Montana Department of Transportation
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Memorandum

To: Nigel Mends, P.E.
Missoula Bridge Area Engineer

From: Jeff Jackson, P.E.
Geotechnical Operations Manager

Richard Jackson, P.E.
Geotechnical Engineer

Date: December 3, 2008

Subject: Geotechnical Engineering Structure Report (466)
Lonepine N&E
STPP-F 36-1(4)26
UPN 1289

A recommended foundation design was originally provided by The Geotechnical Section in a memorandum dated August 20, 1999 and recommended 14 inch fluted piles for support of the bridge foundation. We are providing this current memorandum to update recommendations to the 1999 memorandum for the bridge foundation based upon additional design analyses and recent experience of pile driving in soft glacial lake Missoula sediments. The original report from 1999 recommended one pile at each bent should be driven and monitored with a PDA, a re-strike performed, and the production piling lengths would then be ordered based upon capacities obtained from the re-strike information. This report also assumed that embankments would be constructed and allowed to settle for two months prior to pile driving. The original design used a factor of safety of 2.0 for the pile capacity. This factor of safety is only recommended by FHWA if a static compression load test is performed. Higher factors of safety are required if pile capacity during construction is estimated by dynamic testing (PDA) or wave equation analysis. Based upon these factors we are providing updated recommendations for the bridge.

This report includes the results of the subsurface exploration, laboratory tests, analyses, and geotechnical recommendations in relation to the design of the bridge foundation.

Project Location and Information

The project is located on MT Hwy 28 in Sanders, Lake and Flathead Counties. The project begins at about RP 26.0 just north of Lonepine and extends north to Niarada and then curves east about 2 miles to RP 35.8. The project is located within the Flathead Indian Reservation.

The scope of the project is to reconstruct approximately 9.8 miles of MT Hwy 28. The road is severely deteriorated and will be upgraded to meet modern geometric standards. The work includes grading, gravel surfacing, plant mix surfacing, culverts, the replacement of one bridge over the Little Bitterroot River and miscellaneous other features.

The proposed new bridge on the Project crosses the Little Bitterroot River at approximate Station 64+00 (R.P. 26.5). It will replace an existing timber bridge with a new single span concrete bridge which will be about 82 feet long. The new bridge will be set so that it has a low beam elevation of 2763 ± feet and it will be located within a vertical curve.

Physiography and Geology.

The section of Hwy 28 from Lonepine north to Niarada traverses the Little Bitterroot River Valley west of the Salish Mountains. Northeast of Niarada, the highway follows the Big Draw, a pre-glacial valley of the Flathead River (Hyndman, 1986). The entire length of the project crosses over deep (100-300 feet thick) silt and clay sediments that settled out of the Glacial Lake Missoula Water column during the last ice age. The silt and clay deposits are exposed in the cuts along the highway. Erosion of the sediments has given the area badlands topography. The land adjacent to the highway is primarily used for grazing.

The site lies within the Intermountain Seismic Belt and is mapped as a zone of potentially moderate seismic ground movement. Based on the fine grained nature of the soils, we anticipate that the risk of liquefaction or of lateral spreading is low in most areas. Earthquake induced slope movement at the bridge ends is possible due to loss of strength during dynamic loading in the fine grained soils. The cost of mitigating this condition is usually excessive and we anticipate that some minor slope deformations or settlement at the bridge approaches will be acceptable in the event of a seismic event.

Subsurface Conditions

Subsurface soil conditions at the bridge generally consist of interbedded silt and clay soils. Consistency of the soils ranged from very soft to very stiff and are considered highly compressible when subject to loading from the approach embankment. A relatively hard or very dense stratum was not encountered during the investigation nor was any appreciable amount of coarse grained sand or gravel.

Groundwater levels at the time of our investigation ranged from near the ground surface to approximately 10 feet below the ground surface. Our subsurface investigation was performed in late February 1999 and groundwater levels are likely higher during the spring and early summer months.

Additional subsurface information is provided on the attached boring logs.

Bridge Foundation Design and Construction Recommendations

Design parameters

Based upon the predominant soft soils present, a deep foundation consisting of driven piles is recommended. The capacity of the piles will mostly be obtained through skin friction along the length of the pile.

Potential scour at the bridge abutments is to be mitigated with the placement of riprap at both abutment locations.

The soils at the bridge location are highly compressible when subject to loading from the approach embankments. The amount of settlement expected to occur based upon the proposed fill height, ground water elevations, and corresponding stress distribution at depth, will be large enough to mobilize negative skin friction along the piles to depths of approximately 30 feet.

Negative skin friction (also called negative shaft resistance or drag load) will increase the loading on the piles as the subsurface soils settle in relation to the pile. This settlement results in increased loading on the pile, thus resulting in larger piles, longer piles, and higher required ultimate capacities during driving.

To somewhat reduce the potential negative skin friction, we recommend constructing the approach embankments prior to driving the piles and the design recommendations contained within the rest of this report assume that the approach embankments will be constructed, the subsurface soils allowed to settle for a minimum of 30 days (and longer if possible), and the piles will be driven through the approach embankments. It should be noted that this will reduce some of the negative skin friction and potential for settlement, but due to the extent of the soft soils, significant long term negative skin friction is still expected.

In the event the approach embankments cannot be constructed and allowed to settle prior to pile driving for unforeseen reasons at this time, piles driven to greater depths and higher ultimate capacities during driving will be required to resist the down drag loads, larger diameter piles may also be required. The Geotechnical Section should be notified to re-assess our recommendations in the event the approach embankments can not be constructed and subsurface soils allowed to settle prior to pile driving.

Bridge Foundation Recommendations

Loading Information:

Preliminary axial loads (per abutment) were provided by the Bridge Bureau via email on August 1, 2008 for this bridge. Based upon the information provided each abutment is expected to experience 438 kips of dead load and 177 kips of live load for a total of 615 kips.

These loads are unfactored service loads. The existing bridge layout (using the previously recommended fluted piles) includes 8 piles per bent.

Based upon information provided in your email dated October 15, 2008, we understand that AASHTO does not require lateral analyses resulting from a seismic event for a single span bridge such as the one on this project. Therefore, the Geotechnical Section has not evaluated the lateral capacity of the piles.

Bridge Foundation:

Based upon the factors listed at the beginning of this memorandum the fluted piles are not adequate as designed.

Fluted piles are a proprietary pile type, are very difficult to splice in the field, and predicting the available capacity has more uncertainty than a conventional pipe pile, therefore we do not recommend using fluted piles on this project. These types of piles are ordinarily more applicable for very large projects where hundreds of pile might be required and these piles can then be economical by driving and load testing some indicator piles after which the production pile can be ordered to minimize field splicing and production driving begin.

Instead of fluted piles we recommend the use of conical tip closed end pipe piles to support structural loads. Based upon recent experience with the difficulty of obtaining adequate capacity from highly loaded friction piles in Glacial Lake Missoula sediments, we strongly recommend using 8 piles per bent as shown in the plans. Using fewer piles increases the chances of having the piles not obtain capacity or "running" beyond the design tip elevation as they will need to be driven to higher capacities in the poor soils to support higher loads.

Due to the soft soils present, potential for negative skin friction, and length of piles that will be required to support loading conditions, we recommend that a compression static load test be performed on this bridge. Performing a load test allows the use of lower factors of safety as recommended by FHWA (FOS = 2.0) versus using a FOS of 2.75 when wave equation analyses are used to accept pile capacity. The use of a lower factor of safety ultimately leads to using shorter piles as capacities do not need to be as high. The additional cost of the load test can often offset the associated costs of longer piles. Recent PDA testing in similar soil conditions has produced results that have been questioned by both the Geotechnical Section and Construction Personnel. The static load test can also be used to verify the PDA pile capacity predictions which are indirect estimates of capacity. Performing static load tests is the only way to verify the pile capacity and helps to refine pile design methods in difficult soil conditions such as the fine grained glacial lake Missoula sediments.

The Department has not performed any load tests in recent years, but it should be noted that a static load test is being planned for a project in the Glendive District to be let in early 2009. General information obtained from this project (costs, scheduling, any issues, etc.) can be used to refine the load test requirements for this project.

The Minimum Required Axial Ultimate Capacity During Driving and design tip elevations are provided in the following Table. These capacities and design tip elevations assume that a static load test will be performed and 8 piles per bent will be used. In the event a load test is not performed, the piles will need to be driven to greater depths and to higher ultimate capacities.

Bent	Pile Type and Size	Approximate Bottom of Pile Cap Elevation (ft)	Design Pile Tip Elevation** (ft)	Minimum Required Axial Ultimate Pile Capacity During Driving* (kips)
1	14 x 0.5 inch	2761.2	2668	225
2	14 x 0.5 inch	2759.8	2672	225

* This is the minimum required capacity during initial driving, the static load test will be used to verify that adequate set-up has occurred and the required additional capacity is obtained.
** If production piling is ordered after the load test is performed, the results of the load test will be used to determine the pile tip elevation.

Pile spacing should be a minimum of 3 pile diameters center to center. The standard note indicating a 24-72 hour time window for restrike should be modified to indicate that a minimum of 7 days should elapse before performing a restrike. This is to allow the piles to set up and gain capacity as excess pore water pressures dissipate in the fine grained soils.

Pile Driveability

Pile driveability analyses were performed using wave equation analyses assuming the use of a manufactures recommended driving system for a given hammer size typical of hammers used on MDT projects. Our driveability analyses indicates the piles can be driven to design tip elevations without overstressing the piles.

Compression Static Load test

Allowable Stress Design (ASD) methodology allows for lower factors of safety and Load Resistance Factor Design (LRFD) methodology allows the use of higher resistance factors when static load tests are performed, this would result in pile sizes that are smaller or shorter due to the modified factors of safety/resistance factors that are required. FHWA has mandated the use of LRFD for foundation design for all projects where preliminary engineering (i.e. PFR stage of a project) was initiated after October 1, 2007. One element of LRFD with respect to substructure design to realize cost savings is to perform the "calibration" of the resistance factors to local subsurface conditions in lieu of using the standard "generic" resistance factors. Static load testing will help to begin the process of calibrating resistance factors used in LRFD design methodology to subsurface conditions encountered in Montana and more specifically for the Missoula District to piles located in fine grained glacial lake Missoula sediments.

The static load test should be performed to a capacity of 300 kips. This ultimate capacity is based upon a factor of safety of 2.0 as recommended by FHWA and also includes the additional loads for negative skin friction under long term conditions. We have recommended that the capacity during driving be lower than the required static load test capacity predicated upon the "set-up" (gain in capacity) expected to occur as excess pore water pressures generated during driving in the clay soils dissipate. Performing the static load

test will verify if the required set-up has been obtained.

We recommend that design and construction of the reaction frame and testing apparatus be performed by the contractor or their consultant. The Geotechnical Section will monitor the actual load testing and collect and analyze the data.

Static load tests are covered by the Standard Specifications and we will provide a draft Special Provision to supplement the requirements contained in the standard specifications after we receive any comments on this memorandum.

Pile lengths can be optimized and potential cost savings in pile length maximized if the load test is performed prior to ordering production piling, however, this requires the waiting time for the production piling to arrive on site and may impact the construction schedule and it is nearly impossible to quantify the additional costs associated with the waiting.

Alternatively, the production piling lengths can be specified as part of the contract documents, as estimated from design analyses but this option negates the opportunity to reduce pile lengths. It should also be noted that performing a static load test reduces pile lengths whether the piling is ordered before or after the load test, because of the smaller factors of safety that can be used when a load test is performed.

We request review and comments from those on the distribution list to this memorandum as well as concurrence to performing the static load test if no comments are provided. We will provide additional details for the load test after we receive comments/concurrence.

Other Design and Construction Recommendations.

We recommend select backfill consisting of A-1-a material at the bridge ends. A Special Provision, *Bridge End Backfill* has been attached.

Approach embankment end slopes are shown as 1.5H:1V at Bent 1. Stability analyses indicate that 1.5H:1V end slopes are marginally stable, the option of using 2H:1V end slopes should be evaluated to improve slope stability.

We recommend using High Survivability Class C Permanent Erosion Control Geotextile (table 716-5 of the Standard Specifications) below the riprap at the abutments.

Possible Further Subsurface Investigation

Due to the soft, fine grained soils present throughout this project, we are tentatively planning to perform some Cone Penetration Testing (CPT) at the bridge foundation locations, pending work load of the Field Investigation Unit. The Geotechnical Section has recently obtained a truck mounted CPT rig that will allow us to perform this testing. Based upon the results of this testing, there is a possibility that we may slightly modify the design tip elevations required capacities or minimum time for restrike of the recommended piles. We anticipate

that the piling size or required number of piles per bent would not change based upon results of the CPT testing.

Questions regarding this memorandum or project may be directed to Jeff Jackson, MDT Geotechnical Section, 444-3371 or via email, jejackson@mt.gov.

JGJ::jgj

Attachments: Boring Logs (Bridge Area only)
Bridge End Backfill Special Provision
Settlement waiting period Special Provision.

Original: Geotechnical Project File

Copies: Dwane Kailey, P.E, District Administrator - Missoula
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Geotechnical Correspondence File