Offset:	~	Other Comments:	Boring located in EB lane.

Hole Diam.:	6 in.
Rod Type.:	AWJ
Hammer Type:	Automatic

SOIL DESCRIPTION

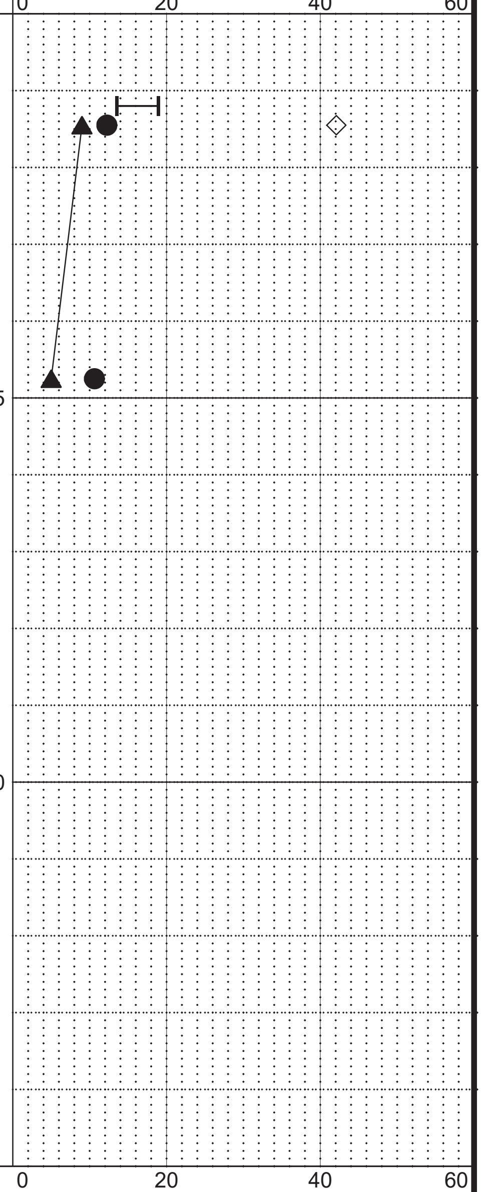
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.	Symbol	Samples	Ground Water
8.0 inches of asphalt pavement.	0.7			Groundwater not encountered during drilling.
Loose, brown to red-brown, *Silty, Clayey Sand (SC-SM);* moist. [A-4] - Trace gravel above 3 feet.	5.5		B-1 S-1 S-2	
BOTTOM OF BORING COMPLETED ON 07/15/2021				

PENETRATION RESISTANCE **(blows/foot)**

▲ Hammer Wt. & Drop: *lbs / inches*

140 lbs / 30 inches



LEGEND

* Sample Not Recovered

G Grab Sample

I Standard Penetration Test

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

KS FLAP KIN 50(1)
Cheney Reservoir Access
Kingman County, Kansas

NOTES

1. Refer to Figures A-1 for explanation of symbols, codes, abbreviations, and definitions.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
3. Groundwater level, if indicated above, is for the date specified and may vary.
4. USCS designation is based on visual-manual classification and selected lab testing.

LOG OF BORING SW-05

June 2022 105834-001

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FIG. A-6

QCL 3

Top of Core

Core SW-01 - Eastbound lane, Measured Thickness of 5-1/2 inches

Top of Core

Core SW-02 - Westbound lane, Measured Thickness of 6 inches

KS FLAP KIN 50(1)
Cheney Reservoir Access
Kingman County, Kansas

PAVEMENT CORE PHOTOGRAPHS

June 2022 105834-001

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FIG. A-7
Sheet 1 of 3

Top of Core

Core SW-03 - Eastbound lane, Recovered Thickness of 5.5 inches, 9 inches cored, Completely Stripped asphalt 3.5 to 6.5 inches.

Top of Core

Core SW-04 - Westbound lane, Measured Thickness of 5 inches

KS FLAP KIN 50(1)
Cheney Reservoir Access
Kingman County, Kansas

PAVEMENT CORE PHOTOGRAPHS

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FIG. A-7
Sheet 2 of 3

Top of Core

Core SW-05 - Eastbound lane, Measured Thickness of 8 inches

KS FLAP KIN 50(1) Cheney Reservoir Access Kingman County, Kansas	
PAVEMENT CORE PHOTOGRAPHS	
June 2022	105834-001
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. A-7** Sheet 3 of 3

KS FLAP KIN 50(1)
Cheney Reservoir Access
Kingman County, Kansas

BORING SW-01 EXPLORATION PHOTOGRAPH

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FIG. A-8

KS FLAP KIN 50(1)
Cheney Reservoir Access
Kingman County, Kansas

BORING SW-02 EXPLORATION PHOTOGRAPH

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FIG. A-9

KS FLAP KIN 50(1)
Cheney Reservoir Access
Kingman County, Kansas

BORING SW-03 EXPLORATION PHOTOGRAPH

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FIG. A-10

KS FLAP KIN 50(1)
Cheney Reservoir Access
Kingman County, Kansas

BORING SW-04 EXPLORATION PHOTOGRAPH

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FIG. A-11

KS FLAP KIN 50(1)
Cheney Reservoir Access
Kingman County, Kansas

BORING SW-05 EXPLORATION PHOTOGRAPH

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FIG. A-12

APPENDIX B: LABORATORY TEST RESULTS

Appendix B
Laboratory Test Results

CONTENTS

Tables

Figures

B.1 INTRODUCTION

Laboratory tests were completed on soil samples retrieved from the borings in general accordance with the American Association of State Highway and Transportation Officials (AASHTO) and American Society of Testing and Materials International International (ASTM) testing methods. The laboratory testing program was performed to classify the materials into similar geologic groups and provide data that can be used for design of the project. The geotechnical laboratory testing was performed by our laboratory in Denver, Colorado, Vine Laboratories, Inc. of Commerce City, Colorado, and Colorado Analytical Laboratories, Inc. of Commerce City, Colorado. A summary of the laboratory test results is presented in Table B-1. The following sections describe the laboratory testing procedures.

B.2 GEOTECHINCAL INDEX TESTS

B.2.1 Water Content

Water content was determined for selected samples in general accordance with AASHTO T265, Laboratory Determination of Moisture Content of Soils. To perform this test, a sample was weighed before and after oven-drying, and the water content was calculated. Water content determinations are shown graphically on the boring logs and are also summarized in Table B-1.

B.2.2 Grain Size Analysis

The grain size distribution of selected samples was determined in general accordance with AASHTO T311, Standard Method of Test for Grain-Size Analysis of Granular Soil Materials. Results of these analyses are presented as grain size distribution curves by boring number series on Figure B-1 and summarized in Table B-1.

Selected samples were tested for the percentage of material passing the No. 200 sieve in general accordance with ASTM D1140, Standard Test Method for Amount of Material in Soils Finer than the No. 200 (75-µm) Sieve. The percent fines (silt- and clay-sized particles passing the No. 200 sieve) are shown graphically on the boring logs in Appendix A and are also summarized in Table B-1.

B.2.3 Atterberg Limits

Soil plasticity was determined by performing Atterberg limits tests on selected fine-grained samples. The tests were completed in general accordance with AASHTO T89, Standard Test

Method for Determining the Liquid Limit of Soils and AASHTO T90, Standard Test Method for Determining the Plastic Limit and Plasticity Index of Soils. The Atterberg limits include liquid limit (LL), plastic limit (PL), and plasticity index (PI equals LL minus PL) and are generally used to assist in classification of soils, to indicate soil consistency (when compared to natural water content), and to provide correlation to soil properties. The results of the Atterberg limits tests are plotted on a plasticity chart on Figure B-2, shown graphically on the boring logs in Appendix A, and summarized in Table B-1.

B.3 GEOTECHNICAL ENGINEERING PROPERTY TESTS

B.3.1 Corrosion

Corrosion testing of select samples was performed for pH, resistivity, sulfate content, and chloride content by Colorado Analytical Laboratories, Inc. Testing for pH was completed in general accordance with AASHTO T289-91, Standard Test Method for Measuring pH of Soil for Use in Corrosion Testing. Resistivity testing was completed in accordance with AASHTO T288-91, Standard Method of Test for Determining Minimum Laboratory Soil Resistivity. Sulfate content testing was completed in general accordance with AASHTO T290-91, Standard Method of Test for Determining Water-Soluble Sulfate Ion Content in Soil. Chloride content testing was completed in general accordance with AASHTO T291-91, Standard Method of Test for Determining Water-Soluble Chloride Ion Content in Soil. Test results for sulfate and chloride content are given in units of percent by weight. The test results are summarized in Table B-1.

B.3.2 R-Value

Hveem Staboilometer (R-value) test was completed by Vine Laboratories, Inc., of Commerce City, Colorado on a bulk subgrade sample. The test was completed in general accordance with AASHTO T190, Standard Method of Test for Resistance R-Value and Expansion Pressure of Compacted Soils. The R-value test results are summarized in Table B-1 and presented on Figure B-3.

Table B-1 - Summary of Laboratory Test Results by Boring

SAMPLE DATA							GRAIN SIZE ANALYSIS[3]			ATTERBERG LIMITS			CORROSION				R-Value	
Boring	Sample	Depth (feet) Top	Depth (feet) Bottom	USCS Symbol[1]	AASHTO Soil Classification and Group Index[2]	Natural Moisture Content (%)	Gravel (%)	Sand (%)	Fines (%)	Liquid Limit	Plastic Limit	Plasticity Index	pH	Resistivity (ohm-cm)	Sulfate Content (%)	Chloride Content (%)	R-Value	Exudation Pressure (psi)
SW-01	S-1	0.5	2.0	SM	A-2-6 (0)	9.6			24	24	13	11						
	S-2	4.0	5.5			8.6												
SW-02	S-1	0.5	2.0	SC	A-6 (1)	13.7			36	24	12	12						
	S-2	4.0	5.5			12.3												
SW-03	B-1	0.8	4.0	CH	A-7-6 (34)	27.4	1	22	78	60	17	43	7.6	820	0.026	0.006	4.1	300
	S-1	0.8	2.3			27.5												
	S-2	4.0	5.5			15.6												
SW-04	S-1	0.4	1.9	CL	A-6 (6)	13.0			62	28	13	15						
	S-2	4.0	5.5			15.6												
SW-05	S-1	0.7	2.2	SC	A-4 (0)	12.2	0	58	42	19	14	5						
	S-2	4.0	5.5			10.6												

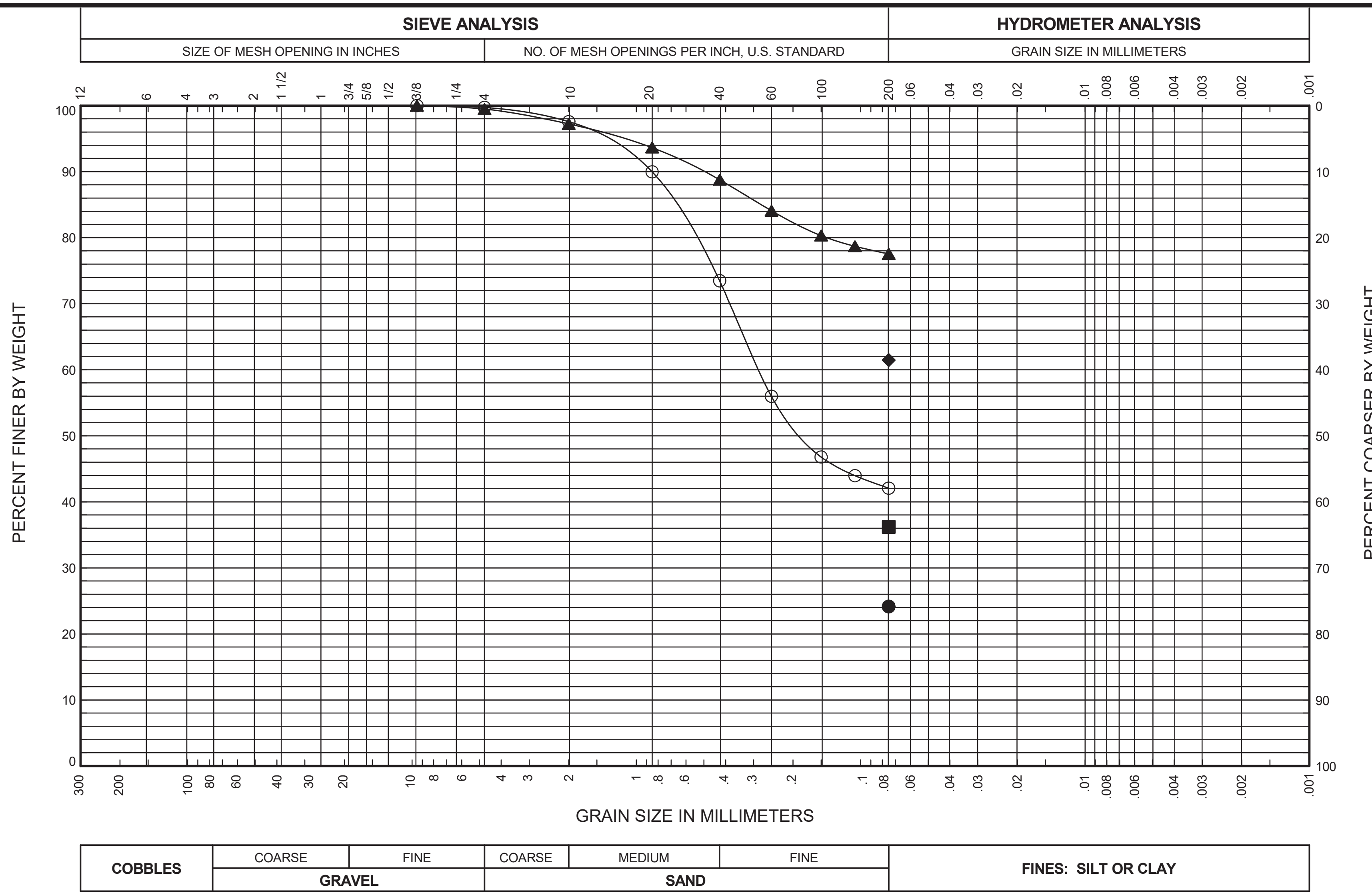
NOTES:

1 Refer to Appendix A, Figure A-1 for definitions.

2 AASHTO soil classification and group index indicated in parenthesis

3 Gravel defined as particles larger than the No. 4 sieve size, Sand as particles between the No. 4 and No. 200 sieve sizes, and Fines as particles passing the No. 200 sieve.

ohm-cm = ohm centimeters; psi = pounds per square inch.



SAMPLE ID	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● SW-01, S-1	0.5	SC	Clayey Sand [A-2-6]	24.1	9.6	24	13	11
■ SW-02, S-1	0.5	SC	Clayey Sand [A-6]	36.2	13.7	24	12	12
▲ SW-03, B-1	0.8	CH	Fat Clay with Sand; trace gravel. [A-7-6]	77.6	27.4	60	17	43
◆ SW-04, S-1	0.4	CL	Sandy Lean Clay [A-6]	61.5	13.0	28	13	15
○ SW-05, S-1	0.7	SC	Clayey Sand [A-4]	42.1	12.2	19	14	5

KS FLAP KIN 50(1)
Cheney Reservoir Access
Kingman County, Kansas

GRAIN SIZE DISTRIBUTION

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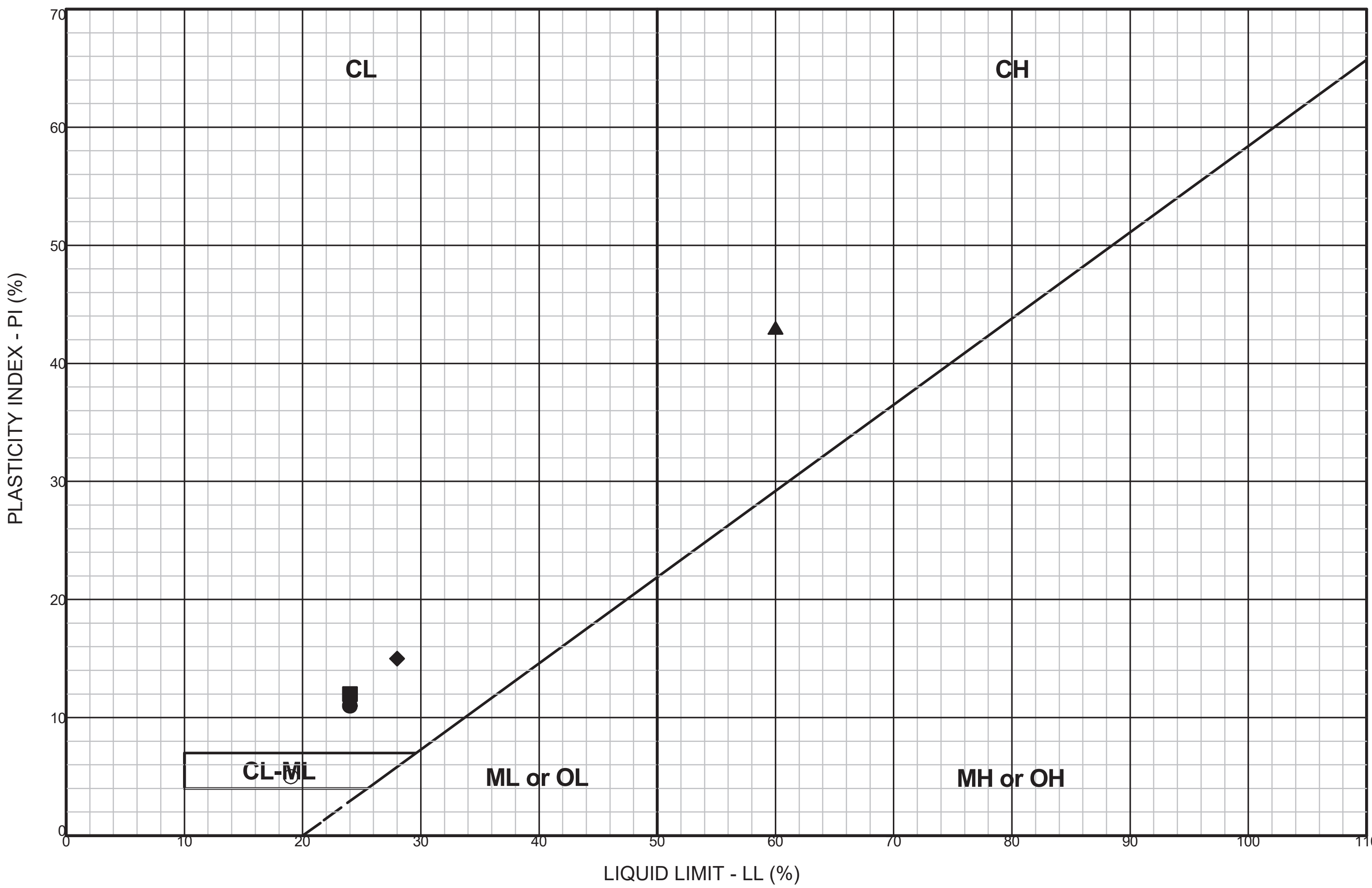
FIG. B-1

HDR

GSA_MAIN 105834 CFL KS CHENEY RESERVOIR.GPJ 8/31/21

FIG. B-1



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

SAMPLE ID	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT. W.C. %	PASS. #200, %
● SW-01, S-1	0.5	SC	Clayey Sand [A-2-6]	24	13	11	9.6	24.1
■ SW-02, S-1	0.5	SC	Clayey Sand [A-6]	24	12	12	13.7	36.2
▲ SW-03, B-1	0.8	CH	Fat Clay with Sand; trace gravel. [A-7-6]	60	17	43	27.4	77.6
◆ SW-04, S-1	0.4	CL	Sandy Lean Clay [A-6]	28	13	15	13.0	61.5
○ SW-05, S-1	0.7	SC	Clayey Sand [A-4]	19	14	5	12.2	42.1

KS FLAP KIN 50(1)
Cheney Reservoir Access
Kingman County, Kansas

PLASTICITY CHART

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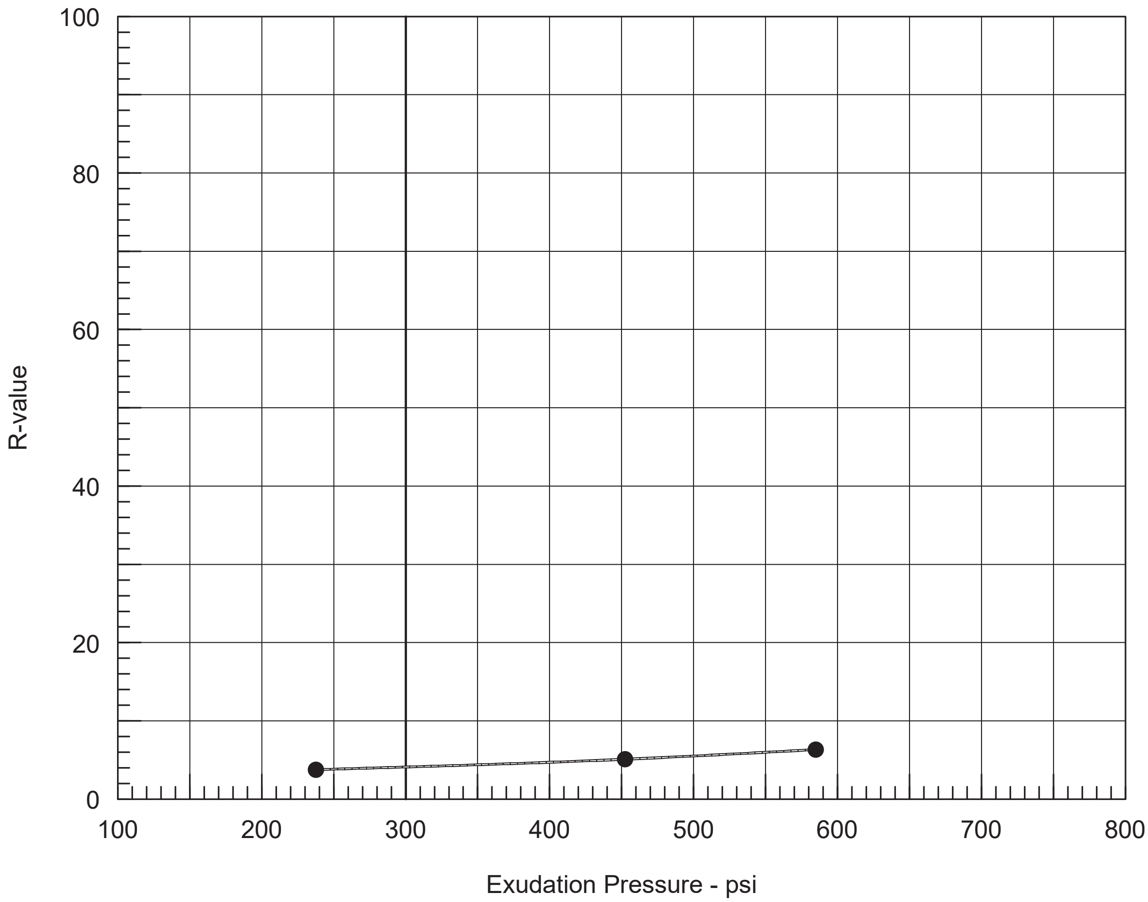
FIG. B-2

HDR

FIG. B-2

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - AASHTO T 190

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	50	91.8	29.2	0.00	147	2.59	452	4.9	5.1
2	60	92.8	28.8	0.00	144	2.56	585	6.1	6.3
3	30	87.2	33.1	0.00	150	2.47	238	3.8	3.8

Test Results	Material Description
R-value at 300 psi exudation pressure = 4.1	SW-03 B-1 Bulk / 0.8-4.0'

Project No.: 105834-001

Project: CFL KS Cheney Reservoir

Location: SW-03 B-1 Bulk / 0.8-4.0'

Sample Number: S2608 **Depth:** 0.8-4.0'

Date: 7/30/2021

Tested by: Juan Romero

Checked by: Clay Hollowell

Remarks:

R-VALUE TEST REPORT

Vine Laboratories

FIG. B-3

APPENDIX D: PAVEMENT DESIGN FILES

Appendix C

Pavement Design Calculations

Exhibits

Exhibit C-1: Flexible Pavement 18-kip [Equivalent Single-Axle Loading (ESAL) Worksheet

Exhibits C-2: Flexible Pavement Design Worksheets

Flexible Pavement 18-kip Equivalent Single-Axle Loading (ESAL) Worksheet

SHANNON & WILSON, INC.

Project No: 105834-001

Location: NE 50 St

Comment: Analysis based on Table D.21 of the 1993 AASHTO Guide for the Design of Pavement Structures

20 year Design Life

Traffic Study Year:	2021	
Paving Year:	2023	
Pavement Design Life (D):	20	years
2021 Two-way Average Daily Traffic (ADT):	650.0	vehicles per day (vpd)
2023 ADT:	676.3	vpd = 2021 ADT (1+r/100)^2
Estimated 2043 ADT:	1,005	vpd = 2021 ADT (1+r/100)^22
Growth Rate (r) :	2.00	%

Equations

b = 2023 ADT * (a/100)

c = b * 365

d = [(1+r/100)^20-1]/(r/100)

e = c * d

g = e * f

j = g * h * i

	a	b	c	d	e	f	g	h	i	j
FHWA Vehicle Classification and Description	**Traffic Percentage**	**2023 ADT**	**2023 Total Traffic**	**Growth Factors**	**20 yr Design Traffic Volume (total two-way volume)**	**Flexible Pavement Equivalency Factor**	**Roadway Design 18k ESAL**	**Directional Distribution Factor**	**Traffic Lane Factor**	**Design Lane 18k ESAL**
1. Motorcycles	0	0	0	24.30	0	0	0	0.60	1.0	0
2. Passenger Cars	80.0	541.0	197,468	24.30	4,797,951	0.0006	2,879	0.60	1.0	1,727
Pickup, Single Axle Boat Trailer	17.5	118.3	43,196	24.30	1,049,552	0.0180	18,892	0.60	1.0	11,335
3. Pickups, vans	0	0	0	24.30	0	0.0022	0	0.60	1.0	0
4. Busses	0	0	0	24.30	0	1.250	0	0.60	1.0	0
5. 2-axle, 6-tire Single-Unit Truck	0	0	0	24.30	0	0.50	0	0.60	1.0	0
6. 3-axle, Single-Unit Truck (Delivery)	1.0	6.8	2,468	24.30	59,974	0.75	44,981	0.60	1.0	26,989
Large RV	1.25	8.5	3,085	24.30	74,968	0.50	37,484	0.60	1.0	22,490
Trash Truck	0.25	1.7	617	24.30	14,994	1.50	22,490	0.60	1.0	13,494
7. 4 or more axle Single-Unit Truck	0	0.0	0	24.30	0	1.50	0	0.60	1.0	0
8. 4 or less axle Single-Trailer Truck	0	0	0	24.30	0	1.75	0	0.60	1.0	0
9. 5 axle Single-Trailer Truck	0	0	0	24.30	0	2.15	0	0.60	1.0	0
10. 6 or more axle Single-Trailer Trucks	0	0	0	24.30	0	2.15	0	0.60	1.0	0
11. 5 or less axle Multi-Trailer Truck	0	0	0	24.30	0	3.00	0	0.60	1.0	0
12. 6 axle Multi-Trailer Truck	0	0	0	24.30	0	3.00	0	0.60	1.0	0
13. 7 or more axle Multi-Trailer Truck	0	0	0	24.30	0	3.00	0	0.60	1.0	0
Total	100.0	676	246,835		5,997,439		126,726			**76,035**

Calculated ESAL	77,000
Design ESAL	77,000

Notes

1. The ADT is from 2018 traffic study provided by HDR.
2. Exhibit 11.2-A of the Federal Lands Highway, Project Development and Design Manual (PDDM), provides common truck factor ranges for each FHWA vehicle classification. The average value was assumed in our analysis.
3. The minimum design life for an R-3 project is 20 years based on the PDDM
4. Traffic distribution for NE 50 St. (percent of passenger cars, single unit trucks, vehicles with boats, large RVs, and trash trucks) were assumed.

Flexible Pavement Design Worksheet

Job No.: 105834-001
Location: NE 50 St. (NE 150 Ave. to N 407 St. W.)
Comment: Analysis follows design procedures form 1993 AASHTO Guide for Design of Pavement Structures
Alt. 4, HMA over FDR

1. Pavement Design Life: (PDDM) for Reconstruction Projects — 20.0 years

2. Traffic Loading (W_{18}): (PDDM minimum) — ESALs: 77,000 per lane

3. Serviceability:
   - p_0: 4.2 (PDDM)
   - p_t: 2.5 (PDDM) for ADT between 500 and 5,000 vehichles per day
   - ΔPSI: 1.7

4. Subgrade Resilient Modulus (M_R):

   Eqn 1.5.3 of the 1993 AASHTO Guide for Design of Pavement Structures
   M_R = 1,000 + 555 (R-value) = 3,276 psi

   R-value 4.1 — M_R: 3,276 psi

5. Reliability:
   R: 75 % (PDDM) for ADT less than 2500 vehicles per day — Z_R: -0.674

6. Design Standard Deviation (S_o):
   S_o: 0.49 (PDDM)

7. Required Structural Numbers (SN_i):

Analysis M_R		
30,000	SN_1:	**1.149**
3,276	SN_2:	**2.845**
-NA-	SN_3:	**-NA-**

$$\log_{10}(W_{18}) = Z_R\ S_o + 9.36\ \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32\ \log_{10}(M_R) - 8.07$$

Layer Analysis

8. Pavement Materials Characterization:

Layer	Material	Structural Layer Coefficients	Drainage Coefficients	Layer Modulus (psi)
1	ACP (Sec. 403)	a_1: 0.39	-	-
2	FDR	a_2: 0.12	m_2: 1.00	30,000
3		a_3:	m_3:	

9. Solutions for Thicknesses: [Figure 3.2, Part II of 1993 AASHTO]

$SN^*_1 = a_1D^*_1 >= SN_1$

$SN^*_2 = a_1D^*_1 + a_2D^*_2m_2 >= SN_2$

$SN^*_3 = a_1D^*_1 + a_2D^*_2m_2 + a_3D^*_3m_3 >= SN_3$

Recommended Thicknesses				
Layer	Material	Thickness (D^*_i)	SN^*_i	SN_i
1	ACP	5.5 inches	2.145	1.149
2	FDR	6.0 inches	2.865	2.845
3		inches		

Note: Required SN <= Pavement SN, Design is Acceptable

Flexible Pavement Design Worksheet

Job No.: 105834-001
Location: NE 50 St. (NE 150 Ave. to N 407 St. W.)
Comment: Analysis follows design procedures form 1993 AASHTO Guide for Design of Pavement Structures
Alt. 2, HMA over FDR with Cement

1. Pavement Design Life: (PDDM) for Reconstruction Projects — 20.0 years

2. Traffic Loading (W_{18}): (PDDM minimum) — ESALs: 77,000 per lane

3. Serviceability:
   - p_0: 4.2 (PDDM)
   - p_t: 2.5 (PDDM) for ADT between 500 and 5,000 vehichles per day
   - ΔPSI: 1.7

4. Subgrade Resilient Modulus (M_R):

   Eqn 1.5.3 of the 1993 AASHTO Guide for Design of Pavement Structures
   M_R = 1,000 + 555 (R-value) = 3,276 psi

   R-value 4.1 — M_R: 3,276 psi

5. Reliability:
   R: 75 % (PDDM) for ADT less than 2500 vehicles per day — Z_R: -0.674

6. Design Standard Deviation (S_o):
   S_o: 0.49 (PDDM)

7. Required Structural Numbers (SN_i):

Analysis M_R		
30,000	SN_1:	**1.149**
3,276	SN_2:	**2.845**
-NA-	SN_3:	**-NA-**

$$\log_{10}(W_{18}) = Z_R\ S_o + 9.36\ \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32\ \log_{10}(M_R) - 8.07$$

Layer Analysis

8. Pavement Materials Characterization:

Layer	Material	Structural Layer Coefficients	Drainage Coefficients	Layer Modulus (psi)
1	ACP (Sec. 403)	a_1: 0.39	-	-
2	FDR with Cement	a_2: 0.17	m_2: 1.00	30,000
3		a_3:	m_3:	

9. Solutions for Thicknesses: [Figure 3.2, Part II of 1993 AASHTO]

$SN^*_1 = a_1D^*_1 >= SN_1$
$SN^*_2 = a_1D^*_1 + a_2D^*_2m_2 >= SN_2$
$SN^*_3 = a_1D^*_1 + a_2D^*_2m_2 + a_3D^*_3m_3 >= SN_3$

Recommended Thicknesses				
Layer	Material	Thickness (D^*_i)	SN^*_i	SN_i
1	ACP	4.0 inches	1.560	1.149
2	FDR with Cement	8.0 inches	2.920	2.845
3		inches		

Note: Required SN <= Pavement SN, Design is Acceptable

IMPORTANT INFORMATION

Important Information

About Your Geotechnical Report

IMPORTANT INFORMATION

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors that were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining

IMPORTANT INFORMATION

your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary, because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland.