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Montana Department of Transportation

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Memorandum

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From: John Sharkey
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Date: November 25, 2020

Subject: NH 1-7(55)417
E of Zurich - Harlem
UPN 9221000
Geotechnical Engineering – Alignment Report (Activity 464)

The Geotechnical Bureau has completed an engineering assessment of the proposed alignment and reconstruction project of US Highway 2 between Zurich and Harlem, MT. This report provides information about the investigations, details of conditions within the corridor, and recommendations for design and construction based on the work performed to date.

A 2017 preliminary geotechnical evaluation report provided information on subsurface conditions on the south side of the PTW. At that time, a southward alignment shift was being considered. The revised alignment will instead shift significant portions of the roadway to the north. However, much of the information obtained during the preliminary evaluation is relevant to the updated alignment. As such, the conclusions and recommendations provided here are based in part on the preliminary evaluation as well as the more recent efforts focusing on the revised alignment.

1.0 LOCATION AND PURPOSE

The project is in Blaine County on US Highway 2, beginning at RP 416.7 near the intersection with Eight Mile Road and ending at RP 424.0 just east of the intersection with Thirty Mile Road. It will reconstruct approximately 7.3 miles of US-2 between the towns of Zurich and Harlem, MT.

It is scoped to improve the operational efficiency and safety by providing dedicated passing and turning lanes, and to provide a design meeting current standards. Improvements include a wider finished top with 8' shoulders, and flatter, traversable inslopes. New pavement surfacing, signing, and updated delineation will be included. Many of the culverts will remain in place, however, as they will have either been lined or replaced with the preceding Chinook – Harlem Culverts (UPN 922000) project.

2.0 GEOLOGY AND SOILS

The alignment lies within Quaternary-age alluvium deposited by the Milk River and the glacial-era Missouri River. During the last stages of glaciation, the ice mass advanced south toward present-day Virgelle MT blocking the flow of the Missouri River and diverting it eastward to create its current channel. The Milk River now flows within this older abandoned Missouri River channel between Havre and Glasgow, and the valley is filled with alluvium – fine-grained sand, silt, and clay – deposited by both rivers.

Bordering the valley to the north and south, Cretaceous-age bedrock rises to gently rolling plains above the valley floor. Bedrock units include the Judith River Formation – a brown to grey sandstone with interbedded black shale, and the Bearpaw Formation – a dark gray shale with bentonite interbeds. Geologic maps show the bedrock is overlain with residual glacial deposits, but these are generally thin veneers within the project corridor. A map of the regional geology and the locations drilled is included as an electronic link to this report.

Soils maps show the existence of fine-grained sands, silts, and clays likely derived from the nearby bedrock and glacial deposits. The smaller drainages contain water-deposited silt, sand, and fine gravel of varying depths. Many of the soils within the corridor possess at least some potential for swelling, are poorly consolidated, and will frequently exhibit high moisture sensitivity and low bearing capacity in a high water table environment.

3.0 GEOTECHNICAL INVESTIGATIONS AND TESTING

3.1 Drilling and In-Situ Testing - Preliminary Geotechnical Evaluation

Twelve borings were drilled in August 2017 to obtain information used to prepare the Preliminary Geotechnical Evaluation Report. Boring locations were chosen to evaluate subsurface soils and conditions on the south side of the PTW as a southward alignment shift was being considered at the time to accommodate the project intent.

Borings for the preliminary evaluation were drilled with the CME 850 rig using 8-inch hollow-stem augers, and each attained a depth of 21.5 feet below the ground surface. Standard Penetration Testing (SPT) was conducted to determine in-situ material density or consistency. Split spoons were used to collect samples for laboratory testing.

Cone Penetrometer Test (CPT) soundings were performed adjacent to 7 of these borings. CPT employs a rig-mounted direct-push system that advances small-diameter steel rods with an instrumented tip, or "cone", into the ground. The instruments record the tip pressure necessary to penetrate the soil, the interface skin friction along the perimeter of the cone, and the pore water pressure within the soil. Readings occur in real-time, and the data provide more detailed and potentially more accurate representations of subsurface conditions compared to traditional methods because soil parameters are measured every few centimeters.

Pore pressure dissipation tests, useful in predicting how rapidly an excess pore pressure condition returns to hydrostatic, or stable baseline state, were also performed with the CPT. This information is used along with laboratory data to evaluate the time-rate of settlement resulting from the placement of fill onto soft, fine-grained soils where a high water table is present. In such cases, the increased load is initially "carried" by the pore water, and an excess pore pressure condition develops. As the excess pore pressure condition dissipates, the subgrade soils respond to the added load through consolidation, a non-linear reduction in volume and reorganization of the soil structure. The magnitude of settlement is dependent on the mass of the applied load and the modulus of the 'receiving' soils. The time-rate of consolidation, and thereby the duration of settlement, depends on the permeability of the receiving soils. For soils with low permeability and high water content, consolidation (and thereby settlement) often continues for a period of months, years, or in extreme cases, decades.

A separate investigation was conducted in 2017 for the Chinook – Harlem Culverts (UPN 922000) project. Fourteen borings were drilled for the culverts project, and borings 9220-3 through 9220-8 are relevant to this project. These borings ranged in depth from 11.5 to 26.5 feet below the asphalt surface and were drilled with the CME 850 rig using 8-inch hollow-stem augers. Similar sampling and testing protocols were performed for the culverts project investigation, but the CPT equipment was not used.

3.2 Drilling and In-Situ Testing – Geotechnical Engineering Alignment Investigation
Twenty-two additional borings were drilled by the MDT Field Investigation Unit in 2020. Boring locations were chosen to evaluate subsurface soils and conditions specific to the updated alignment, approximately half of which will shift northward toward the BNSF Right of Way.

Eight off-road boring locations were chosen to determine material properties and ground water levels within the proposed fill segments. This information was used to assess soil shear strengths, settlement potential (including probable magnitudes and rates), and the overall constructability of the new alignment.

Fourteen on-road borings were drilled in the areas where the new alignment will coincide, wholly or partially, with the existing alignment. Information from these locations was used along with the that from the culverts project to assess the conditions of the PTW, evaluate alignment constructability, and determine the potential for subgrade-related difficulties during construction.

Drilling was completed with the CME 55 and CME 850 rigs using 8-inch hollow-stem augers. The borings ranged in depth between 9 and 21.5 feet below the asphalt or ground surface and Standard Penetration Testing (SPT) was conducted to determine relative soil densities and consistencies. Split spoons and Shelby tubes were used to obtain samples within discrete intervals for lab testing. At some on-road locations, grab samples were collected from the auger flights to supplement recovery of the base course materials.

Subsurface soils were also evaluated using a Dynamic Cone Penetrometer (DCP) at six of the on-road boring locations. DCP testing allows a comparative evaluation of the relative strengths of near-surface materials (base course and shallow subgrade to an approximate depth of 36 inches) in much higher resolution than SPT. The DCP results are then correlated to each respective material's bearing capacity (CBR), Resilient Modulus (M_r), and R-value. The correlated data is shared with MDT Pavement Design staff to complement other information used when designing the new surfacing section. The CBR, Mr, and R-Value graphs from DCP testing are included as electronic links with this report.

3.3 Laboratory Testing

Sample testing conducted in the MDT Geotechnical and Materials labs included soils classifications with gradations and Atterberg limits, consolidation, shear strength, and determination of natural moisture content. The MDT Chemistry laboratory analyzed base course and subgrade samples to determine soluble soil sulfate concentrations.

Relative soil strength is typically estimated with good reliability in the field using standard penetration testing methods. The CPT data (south of PTW only) supplement the SPT data but provide higher resolution and better confidence of soil strength when evaluating fine-grained materials. The CPT graphs and boring logs, including those relevant from the Chinook – Harlem Culverts project, are included as links with the distribution of this report. The graphs and logs provide detailed information on corridor conditions and testing results.

4.0 CORRIDOR CONDITIONS

4.1 Corridor Conditions: Off-Road Locations

Soils on the south side of the PTW are consistent throughout the areas investigated. The topsoil is generally sandy silt (A-4) that ranges between 0.3 feet and 3.0 feet in approximate thickness. This material is underlain in most locations by lean clay and/or fat clay (A-6 and A-7) with varying amounts of sand. The clays were the predominant soil types encountered during the preliminary investigation, but other subsurface soil types were observed, including clayey sand (A-6), silt or sandy silt (A-4), and silty clay (A-6), in the order of apparent prevalence. Clayey sand layers were often wet with visible free water.

Similar soils were encountered on the north side of the PTW, but the topsoil more often consisted of lean or fat clay. Of particular note is a segment of the proposed alignment between approximate mile posts 418 and 418.5, or approximate stations 270+00 to 280+00, that exhibits significantly poorer subsurface conditions – lean and fat clay, and clayey sand, that is very soft to soft (or very loose) in a high water table setting. Borings 18 and 19 represent the subsurface conditions within this segment. Information from boring 4, located on the south side of the PTW at station 279+93, suggests that poor subsurface conditions are laterally extensive within this area. Information obtained from borings 8

(369+75, Rt), 9 (407+65, Rt), and 10 (444+67, Rt) indicates that relatively poor subsurface conditions can be anticipated, at least sporadically, between approximate stations 270+00 and 445+00. Loose, wet, heaving sand was encountered below a depth of 6 feet near station 314+03 (boring 22, approximate mile post 419.9).

Overall, soil consistencies for the cohesive materials ranged from very soft to very stiff, while densities for the cohesionless materials were generally very loose to loose. There was not a consistent trend of increasing material consistency (or density) with depth. Some organic soils were observed, but bedrock was not encountered in any of the borings.

4.2 Corridor Conditions: On-Road Locations (PTW)

Including the relevant borings from the culverts project, a total of twenty borings provide the following summary of the existing surfacing section and embankment conditions. Asphalt thickness ranges between 0.5 and 1.0 feet, with an average thickness of 0.6 feet. The base course classifies as well- to poorly-graded gravel with sand, or as silty sand with gravel, and predominantly within the AASHTO A-1 or A-2 soils groups. Base course thickness ranges from 0.9 to 1.9 feet, averaging approximately 1.4 feet. The base course percent fines are between 3 and 43 percent, with an average of 13 percent.

The above reported base course thicknesses (and average thickness) do not include results from borings 9221-30, 31, and 32 where the observed gravel is significantly thicker than what was typically seen. The gravel thickness at these locations is approximately 2.9, 3.9, and 4.3 feet, respectively. Additional gravel may have been used at these locations to replace poor subgrade soils during construction of the current alignment.

The base course is underlain by a variety of materials including, in the order of apparent prevalence, lean and fat clay (A-6 to A-7), silty or clayey sand (A-2, A-4, and A-6), silt (A-4), clayey or silty gravel (A-6 and A-2), poorly graded sand (A-1), and silty clay (A-6). The clays often contain at least some percentage of sand.

4.3 Groundwater Conditions

Groundwater was encountered at all but four of the off-road locations, recorded at depths between 3 feet and 20.5 feet below the ground surface. It was encountered at seven of the on-road locations (including those from the culverts project), ranging from 7.5 to 15.7 feet below the asphalt surface. However, most of the on-road borings attained a maximum depth of only 9 feet, and we suspect that water would have been encountered in many more had these borings been a few feet deeper. The water levels generally either rose or fell after first being intersected, and the levels reported on the logs should not be interpreted as hydrostatic levels. If given a longer wait period, water levels may have continued to change to some degree until equilibrium was reached. Recorded water levels are representative of the time and location of drilling and will fluctuate with seasonal events.

4.4 Corridor Condition Summaries

Tables 1 and 2 provide summaries of the corridor information given above.

Table 1. Summary of observed conditions: off-road boring locations.

Boring	Appx. Station	Topsoil Type: USCS (AASHTO)	Appx. Topsoil Thickness (ft)	Subgrade Soil Types: USCS (AASHTO A-)	Water Depth (ft below surface)
9221-1	236+43	ML (A-4)	1.0	CH, CL (7, 6)	-
9221-2	249+84	CH (A-7)	0.6	CH, CL, SC (7, 6)	8.9
9221-3	253+81	ML (A-4)	1.0	CL (7, 6)	-
9221-4	271+93	ML (A-4)	2.0	CL, CH, SC, ML (6, 7, 4)	9.4
9221-5	302+32	ML (A-4)	0.7	ML, SC, CL (4, 6)	15.7
9221-6	333+77	ML (A-4)	1.0	CH, CL (7)	5.9
9221-7	350+48	ML (A-4)	1.5	CL, CH, SC (6, 7)	8.2
9221-8	369+75	ML (A-4)	0.6	CH, CL (7, 6)	3.7
9221-9	407+65	ML (A-4)	0.3	CL-ML, CL, CH (6, 7)	4.8
9221-10	444+67	ML (A-4)	3.0	CH, ML (7, 4)	5.5
9221-11	516+60	ML (A-4)	1.0	CH (7)	11.3
9221-12	563+96	ML (A-4)	1.0	CL (7)	8
9221-13	218+26	CH (A-7)	0.5	CH (7)	5.2
9221-14	227+97	CL (A-6)	1.0	CL, CL-ML (6)	-
9221-16	251+89	OH (A-8)	0.4	CL (6)	-
9221-18	271+99	CH (A-7)	3.0	CL (6)	4.8
9221-19	279+04	CL (A-6)	1.8	SC, CL (6)	4.7
9221-21	304+05	CL (A-6)	0.5	SM, CL-ML (2, 6)	20.5
9221-22	314+03	CH (A-7)	-	SW, SM, SP (1, 2)	3
9221-24	359+43	ML (A-4)	1.0	CL, ML (6, 4)	10.6

Notes: (-) Not tested, not determined, or not encountered.

Table 2. Summary of observed conditions: on-road boring locations.

Boring	Appx. Station	Asphalt Thickness (ft)	Base Course Thickness (ft)	Base Course Type: USCS (AASHTO)	Base Course Fines (%)	Subgrade Soil Types: (AASHTO)	Water Depth (ft below surface)
9221-15	237+81	0.6	1.5	GP (A-1)	4	A-4, A-6	-
9221-17	262+08	0.7	1.3	SM (A-1)	-	A-6	-
9221-20	290+86	0.5	1.4	SM (A-1)	-	A-6, A-4	-
9221-23	342+99	0.6	1.5	GW (A-1)	3	A-2, A-6	-
9221-25	388+89	0.6	1.8	GW (A-1)	3	A-6, A-1	-
9221-26	426+92	0.6	1.5	GP (A-1)	4	A-6, A-4	-
9221-27	444+05	0.5	1	SM (A-1)	-	A-4	-
9221-28	458+96	0.6	0.9	GP (A-1)	5	A-4, A-7	9
9221-29	473+92	0.7	1.1	SM (A-1)	21	A-2, A-4	-
9221-30	488+90	0.6	2.9	SM (A-1)	-	A-2, A-4	-
9221-31	504+07	0.6	3.9	GP (A-1)	4	A-4	9
9221-32	519+01	0.7	4.3	GC (A-2)	-	A-6	-
9221-33	533+94	0.6	1.1	GW (A-1)	7	A-6, A-2	-
9221-34	549+04	1.0	1.3	SM (A-4)	43	A-4, A-6	-
9220-3*	211+06	0.6	1.9	GP (A-2)	4	A-6, A-7, A-2	10.7
9220-4*	283+30	0.6	1.4	SM (A-1)	-	A-6, A-2	-

9220-5*	359+79	0.6	1.6	SM (A-1)	-	A-6, A-7	7.5
9220-6*	481+02	0.5	1.5	SM (A-2)	43	A-4, A-6	15.7
9220-7*	480+77	0.6	1.6	SM (A-1)	-	A-6	11.5
9220-8*	578+16	0.7	1.5	SM (A-1)	-	A-2, A-4	11.2
Averages		**0.6**	**1.4^**		**12.8**		

Notes: * Relevant borings from the Chinook – Harlem Culverts project. (-) Not tested, not determined, or not encountered. ^ Average value does *not* include results from borings 9221-30, 31, and 32 where unusually thick base materials were observed during drilling.

4.5 Chemical Soil Stabilization and Sulfate Test Results

Calcium-based additives are often used to improve the engineering properties of poor soils. Cement, lime, and fly ash added to weak, fine-grained soil can dramatically improve the shear strength, and reduce the compressibility, plasticity, and swelling potential of the soil. The treatment is used regularly in some states to improve constructability and the long-term performance of a reconstructed roadway. However, where soil sulfate concentrations are too high, the technique can cause an undesirable consequence known as sulfate-induced heave; a condition of extreme soil swelling that can lead to catastrophic pavement failure.

Sulfate occurs naturally in many of the native soils throughout Montana. Most of the near-surface soils of central and eastern Montana contain at least a small percentage of sulfate. But within the typical length of a project, there are often isolated segments where the sulfate concentrations are high. Sulfate is an ion that combines with other natural elements in the soil to form mineral compounds. Gypsum is a commonly occurring example.

For sulfate concentrations between 0.3 and 0.5 percent, mix design, construction techniques, and quality control protocols become vital to the success of treatment. At higher levels, more rigorous drilling, sampling, testing, and mix design evaluations are required, as is enhanced QA/QC during construction. At concentrations of 0.8 percent and above, calcium-based soil stabilization is not advised due to an elevated risk for failure.

Percent soluble sulfate was evaluated in nineteen project samples representing various depths within the soil profile. Results ranged from a minimum value of 0.005 percent to a maximum of 0.9 percent. The data suggest a general trend of increasing sulfate content with depth, and significantly higher values were obtained from subgrade samples on the east end of the project. However, sulfate is water-soluble and can be mobilized in significant events (floods, ponding), resulting in changes in concentrations in the soil profile. This may be why the base course near MP 422.3 is high in sulfate. Table 3 provides the test results on soil samples collected during the geotechnical investigation:

Table 3. Percent Soluble Soil Sulfate

Boring	Appx. Station	Sample Interval (ft)	Sample Material	Percent Soluble Sulfate (%)
9221-15	237+81	0.7 - 1.0	Base Course	0.006
9221-15	237+81	2.5 - 4.0	Subgrade	0.007
9221-17	262+08	2.0 - 3.5	Subgrade	**0.4**
9221-20	290+86	0.8 - 2.3	Combined	0.005
9221-20	290+86	5.0 - 6.5	Subgrade	0.005

9221-23	342+99	0.8 - 1.0	Base Course	0.009
9221-23	342+99	5.0 - 6.5	Subgrade	0.01
9221-25	388+89	0.6 - 1.0	Base Course	0.01
9221-26	426+92	0.9 - 1.0	Base Course	0.01
9221-26	426+92	2.5 - 4.0	Subgrade	0.7
9221-27	444+05	1.5 - 3.0	Subgrade	0.2
9221-28	458+96	0.7 - 0.9	Base Course	0.03
9221-28	458+96	1.5 - 3.0	Subgrade	0.01
9221-29	473+92	1.8 - 3.3	Subgrade	0.1
9221-30	488+90	0.6 - 1.5	Base Course	0.3
9221-30	488+90	3.0 - 4.5	Subgrade	0.7
9221-31	504+07	2.5 - 4.0	Subgrade	0.9
9221-33	533+94	2.5 - 4.0	Subgrade	0.8
9221-34	549+04	2.3 - 3.8	Subgrade	0.5

Notes: Orange font indicates values where additional efforts would be required to ensure the success of calcium-based treatment. Red font indicates results that are above recommended thresholds for calcium-based soil stabilization.

5.0 DISCUSSION

The project corridor lies within mostly level agricultural lowlands of the Milk River Valley. In many locations, the existing alignment has steep, non-traversable inslopes leading to ditches that regularly pond with water for extended periods. Groundwater levels are consistently high throughout the corridor and surface drainage is hindered by minimal regional gradients. Areas of alkali soils are widespread and numerous wetlands are present in the ditches bordering the highway embankments. Several canals and irrigation facilities exist near or pass beneath the roadway.

Substantial fills are required on the western end to construct the alignment shift, but cuts are limited in size and extent to improve inslopes and ditches. Only slight modifications are planned to the existing vertical alignment. Most of the culverts will have been addressed with the preceding project, and no structures will be replaced.

Between approximate stations 204+50 to 307+50 left, and 325+00 to 387+00 right, fills up to maximum height of 8 or 10 feet will be required to build partial-width embankment segments to accommodate the alignment shift and wider roadway. In many places, the fill will be placed on soft, fine-grained soils in a high water table setting. Settlement will occur within the native subgrade soils due to the additional load of fill. Long-term differential settlement can result in undesirable impacts to a roadway, especially where a portion of the roadway width is constructed on soft, undisturbed ground, and the remainder is built upon preexisting embankment. The impacts can include substandard pavement geometry (e.g. extreme crowns), dips, heaves, longitudinal cracking, poor drainage, poor ride quality, poor pavement performance, and increased maintenance requirements.

Beyond station 387+00, smaller wedge-shaped fills will be placed on both sides of the highway as the new alignment returns to essentially coincide with the existing alignment, eventually connecting to the PTW near Harlem, MT. As presently designed with 6:1 fill slopes, we expect no slope stability issues and cuts should be relatively easy to excavate to proposed lines and grades with standard construction equipment.

6.0 DESIGN AND CONSTRUCTION RECOMMENDATIONS

6.1 Moisture Sensitive Soils

Soil liquidity indices can be used to determine the relative sensitivity of soils to in-situ and introduced moisture. At values greater than 0.3, soils can be difficult to handle, place, or compact with modern construction machinery. At values above 0.5, handling, placement, and compaction becomes extremely difficult. Lab results indicate the existence of moisture sensitive soils throughout the corridor. The notable liquidity indices are provided below.

Table 4: Notable Liquidity Indices based on laboratory results of collected soil samples.

Boring	Appx. Station	Sample Depth (ft)	In-Place Moisture Content (%)	Liquidity Index	Moisture Content Plus 4 (%)	Liquidity Index: @ Plus 4% MC
9221-1	236+43	10.8	23	0.21	27	0.34
9221-2	249+84	5.8	26	0.22	30	0.34
9221-3	253+81	5.8	24	0.21	28	0.36
9221-4	271+93	3.3	30	0.78	34	1.00
9221-6	333+77	5.8	28	0.24	32	0.35
9221-8	369+75	10.8	30	0.94	34	1.19
9221-9	407+65	5.8	27	0.41	31	0.59
9221-12	563+96	10.8	32	0.56	36	0.72
9221-13	218+26	2.5	32.6	0.15	36.6	0.28
9221-13	218+26	10	31.4	1.49	35.4	1.93
9221-14	227+97	7.5	28.5	0.97	32.5	1.23
9221-15	237+81	2.5	17.4	0.06	21.4	0.63
9221-17	262+08	5	18	0.18	22	0.55
9221-18	271+99	3.3	32	0.88	36	1.13
9221-18	271+99	13.3	30	0.93	34	1.21
9221-21	304+05	7.5	37	0.38	41	0.50
9221-22	314+03	2.5	24.5	0.09	28.5	0.23
9221-23	342+99	5	20	0.11	24	0.32
9221-24	359+43	15.8	29	0.50	33	0.68
9221-25	388+89	5	17.4	0.15	21.4	0.40
9221-26	426+92	2.5	13.9	-0.08	17.9	0.21
9221-31	504+07	5	19.3	0.26	23.3	0.70
9221-34	549+04	7.5	32.3	0.78	36.3	1.02
9220-3	211+06	5.8	18	0.40	22	0.80
9220-6	481+02	4.8	23	0.56	27	1.00
9220-6	481+02	14.8	27	0.42	31	0.63
9220-7	480+77	5.8	24	0.43	28	0.62
9220-7	480+77	18.3	31	1.00	35	1.29
9220-8	578+16	9.8	37	1.70	41	2.10

The lower numbers indicate lower moisture sensitivity. However, small increases in moisture content will be detrimental to soil shear strengths and construction difficulties may arise. The column on the right demonstrates how moisture sensitivity is affected by a

4 percent increase in soil moisture content. As the moisture content approaches a soils inherent plastic limit, the soil becomes easy to fail under loading. Once failed, the shear strength is reduced to residual values and loading-induced deformation intensifies. Construction sequencing is necessary to minimize soil exposure during precipitation events. The Moisture Sensitive Soils Special Provision will be included in the contract to provide guidance if difficulties do arise.

6.2 Reuse of Surfacing Materials

Among the locations investigated, the approximate average pavement thickness is 0.6 feet. The plant mix appears to be in fair condition and we believe it can be milled and/or pulverized throughout the entire project length and reused as borrow or as part of the new surfacing section. However, if the pavement is reused for new CAC material, we recommend a maximum 50% RAP blended with virgin aggregate. Additionally, it should be processed to ensure no oversize material is incorporated, and pug-milled to attain a well-blended, uniform mixture.

Excluding borings 9221-30, 31, and 32 where unusually thick gravel was observed beneath the pavement, the existing base course averages 1.4 feet in thickness. Laboratory results indicate the materials classify as well- to poorly-graded gravel with sand, or silty sand with gravel, and the majority are within the AASHTO A-1 or A-2 groups. With some exceptions, the base course samples contained low percentages of fines. Where practical and beneficial, we believe this material could be reclaimed for borrow, traffic gravel, or for incorporation into the new surfacing section. Moreover, we believe that the existing base course could be left in place on the eastern end of the project (i.e. ~ MP 422 on) where the new alignment essentially coincides with the PTW at a similar or higher elevation. If left in place on the eastern end, and construction methods are modified accordingly, the existing base material may provide sufficient support for the temporary passage of vehicles and construction equipment prior to completion of the new surfacing section. Pavement milling or pulverization would provide additional material for use in the areas where a grade raise is proposed. This may reduce the cost of imported traffic gravel and help reduce the need for digouts to repair soft spots that commonly develop and expand with vehicular loading during construction.

6.3 Subexcavation and Replacement - More Data Required

As noted above, the existing base course appears to be relatively clean, but high fines were observed in a few of the locations investigated (MP 422, 422.2, and 423.5). The MDT Non-Destructive Testing (NDT) Unit conducts periodic Falling Weight Deflectometer and Ground Penetrating Radar surveys of the state's highway system. The data can be used to corroborate geotechnical observations, and possibly help delineate the spatial extents of the areas with high base fines and poor subgrade. This is often a more efficient means of bounding the areas of subexcavation and replacement within the existing surfacing prism.

Unfortunately, only network-level NDT data exist for this project corridor. At a sampling interval of 250 meters, we do not believe the data resolution is high enough to identify the limits of high fines in the base course. However, we will review the data after it has been processed, and we have requested a project-level (≤ 100-meter) NDT survey to be conducted in the spring of 2021. If the high-fines boundaries can be delineated, we will provide appropriate stationing for partial- or full-width subexcavation and replacement, according to Subsection 203.03.1.H, in a supplemental report.

6.4 Global Placement of Separation Geotextile CAC Underlayment

We agree with the Pavement Section's recommendation for the global placement of separation geotextile between the subgrade and the new CAC material for the entire project length (Option #8). Considering the typical borrow sources in the region, it is likely that the new embankment segments will be constructed with material containing high percentages of fines. Too, the existing PTW subgrade material is predominantly clay. Separation geotextile underlayment will help prevent fines contamination of the new CAC material, and may also add some dimensional stability to the typical section prism. The CAC layer should therefore maintain better long-term drainage and load distribution characteristics, which should in-turn result in improved long-term pavement performance. However, if Cement-Treated Base (CTB) is used for any portion of the project, the geotextile can be eliminated as CTB should effectively prevent fines migration.

6.5 Ground Improvement

As discussed above, weak, wet, fine-grained subgrade soils are prevalent in many places near the existing highway embankments. These soils will be problematic during construction, especially if exposed to introduced moisture and repeated loading from construction and vehicular traffic. We recommend Embankment Foundation Treatment, Benched Embankment Widening, and inclusion of the Construction Traffic Special Provision as discussed in the following sections. These should be beneficial to construction and long-term roadway performance.

6.5.1 Embankment Foundation Treatment - Shear Failure Mitigation

In many of the soil samples tested, the subgrade soil moisture content was near or exceeded the material's natural plastic limit, and the likelihood of soil shear failure during construction is high, especially where heavy equipment will repeatedly traverse these materials. Additional moisture during construction will intensify the problem and increase the odds of sinking the construction equipment. Because many of the contractor's means and methods are beyond MDT control, Construction personnel will need appropriate special provisions and support from Management to halt construction if necessary.

The Engineering Project Manager for the Lohman E & W project indicated that embankment foundation treatment, along with favorable weather conditions, helped eliminate many of the short-term construction issues that were problematic on the Havre – E project. We believe that this treatment will also help alleviate some of the long-term differential settlement issues that continue to plague the Havre – E segment by providing a pathway for the escape of water within and beneath the embankments should weather and groundwater conditions be unfavorable during construction. Settlement may therefore proceed more rapidly and uniformly.

Similar to the Lohman project design, we suggest a slightly reduced thickness of 24 inches of A-1 material placed on top of stabilization geotextile to provide a stable construction platform and reduce the chance for shear failure with equipment passage. To simplify staking and construction, we suggest using a uniform elevation for the top of the embankment foundation treatment and varying the material thickness as the natural topography dictates. The remainder of the embankment can be constructed with unclassified material. At a minimum, we suggest consideration of the treatment between stations 195+00 and 203+50 RT, 206+00 to 275+50 LT, 290+50 and 305+00 LT, and 327+50 and 359+00 RT.

6.5.2 Embankment Widening – Benched Geometry and Meet-Line Special Borrow

We recommend including plan detail sheets, cross-sections, and special provisions that direct benching of existing embankments according to Section 203.03.2.C in areas where taller sections will be built. We also suggest estimating the required material volumes, including unclassified borrow, using a neat-line benched geometry.

We recommend including a surfacing detail that provides a minimum of 8 inches of A-1-a special borrow, with a PTW sub-cut if necessary, throughout the longitudinal 'meet-line' of the existing embankments. This will ensure that a minimum of one lift of quality granular material exists to support temporary traffic and the final surfacing section.

These recommendations should reduce the need for change orders during construction. More importantly, they should eliminate an otherwise undesirable condition where poor soils, once part of the preexisting inslope and shoulder, directly underlie the new traveling surface at the meet-line between the new and old embankment segments.

6.5.3 Settlement Estimate - Special Embankment Construction (Preloading) Optional

An evaluation of the consolidation testing data reveals mixed results. In most locations tested, the data suggest that the subsurface soils are over-consolidated. In other words, the soil has previously experienced greater overburden pressures than the proposed fills will induce. This is a relatively common condition in glaciated areas where loading from an ice mass has pre-consolidated subsurface soils. We therefore anticipate negligible settlement after placement of fill according to current design within these areas.

However, the test results from samples collected near station 272+00 (boring 18) suggest that the opposite condition exists. Consolidation data from this location suggest that the native subgrade soils are under-consolidated, meaning the degree of consolidation is less than what would be expected from the existing, in-place overburden pressure. This condition is not as common in glaciated areas, but it is common in fluvial deposits and in some soils where the water table fluctuates. In such cases, the magnitude of settlement is compounded by the addition of fill, and it is proportional to the pressure of the added fill plus a predictable percentage of the existing overburden pressure.

Consolidation testing was performed on samples collected from only five locations (proposed fills only) of the new alignment. An exhaustive testing regimen of the entire alignment is impractical due to many constraints, including limited time and resources, and excessive cost. We therefore use engineering judgement to focus efforts on areas where we believe the likelihood for settlement is greatest and the potential for negative impact is highest. We suspect there are other areas of the new alignment, besides station 272+00, where significant settlement is likely. Furthermore, because some of the native subgrade soils are pre-consolidated and some are not, we believe the likelihood for differential settlement is high and adverse impacts to the new traveling surface are possible.

A comprehensive analysis was performed to estimate the magnitude and time-rate of settlement near station 272+00 using information obtained from the field, the lab, and the current set of plans. The results predict approximately 0.5 feet (6 inches) of settlement occurring within the native subgrade soils due to the additional load from 8 feet of fill. However, the time-rate calculations provide an estimate of 36 days to achieve 90% consolidation. Therefore, 90% of the predicted settlement is likely to occur before the final surfacing materials (CAC and pavement) are in place. We therefore believe that Special Embankment Construction (Preloading) is optional since construction of this magnitude would typically require a duration much longer than 36 days, and grade deficiencies caused by settlement can be corrected during placement of the CAC lifts.

Conversely, because contractor means, methods, and sequencing are impossible to predict, Special Embankment Construction (Preloading) may be preferred to ensure adequate time is provided to allow the majority of the predicted settlement to occur. If so, we recommend the areas outlined in section 6.5.1 be considered for special embankment construction. The embankment segments can be preconstructed up to the approximate subgrade elevation, or to the height of the existing PTW subgrade, and sloped to drain from the PTW. To perpetuate drainage, gaps can be left at approaches and culvert locations; the adjacent preloaded areas will have a beneficial effect. The Geotechnical Bureau will prepare a special provision to direct this work if a decision is made to proceed with special embankment construction.

Based on observed conditions and test results, we expect no bearing capacity issues if the remaining fill sections are constructed as currently designed. If significant plan changes are required following the distribution of this report, please notify Geotechnical Bureau for an assessment of the potential impacts.

6.5.4 Chemical Subgrade Stabilization Not Recommended

As discussed in section 4.5, chemical stabilization techniques can be used to improve the engineering qualities of poor soils. However, because of the elevated sulfate concentrations in corridor, we do not recommend the use of calcium-based additives to stabilize the subgrade soils of the PTW. With sulfate concentrations above recommended thresholds, the risk for sulfate-induced heave and pavement failure is too high.

We also caution that the use of Cement Treated Base (CTB) is not without some risk. We note that CTB has been discussed as options to 1) reduce the surfacing section thickness, and 2) to reduce the depth of subexcavation and replacement that would otherwise be required in areas where a lower grade is necessary.

Ideally, more time would allow an assessment of the long-term performance of the CTB sections constructed on the nearby Lohman E&W project. Since this is not an option, we recommend a discussion among MDT staff on the advantages and potential pitfalls of CTB, and we suggest inclusion of members from Construction, Design, Geotechnical, Preconstruction, and Surfacing, and others with experience in its use. The final decisions on the use of CTB should include an awareness and acceptance of the risks and the possible expense of future treatments to fix problems that arise. The use of sulfate-resistant cement could be considered to mitigate the risks on the eastern end of the project where sulfate concentrations are high in the near-surface soils.

6.6 Construction Traffic and Temporary Traffic - Roadway Subgrade

A relatively new special provision called Construction Traffic and Temporary Traffic - Roadway Subgrade was included in the Lohman E&W contract. This special provision assigns responsibility for PTW maintenance to the contractor during construction but requires additional effort in contract administration by the EPM. We believe the assignment of responsibility was beneficial on the Lohman project because many of the means, methods, sequencing, and equipment choices were beyond MDT control. We recommend inclusion of the special provision with this project for the same reasons and will edit the document as appropriate and provide it with other Geotechnical Special Provisions if this recommendation is accepted.

6.7 Harlem Canal - Impact Avoidance

The Harlem Canal was modified during construction of the current alignment, and as a result, much of it lies within MDT Right of Way between stations 457+00 and 492+00. To fit standard inslopes and ditches, portions of the northern berm of the canal will have to be narrowed and slightly steepened.

We recommend avoiding impacts to the canal berm due to concerns that modifications may increase leakage and reduce berm stability, possibly leading to failure. We instead suggest consideration of slightly steepened inslopes and reduced ditch widths to avoid impact to the berm. If the modified inslope and ditch geometries would have to be too severe, we suggest consideration of a low-height (i.e. 3-4' high) retaining system throughout this area.

The preliminary retaining system options for consideration, in order of preference, include a Reinforced Soil Slope and a precast gravity block (aka. Big Block) wall. A Reinforced Soil Slope (RSS) would provide a relatively low-cost means of reducing the inslope width on the south side of the highway if a final, lower slope no steeper than 1:1 (H:V) is acceptable. Standard equipment is used in construction, and most of the materials will be common bid items in the project contract. The RSS facing can be enhanced to resist erosional forces if necessary.

A precast gravity block wall should be considered if final slopes steeper than 1:1 are necessary, and a vertical face is feasible with modest wall heights. If speed of construction is a concern, the precast block wall option may offer an advantage over RSS. However, precast gravity walls are generally proprietary systems designed by the manufacturer and would likely be the more costly option in this setting.

If the combination of steeper slopes and narrower ditches, with or without a low-height retaining system, does not satisfy the clear zone geometric needs of the new alignment, we will analyze the effects of canal berm modification with slope stability software. We have the survey information necessary to begin analysis in two critical locations but will need an approximate maximum canal water elevation to proceed. If berm modification is unavoidable, please contact the Geotechnical Bureau to request an evaluation. The results will be distributed in supplemental report.

6.8 Surfacing Section Drainage

Drainage is a primary factor considered in surfacing section design and dramatic reductions in pavement life occur when the section remains in a saturated state. FHWA research has shown 40 to 50% reductions in pavement life occurring if the section remains saturated for only 1 month per year. Because MDT standard practice directs the placement of several inches of topsoil over the portion of the inslope where the base course would otherwise daylight, surfacing section drainage is impeded. Reduced drainage therefore necessitates an increased asphalt thickness to provide an equivalent pavement design life. We recommend a comparative typical section analysis conducted by Surfacing Design to demonstrate the impacts and potential costs associated with reduced section drainage.

6.9 Construction Sequencing and Project Timing

We believe that construction sequencing and project timing need careful consideration to reduce the likelihood of construction-related difficulties, digouts, and costly change orders. When the existing pavement is removed, there is a high probability for base course/subgrade failure caused by normal traffic and construction traffic passage. Subgrade failure will lead to additional necessary actions and costs to correct. We therefore recommend that the existing pavement be left in place until the final stages of construction if possible, to maintain travel through the corridor and reduce the need for change orders.

6.10 Design Project Shrink/Swell Factor Estimate

We recommend using a design shrink factor of 30% for the purpose of estimating earthwork volumes during plan development.

6.11 Other

We currently do not anticipate areas of difficulty other than those noted above. Questions and comments regarding this report can be directed to John Sharkey by e-mail at jsharkey@mt.gov.

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Attachments by Link:

- Geotechnical Boring Locations Map
- Final Geotechnical Boring Logs
- CPT Sounding Logs
- DCP Soil Strength Correlation Graphs
- Relevant Geotechnical Boring Logs Chinook - Harlem Culverts Project

Special Provisions Currently Proposed:

- Moisture Sensitive Soils (to be inserted by Contract Plans)
- Embankment Foundation Treatment (with stationing specified)
- Construction Traffic and Temporary Traffic - Roadway Subgrade