Kansas Department of Transportation

nc #5
SM
8-31-01

MEMO TO: Jim L. Kowach, P.E.,
Chief, Bureau of Design

ATTN: Kenneth F. Hurst, P.E.
Engineering Manager, State Bridge Office

FROM: Delmar Thompson, P.G.,
Regional Geologist, Lawrence

DATE: August 23, 2001

SUBJECT: Bridge Foundation Geology Report

RE: Project 24-75 K-7642-01
Bridge Widening Project
US-24 over the Vermillion River
Bridge 24-75-18.8 (008)
6.1km (3.8mi.) east of K-99 Junction
Pottawatomie County

Three copies of the above report are attached to this memorandum. An Engineering Geology Bridge Sheet has been drawn for this bridge on the Microstation Workstation. This file has been placed on the Design file server under the filename Dt06ft05/Geology/76421880.dgn. Two copies of the drill sounding logs are attached to this report.

If questions arise over the contents of this report, please contact the Lawrence Regional Geology Office.

LSI:GNC:GRK:DLT:jmc
Attachments
cc: Bureau of Construction and Maintenance
District I
Regional Geology Offices
Project File

BUREAU of MATERIALS and RESEARCH

GEOTECHNICAL UNIT
GEOLOGY SECTION

BRIDGE FOUNDATION GEOLOGY REPORT

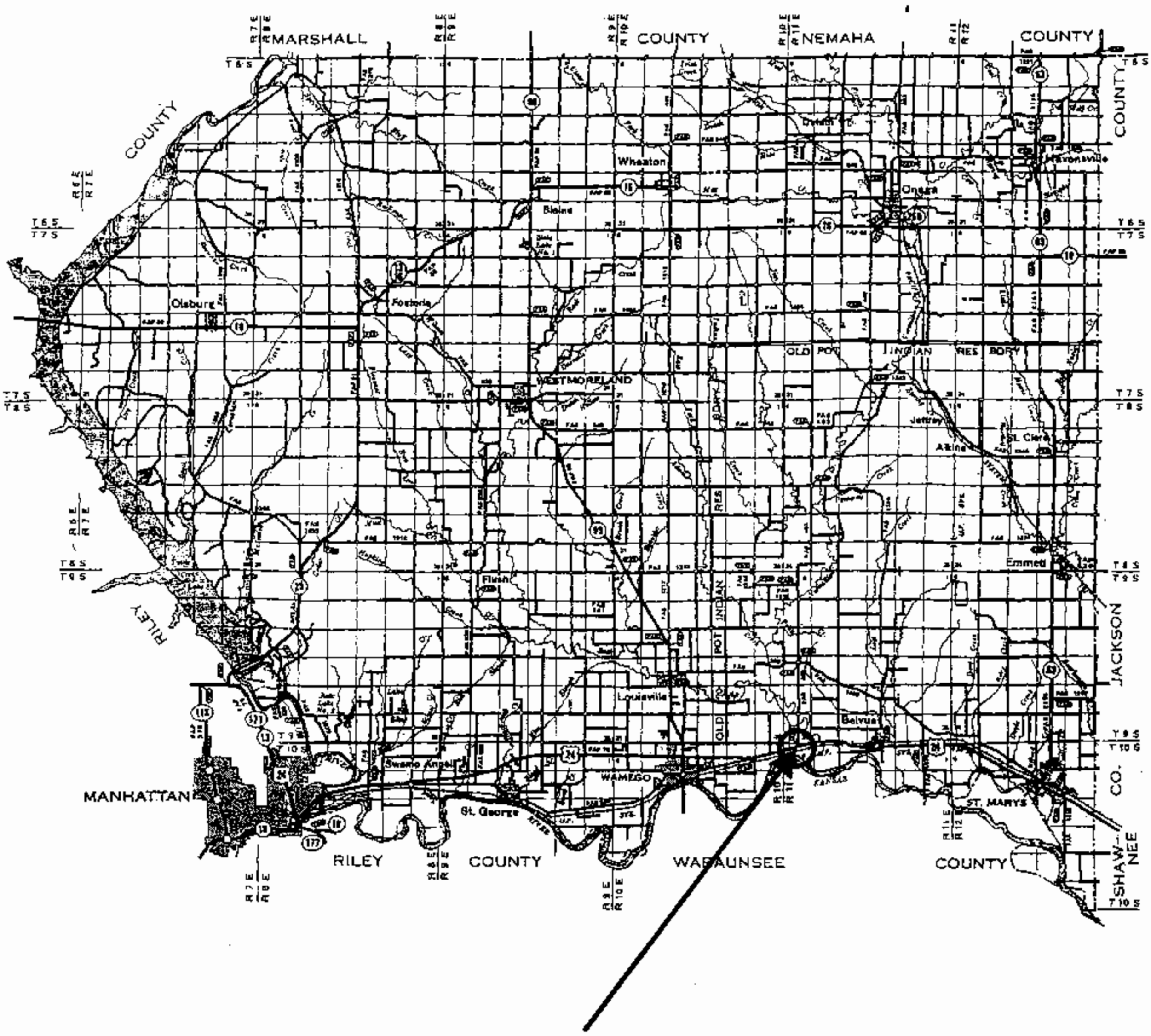
24-75 K-7642-01
Bridge Widening Project
US-24 over the Vermillion River
Bridge No. 24-75-18.80 (008)
6.1km (3.8mi.) east of K-99 Junction
Pottawatomie County

GARY R. KOONTZ, P.G.
CHIEF GEOLOGIST

BY

Randy Billinger, P.G., Geologist II
Delmar Thompson, P.G., Regional Geologist

August, 2001



24-75 K-7642-01
US-24 over the Vermillion River

Br. No. 24-75-18.80 (008)
Pottawatomie County

INTRODUCTION

This report details the geologic setting and footing recommendations associated with the proposed bridge widening on US-24 over the Vermillion River. The bridge site is located 6.1 km east of the K-99 and US-24 Junction. This junction lies at the north edge of the city of Wamego. The project consists of redecking and widening the bridge 2.74 m on each side. The bridge spans the Vermillion River just north of the confluence of the Vermillion River with the Kansas River.

GEOLOGY AT THE BRIDGE SITE

Soil Mantle

The soil mantle at this bridge location consists of all unconsolidated material above bedrock. At the abutment locations, the soil mantle is composed of alluvial material approximately 15.0 m thick which consists of silty clay, gravel, and sand. The upper 2.5 m of the mantle consists of gray silty clay with some light gravel. Below this layer, is a heavy gravel bed approximately 2.5 m thick. Below this gravel bed, the alluvium consists of gray, sandy, silty clay with light gravel. The mantle material grades to coarse sand with small gravel in the lower 4.5 to 5.0 m above the bedrock interface.

At the pier locations, the mantle consists of sandy, silty clay and coarse sand with small gravel. The total thickness of the mantle at the pier locations varies from 5.0 to 9.0 meters.

Permian System
Admire Group
Janesville Shale Formation
Hamlin Shale Member

We believe that the bedrock unit that will be encountered at this bridge site is the Hamlin Shale Member. A much larger scale and more involved investigation would be necessary to positively confirm the shale unit's identity. It is possible that the member is misidentified; however, for construction purposes, the identity of this shale member will make no difference.

The Hamlin Shale Member consists of gray, green, and maroon, sandy shale, thin sandstone lenses, and a thin limestone bed. The upper 6.7 to 7.0 m of this shale member consist of gray, firm, clayey to sandy shale. Below this portion of the member is a thin, gray, fossiliferous limestone bed approximately 0.3 m thick. Underlying this limestone bed, is a green and maroon, sandy shale, with thin sandstone lenses. One sandstone bed was noted as being approximately 0.5 m thick. Approximately 3.4 m of the lower portion of the Hamlin Member was cored. The total thickness of the Hamlin Shale Member is over 11.0 meters.

FOUNDATION RECOMMENDATIONS

Abutment Foundations
Pile Footings

General Notes on Pile Foundations

The following notes should be applied to the construction of the proposed bridges.

1) Loads on individual piles should not exceed 55 tons, 490 kN, or 9 ksi for a 10 X 42 H-pile.

2) Because slight variations in the final pile tip elevations may occur, the following note should be placed in the Construction Plans:

Pile Note:
"Final pile tip elevations should be established by the field engineer, based on observed blow counts. Once bearing has been obtained within the bedrock, driving should cease to avoid damage to the pile."

Notes regarding Predrilled Piles (from the Standard Specs. page 431 metric version)

The following notes should be applied to all predrilled piles utilized on this project.

Pre-drilled holes shall be drilled to an accuracy that will permit the pile to be set in its true position and not tilted in such a manner as to drive the pile out of position.

All pre-drilled holes shall be drilled to the depth as shown on the plans, unless otherwise directed by the Engineer in charge.

After the pile has been driven to final position in the pre-drilled boring, the annular space around the pile should be completely filled with loose sand or the type of material as specified in the plans.

Abutment 1, Station 7+034.798
Abutment 2, Station 7+168.910

We recommend driven H-piles at the abutment locations to ensure penetration through the sand and gravel and maximum penetration into the Hamlin Shale Member. The piles should be driven to bearing at design tip elevations 277.4 at Abutment 1 and elevation 276.7 at Abutment 2. These elevations will allow for 1.8 meters of penetration into the shale bedrock material.

At this location, the Hamlin Shale Member is firm and gray at the mantle bedrock contact. Unconfined compression test results indicate that the upper shale of this member has an unconfined strength of 2319.3 kPa. Because of this, we predict the piles will obtain bearing near the mantle bedrock contact with only minimal penetration into the shale. If the piles obtain bearing at a higher elevation within the Hamlin Shale Member, some pile cut off will be required.

Pier Foundations
Drilled Shaft Footings

We recommend that drilled shaft footings be utilized at the pier locations of this bridge widening project. Due to the close proximity of the new footings to the existing footings, it is thought that drilled shafts can be placed with less disturbance to the existing footings. The shafts should be constructed with care, so as not to damage the existing footings. The shaft should be advanced slowly and special care should be observed to ensure the shaft is constructed as straight as possible.

Pier 1, Station 7+065.278
Pier 2, Station 7+101.854
Pier 3, Station 7+138.430

Deep Drilled Shaft Footing Option

We recommend a drilled shaft socket elevation of 270.0 for the shafts at Pier 1, Pier 2, and Pier 3. These elevations will place the drilled shaft sockets approximately 8.4 to 8.8 m into the Hamlin Shale Member. A maximum footing pressure of 1437 kPa (15 tsf) should not be exceeded.

We recommend placing permanent casing to an elevation of 277.0. This will place the casing below the existing footing elevations and help to protect them during the construction of the new drilled shaft footings.

Shallow Drilled Shaft Footing Option

If the designer feels that a shallower drilled shaft is feasible, the drilled shaft socket elevations can be raised. However, shaft embedment into the bedrock will be reduced with a shallow socket elevation, and may be cause for concern should an abnormal scour event occur. With shallower socket design, lateral loading may be of concern during high water events which could result in large amounts of debris collecting at the pier locations causing a damming effect of the river. Information for shallow socket design elevations are as follows should this option be considered.

If shallow socket elevations are to be designed, we recommend a drilled shaft socket elevation of 275.0 for the shafts at Pier 1, Pier 2, and Pier 3. These elevations will place the drilled shaft sockets approximately 3.4 to 3.8 m into the Hamlin Shale Member. A maximum footing pressure of 862 kPa (9 tsf) should not be exceeded.

Pile Footing Option

The designer has requested an option for pile footings at the pier locations similar to those in place on the existing bridge structure. The current stream channel profile on the Vermillion River indicates that approximately 6.0 m of scour has occurred since the bridge was built in 1945. Due to the high probability of continued scour at the pier locations, a series of driven H-pile surrounded by protective driven sheet piling can be utilized as a footing option at the piers. This will eliminate the need for a deep excavation to construct pile caps. Details for the combination driven H-pile/driven sheet pile design is given below.

It is recommended that a series of H-pile be driven to bearing in the shale of the Hamlin Member at each of the widening locations. Once the H-piles are in place, sheet piling should be driven to bearing, again into the Hamlin Shale Member, around each H-pile grouping in a way that completely encloses the grouping, protecting the piles from scour. The enclosed area should be filled and compacted to the pile cap elevation with sand or material indicated by the designer. A pile cap can then be built in such a way that incorporates both the H-piles and the sheet piling into the design of the cap. Bridge support columns will be built into and on top of this cap. This type of footing design is in place at the existing pier locations on this bridge. Recommended bearing elevations for the H-pile and the sheet piling are listed in the table below.

Location	Station	Approximate Bedrock Contact Elevation	Recommended H-pile Design Bearing Elevation	Recommended Sheet Piling Elevation
Pier 1	7+065.278	278.8	277.0	277.6
Pier 2	7+101.854	278.5	276.6	277.3
Pier 3	7+138.430	278.4	276.6	277.3

These design elevations allow for approximately 1.8 m (6 feet) of H-pile penetration into the shale and approximately 1.2 m (4 feet) of sheet pile penetration into the shale. We anticipate that neither the H-pile nor the sheet pile will penetrate into the shale to this extent and will more likely obtain bearing above the listed elevations. However, we are allowing extra pile length to account for minor variances in the bedrock contact elevation across the pier locations, or in the event that piles penetrate deeper than expected. If the piles obtain bearing at a higher elevation within the Hamlin Shale Member, some pile cut off will be required.

Lateral Load Parameters

Soil and rock parameters for laterally loaded pile design are as follows:

NPY=3 NPPY= 3 b = 5.0 ft. diameter shaft

Soil Mantle

Silty clay, brown.
Soft clay. Effective Unit weight = 53.713 pcf
Sample is below the water table.

Average Qu	0.27 tsf	C1 = 0.027 ksf
Average Dry Weight	96.2 lbs/ft^3	EE50 = 0.20
Average Moisture	20.7 %	K = 864.00 kcf
GAM1	0.054 kcf	

Hamlin Shale Member
Shale, gray, sandy.
Elevation 279 to 273.

		YP(I,J)	PP(I,J)
Average Qu	18.8 tsf		
Average Dry Weight	128.0 lbs/ft^3	0.0000	0.0000 k/ft
Average Moisture	12.1 %	0.0350	52.640 k/ft
GAM1	0.143 kcf	0.1017	94.000 k/ft

Hamlin Shale Member
Shale, gray, sandy.
Elevation 273 to 270.2.

		YP(I,J)	PP(I,J)
Average Qu	6.16 tsf		
Average Dry Weight	129.0 lbs/ft^3	0.0000	0.0000 k/ft
Average Moisture	11.5 %	0.0350	17.248 k/ft
GAM1	0.144 kcf	0.1017	30.800 k/ft

Hamlin Shale Member
Sandstone, green and maroon.
Elevation 270.2 to 269.0.

		YP(I,J)	PP(I,J)
Average Qu	452.7 tsf		
Average Dry Weight	141.5 lbs/ft^3	0.0000	0.0000 k/ft
Average Moisture	6.5 %	0.0020	1810.80 k/ft
GAM1	0.151 kcf	0.0120	2263.50 k/ft

Hydrology

The groundwater elevation at the proposed bridge location during our geologic investigation in June 2001 was measured and recorded at elevation 289.77. Any excavation below the ground water table will require sheeting and dewatering equipment.

Investigative Procedure

Information from two power auger soundings, and one core drill sounding at the proposed bridge location, as well as information from the existing as-built bridge plans, were used to develop the foundation geology at this bridge site. Selected samples of the cores were submitted to our testing facility for unconfined compression testing. The results of these tests and logs of the soundings are included with this report.

Acknowledgments

The following individuals should be recognized for their help in conducting the foundation investigation for this project: Rob Vervynck, ET. Senior; and Willard Trout, ET.