Final Report

GEOTECHNICAL INVESTIGATION OF SELECTED FEATURES: BEARTOOTH HIGHWAY, U.S. 212, WYOMING

Prepared for:
Department of Transportation
Federal Highway Administration
Lakewood, Colorado

March, 2001

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This report presents the results of geotechnical investigations conducted at eleven areas along the Beartooth Highway FH4-1(0) Park County, Wyoming. The areas are sites of geologic hazard or significant structures. A soil survey of the alignment was also conducted. The following paragraphs describe the sites, the investigation conducted, and geotechnical recommendations. Table ES-1 summarizes foundation recommendations and Table ES-2 summarizes slope recommendations. Detailed recommendations are presented in the relevant sections.

Table ES-1

FOUNDATION RECOMMENDATIONS

Site	Foundation Type	Criteria	Axial Capacity (allowable)	Earth Pressure
Beartooth Ravine Bridge	Micropiles	• 0.5 m minimum plunge • 1 deep (probe) pile at each abutment/pier	0.5 MPa bond	γ_{ef} = 5.9 kPa/m
	Drilled Piers	• 1.5 m minimum socket • 1 probe hole at each abutment/pier	0.5 MPa skin 5 MPa end	
	Footings	• 1.5 m minimum embedment	1 MPa bearing	
Site 2 Wall	Micropiles		0.5 MPa bond	γ_{ef} = 5.9 kPa/m
Beartooth Lake Outlet Bridge	Driven Piles	• 6 m into silty clay	20 kPa skin	γ_{ef} = 5.9 kPa/m
	Micropiles	• 2 m minimum plunge • 6 m into silty clay	19 kPa bond	
Little Bear Creek Bridges	Driven Piles (H-Piles)		82.8 Mpa Working Stress	γ_{ef} = 5.9 kPa/m
Long Lake Bridge	Driven Piles (H-Piles)		82.8 Mpa Working Stress	γ_{ef} = 5.9 kPa/m
Fen Mitigation Bridge	Micropiles	• 0.5 m plunge	0.15 – 0.25 MPa bond	γ_{ef} = 5.9 kPa/m
	Drilled Piers	• 1.5 m minimum socket	0.15 - 0.25 MPa skin 1.5 - 2.5 MPa end	
	Driven Piles (H-Piles)		82.8 MPa Working Stress	
	Footings (Abutment No. 2)	• 1.5 m minimum embedment	0.5 MPa bearing	

Table ES-2

ROCK CUT SLOPE RECOMMENDAITONS

Site	Station	Max Slope Ratio (Vertical:Horizontal)	Ditch Criteria	Other Criteria	Blasting Requirements	Scaling Requirements	Bolting Requirements
Site 2 Knob	41+540 – 41+580	2:1	Ritchie or 3 m minimum	0-6 m horizontal cut	Controlled Blasting	Bar, occ. Light blasting	Use as needed in lieu of scaling
Site 2 Wall	41+580 – 41+720	2:1	Ritchie or 2 m minimum	0-2 m horizontal cut	Controlled Blasting	Bar, occ. Light blasting	Use as needed in lieu of scaling
	41+900 – 41+980	2:1	Ritchie or 2 m minimum	Reinforce steep colluvium cuts (soil nail wall or dry stack rockery)	Controlled Blasting (rock)	Bar, occ. Light blasting	Use as needed in lieu of scaling
Site 2 Through-Cut	41+980 – 42+060	1:1	Ritchie or 2 m minimum	5-7 m cut height	Controlled Blasting	Bar, occ. Light blasting	Use as needed in lieu of scaling (not likely needed)
Site 3	53+200 – 54+800	4:1	Ritchie or 2 m minimum		Controlled Blasting	Bar, occ. Light blasting	Use as needed in lieu of scaling (not likely needed)

Site 1

Site 1 is an existing slide area. Investigation consisted of auger drilling and sampling, and testing for index soil properties. An inclinometer and piezometer were installed, and baseline readings were taken. Recommendations for Site 1 are to improve surface drainage and install a trench drain along the inside shoulder of the road. Additionally, the embankment slope should be flattened to 1:4 (vertical:horizontal), or a blanket drain should be installed and the embankment should be reconstructed to its current slope ratio (approximately 1:3).

Beartooth Ravine Bridge

The Beartooth Ravine bridge is a planned new bridge that will span a talus-filled ravine and improve road alignment. Investigation consisted of core drilling at both abutments and both piers of the 50 km/hr alternative, testing of bedrock, and seismic tomography. Recommendations are for abutment foundations on spread footings or micropiles and pier foundations on micropiles. Alternatives are reviewed and recommendations for drilled piers are also provided, as shown in Table ES-1.

Site 2

Site 2 is an area adjacent to the east abutment of the Beartooth Ravine bridge, where the alignment passes hazardous existing rock cut slopes and high natural rock slopes. A concrete

"Z-wall" and rock cut are planned to gain additional width. Investigation consisted of core drilling and testing of bedrock, seismic tomography, and mapping of selected sections of rock slopes. Recommendations for rock cut vary from 0 to 6 meters horizontally, a slope ratio of 2:1 (vertical:horizontal) and spot or pattern bolting to stabilize portions of the slope, as shown in Table ES-2. Rockfall collection ditches should be incorporated to the extent practical. The "Z-wall" foundation should consist of micropiles bearing on bedrock. Uncertainty regarding the suitability of the talus and existing fill as the bearing strata was judged to be too high. Differential settlement could cause significant distress to the wall and road.

Beartooth Lake Outlet Bridge

The Beartooth Lake Outlet Bridge is a replacement of the existing bridge prominently located at the Beartooth Lake Outlet. Investigation consisted of core drilling through deep soil deposits comprised of boulders through clay material types. Index properly tests were conducted. Recommendations are for pile foundations bearing in the stiff fine-grained soils underlying the loose granular soils. Driven piles or micropiles are recommended, as shown in Table ES-1.

Little Bear Creek Bridges

Two single span bridges crossing Little Bear Creek near the Top-of-the-World Store will be replaced. Investigation consisted of one core hole at each bridge and testing of bedrock. Driven H-piles are the recommended foundation type, as shown in Table ES-1.

Fen Mitigation Bridge

The Fen Mitigation Bridge is a planned, low multispan bridge in a wetlands area. Investigation consisted of a core hole at one pier and each abutment, and testing of bedrock. Foundation conditions vary widely along the bridge and multiple foundation types may be used. Recommendations for spread footings, driven H-piles, micropiles and drilled piers are provided, as shown in Table ES-1.

Long Lake Outlet Bridge

The Long Lake Outlet bridge is a planned single span bridge with an adjacent retaining wall on the side of Long Lake. Investigation consisted of two core holes and testing of bedrock. Driven piles are the recommended foundation type, as shown in Table ES-1.

Site 3

Site 3 is 1.6 kilometers of planned rock cut in an area where lower rock cuts currently exist. Investigation consisted of face mapping of major cut areas and three line surveys. Recommendations are for a 1:1 (vertical:horizontal) slope ratio and spot bolting, if needed, as shown in Table ES-2. Rockfall collection ditches should be incorporated to the extent practical.

Site 4

Site 4 is an area between hairpin turns, above timberline, where an MSE wall is planned. Investigation consisted of three cored test holes and a seismic refraction survey. Recommendations are to found the MSE wall on the colluvium, talus, and weathered bedrock foundation. Voids and loose areas at the foundation grade may need foundation preparation work, as discussed.

Bar Drift Area

The Bar Drift area is an area above timberline on a north facing slope where past frost damage has been evident. Investigation consisted of three test pits and laboratory testing for gradation. Recommendations are for improved drainage in the drift area and replacement of finer grained soils and weathered bedrock with gravel and cobble fill.

1.1 PROJECT BACKGROUND

The Beartooth Highway (U.S. 212) is a 110.5 kilometer route that begins at the northeast entrance to Yellowstone National Park, near Silver Gate, Montana, and ends at Red Lodge Montana. The highway is within the Shoshone National Forest and is maintained by the National Park Service at Yellowstone National Park. In addition to being a Forest Highway, the route carries a special designation as a National Park Approach Road. Improvement to the Beartooth Highway through realignment and widening are proposed in Wyoming from MP 24.5 to MP 43.2 (the Wyoming/Montana state line). Proposed improvements include widening the existing road from a 5.48 meter pavement width to a 9.75 meter pavement width, straightening curves, realigning in areas, building new structures (bridges and walls) and modifying pullout areas. Ten sites between Station 39+800 to 67+600, where substantial cuts, fills, widening, or structures are planned, are the subject of this report.

1.2 GEOLOGIC SETTING

The Beartooth highway passes through the southeastern portion of an area known as the Beartooth uplift. This uplift resulted from the mountain building of the Laramide orogeny and covers an approximately 7,700 square kilometer area. The uplift area is 1,200 to 1,500 meters higher than the surrounding basins, and relief within the uplifted area varies from 1,675 meters elevation along the lower Stillwater River, to 3,901 meters elevation at Granite Peak, Montana. The Beartooth uplift generally consists of glaciated Precambrian granite and metamorphic rock which is overlain in places by Tertiary volcanics and occasional flat-lying sedimentary rock remnants. The geology, geologic history, and seismicity of the area are discussed in greater detail in the 1998 URS report identified in Section 1.3.

1.3 PREVIOUS WORK

Previous phases of investigation are summarized in the following reports.

- Initial Geohazards Evaluation and Geological Study: Beartooth Highway, U.S. 212, Wyoming, Woodward-Clyde Federal Services (URS), October 27, 1998.
- Initial Geotechnical Investigation: Beartooth Highway, U.S. 212, Wyoming, URS Greiner Woodward Clyde Federal Services (URS), October 29, 1999.
- Beartooth Highway MP 24.5 to MP 43.1 Preliminary Materials Report (Report 98-16) FHWA Geotechnical Services, October 5, 1998.
- Beartooth Highway Material Source Investigation, Report 99-14, FHWA Geotechnical Services September 20, 1999.

During the initial geohazards evaluation in 1998 (URS, 1998), six sections of the highway were identified and called Site 1 through Site 6. The same terminology is used herein. No drilling or subsurface exploration, other than seismic refraction, was conducted as part of the previous work. The previous work is referenced to stationing that may not correspond to the stationing used herein.

1.4 OBJECTIVES OF REPORT

The purpose of this report is to present subsurface geotechnical and surface geological data required for the design of slopes, walls, and bridges, and to provide geotechnical recommendations for these project features. The investigation was conducted in late August and September, 2000, and consisted of geologic outcrop mapping of proposed cutslopes, drilling for retaining walls and bridges, seismic refraction and 3-D tomographic surveying of the larger proposed structure and cut locations. A summary of the investigation methods used at each site is presented in Table 1-1. Recommendations for slope stability, cutslope ratios, wall types and locations, and bridge foundations are included for the selected features in this report.

A soil survey was conducted for the entire alignment, from Station 39+450 to Station 67+340, including the reaches between the sites and structures. The soil survey is presented in Appendix A. A separate soil survey and summary of water problem areas conducted in early July, 1998, is presented in the FHWA Preliminary Materials Report (Report 98-16).

Table 1-1

SUMMARY OF INVESTIGATIONS

Report Section	Location	Approximate Stationing	Soil Survey	Auger Drilling	Core Drilling	Test Pits	Seismic Tomography	Seismic Retraction	Rock Cut Mapping	Laboratory Testing	Inclinometer	Piezometer
2	Site 1 (slide area)	39+800 – 40+000	X	X						X	X	X
3	Beartooth Ravine Bridge	41+365 – 41+540	X		X		X			X		
4	Site 2 (rock cut and wall)	41+540 – 42+000	X		X				X	X		
5	Beartooth Lake Outlet Bridge	42+220 – 42+300	X		X					X		
6	Little Bear Creek Bridges	45+280 – 46+620	X		X					X		
7	Fen Mitigation Bridge	49+100 – 49+300	X		X					X		
8	Long Lake Outlet Bridge	50+240 – 50+300	X		X					X		
9	Site 3 (rock cut)	53+200 – 54+800	X						X			
10	Site 4 (MSE wall)	56+800 – 57+200	X		X			X		X		
11	Bar Drift Area (frost damage)	60+960 – 61+080	X			X				X		

1.5 REPORT ORGANIZATION

The report is organized by sections that address the sites identified in 1998 and the proposed bridge and wall structures. The approximate locations of these sites and structures are shown on Figure 1-1, and included in the section titles.

There were three sets of stationing staked at time of investigation. Test pits, test holes, and mapping in this investigation used the stationing in the main Plan & Profile sheets (A series sheets) from Segment 4A, 4B, and 4C of the Forest Service Project 4-1(0), Beartooth Highway dated July 2000, with the exception of THB-1, THB-2, THB-4, and THB-5. These four test holes used stationing from Beartooth Ravine Bridge 50 km/hr Option, Segment 4A (sheets C1-C2), Forest Service Project 4-1(0), Beartooth Highway dated July 2000.

Data from the investigation and analyses in support of the recommendations are presented in the following appendices. Where not presented elsewhere, a brief summary of the means and methods used to collect or analyze the data is provided in appropriate appendices.

Appendix A Soil Survey

Appendix B Test Hole Logs and Photographs

Appendix C Test Pit Logs and Photographs

Appendix D Laboratory Test Results

Appendix E Seismic Tomography

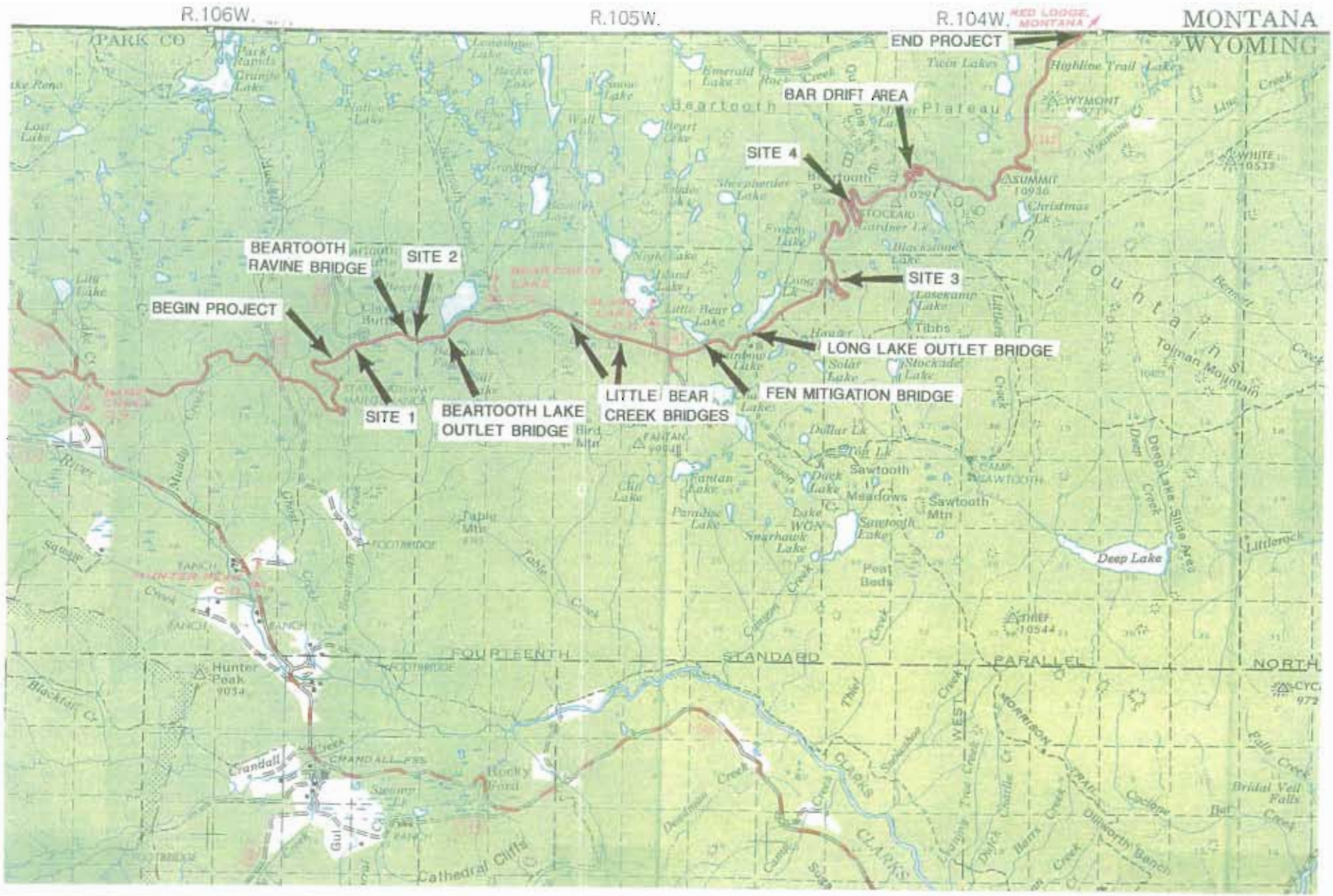
Appendix F Seismic Refraction

Appendix G Cut Mapping

Appendix H Slope Stability Analysis

Appendix I Monitoring Data

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/8/01	FH 4-1(10) BEARTOOTH HIGHWAY	1-1	
SITE MAP				



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2.1 INVESTIGATION

Two test holes were drilled from the outboard shoulder of the road, and one from the inboard shoulder at Site 1, as summarized in Table 2-1. Test holes TH1-1, TH1-3, and TH1-4, were drilled using the hollow stem auger method. Sampling was performed on 1.5 meter intervals with California tubes, Shelby tubes, and split spoon samplers for Standard Penetration Tests (SPTs). Once the holes were advanced to final depth, one inclinometer and two piezometers were installed. The hole locations and summary logs are shown on Figure 2-1, and individual logs and photos are provided in Appendix B. Instrument monitoring data is provided in Appendix I.

Table 2-1

TEST HOLE SUMMARY, SITE 1

Test Hole Number	Stationing	Offset (m)	Elevation (m)	Depth to Bedrock (m)	Depth to Water(1) (m)	Test Hole Depth (m)
TH1-1	39+980	1.8 R	2733.2	14.9	11.7	15.3
TH1-3	39+980	5.4 L	2733.8	N.E.	12.0	12.5
TH1-4	39+870	2.5 R	2727.9	N.E.	N.E.	10.8

Notes: (1) Encountered in test hole at time of drilling. N.E. = Not Encountered

2.1.1 Site Characteristics

The existing alignment crosses a slope of approximately 1:4 (vertical:horizontal) in this area. The slope is hummocky and apparently unstable at some times. The road has notable dips in grade and pavement has been repaired. Surface drainage from the slope above the road is collected in an unlined toe ditch which drains easterly to a cross culvert. Thick, green grasses and plants downslope of the road suggest shallow groundwater.

2.1.2 Subsurface Characteristics

TH1-1 was augured to a depth of 15.5 meters. The soils encountered were stiff to very stiff clays, with California (76 mm O.D.) sampler blowcounts ranging from 12 to 24, overlying medium dense to dense sand, with California sampler blowcounts of 38 to 50. Apparent density and consistency descriptors used in the text are based on the Key on page B-52 of Appendix B. Because of the larger sampler size, California sampler blowcounts were multiplied by 0.8 to correct to apparent SPT N-values. Weathered shale was encountered in the last 0.5 meters of the hole. An inclinometer was installed to the full depth of the test hole.

TH1-3 was augured to a depth of 12.5 meters. The soils encountered in the upper 12 meters of the hole were stiff to hard clay, with California sampler blowcounts of 20 to 68. The last 0.5 meter of the hole was very dense gravel to sand. TH1-4 was augured to a depth of 10.8 meters. The clay soils encountered to 7.5 meters depth were stiff, with California sampler blowcounts of 11 to 13. Dense sand was encountered from 7.5 meters to a depth of 9.2 meters. Shale boulders were encountered from 9.2 meters to the end of the hole.

TH1-1 and TH1-4 encountered man-placed fill and natural soils. The materials had similar characteristics except the fill was less stiff (California sampler blow counts of 11 to 14) than the natural soils (California sampler blow counts of 19 to 68).

TH1-3 and TH1-4 had piezometers installed as shown on the logs in Appendix B. At the time of drilling, the water level was encountered 11.7 meters below the ground surface of TH1-1 and 11.95 meters below the ground surface at TH1-3. Water was not observed in TH1-4.

The results of the laboratory testing are presented in Appendix D and summarized in Table 2-2. In general, half the samples tested were plastic, with a plasticity index of 3 to 14 and a liquid limit of 19 to 35. Only two samples had essentially 50 percent fines, or more. Moisture content varied from 4.5 to 17.6 percent, without apparent correlation to grain size or depth.

Table 2-2

SITE 1 LABORATORY TEST RESULTS

Test Hole	Depth (m)	Dry Density (kg/ cu. m)	Moisture Content (%)	USCS[1]	% Gravel[2]	% Sand[3]	% Fines[4]	Plasticity Index	Liquid Limit
TH1-1	7.60	1823	17.6	SM	22	29	49	3	19
TH1-1	12.20	2204	7.8	SM	40	45	15	NP	
TH1-1	15.25	1804	14.8	SM	0	67	33	NP	
TH1-3	4.55	1805	10.0	CL	3	37	60	11	35
TH1-3	9.15	2098	9.3	SM	31	40	29	NP	
TH1-3	12.20	1607	17.0	SW-SM	3	88	9	NP	
TH1-4	6.10	2031	7.0	GC	46	21	33	13	30
TH1-4	7.60	2129	4.8	GC	70	16	14	14	35
TH1-4	9.15	1783	4.5	SP-SM	20	69	11	NP	

[1] USCS = Unified Soil Classification System, based on visual observation when grain size data and plasticity data not available

[2] Percent greater than the No. 4 sieve size.

[3] Percent finer than the No. 4 sieve size and coarser than the No. 200 sieve size.

[4] Percent finer than the No. 200 sieve size. Includes silt and clay sizes.

2.2 INTERPRETATION AND ANALYSIS

The inclinometer installed in TH1-1 was monitored four times between September 7 and 15, 2000, and no movement was observed during the monitoring period. The variation in measurements shown on the data records (see Appendix I) is normal scatter inherent to the instrument and measurement method. The measurements were made to establish a baseline for future observations. This is normally a dry season and a time when observable slope movement is not likely.

Water was encountered in TH1-3 at an elevation of 2721.5 meters at the time of drilling. The water level dropped to elevation 2720.2 after drilling. Water was not encountered in TH1-4 at the time of drilling. Water level readings were taken between August 28 and September 17, 2000 and are presented in Appendix I.

The evidence of movement visible on the slope and in the roadway indicates that there are times when the factor of safety against slope failure is approximately 1.0. This probably occurs during

the spring when the water table is elevated due to either snowmelt or rainfall, or a combination of the two. It is possible that movement occurs every spring or only in certain years, when water conditions are particularly adverse.

Index properties measured at the site were used to select shear strength parameters (c, $^{s}_{2}$') for limit equilibrium analysis of the slope above and below the road. Because movement has been occurring for some time and the clayey materials present at the site are subject to weakening with this type of movement, peak values for strength are not appropriate. The values used are believed to be representative of a condition between the "fully softened" state and "residual" state for the soil. These values are related to plasticity and grain size, and independent of consistency (blow counts). Soil parameters of c = 0 kPa, ϕ' = 28 degrees, and a total unit weight of 21 kN/m^3, were selected for use in the analyses based on laboratory test results.

Emphasis for analysis was placed on the portion of the slope below the road because it has been impacting the road, whereas observations indicated no evidence that movement above the road was impacting the road. This is probably because sliding surfaces are shallow, and of limited extent, and movement only occurs sporadically.

Failures below the road and including the road are impacted by the road embankment and the drainage conditions imparted by the road and drainage ditch. Back calculation for a factor of safety of 1.0, using soil parameters of c = 0 kPa, ϕ' = 28 degrees, and a total unit weight of 21 kN/m^3, indicates the water table must rise to within 1 meter of the ground surface for failure to occur. Given the site location, the existing drainage conditions, and the materials at the site, this condition appears realistic for at least some spring seasons (e.g., higher than average precipitation or faster than average snow melt). Slope stability results are presented in Appendix H.

2.3 RECOMMENDATIONS

Based on these observations and analyses, we recommend the road improvements between Stations 39+800 and 40+000 include the following:

1. No cuts on the left side of the road.
2. Embankment slopes of 1:3 (vertical:horizontal) or flatter, if on site materials are used and a blanket drain is installed. Slopes of 1:4 (vertical:horizontal) or flatter if a blanket drain is not installed.
3. Improve surface drainage with an inboard paved ditch, and cross culverts.
4. Improve subsurface drainage by installation of a trench drain at the left shoulder and (optionally) a blanket drain beneath new or existing embankment fill.
5. Use of granular, coarse grained fill from elsewhere on the project to construct any new embankment. Satisfy Select Borrow requirements, FP-96, Section 704.07.

These recommendations are aimed primarily at suppressing an apparently high seasonal water table. The analyses presented in Appendix H show that if the water table is kept 2 meters or more below the surface, the factor of safety will be 1.25 or greater, which satisfies the minimum factor of safety criteria for landslides in the FHWA-CFLHD Project Development and Design Manual (1996). Despite the low theoretical factor of safety for the global surface shown, it is not

necessary, at this time, to implement specific measures to improve the global stability of the slope, or the slope above the road, because the distress observed on the road is consistent with failure occurring at the road level, and below, as shown by the smaller failure surface on Figure H-1, Appendix H (also see the central photo on the report cover). The theoretical factor of safety for the global surface is probably lower than actually exists because the surface is deep enough to encounter higher strength materials and the water table is probably not as consistently close to the ground surface as shown. Nevertheless, additional cuts into the slope above the road are not recommended because they may initiate slope failures that will impact the road.

Mitigation measures recommended herein are intended to lower the water table to 2 meters below the ground surface in the vicinity of the road and road embankment. We recommend a 2 meter deep trench drain be installed near the left (inboard) shoulder of the road, as shown on Figure 2-2. The design should include a minimum grade of 2 percent and a perforated drain pipe to facilitate drainage. Geosynthetics and/or granular filter criteria may be used to protect against contamination and clogging of the drain. Trench construction should follow FP-96, Section 605 and applicable health and safety standards, such as OSHA. The length of open trench, and the time it is left open, should be limited to avoid inducing slope movement above the trench. The recommended maximum length of open trench is 15 meters and the recommended maximum time the trench is left open is 2 days.

The blanket drain, if installed, should be 0.5 meters thick and should be installed as low as practical in the embankment section. The drain should extend into the embankment far enough so that the bottom of the drain is 2.0 meters (minimum) below the ground surface, as shown on Figure 2-2. The blanket drain should have a minimum slope of 2 percent perpendicular to the road alignment. Measures will be required where the drain intersects the ground surface to ensure the drain is not blocked. These may include lateral drain pipes and rock armoring of the slope, as shown on the figure. Excavation for the drain may need to be staged to avoid destabilizing the slope by removing too much of the embankment toe and oversteepening the slope during construction.

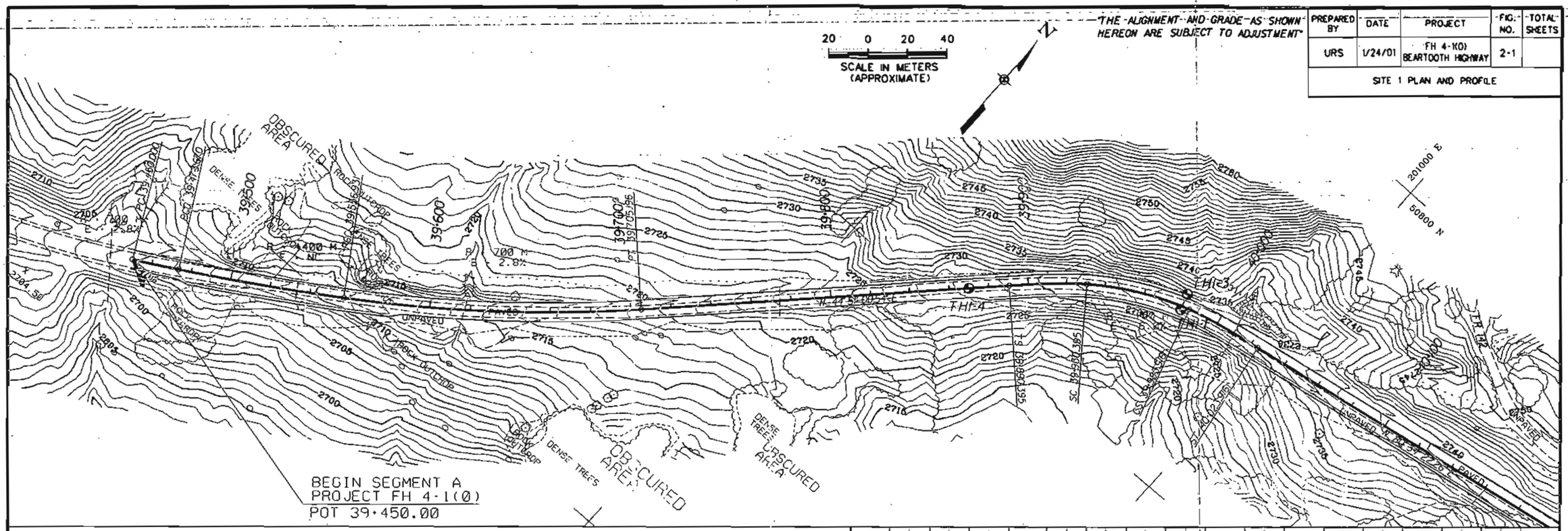
An alternative to installing the blanket drain is the use of flatter embankment slopes of 1:4 (vertical:horizontal), as shown on Figure 2-2. The analyses presented in Appendix H show the minimum theoretical factor of safety for a 1:4 embankment slope is 1.4 if the water table is as shown on Figure H-2. This is the water table assumed to exist with an operating trench drain, but no blanket drain. The lower factor of safety shown on Figure H-2 for global stability is not believed to be realistic for the reasons explained previously.

We recommend that the piezometers and inclinometer be monitored at least twice annually during the design and construction phases of this project. The data should be reviewed and used to evaluate the interpretations and recommendations presented herein. The need for further investigation is not anticipated at this site.

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/01	FH 4-1(0) BEARTOOTH HIGHWAY	2-1	
SITE 1 PLAN AND PROFILE				

"THE ALIGNMENT AND GRADE AS SHOWN HEREON ARE SUBJECT TO ADJUSTMENT"

20 0 20 40
SCALE IN METERS (APPROXIMATE)



LEGEND FOR SITE 1

- ASPHALT AND BASE COURSE PAVEMENT SECTION
- MAN-PLACED FILL, CLAY, SANDY, BROWN, MOIST
- CLAY, CL, MEDIUM STIFF TO VERY STIFF, GRAY-BROWN, BROWN, GRAY, BROWN-GRAY, SILTY, SOME SHALE, LIMESTONE AND GRANITE GRAVEL AND COBBLES, MOIST, INCLUDES SILTY SAND (SM)
- SAND, SC, GP/SP, SM/SC, SM, SW-SM, MEDIUM DENSE TO VERY DENSE, FINE TO COARSE GRAINED, SILTY, CLAYEY WITH SOME UNWEATHERED LIMESTONE, SHALE AND GRANITE INCLUSIONS, SLIGHTLY MOIST TO SATURATED, INCLUDES CLAYEY GRAVEL WITH SAND (GC)
- BOULDERS, WITH POORLY GRADED SAND, SILT AND GRAVEL (SP-SM) MATRIX. (POSSIBLE BEDROCK)
- SHALE, WEAK TO MODERATELY STRONG, HIGHLY WEATHERED, MICACEOUS

THI-4 ⊗ TEST HOLE LOCATION

— 12 INDICATES TOP OF INTERVAL FOR STANDARD PENETRATION TEST (SPT). INDICATES THE BLOWCOUNT (N-VALUE) FOR THE FINAL 300 MM OF 450 MM TOTAL PENETRATION IS 12. THE SPT CONSISTS OF RECORDING BLOWS OF A 63.5 KG HAMMER FALLING 760 MM TO DRIVE A 50 MM O.D. SPLIT SPOON SAMPLER. WHERE TWO VALUES ARE SHOWN SEPARATED BY A SLASH, REFUSAL WAS MET BEFORE 450 MM PENETRATION. THE FIRST NUMBER IS THE BLOWCOUNT, THE SECOND NUMBER IS THE PENETRATION DEPTH.

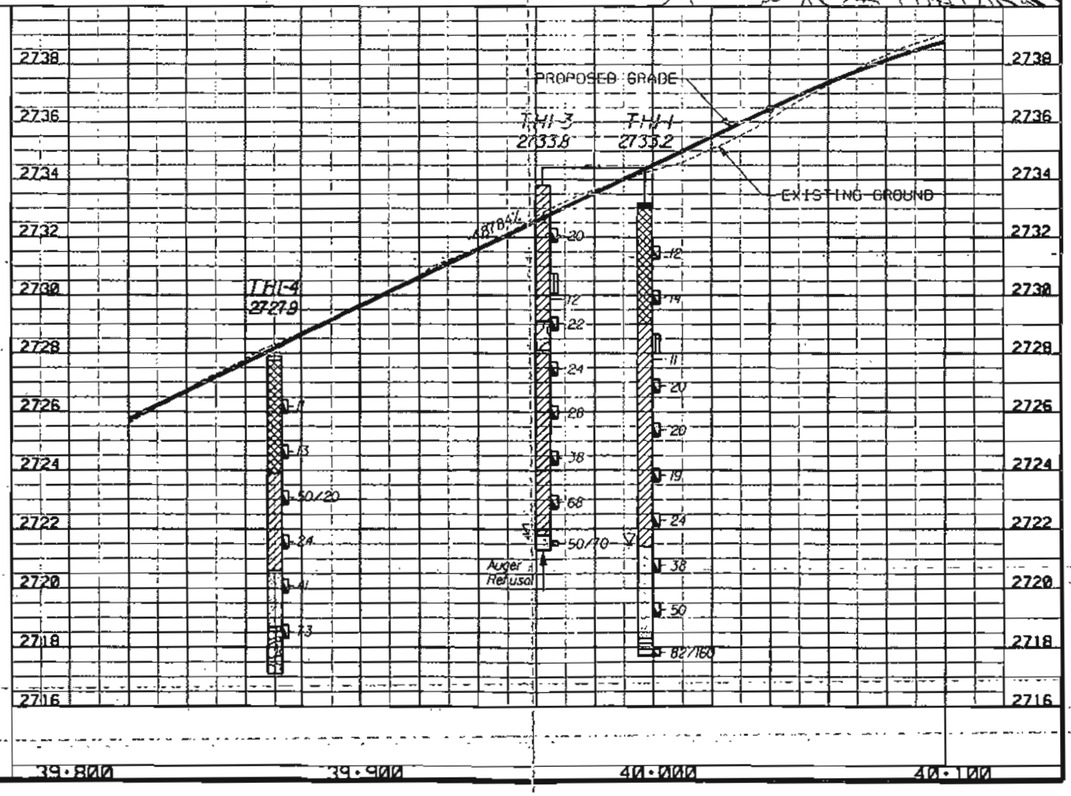
50/20 INDICATES 50 BLOWS OF A 63.5 KG HAMMER FALLING 760 MM WERE REQUIRED TO DRIVE A 75 MM O.D. CALIFORNIA TUBE SAMPLER 20MM. WHERE A SINGLE VALVE IS SHOWN, IT IS THE BLOWCOUNT FOR THE FINAL 300 MM OF 450 MM TOTAL PENETRATION.

INDICATES INTERVAL THROUGH WHICH A 72 MM SHELBY TUBE SAMPLER WAS PUSHED.

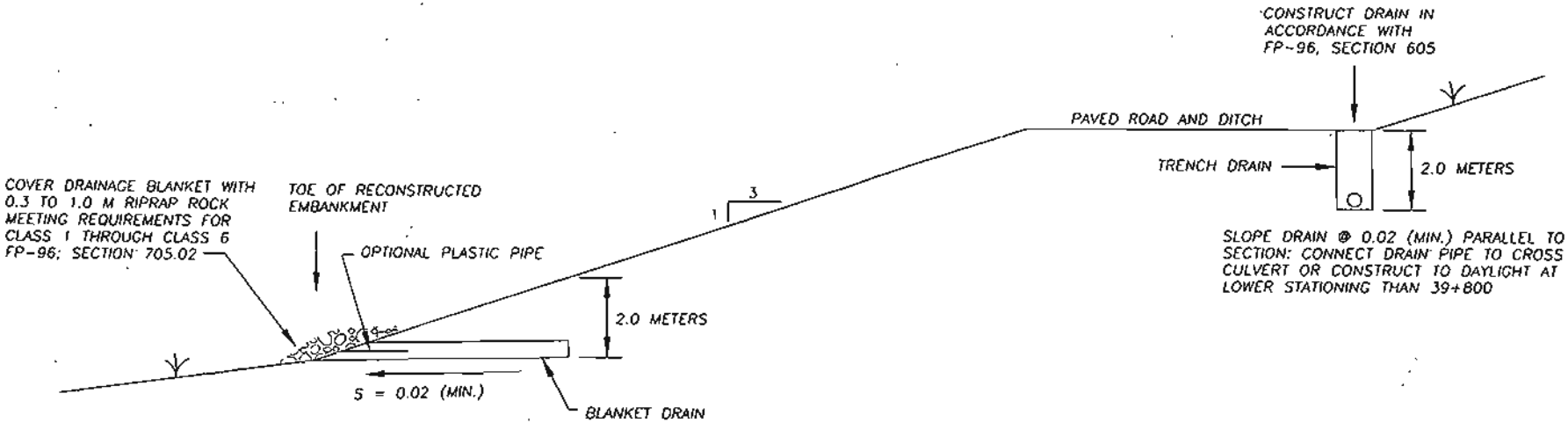
INDICATES WATER LEVEL AT TIME OF DRILLING.

NOTES:

1. TEST HOLES WERE DRILLED ON 8/17/00 THROUGH 9/15/00 BY HAZTECH DRILLING, INC. USING A BK-81 (TRUCK MOUNT) RIG AND A 114 MM I.D. HOLLOW STEM AUGER.
2. BASE MAPS OBTAINED FROM DESIGN FIGURES PROVIDED BY THE FHWA.
3. TEST HOLE LOCATIONS ARE APPROXIMATE AND WERE LOCATED BY TAPE MEASUREMENT ON OR BESIDE THE EXISTING ROAD AT DESIGNATED STATIONS. MEASUREMENTS WERE REFERENCED TO MARKINGS AND STAKES PLACED ALONG THE EXISTING ROAD BY THE FHWA.
4. TEST HOLE LOGS IN THIS REPORT ARE SUBJECT TO LIMITATIONS, EXPLANATIONS, AND CONCLUSIONS OF THIS REPORT. SEE INDIVIDUAL LOGS IN APPENDICES FOR MATERIAL DESCRIPTIONS AT EACH TEST HOLE.

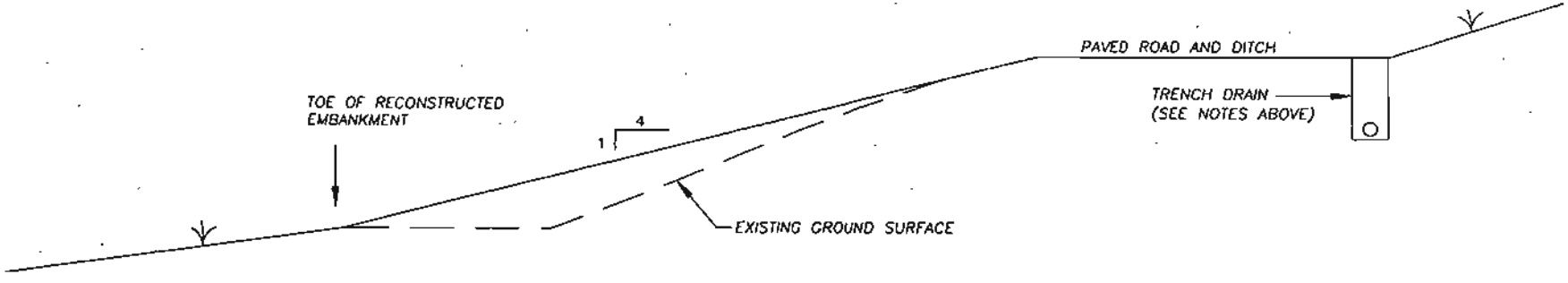


BRTH2-1.DGN



1:3 (V:H) ALTERNATIVE FOR MITIGATING INSTABILITY

NOT TO SCALE



1:4 (V:H) ALTERNATIVE FOR MITIGATING INSTABILITY

NOT TO SCALE

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	3/27/01	FH 4-1(0) BEARTOOTH HIGHWAY	2-2	
SITE 1				

BRTH-2-1.DWG

3.1 INVESTIGATION

Four (4) test holes, THB-1, THB-2, THB-4, and THB-5, were drilled using either H- or N-size core barrels. The holes were located at the abutments and piers for the Beartooth Ravine Bridge alignment for the 50 km/hr option. THB-1 and THB-5 were located at the abutments and drilled with an H-size core barrel. THB-2 and THB-4 were located at the piers and drilled with an N-size core barrel. SPT samples were taken every 1.5 meters except in bedrock or if the core run ended in a boulder.

The holes were completed with 50 mm (2-inch) I.D. schedule 40 PVC pipe grouted in place, in preparation for the cross hole seismic tomography performed at this site. The seismic tomography survey and results are discussed in this section and presented in Appendix E.

3.1.1 Site Characteristics

The Beartooth Ravine bridge is the largest and highest planned bridge, having three spans of 43 meters, 54 meters, and 43 meters, as shown on Figure 3-1. The bridge deck is designed to be between approximately 8 to 15 meters above the ground surface. The bridge spans a broad ravine which has boulder talus and fill exposed on the surface. No evidence of channelized flow was observed in the ravine, which is partially forested. The ravine slopes are approximately 1:2 (vertical:horizontal) at the location of the planned piers. The ground surface elevation varies from 2687 to 2703 across the length of the bridge.

The existing road generally follows the contour around the ravine, so it is several meters higher on the slope than the bridge foundation. The existing road will be abandoned once the bridge is constructed.

3.1.2 Subsurface Characteristics

The test hole locations and summary logs are shown on Figures 3-1 and 3-2, and individual logs and photos are provided in Appendix B. A summary of test hole locations and results is shown in Table 3-1. Laboratory test results are presented in Appendix D.

Table 3-1

TEST HOLE SUMMARY, BEARTOOTH RAVINE BRIDGE

Test Hole Number	Stationing	Offset (m)	Elevation (m)	Depth to Bedrock (m)	Depth to Water (1) (m)	Test Hole Depth (m)
THB-1	41+400	0	2702.8	1.1	N.E.	8.6
THB-2	41+443	0	2675.8	5.7	N.E.	11.0
THB-4	41+497	0	2680.9	4.9	N.E.	10.0
THB-5	41+540	0	2694.2	6.1	N.E.	11.3

Notes: (1) Encountered in test hole at time of drilling. N.E. = Not Encountered

THB-1 was drilled on the existing roadway at the west abutment of the bridge. Slightly weathered granite and gneiss bedrock was encountered approximately 1 meter below the road surface. Bedrock is also present in the cutslope, across the existing road from THB-1. The bedrock was cored with an H-size core barrel and recovery ranged from 70 to 100 percent. The Rock Quality Designation (RQD) varies from 68 to 80. RQD is defined as 100 times the total length of sound core over 100 mm in length divided by the length of core run. Groundwater was not encountered during drilling. Based on the topography, it is anticipated that groundwater is below the planned abutment elevation (2700 m). The unconfined compressive strength obtained in the laboratory using one sample from 7.8 meter depth is 57 MPa. The estimated unconfined compressive strength from point load testing varies from 34 to 200 MPa, as shown in Table 3-2.

THB-2 and THB-4 were drilled at the bridge pier locations. Slightly weathered granite and gneiss bedrock was encountered approximately 4.9 to 5.7 meters below the ground surface. The overburden is talus with particle sizes from sand through boulders. The longest length of boulder cored was 0.5 meters, but larger boulders, up to approximately 5 meters diameter, are present on the talus slope and may be encountered in the subsurface during construction.

The bedrock at THB-2 and THB-4 was cored with an N-size core barrel; recovery ranged from 73 to 100 percent, and was generally 100 percent. The RQD varies from 48 to 100. Groundwater was not encountered during drilling. Based on the topography and observed joint staining, it is anticipated that groundwater may be present in bedrock discontinuities. The unconfined compressive strength of two samples from 7.8 to 10 meters depth is 37 to 86 MPa. The estimated unconfined compressive strength from point load testing varies from 58 to 165 MPa.

THB-5 was drilled at the east abutment of the bridge. Slightly weathered granite and gneiss bedrock was encountered approximately 6 meters below the road surface. Bedrock is also present in the cutslope, across the existing road from THB-5. The overburden is fill and talus with particle sizes from sand through boulders. The longest length of boulder cored was 0.2 meters.

Bedrock was cored with an H-size core barrel and recovery was generally 100 percent. The RQD varies from 38 to 100, and is less than 92 only in the upper 1.5 meter core run. Groundwater was not encountered during drilling. Based on the topography, it is anticipated that groundwater is below the planned abutment elevation. The unconfined compressive strength of a sample from 6.2 meters depth is 71 MPa. The estimated unconfined compressive strength from point load testing varies from 134 to 200 MPa. The results are summarized in Table 3-2.

Table 3-2

BEARTOOTH RAVINE BRIDGE LABORATORY TEST RESULTS

Test Hole	Depth (m)	UCS[1][2] (MPa)	UCS[2] (MPa)
THB-1	4.30	200	
THB-1	6.40	187	
THB-1	7.80	34	57
THB-2	5.80	132	
THB-2	7.00	165	
THB-2	10.80	58	37
THB-4	7.00		86
THB-4	8.60	74	
THB-4	9.60	108	
THB-5	6.17		71
THB-5	8.60	200	
THB-5	10.65	134	

(1) Values determined from point load testing of core (ASTM D5731), as described on data sheets in Appendix D.
(2) UCS = Unconfined Compressive Strength (ASTM D2938), as described on data sheets in Appendix D.

Standard Penetration Tests (SPT's) were generally conducted at the end of each 1.5 meter core run, except where the run ended in a boulder or in bedrock. N-values in the overburden range from 24 to 46 and, where penetration was too little to measure an N-value, blowcounts range from 62 blows for 350 mm to 50 blows for 100 mm. The considerable presence of gravel, cobbles, and boulders has probably impacted some of the SPT results. The presence of some relatively low N-values indicates there are intervals of loose to medium dense soils, or possibly voids. The summary logs show percent core recovery for core runs in the overburden and the SPT results. Core intervals include overcoring (reaming) the interval of the hole sampled with the SPT split spoon sampler. Despite coring through this disturbed material, recovery (based on the full length of the run) varied from 10 to 82 percent and was generally 30 to 60 percent. The zones with higher recovery may be indicative of a high percentage of larger particle sizes in the overburden.

Seismic Tomography

The seismic tomography survey results were used to evaluate variations in the subsurface material properties at locations where test holes were not drilled. In addition, seismic tomography can be used to develop a three-dimensional image, and two-dimensional cross sections, of the subsurface. The seismic tomography survey procedure and complete results are presented in Appendix E.

Our experience with this exploration method suggests that the spatial variation of material types, as indicated by the seismic velocity, can be useful in predicting variability in foundation and excavation conditions. The success of the method, however, in providing accurate velocities is dependent on site conditions, the subsurface seismic raypath coverage achieved, and how the seismic tomography model is constrained. We understand that for these data, the tomography

processing was constrained to eliminate travel times resulting in velocities exceeding 5,000 m/s. Additionally, an input bedrock velocity of 4,000 m/s was applied. These constraints would tend to result in average velocities of the tomogram being computed during the iteration process to be lower than 5,000 m/s since this is an upper bound value. Comparing the tomogram results to the seismic refraction results previously completed in the area (URS, 1999) indicates that the tomogram velocities are lower than those observed from the refraction (3,040 to 6,220 m/s).

The particular tomogram algorithm used for these data also has difficulty mapping transitions between seismic interfaces where a profound and substantial seismic velocity contrast exists. The algorithm tends to smooth the velocities for such a scenario, even though geologically the boundary may be a discrete and distinct interface. Because there is little or no weathering profile at this site (based on drilling results), the talus/bedrock contact could be an example of a discrete interface where smoothing in the tomogram has occurred.

The results show that the higher velocity materials, which are interpreted to be bedrock, are somewhat parallel to the ground surface. There appear to be some troughs perpendicular to the road where bedrock is deeper, but the pier locations are in areas where the depth to higher velocity materials is average or less than average. The velocity tends to range from about 2000 to 3000 m/s at depths indicative of the upper 5 meters of bedrock in the test holes. Seismic velocities as high as 4000 m/s to 5500 m/s would not be surprising for slightly weathered granite with the percent recovery and RQD's observed. Thus, using the test hole observations to calibrate the tomography results, seismic velocities as high as 5500 m/s should be anticipated in the upper 5 meters of foundation bedrock.

3.2 INTERPRETATION AND ANALYSIS

Observations and data on soil and bedrock were interpreted and analyzed to develop geotechnical recommendations. At this site, the soil overburden has been evaluated primarily for temporary excavation slopes and lateral earth pressure. The bedrock has been evaluated for the bridge foundation.

At the pier locations the material overlying the bedrock is talus. Test holes and observations indicate a wide range of particle sizes is present, from sand through large boulders. Due to the nature of the deposit, we expect the finer material to generally fill the voids between the coarser materials. Some voids between larger materials may be completely or partially filled very loosely. Boulders and loose soil may cause difficulty in excavation and may locally present hazardous conditions (with respect to rockfill) in excavation slopes. The boulders and cobbles are granitic, being derived from the outcrop above the road, and probably have unconfined compressive strength similar to or less than the bedrock in this area.

At the abutments, the existing man-placed fill comprises at least part of the overburden. Based on observations from drilling at the abutments and at Site 2, the fill is similar to the talus. The distinction between the two is based on the following general characteristics. The talus is mostly gray and the fill is mostly brown. The talus has a larger mean size, and lower recovery of matrix material by SPT sampling, than the fill. Some talus matrix material has organic characteristics. We expect that the fill was generated from nearby rock cuts, and was placed with little compaction at the time the road was constructed.

The bedrock quality has been evaluated based on the unconfined compressive strength of intact rock, the RQD, and the Rock Mass Rating (RMR). The RMR is a rating system that incorporates strength of intact rock, RQD, the spacing, condition, and general orientation of discontinuities, and groundwater conditions, to characterize the rock mass. Table 3-2 presents the interpretation of bedrock quality and estimated ranges of engineering parameters based on these methods. In general, these characterizations show that the bedrock quality is fair to good. The bedrock quality appears to be nominally better at the abutments than at the piers, but this difference is not significant enough to impact foundation recommendations.

The values of Young's modulus for the rock mass (E_m), and for the cohesive (c) and frictional (ϕ) strength parameters, are based on published correlations to the RMR. They are presented here to assist in characterizing the relative quality of the rock mass. These correlated values are part of the information used to develop the geotechnical recommendations for design presented in Section 3.3.

Table 3-3

INTERPRETATION OF BEDROCK QUALITY
BEARTOOTH RAVINE BRIDGE

Location	Characteristic RQD	Characteristic Rock Quality[1]	RMR[2] Range	Characteristic RMR Class[2]	E_m (GPa)[3]	c (kPa)[2]	ϕ (deg)[2]
West Abutment	72	Fair/Good	68-80	II (Good)	28-56	350-400	40-45
Pier at 41+443	60	Fair	62-80	II (Good)	20-56	300-400	35-45
Pier at 41+497	63	Fair	58-68	II (Good)	16-28	300-350	35-40
East Abutment	90[4]	Good	56-83	II (Good)	16-67	300-400	35-45

Notes:

[1] Based on RQD, Deere (1964). Classification for RQD <25 = very poor, 25 < 50 = poor, 50 < 75 = fair, 75 < 90 = good, 90 < 100 = excellent
[2] Rock Mass Rating, Bieniawski (1989), classification for RMR < 20 = V (very poor), 21 to 40 = IV (poor), 41 to 60 = III (fair), 61 to 80 = II (good), 81 to 100 = I (very good). Cohesion and friction ranges are based on RMR class. For Class V: ϕ < 15°, c < 100 kPa; for Class IV: ϕ = 15-25°, c = 100-200 kPa; for Class III: ϕ = 25-35°, c = 200-300 kPa, for Class II: ϕ = 35-45°, c = 300-400 kPa; for Class I: ϕ > 45, c > 400 kPa.
[3] Young's Modulus of Rock Mass based on RMR, $E_m = 10^{(RMR-10)/40}$, Serafin and Pereira (1983)
[4] Excludes first core run

3.3 RECOMMENDATIONS

3.3.1 Excavation Slopes

Temporary excavation slopes should be designed by the contractor. We recommend temporary slopes of less than 4 meter height be cut no steeper than 1:1 (vertical:horizontal). The slopes may be locally steeper (and flatter) because of boulder inclusions. Precarious boulders and cobbles on the excavation slope should be removed and may result in slopes somewhat less steep than 1:1. Large boulders should be split so that depressions (and local oversteepening) are not caused by removal of the entire boulder. Shallow sloughing along the excavated cut slope should be expected and controlled to prevent hazards to construction workers and highway traffic. Recommended measures for controlling sloughing include the following:

- Loosened boulders and other loose materials should be removed from the slopes. The slopes should be observed routinely so that these materials can be quickly identified and removed.

- Temporary drainage ditches should be constructed at the top of the slope to minimize runoff down the slopes. Runoff down the slope should not be allowed.
- The contract specifications should limit the amount of time a temporary excavation slope is left open. We recommend a maximum of 30 days.
- If signs of slope instabilities still persist after these measures have been implemented, the slope should be excavated to a flatter, more stable inclination, or supported by other means.

Cuts of 4 meter height, or more, should be no steeper than 1:1.5 (vertical:horizontal). All cuts should comply with applicable health and safety standards and should be evaluated for stability. Reinforcement or shoring may be required for some slopes. If steeper slopes are planned than recommended herein, reinforcement or shoring should be included. Reinforcement and shoring should be designed by a licensed engineer.

3.3.2 Foundations

The Beartooth Ravine Bridge site appears suitable for the proposed construction from a geotechnical engineering point of view. The following foundation systems were evaluated for bridge support at the site:

- micropiles drilled into the bedrock;
- straight shaft piers drilled into the bedrock;
- spread footings bearing on bedrock at abutment No. 1.

Micropiles, drilled shafts, and spread footings should be installed in accordance with the Project Specifications.

Micropiles

Micropiles, also known as minipiles or pin piles, are small diameter, high capacity drilled and grouted piles. Micropiles are installed with an air track percussion drill rig that requires a working bench about 4.5 meters wide. Micropiles range in diameter from 127 to 305 mm in diameter and consist of steel pipe (casing), steel reinforcement and cement grout. The piles obtain their capacity from skin friction in the supporting strata and can provide both axial and uplift capacity. Micropiles should be designed and constructed in accordance with FHWA report, FHWA-SA-97-070, Micropile Design and Construction Guidelines Implementation Manual (FHWA, 2000), and the recommendations presented herein. Type A micropoiles are recommended (FHWA, 2000).

The geotechnical design for micropiles should be based on an allowable bond stress of the soil/rock bond of 0.5 MPa. This bond stress is based on reported nominal bond stress values (FHWA, 2000), consideration of acceptable settlement in jointed rock mass using methodology for drilled piers, and a factor of safety of 2.5. Micropiles in the range of 229 to 254 mm diameter will have capacities of 360 to 400 kN/m. At least one micropile at each bridge pier or abutment should be drilled at least 6 meters below the bedrock contact to confirm the interpretation of the top of bedrock, and that piles are not founded on a boulder.

The structural design of micropiles should be based on an allowable load of 0.47 times the lower of the yield stress of the steel casing or reinforcement, and 0.4 times the unconfined compressive strength of the grout. The structural design should also provide for sufficient length for the grout to steel bond.

Micropiles should be constructed with permanent steel casing, with a casing plunge extending at least 0.5 m into the underlying bedrock. The casing should meet the physical properties of ASTM A-252 Grade 3, except that the minimum yield strength should be 552 MPa. Reinforcing steel may be Grade 60, 75 or 80 reinforcing rod (ASTM A615, A616 or A617). The grout should consist of neat cement and water mixed with a high shear colloidal type mixer. The grout should have a fluid consistency, a water/ cement ratio of about 0.45 and a minimum unconfined compressive strength of 27.6 MPa at 28 days.

Micropiles provide lateral resistance from the horizontal response of the adjacent soils and can sustain significant lateral deflection within the available structural pile capacity. Increased bending capacity can be provided by increasing the diameter of the pile or the thickness of the pipe. Lateral resistance can also be provided by battered piles. The lateral design parameters presented in the Drilled Piers section may be used for the design of micropiles.

Drilled Piers

Piers should be designed for axial loads using a combination of end bearing and skin friction in the bedrock. Allowable end bearing pressure of 5 MPa and allowable skin friction of 0.5 MPa for downward, compressive loads, is recommended. Theses values are based on intact rock strength, RQD, and a factor of safety of 2.5. Settlement of drilled pier foundations designed according to the above criteria should be 15 mm or less. Uplift may be resisted by using 50% of the skin friction value.

Straight shaft piers should be drilled a minimum of 1.5 meters into bedrock. Design bedrock penetration should be measured below the bottom of temporary casing. The depth and quality of the bedrock should be evaluated under the direction of a qualified geotechnical engineer. A probe hole is recommended in at least one drilled pier at each bridge pier or abutment. The hole should be drilled from the bottom of the drilled pier for a depth of at least 3.5 meters to confirm that the pier is founded on bedrock, not a boulder. The hole should be probed to identify potential discontinuities and material contacts.

Piers should be considered to work in group action if the horizontal spacing is less than three pier diameters. A minimum practical horizontal spacing between piers of at least three diameters should be maintained, and adjacent piers should bear at about the same elevation. The capacity of individual piers must be reduced when considering the effects of group action. Capacity reduction is a function of pier spacing and the number of piers within a group. If group action analyses are necessary, capacity reduction factors can be provided for the analyses.

All piers should be reinforced full depth for the applied axial, lateral and uplift stresses imposed. Minimum reinforcement of at least one-half percent of the cross-sectional area of each pier is recommended.

Drilling to design depths should be possible with high capacity augers and coring barrels. Granular soil conditions indicate that temporary steel casing may be required to properly drill and clean piers prior to concrete placement. Groundwater, if present, should be removed from

each pier hole prior to concrete placement. Drilling fluids may be needed and fluid loss should be expected. Pier concrete should be placed immediately after completion of drilling and cleaning. If pier concrete cannot be placed in dry conditions, a tremie should be used for concrete placement. Due to potential sloughing and raveling, foundation concrete quantities may exceed calculated geometric volumes. Drilled shaft construction should be in accordance with FP-96, Section 565.05.

Casing should be withdrawn in a slow continuous manner maintaining a head of at least 1.5 meters of concrete to prevent infiltration of water or the creation of voids in pier concrete. Pier concrete should have a relatively high fluidity when placed in cased pier holes or through a tremie. Concrete with slump in the range of 125 to 200 mm is recommended.

Free-fall concrete placement in piers is acceptable if provisions are taken to avoid striking the concrete on the sides of the hole or reinforcing steel. Hoppers with an elephant trunk, or other approved device, are recommended.

Pier bearing surfaces must be cleaned prior to concrete placement. The top of the piers should be smooth and free of protrusions. Drilled shaft installation should be observed under the direction of a qualified geotechnical engineer. The bearing surface and pier configuration should be observed.

Lateral resistance to horizontal forces can be provided by the bending resistance of piers and piles. Lateral resistance can also be provided by battered piles. The vertical and horizontal components of the load will depend on the batter inclinations. Pile batters limited not to exceed 1:0.25 (vertical:horizontal) are recommended.

We understand that resistance to lateral load will be evaluated by the L-Pile or COM 624 computer programs. Soil properties for lateral capacity analysis by using L-PILE Plus 3.0 are presented as follows:

Material	Soil Type*	Subgrade Modulus (GPa/m)	ϕ (degrees)	c (kPa)	D_{50}	Effective Unit Weight (kN/m^3)
Compacted Embankment Fill	4	41	35	0	---	21.2
Sand Gravel Cobbles	4	27	35	0	---	11.8
Bedrock	2	543	0	239	0.007	11.8

*Soil Type 4 is sand
*Soil Type 2 is stiff clay below the water table

Spread Footings

Abutment No. 1 may be founded on spread footings bearing on the underlying bedrock. Footings placed on undisturbed bedrock may be designed for an allowable bearing pressure of 1.0 MPa. Total settlement of footings designed and constructed as discussed in this section is estimated at less than 25 mm. Footings should be placed at least 1.5 m below the surrounding exterior grade to provide adequate frost protection. The lateral resistance of footings placed on the natural bedrock will be a combination of the sliding resistance on the bottom of the footing and the passive earth pressure against the side of the footing. Resistance on the bottom of the

footing can be calculated based on a coefficient of friction of 0.4. Passive pressure against the side of the footing can be calculated using an equivalent fluid unit weight of 9.0 kN/m^3 assuming properly compacted granular backfill.

Abutment Walls and Retaining Walls

The use of free draining granular soils for backfilling behind bridge abutments and retaining walls is recommended. Granular soils result in lower lateral earth pressures and provide better subsurface drainage. Abutments and retaining walls should be provided with an underdrain system consisting of a front drainage zone and an underdrain pipe at the base of the wall. The front drainage zone may consist of a vertical layer of either a granular soil layer meeting FP-96 Section 210 or a geocomposite drain meeting FP-96 Section 605.05. The underdrain pipe should consist of a minimum 100 mm PVC pipe encased in a geotextile wrapped granular layer or a geocomposite sheet drain. The pipe should be sloped to daylight at a minimum slope of 2 percent.

Abutment walls and retaining walls which can be expected to deflect sufficiently to mobilize the full active earth pressure condition should be designed for a lateral earth pressure computed on the basis of an equivalent fluid unit weight of 5.9 kN/m^3. This pressure assumes a free draining backfill. Retaining structures should be designed for appropriate surcharge pressures such as traffic, construction equipment and upward sloping backfill.

3.3.3 Other Considerations

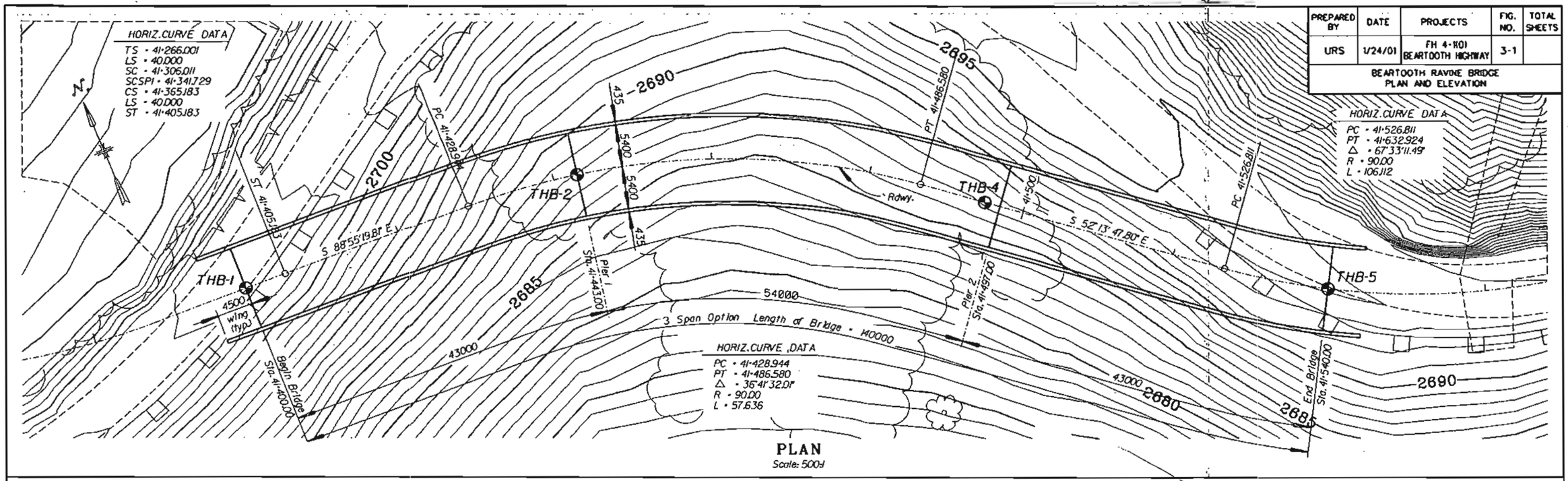
Rockfall protection. Rockfall could damage the bridge, especially on the eastern side of the ravine where rock slopes and the rockfall hazard are both higher than elsewhere. We recommend the abandoned existing road be graded as a berm and ditch to provide for rockfall collection. Access should be provided to allow cleanout and maintenance, as needed.

Talus consistency. Grouting of the PVC casing into the test holes was completed with difficulty. Grout take was not measured, but it was necessary to complete grouting in several stages, allowing grout setup between stages, and to add sand during grouting. This indicates a wide extent of voids in the talus.

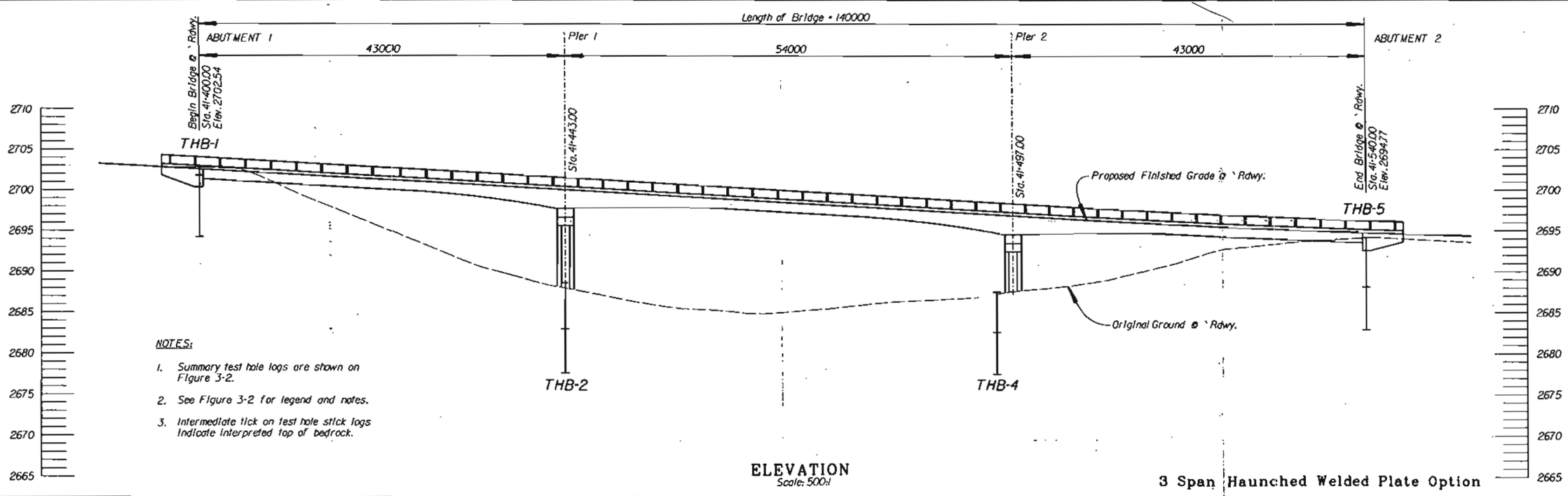
Further investigation. No further investigation is recommended at this site unless pier or abutment locations are changed. If piers or abutments are moved, test holes and testing should be competed at the new locations.

PREPARED BY	DATE	PROJECTS	FIG. NO.	TOTAL SHEETS
URS	1/24/01	FH 4-101 BEARTOOTH HIGHWAY	3-1	

BEARTOOTH RAVINE BRIDGE
PLAN AND ELEVATION



PLAN
Scale: 500:1



NOTES:

1. Summary test hole logs are shown on Figure 3-2.
2. See Figure 3-2 for legend and notes.
3. Intermediate tick on test hole stick logs indicate interpreted top of bedrock.

ELEVATION
Scale: 500:1

3 Span Haunched Welded Plate Option

PREPARED BY MK/CENTENNIAL (Base map only)

SCALE: As Shown

DATE: October 24,2000

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION, DENVER, COLORADO

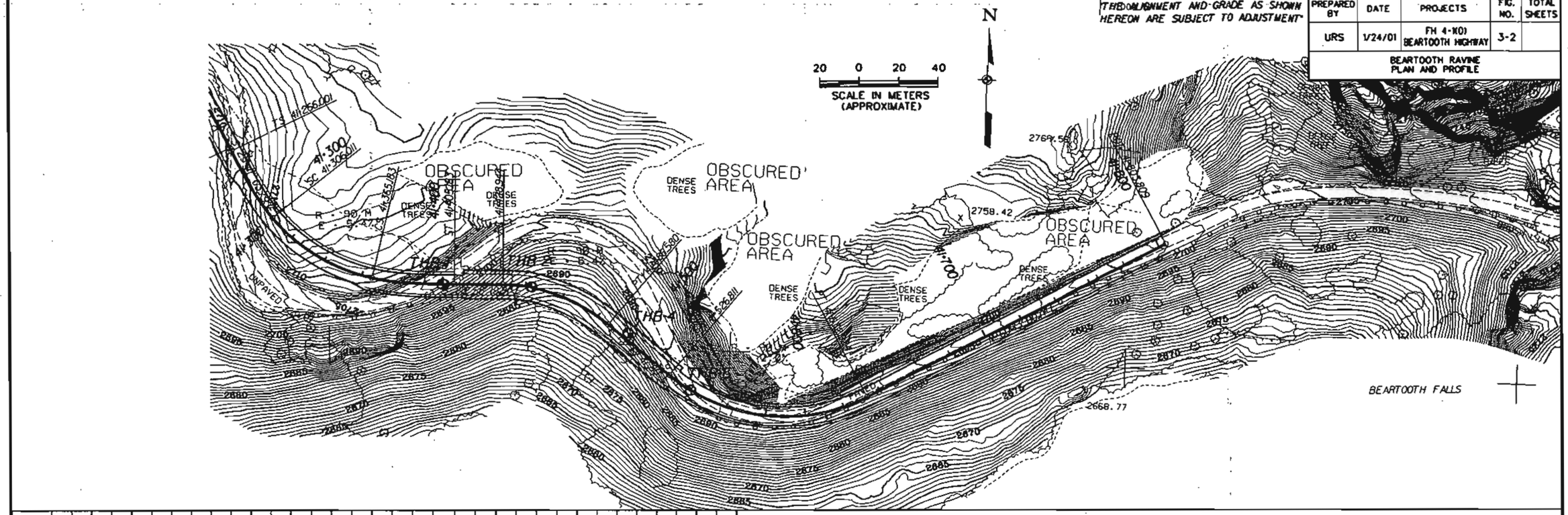
BRIDGE NAME Beartooth Ravine Bridge (50 km/hr alignment)

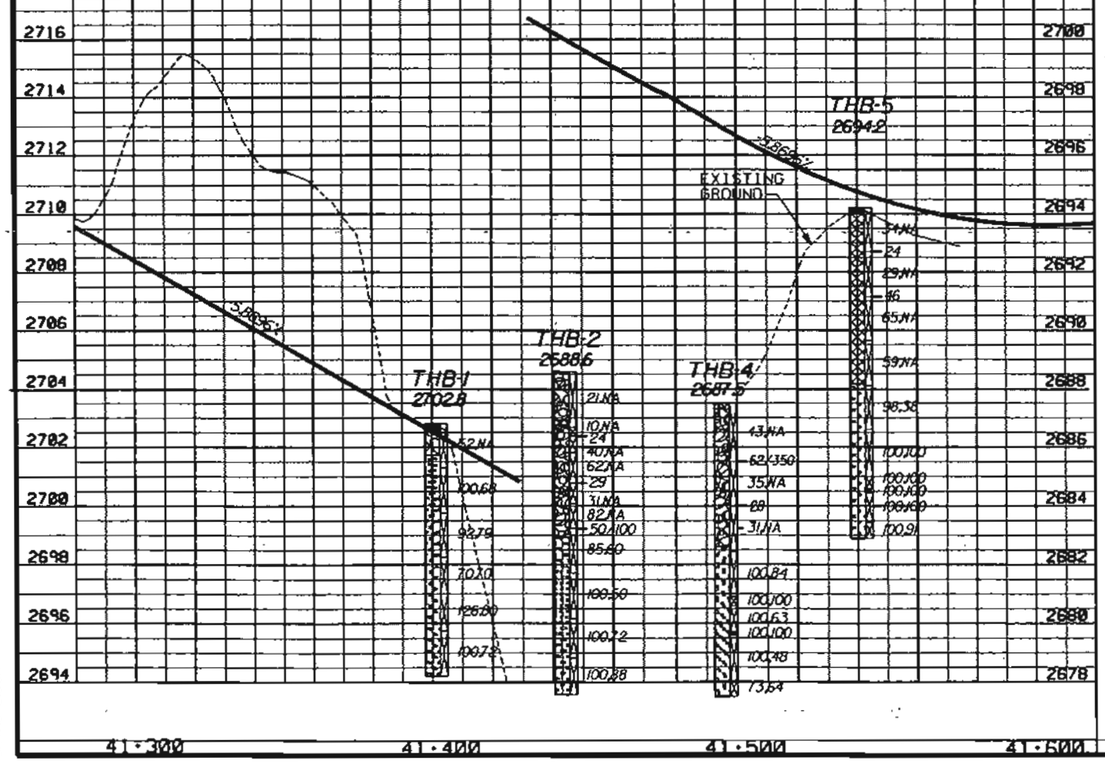
PRELIMINARY LAYOUT

BRTH2-1.DGN

"THE ALIGNMENT AND GRADE AS SHOWN HEREON ARE SUBJECT TO ADJUSTMENT"

PREPARED BY	DATE	PROJECTS	FIG. NO.	TOTAL SHEETS
URS	V24/01	FH 4-1(0)1 BEARTOOTH HIGHWAY	3-2	
BEARTOOTH RAVINE PLAN AND PROFILE				





LEGEND FOR BEARTOOTH RAVINE BRIDGE

- ASPHALT AND BASE COARSE PAVEMENT SECTION
- MAN-PLACED FILL, GRAVEL, COBBLE, AND BOULDERS, GRAY TO BROWN, SANDY TO GRAVELLY MATRIX, MOIST
- GRAVEL, COBBLE, AND BOULDERS, MEDIUM DENSE TO DENSE, (BASED ON BLOWCOUNTS), BROWN, REDDISH-BROWN, NO RECOVERED MATRIX TO SANDY MATRIX.
- GRANITE TO GNEISS, MODERATELY STRONG TO VERY STRONG, PINK TO BLACK, COARSE GRAINED, SLIGHTLY WEATHERED, GNEISS IS GENERALLY WEAKER AND MORE WEATHERED THAN GRANITE
- GRANITE, STRONG TO VERY STRONG, BLACK TO PINK, COARSE GRAINED, SLIGHTLY WEATHERED

THB-1 ⊗ TEST HOLE LOCATION

— 12 INDICATES TOP OF INTERVAL FOR STANDARD PENETRATION TEST (SPT). INDICATES THE BLOWCOUNT (N-VALUE) FOR THE FINAL 300 MM OF 450 MM TOTAL PENETRATION IS 12. THE SPT CONSISTS OF RECORDING BLOWS OF A 63.5 KG HAMMER FALLING 760 MM TO DRIVE A 50 MM O.D. SPLIT SPOON SAMPLER. WHERE TWO VALUES ARE SHOWN SEPARATED BY A SLASH, REFUSAL WAS MET BEFORE 450 MM PENETRATION. THE FIRST NUMBER IS THE BLOWCOUNT, THE SECOND NUMBER IS THE PENETRATION DEPTH.

⊠ 90.39 INDICATES SPLIT TRIPLE-TUBE CORE BARREL WAS USED TO ADVANCE HOLE. 90.30 INDICATES 90% RECOVERY FOR INTERVAL INDICATED AND A ROCK QUALITY DESIGNATION (RQD) OF 30. RQD IS 100 TIMES THE TOTAL LENGTH OF SOUND CORE OVER 100 MM (4 IN.) LONG DIVIDED BY THE LENGTH OF RUN INDICATED. WHERE SPT INTERVALS PRECEDE CORE INTERVALS, THE SPT INTERVAL IS SUBSEQUENTLY OVER-CORED. RECOVERY AND RUN ARE BASED ON A CORE RUN LENGTH THAT INCLUDES THE OVER-CORED LENGTH.

NA NOT APPLICABLE

NOTES:

1. TEST HOLES WERE DRILLED ON 8/17/00 THROUGH 9/15/00 BY HAZTECH DRILLING, INC., USING A BK-81 (TRUCK MOUNT) RIG AND AN HQ SPLIT TRIPLE-TUBE CORE BARREL, EXCEPT AS NOTED HERE. TEST HOLES THB-2, THB-4 WERE DRILLED WITH A DIEDRICH D-25 (SKID MOUNT) RIG AND AN NQ SPLIT TRIPLE-TUBE CORE BARREL.
2. BASE MAPS OBTAINED FROM DESIGN FIGURES PROVIDED BY THE FHWA.
3. TEST HOLE LOCATIONS ARE APPROXIMATE AND WERE LOCATED BY TAPE MEASUREMENT ON OR BESIDE THE EXISTING ROAD AT DESIGNATED STATIONS. MEASUREMENTS WERE REFERENCED TO MARKINGS AND STAKES PLACED ALONG THE EXISTING ROAD BY THE FHWA.
4. STATIONING SHOWN IS FOR THE BEARTOOTH RAVINE 50 KM/HR OPTION, SEGMENT 4A PLANS, JULY 2000.
5. TEST HOLE LOGS IN THIS REPORT ARE SUBJECT TO LIMITATIONS, EXPLANATIONS, AND CONCLUSIONS OF THIS REPORT. SEE INDIVIDUAL LOGS IN APPENDICES FOR MATERIAL DESCRIPTIONS AT EACH TEST HOLE.

8RTH3-2.DGN

4.1 INVESTIGATION

Ten (10) test holes were drilled using either H- or N-size core barrels to investigate conditions below the road and the foundation for the planned fill-side wall. Test holes TH2-2, TH2-5, and TH2-7 were drilled on the talus slope with an N-size core barrel. TH2-1, TH2-3, TH2-4, TH2-6, TH2-8, TH2-9, and TH2-11 were drilled near the outside edge of the existing road with H-size core barrel. SPT samples were taken every 1.5 meters except in bedrock or if the run ended in a boulder. The test hole locations and results are summarized in Table 4-1. The test hole locations are shown on Figure 4-1, summary logs are shown on Figures 4-1, 4-2, and 4-3, and individual logs and photos are provided in Appendix B.

The holes were completed with 50 mm I.D. schedule 40 PVC pipe grouted in place, in preparation for the seismic tomography performed at this site (Appendix E). Geologic outcrop mapping was also performed at this site to augment the mapping conducted in 1999 (URS, 1999). The maps are discussed in this section and presented in Appendix G.

Table 4-1

TEST HOLE SUMMARY, SITE 2

Test Hole Number	Stationing	Offset (m)	Elevation (m)	Depth to Bedrock (m)	Depth to Water[1] (m)	Test Hole Depth (m)
TH2-1	41+610	0	2693.1	3.2	N.E.	7.4
TH2-2	41+610	3.5 R	2691.7	5.2	N.E.	8.6
TH2-3	41+660	1.7 L	2694.6	7.4	N.E.	10.7
TH2-4	41+700	1.5 L	2696.5	9.6	N.E.	12.5
TH2-5	41+700	2.5 R	2695.6	10.7	N.E.	14.0
TH2-6	41+765	0	2699.4	7.6	N.E.	10.7
TH2-7	41+800	7.3 R	2698.0	7.8	N.E.	11.0
TH2-8	41+865	0	2704.4	12.3	N.E.	15.6
TH2-9	41+903	1.6 R	2706.2	3.7	N.E.	7.6
TH2-11	41+964	2.05 R	2708.2	12.9	14.0	16.9

Notes: 1) Encountered in test hole at time of drilling. N.E. = Not Encountered

4.1.1 Site Characteristics

The road is on a cut (left) and fill (right) section through this area. The road travels around a sharp outside corner near Station 41+550 and is relatively straight from 41+600 to 41+850. Bedrock is granite and granite gneiss, fresh to moderately weathered, with several joints as discontinuities. The rock slope between approximate stations 41+500 and 41+600 is about 25 meters high, and the lower 15 meters is cut slope. The existing cut slope has a slope ratio of 2:1 to 5:1 (vertical:horizontal). The cut slope presents a high rockfall hazard. Large blocks have loosened and their release appears imminent.

The rock slope between station 41+600 and 42+000 is about 30 to 50 meters high, and higher in some places. For most of the reach between station 41+600 and 42+000, there is an apron of bedrock with a flatter slope angle near the toe of the slope. The existing rock cut is generally within this apron, however, this topographic detail is not recognizable by the existing topographic survey.

The July 2000 plans for the site show alternatives being considered. The 50 km/hr Beartooth Ravine alternative is understood to be the preferred alternative and is the basis for the sections shown on Figure 4-2 and Figure 4-3 (Sta. 41+800). This alternative calls for rock cuts and a fill-side concrete "Z-wall," to cantilever the outboard lane, through most of the site. The planned cuts have heights ranging to more than 25 meters and the wall has a free-face height of up to about 7 meters.

4.1.2 Rock Slope Characteristics

Face maps of existing rock slopes at approximate stations 41+520, 41+585, 41+775, and 42+020 were prepared to provide information at proposed rock cut slopes. The maps were prepared using photographs of the existing rock slopes to locate and describe discontinuities (joint sets) in the rock mass. Results of the mapping are presented on Figures G-1 to G-3 included in Appendix G. Descriptions of the joint sets are also summarized on the figures, including joint set orientation, width, infilling type and amount, surface shape, roughness, spacing, and RQD, and rock strength estimations. A key to these terms is provided in Appendix G.

Table G-1 is a table of data mapped elsewhere along Site 2 from the 1999 investigation (URS, 1999). The 1999 data were mapped within Areas A through H of the site, which are shown on Figure 4-4. Figure 4-4 is the geologic map of the site (URS, 1999), adapted to reflect the stationing referenced herein. For reference, Photo G-1(a) is in Area H, Photo G-1(b) is in Area G, Photo G-2 is in Area E, and Photo G-3 is in Area B.

Results of the mapping indicate that some joint sets are present in the rock mass that are oriented unfavorably with respect to the proposed cut slope orientations. The cut slope will generally have a strike of 155° to 60°, around the knob between Stations 41+540 and 41+600, and about 60° through Station 41+850. Typical joints are planar to wavy, slightly rough to rough, and have narrow to wide width, with iron staining. Rock mass fracturing ranges from closely to widely spaced.

Seismic Tomography

The seismic tomography results can be used to assess variations in the material properties at locations where test holes were not drilled, such as the knob area between approximate stations 41+540 and 41+640. Seismic tomography can also be used to develop a three-dimensional image of the subsurface. The seismic tomography methodology and results are presented in Appendix E.

There are several limitations of the method applicable to this data that should be kept in mind when assessing the results. We note that our experience with the methodology indicates that spatial variation of subsurface materials, as indicated by the seismic velocity, can be delineated well by tomography. However, the success of the method in providing true seismic velocities is dependent on site conditions, the subsurface seismic raypath coverage achieved, and how the

seismic tomography processing is constrained. For the data near the knob, the tomography processing was constrained to eliminate travel times resulting in velocities exceeding 6,000 m/s. These constraints could tend to result in average velocities of the tomogram being computed during the iteration process to be low if the true velocities exceed 6,000 m/s. Additionally, an input bedrock velocity of 4500 m/s was used. The particular tomogram algorithm used for these data also has difficulty mapping transitions between seismic interfaces where a profound and substantial seismic velocity contrast exists. The algorithm tends to smooth the velocities for such a scenario, even through geologically the boundary may be a discrete and distinct interface.

Tomography results of the knob area are depicted on Figures 14 and 15 of Appendix E. A vertical plane tomogram through THB-5 is shown on Figure 14, and a horizontal plane tomogram at the existing highway elevation is shown on Figure 15. The results shown on Figures 14 and 15 indicate seismic velocities are less than 2,500 m/s in the outer (semi-parallel to the surface topography) 4 to 6 meters. Deeper within the rock mass than 4 to 6 meters, the seismic velocity rapidly increases, ranging from 3000 m/s to 4500 m/s. Based on the method limitations previously discussed, these seismic velocities may be lower than the velocities measured by seismic refraction.

4.1.3 Subsurface Characteristics

This section summarizes the investigation results for the proposed retaining wall, and the in situ materials to be retained by the wall. The individual test hole logs are presented in Appendix B, the laboratory test results are presented in Appendix D, and the seismic tomography results are presented in Appendix E.

TH2-1, TH2-3, TH2-4, TH2-6, TH2-8, TH2-9, and TH2-11 were drilled from the existing road, as far towards the outboard (right) shoulder as possible. Slightly to highly weathered granite and gneiss bedrock was encountered approximately 3 to 13 meters below the road surface. Despite this large variation, there appears to be no particular trend in the slope of the bedrock along the highway. Bedrock is also generally present in the cutslope above the inboard (left) shoulder of the existing road and there appears to be some correlation of bedrock depth with outcrop topography. Where bedrock is shallow (TH2-1 and TH2-9) there is a buttress of bedrock projecting to near the inboard shoulder. Where bedrock is deepest (TH2-8 and TH2-11) there appears to be a minor ravine in the bedrock above the highway, see Figure 4-1 for summary logs and Figure 4-4 for hole locations in relation to surface geology.

The bedrock was cored with an H-size core barrel and recovery ranged from 87 to 100 percent. The rock quality designation (RQD) varies from 50 to 100. Groundwater was not encountered and, based on the topography, it is anticipated that groundwater is below the planned wall elevation. It is anticipated that some water could be encountered at the bedrock contact. The unconfined compressive strength of samples of bedrock from 8.2 to 15.9 meter depth below the road is 31 to 78 MPa. The unconfined compressive strength estimated from point load testing varies from 66 to 200 MPa. These results are summarized in Table 4-2.

Table 4-2

SITE 2 LABORATORY TEST RESULTS

Test Hole	Depth (m)	UCS[1][2] (MPa)	UCS[2] (MPa)
TH2-11	14.50	199	
TH2-11	15.30	200	
TH2-11	15.90		65
TH2-2	7.35	147	
TH2-2	7.95		90
TH2-2	8.25	198	
TH2-3	8.20		78
TH2-3	8.40	160	
TH2-3	9.60	187	
TH2-5	11.10		51
TH2-5	12.40	198	
TH2-5	13.27	229	
TH2-6	8.85	121	
TH2-6	9.20	66	
TH2-6	10.45		43
TH2-7	7.90		98
TH2-7	10.00	82	
TH2-7	10.70	196	
TH2-8	13.80		31
TH2-8	14.10	54	

(1) Values determined from point load testing of core (ASTM D5731), as described on data sheets in Appendix D.
(2) UCS = Unconfined Compressive Strength (ASTM D2938), as described on data sheets in Appendix D.

TH2-2, TH2-5, and TH2-7 were drilled from the slope below the existing road, closer to the planned face of the retaining wall than the other holes. Slightly to highly weathered granite and gneiss bedrock was encountered approximately 5.2 to 10.7 meters below the ground surface. This bedrock contact elevation is 2 to 3 meters less than the elevation of the contact at the nearest hole from the existing road shoulder, indicating the bedrock contact dips in the direction of the slope. Where test holes were drilled, the bedrock elevation is generally 2685 to 2695, which is at or below the planned wall foundation elevation. Given the slope of this contact, bedrock is expected to be several meters (approximately 3 to 9 meters at the sections shown on Figures 4-2 and 4-3) below the planned foundation at the face of the wall.

The bedrock was cored with an N-size core barrel in TH2-2, TH2-5 and TH2-7 and recovery ranged from 79 to 100 percent. The rock quality designation (RQD) varies from 34 to 95, increasing with depth in each hole. Groundwater was not encountered. The unconfined compressive strength of samples of bedrock from 7.9 to 4.1 meter depth is 51 to 98 MPa. The unconfined compressive strength estimated from point load testing varies from 82 to 229 MPa.

The overburden is fill and talus with particle sizes from sand through boulders. The longest length of boulder cored was about 1.6 meters. Many larger boulders, some up to about 5 meters diameter, are evident on the ground surface below the road. Standard Penetration Tests (SPT's)

were generally conducted at the end of each 1.5 meter core run. N-values range from 3 to 35 and, where penetration was too little to measure an N-value, blowcounts range from 56 blows for 250 mm to 50 blows for 10 mm. The considerable presence of gravel, cobbles, and boulders has likely impacted some of the SPT results, especially the higher values. The presence of some relatively low N-values indicates there are intervals of loose to medium dense soils, or possibly voids. The summary logs show recovery for core runs in the overburden, as well as the SPT results. Core intervals include overcoring (reaming) the interval of the hole sampled with the SPT split spoon sampler. Despite coring through this disturbed material, recovery (based on the full length of the run) varied generally from 20 to 90 percent, and was often over 50 percent. The zones with higher recovery may be indicative of a high percentage of larger particle sizes in the overburden.

Seismic Tomography

The seismic tomography results can be used to evaluate variations in the material properties at locations where test holes were not drilled. In addition, seismic tomography can be used to develop a three-dimensional image of the subsurface. The seismic tomography methodology and complete results are presented in Appendix E. Limitations of the method applicable to this project are discussed in Section 3.1.2, under the Seismic Tomography subsection.

The results include an elevation at the face of the proposed wall (Appendix E, Figure 20). The elevation view shows that at elevation 2673, at the face of the wall, the velocity is about 1000 to 3000 m/s, and is at the lower end of this range (about 1000 m/s) between Stations 41+620 and 41+680, where the ray-path coverage is best. It is possible that the interpreted velocity elsewhere along this section would be lower if the ray-path coverage was similar to that between 41+620 and 41+680. Based on the tomography findings, the foundation beneath the wall may consist of materials with seismic velocities as low as 1000 m/s. The plans and sections included in Appendix E support this interpretation.

The seismic refraction surveys that were conducted by URS (1999) along the outboard shoulder (generally just outside the guardrail) show a two-layer subsurface profile. The upper layer is about 3 to 9 meters thick and has a seismic velocity of 340 m/s to 630 m/s. The lower layer has a seismic velocity of 3,040 m/s to 6,220 m/s. These values are consistent with observations from the test holes of fill/talus and bedrock reported herein; they are believed to be more representative seismic velocities than the velocities resulting from the tomographic survey because they agree with our experience of measured seismic velocities in similar materials. The additional characterization of the subsurface materials resulting from the tomography and drilling suggests that the thickness of Seismic Layer 1, delineated from the 1999 seismic work, may not be a vertical thickness. The thickness likely represents the shortest distance between the detecting geophones and the refractor layer. The actual position of the refractor unit mapped with the seismic, considering the sections shown on Figures 4-2 and 4-3, appears to be beneath the inboard travel lane. Figures presenting the results of the 1999 seismic refraction survey have been revised to reflect this understanding and they are included in Appendix F as Figures F-6 to F-10.

4.2 INTERPRETATION AND ANALYSIS

Observations and data on soil and bedrock have been interpreted and analyzed to develop geotechnical recommendations. At this site the soil overburden has been evaluated primarily for temporary excavation slopes and lateral earth pressure. The bedrock has been evaluated for slope stability above the road and for wall foundations, below the road.

4.2.1 Rock Slope

An analysis of the joint set data shown on the photo maps and Table G-1 (Appendix G) was conducted to interpret possible modes of failure for the rock mass to be exposed in the proposed rock cut slopes. The analysis included a survey of wedges that might form, bounded by joints in the rock mass, that could slide out of the proposed cut slopes. Stereographic projections and a computer program called SWEDGE were used to conduct these kinematic analyses. The analyses included blocks of rock that could slide on 1 or 2 joint surfaces and separate from the rock mass along a hypothetical nearby vertical tension crack located behind a cut slope face. Results of the wedge analysis are summarized in Table G-3.

Results of the wedge analysis indicate potentially unstable blocks of rock exist at a number of locations along the proposed cut slopes. Note that the dips of the sliding surfaces range from 42 to 61° and that a number of modes of failure will be almost planar, or slide on a single joint surface. A possible explanation for the joint sets that dip out of slope is that they are the result of stress relief and exfoliation processes that result in the formation of joints oriented parallel to the topographic slopes.

Not all discontinuities have been mapped or analyzed. The nature of the mapping conducted is such that the most prominent features were identified. These features have a dominating influence on the existing rock slope and it is believed that they will have a similar influence on new rock slopes at the site. Discontinuities of differing characteristics and orientation will likely be encountered during excavation and these discontinuities may form failure wedges or modes that have not been identified herein. For these reasons, rock slope design should be based on the prominent features identified here and should have the flexibility to be adapted to additional modes of failure that may be identified during construction. The contractor should expect the identified discontinuities to have a large impact on the excavation. However, the contractor should also expect to encounter some discontinuities and failure modes not identified here.

The rock slope mapping was also used to evaluate the rock mass as a whole. Estimates of RQD and unconfined compressive strength were used with discontinuity data to evaluate RMR for the slope. The base RMR is estimated as 65 to 70. Because of the prevalence of slope parallel joints, the adjustment to correct for discontinuity orientation is -40 (Romana, 1985). The corrected RMR is, therefore, 25 to 30, indicating Poor Rock (Class IV) with c = 100 to 150 kPa and ϕ = 15 to 20 degrees (Bieniawski, 1989).

The mean parameters of c = 125 kPa and ϕ = 17.5 degrees, when used with charts from Hoek and Bray (1974) suggest the minimum theoretical factor of safety of a dry 1:0.5 (vertical:horizontal) slope decreases from 2.3 for a 12 meter slope to 1.4 for a 24 meter slope. These results provide an indication of the stability of the proposed cut slopes, but they must be used with caution because they are based on several assumptions, including the following:

1. The rock mass is so closely jointed by multiple joint sets that it behaves similar to a granular media.
2. Failure occurs along a circular surface comprised of discontinuities and intact rock over the full height of the slope.
3. The slope is dry enough so that no water pressure is exerted on joint surfaces or the hypothetical failure surface. The factor of safety will be lower if wetter conditions are realized.

There may be locations in the planned cuts where these assumptions are reasonable and accurate, but at most locations slope performance will be governed by the stability of individual wedges of rock, bound by discontinuities. Therefore, both the kinematic analysis of wedges and this global stability analysis, and other observations, such as the current stability of existing slopes, have been considered in the development of the recommendations presented in Section 4.3.

4.2.2 Retaining Wall

At the wall face location the material overlying the bedrock is fill and talus. Test holes and observations indicate a wide range of particle sizes is present, from sand through large boulders. Due to the nature of the origin of talus deposit, we expect the finer material to generally fill the voids between the coarser materials. Some voids between larger materials may be filled very loosely, or only partially. Boulders and loose soil may cause difficulty in excavation and may locally present hazardous conditions in excavation slopes. The boulders and cobbles are granitic, being derived from the outcrop above the road, and probably have unconfined compressive strength similar to or less than the bedrock in this area.

The existing man-placed fill comprises part of the overburden. Based on observations from drilling and mapping, the fill is similar to the talus. The distinction between the two is based on the following general characteristics. The talus is mostly gray. The fill is mostly brown. The talus has a larger mean size, and lower recovery of matrix material by SPT sampling, than the fill. Some talus matrix material has organic characteristics. We expect that the fill was generated from nearby rock cuts, and was placed with little compaction. The total depth of fill and talus at the planned wall face location varies from approximately 5 to 15 meters based on the data presented in Section 4.1.

The bedrock quality has been evaluated based on the unconfined compressive strength of intact rock, the RQD, and the RMR. Table 4-3 presents the interpretation of bedrock quality and estimated ranges of engineering parameters based on these methods. In general, these characterizations show that the bedrock quality is fair.

The rock mass Young's modulus (E_m), and the cohesive (c) and frictional (ϕ) strength parameters, are based on published correlations to the RMR. They are presented here to assist in characterizing the relative quality of the rock mass. These correlated values are part of the information used to develop the geotechnical recommendations for design presented in Section 4.3.

Table 4-3

INTERPRETATION OF BEDROCK QUALITY
SITE 2 WALL

Location	Characteristic RQD	Characteristic Rock Quality[1]	RMR[2] Range	Characteristic RMR Class [2]	E_m (GPa)[3]	c (kPa)[2]	ϕ (deg)[2]
41+600 to 41+700	60	Fair	57-73	II (Good)	15-38	300-350	35-40
41+700 to 41+850	65	Fair	53-77	II (Good)	12-47	250-400	30-45
41+850 to 42+000	70	Fair	51-79	II (Good)	11-53	250-400	30-45

Notes:

[1] Based on RQD, Deere (1964). Classification for RQD <25 = very poor, 25 < 50 = poor, 50 < 75 = fair, 75 < 90 = good, 90 < 100 = excellent

[2] Rock Mass Rating, Bieniawski (1989), classification for RMR < 20 = V (very poor), 21 to 40 = IV (poor), 41 to 60 = III (fair), 61 to 80 = II (good), 81 to 100 = I (very good). Cohesion and friction ranges are based on RMR class. For Class V: ϕ < 15°, c < 100 kPa; for Class IV: ϕ = 15-25°, c = 100-200 kPa; for Class III: ϕ = 25-35°, c = 200-300 kPa, for Class II: ϕ = 35-45°, c = 300-400 kPa; for Class I: ϕ > 45, c > 400 kPa.

[3] Young's Modulus of Rock Mass based on RMR, $E_m = 10^{(RMR-10)/40}$, Serafin and Pereira (1983)

4.3 RECOMMENDATIONS

4.3.1 Rock Slope

Rock cut is recommended on the east side of the Beartooth Ravine, through approximately Station 41+580, where the proposed alignment is adjacent to the existing cut slope along the rock knob. This will reduce the rockfall hazard associated with the existing slope. A rock cut of about 4 meters into the slope (horizontally) will probably remove most of the blast damaged and weathered rock observed on the surface. Tomography and site observations suggest that rock quality continues to improve with further distance into the knob. Recommendations provided herein are based on assumed cuts of up to 6 meters horizontally into the knob area.

Rock cut can also be used to widen the road from Station 41+580 to approximately Station 41+720, where a bedrock rib extends essentially to the edge of pavement. We recommend the horizontal cut width be limited to about 2 meters in this area. The bedrock rib should not be cut because of the potential to destabilize large wedges above the road.

Cutting is generally not as desirable between approximate Station 41+720 and the through-cut starting at about Station 41+980 because the slope alternates between colluvium/talus and bedrock, and alternating cut widths and slope ratios could lead to a cut with considerable aesthetic impact. We understand cuts may be required beyond Station 41+900 to maintain an alignment that avoids a fen further upstation. In this case, rock cuts can be made between 41+900 and the throughcut according to the recommendations in this section. Unreinforced cut slopes in colluvium should be no steeper than 1:1.5 (vertical:horizontal). Steeper cuts should be reinforced with methods such as a soil nail wall or dry-stack rockery. Rockery walls to 3 meters height should be no steeper than 1.33:1 (vertical:horizontal) and rockery walls of 3 to 6 meters height should be no steeper than 1:1. Dry stack rockery walls are not recommended for cuts higher than 6 meters in height. Rock walls should be constructed according to FP-96, Section 252.

Rock slope recommendations are based on observations of existing slopes and the analyses presented in Section 4.2 and Appendix G. We recommend that slopes be cut to a ratio of 2:1 (vertical:horizontal) using controlled blasting methods. Existing discontinuities should be used as much as practical to form the back slope and limits of the cuts. Kinematic analysis (Appendix G) shows a failure mode on discontinuities essentially parallel to the cut face. Scaling and spot bolting should be performed as the cut is made and, as needed, when the cut is completed. Scaling by hand methods using bars and occasional light blasting is expected to be sufficient. Bolting of blocks should be used in lieu of scaling when block removal is judged to be undesirable because scaling may destabilize other blocks, or for aesthetic reasons. It is estimated that few spot bolts will be required and bolts will be approximately 4 meters in length.

The rock quality of the through-cut upstation of the Site 2 wall is less than elsewhere at Site 2, and wedge failures are identified by kinematic analysis (see Table G-3 in Appendix G). Cut heights of 5 to 7 meters are planned. A maximum slope ratio of 1.0:1.0 (vertical:horizontal) is recommended. Scaling and spot bolting should be performed as described in this section. Additionally, wedges of the form defined by the kinematic analysis should be removed or bolted. The need for bolts is not anticipated. However, as a contingency, a provision for bolts and bolting is recommended to be included in the construction contract.

A rockfall collection ditch should be provided to catch and store as much of the rockfall as practical. Where practical, the Ritchie criteria should be followed, or design should be based on a criteria related to probability of rockfall retention. For reference, the Ritchie criteria, as presented in Federal Lands Highway, Project Development and Design Manual, 1996, for a ditch beyond the useable shoulder width, and slopes of 2:1 (vertical:horizontal), is as follows:

Slope Height (m)	Ditch Width (m)	Ditch Depth (m)
4.5 to 9.0	2.4	1.2
9.0 to 18.0	3.6	1.8
18 to 30	5.5	1.8

Steeper or flatter slopes require equal or lesser width and depth, according to the Ritchie criteria. If the Ritchie criteria are not attainable, we recommend a minimum collection ditch width of 3 meters around the rock knob, to approximately Station 41+570, and 2 meters elsewhere.

An alternative design that could be used to increase ditch width or reduce cut volume is to construct the lower 3 to 4 meters of rock cut at 10:1 (vertical:horizontal) and to reinforce this part of the slope with pattern bolting, and a facing, if needed, to span between the bolts.

4.3.2 Excavations

Temporary excavation slopes should be designed by the contractor. Temporary slopes of less than 4 meter height are recommended to be cut no steeper than 1:1 (vertical:horizontal). Higher cuts should be no steeper than 1:1.5 (vertical:horizontal). Cuts should comply with applicable health and safety standards and should be evaluated for stability. Reinforcement or shoring should be installed where steeper slopes are required and if 1:1 slopes are required to remain for more than one season. Pile and soil nail walls are suitable for excavation support. Micropiles installed from the road to reinforce the soil mass may be applicable as part of an active retaining

system. Additional recommendations on excavation slopes provided in Section 3.3.1 for the Beartooth Ravine bridge are applicable to Site 2, as well.

4.3.3 Foundations

The proposed retaining wall may be founded on micropiles bearing in the underlying bedrock. The existing fill and talus are not recommended for the support of the retaining wall because of the potential for excessive settlement. The varying character of the fill and talus suggests that considerable differential settlement could be experienced if the wall foundation is above the bedrock. Differential settlement could be limited by pre-grouting and/or pre-stressing the foundation with ground anchors. The required extent and difficulty of this foundation treatment, and the uncertainty that would remain following treatment, are the reasons why shallow foundations are not recommended.

Micropiles should be designed and constructed in accordance with FHWA report FHWA-SA-97-070, Micropile Design and Construction Guidelines Implementation Manual, 2000, and the recommendations presented herein. Type A micropoiles are recommended (FHWA, 2000).

The geotechnical design for micropiles should be based on an allowable bond stress of the soil/rock bond of 0.5 MPa. This bond stress is based on reported nominal bond stress values (FHWA, 2000), consideration of acceptable settlement in jointed rock mass using methodology for drilled piers, and a factor of safety of 2.5. Micropiles in the range of 229 to 254 mm diameter will have capacities of 360 to 400 kN/m. Ten percent of the micropiles should be drilled at least 6 meters below the bedrock contact to confirm the interpretation of the top of bedrock, and that piles are not founded on a boulder.

The structural design of micropiles should be based on an allowable load of 0.47 times the lower yield stress of the steel casing or reinforcement, and 0.4 times the unconfined compressive strength of the grout. The structural design should also provide for sufficient length for the grout to steel bond.

Micropiles should be constructed with permanent steel casing, with a casing plunge extending at least 0.5 m into the underlying bedrock. The casing should meet the physical properties of ASTM A-252 Grade 3, except that the minimum yield strength should be 552 MPa. Reinforcing steel may be Grade 60, 75 or 80 reinforcing rod (ASTM A615, A616 or A617). The grout should consist of neat cement and water mixed with a high shear colloidal type mixer. The grout should have a fluid consistency, a water/ cement ratio of about 0.45 and a minimum unconfined compressive strength of 27.6 MPa at 28 days.

Micropiles provide lateral resistance from the horizontal response of the adjacent soils and can sustain significant lateral deflection within the available structural pile capacity. Increased bending capacity can be provided by increasing the diameter of the pile or the thickness of the pipe. Lateral resistance can also be provided by battered piles. The lateral design parameters presented in the Drilled Piers section of Section 3.3.2 may be used for the design of micropiles.

Retaining Walls

The use of free draining granular soils for backfilling behind bridge abutments and retaining walls is recommended. Granular soils result in lower lateral earth pressures and provide better

subsurface drainage. Abutments and retaining walls should be provided with an underdrain system consisting of a front drainage zone and an underdrain pipe at the base of the wall. The front drainage zone may consist of a vertical layer of either a granular soil layer meeting FP-96 Section 210 or a geocomposite drain meeting FP-96 Section 605.05. The underdrain pipe should consist of a minimum 100 mm PVC pipe encased in a geotextile wrapped granular layer or a geocomposite sheet drain. The pipe should be sloped to daylight at a minimum slope of 2 percent.

Abutment walls and retaining walls which can be expected to deflect sufficiently to mobilize the full active earth pressure condition should be designed for a lateral earth pressure computed on the basis of an equivalent fluid unit weight of 5.9 kN/m^3. This pressure assumes a free draining backfill. Retaining structures should be designed for appropriate surcharge pressures such as traffic, construction equipment and upward sloping backfill.

4.3.4 Other Considerations

- **Talus consistency.** Grouting of the PVC casing into the test holes was completed with difficulty. Grout take was not measured, but it was necessary to complete grouting in several stages, allowing grout setup between stages, and to add sand during grouting. This indicates a wide extent of voids in the talus.
- **Rock slope cut support.** Recommendations should be re-evaluated when the alignment is finalized and cut slope aspects and heights can be determined by survey. This will enable a cut design that matches closely with the existing topography.
- **Further investigation.** The density of test holes conducted for this study is less than standard for final design of walls of this size. When the final wall face location is determined, at least three test holes are recommended to be conducted along, or nominally outside, the wall face. Drilling and testing should be in general accordance with the procedures described herein.

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/01	FH 4-1(0) BEARTOOTH HIGHWAY	4-1	
SITE 2 PLAN AND PROFILE				

"THE ALIGNMENT AND GRADE AS SHOWN HEREON ARE SUBJECT TO ADJUSTMENT"

N

20 0 20 40
SCALE IN METERS (APPROXIMATE)

STATIONING USED FOR BEARTOOTH RAVINE 50 KM/HR OPTION

PRIMARY STATIONING

OBSCURED AREA

DENSE TREES

UNPAVED

TH2-1 TH2-2 TH2-3 TH2-4 TH2-5 TH2-6 TH2-7 TH2-8 TH2-9 TH2-10 TH2-11

PROPOSED RETAINING WALL

SEISMIC REFRACTION SURVEY

BEARTOOTH FALLS

2769.56 2758.42 2668.77

NOTES:

1. See Figure 4-2 for legend and notes.
2. Seismic layer interface based on vertical distance from the source and reservoir. The vertical assumption is used only for representation of the data and may not be accurate. See figures 4-2 and 4-3.
3. Primary stationing used for elevation (this sheet) and general reference in report.

TH2-1 2693.1
TH2-2 2691.7
TH2-3 2694.6
TH2-4 2696.5
TH2-5 2695.6
TH2-6 2699.4
TH2-7 2698.0
TH2-8 2704.4
TH2-9 2706.2
TH2-11 2708.2

PROPOSED GRADE

EXISTING GROUND

SEISMIC LAYER 1

SEISMIC LAYER 2

SEISMIC LAYER INTERFACE (SEE NOTE 2)

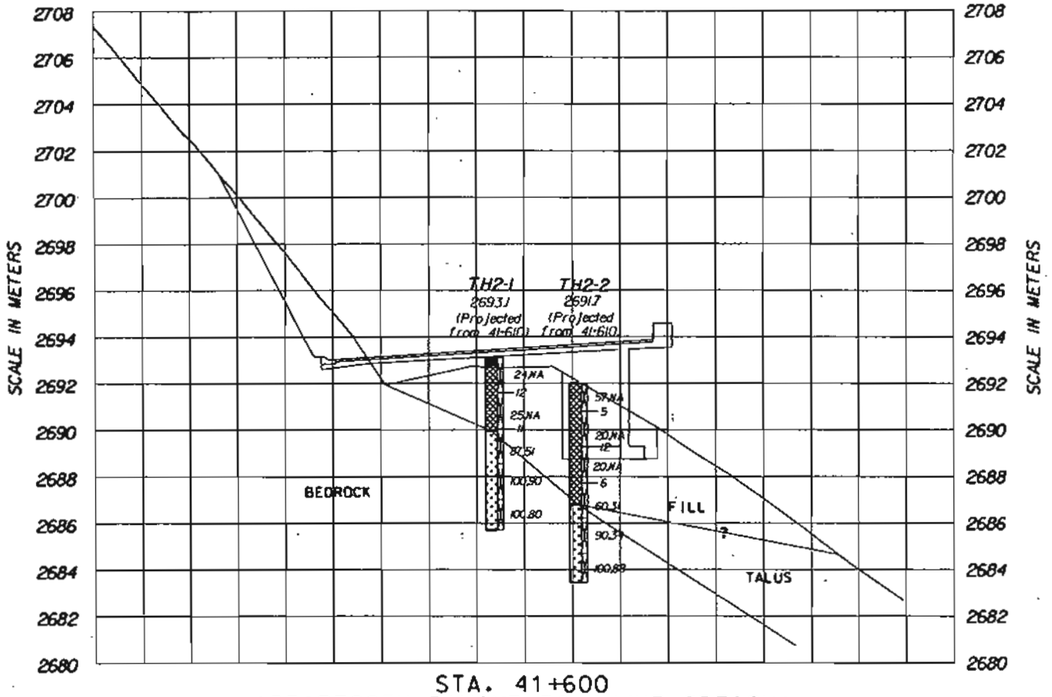
2700 2698 2696 2694 2692 2690 2688 2686 2684 2682 2680 2678

2710 2708 2706 2704 2702 2700 2698 2696 2694 2692 2690 2688

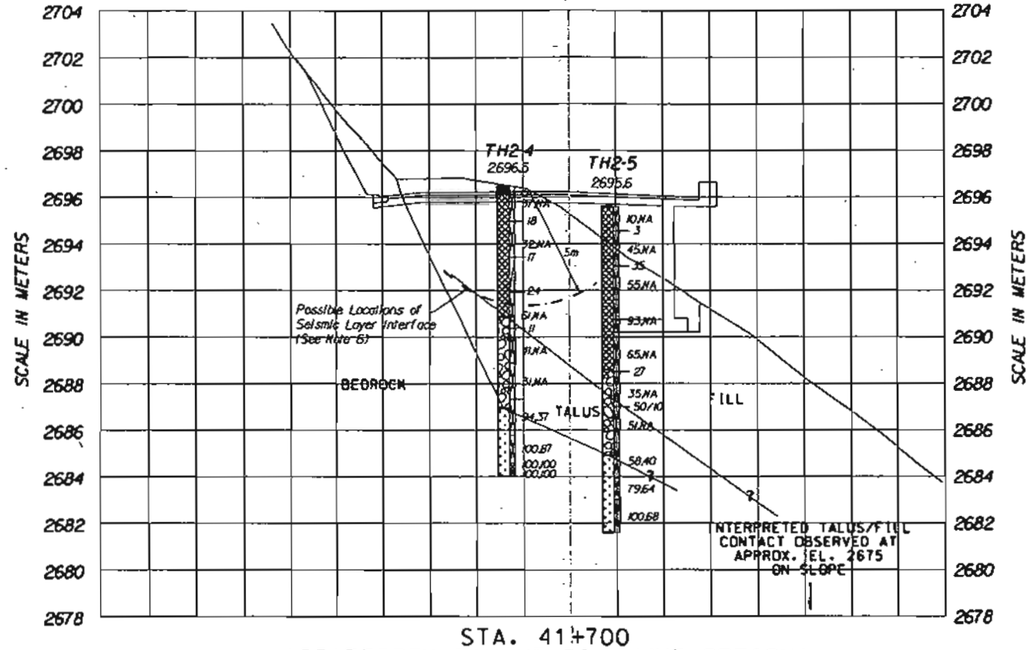
41+300 41+400 41+500 41+600 41+700 41+800 41+900 42+000

BRTH4-1.DGN

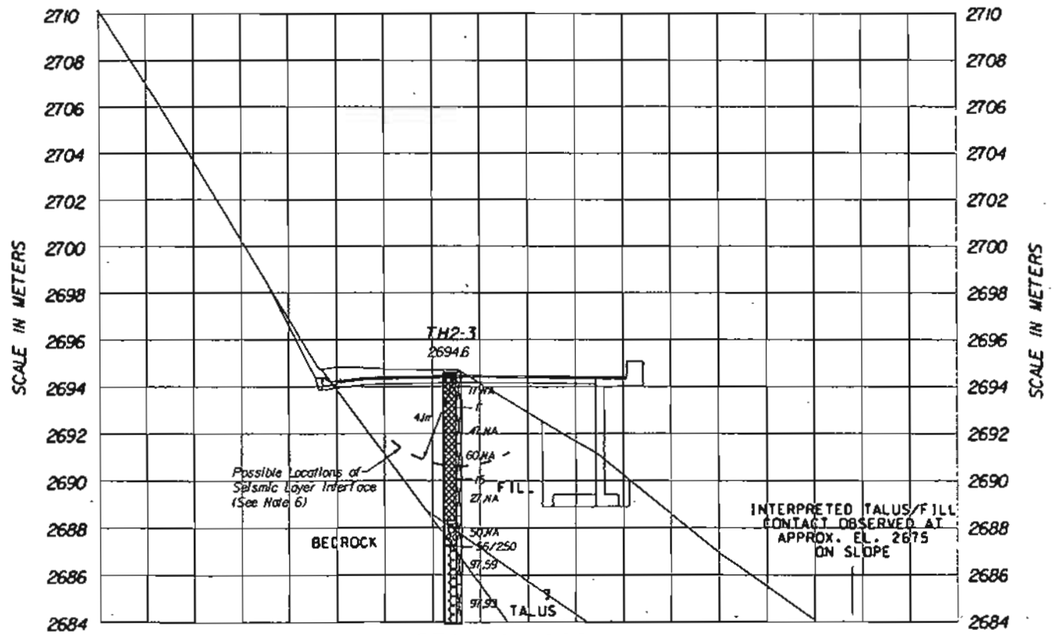
PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/01	FH 4-(10) BEARTOOTH HIGHWAY	4-2	
SITE 2 SECTIONS 41+600, 41+660, 41+700				



STA. 41+600
(BEARTOOTH RAVINE 50 KM/HR OPTION)



STA. 41+700
(BEARTOOTH RAVINE 50 KM/HR OPTION)



STA. 41+660
(BEARTOOTH RAVINE 50 KM/HR OPTION)

LEGEND FOR SITE 2

- ASPHALT AND BASE COARSE PAVEMENT SECTION
- MAN-PLACED FILL, GRAVEL, COBBLE, AND BOULDERS, GRAY TO BROWN, SANDY TO GRAVELLY MATRIX, MOIST
- GRAVEL, COBBLE, AND BOULDERS, LOOSE TO DENSE, (BASED ON BLOWCOUNTS), GRAY, BROWN, REDDISH-BROWN TO BLACK, NO-RECOVERED-MATRIX TO SANDY-TO-GRAVELLY-MATRIX, OCCASIONAL CLAY LENSES
- GRANITE TO GNIESS, MODERATELY STRONG, VERY STRONG, PINK TO BLACK, COARSE GRAINED, SLIGHTLY TO HIGHLY WEATHERED, GNEISS IS WEAKER AND MORE WEATHERED THAN GRANITE
- GRANITE, MODERATELY STRONG TO VERY STRONG, RED-GRAY, RED, GRAY, PINK, BLACK, BLACK TO PINK, COARSE GRAINED, WEAKLY FOLIATED, SLIGHTLY WEATHERED

TH2-1 — TEST HOLE LOCATION

12 — INDICATES TOP OF INTERVAL FOR STANDARD PENETRATION TEST (SPT). INDICATES THE BLOWCOUNT (N-VALUE) FOR THE FINAL 300 MM OF 450 MM TOTAL PENETRATION IS 12. THE SPT CONSISTS OF RECORDING BLOWS OF A 63.5 KG HAMMER FALLING 760 MM TO DRIVE A 50 MM O.D. SPLIT SPOON SAMPLER. WHERE TWO VALUES ARE SHOWN SEPARATED BY A SLASH, REFUSAL WAS MET BEFORE 450 MM PENETRATION. THE FIRST NUMBER IS THE BLOWCOUNT, THE SECOND NUMBER IS THE PENETRATION DEPTH.

90.30 — INDICATES SPLIT TRIPLE-TUBE CORE BARREL WAS USED TO ADVANCE HOLE. 90.30 INDICATES 90% RECOVERY FOR INTERVAL INDICATED AND A ROCK QUALITY DESIGNATION (RQD) OF 30. RQD IS 100 TIMES THE TOTAL LENGTH OF SOUND CORE OVER 100 MM (40 IN.) LONG DIVIDED BY THE LENGTH OF RUN INDICATED. WHERE SPT INTERVALS PRECEDE CORE INTERVALS, THE SPT INTERVAL IS SUBSEQUENTLY OVER-CORED. RECOVERY AND RUN ARE BASED ON A CORE RUN LENGTH THAT INCLUDES THE OVER-CORED LENGTH.

NA — NOT APPLICABLE

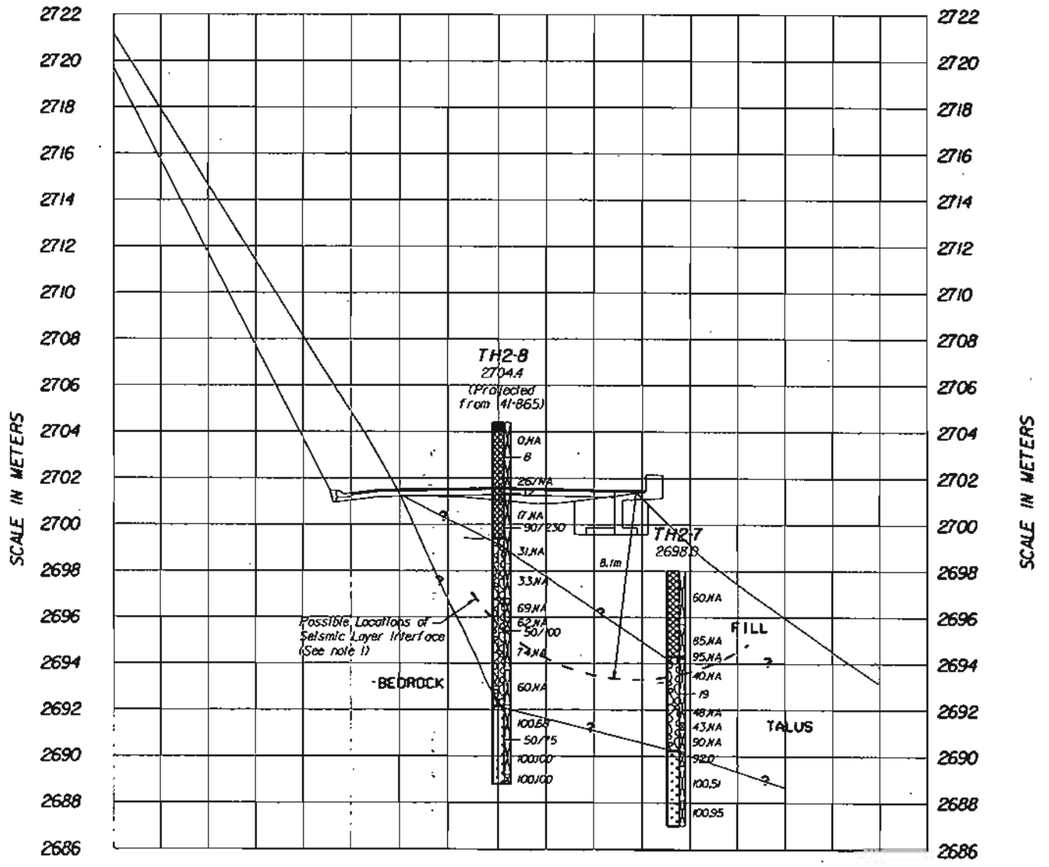
INDICATES WATER LEVEL AT TIME OF DRILLING

NOTES:

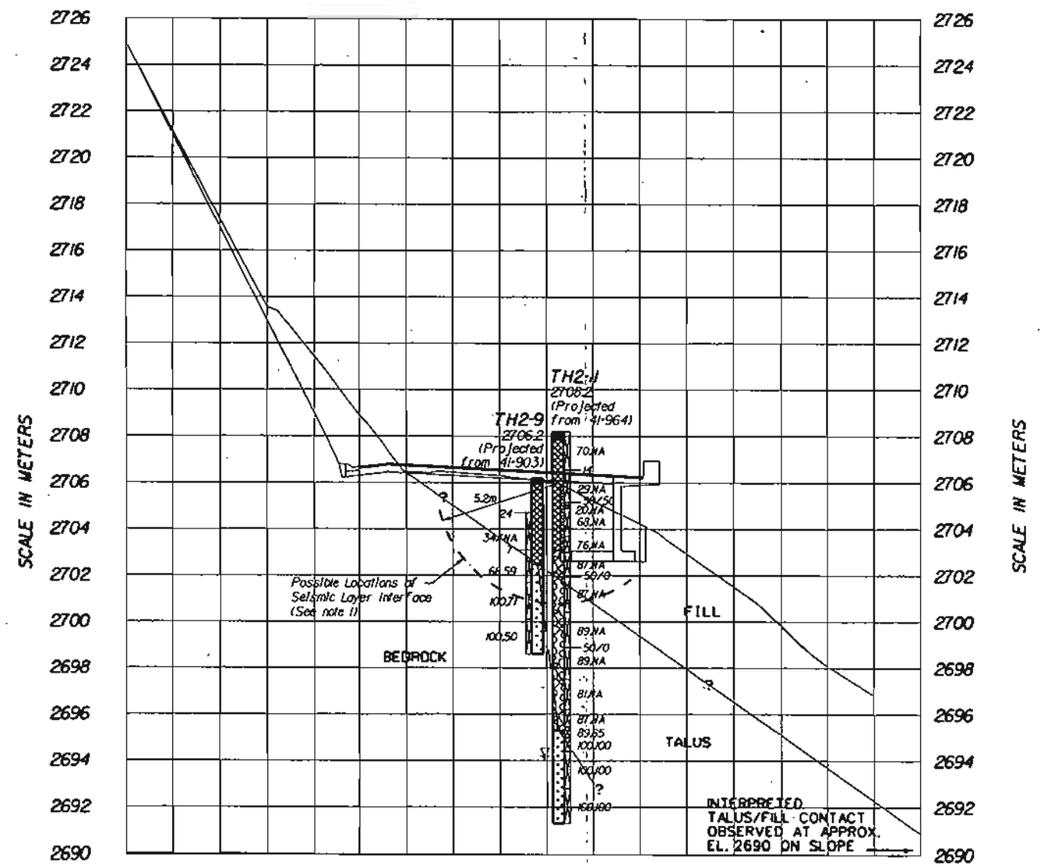
1. TEST HOLES WERE DRILLED ON 8/17/00 THROUGH 9/15/00 BY HAZTECH DRILLING, INC., USING A BK-81 (TRUCK MOUNT) RIG AND AN HQ SPLIT TRIPLE-TUBE CORE BARREL, EXCEPT AS NOTED HERE. TEST HOLES TH8-2, TH8-4, TH2-2, TH2-5, TH2-7 WERE DRILLED WITH A DIEDRICH D-25 (SKID MOUNT) RIG AND AN NQ SPLIT TRIPLE-TUBE CORE BARREL.
2. BASE MAPS OBTAINED FROM DESIGN FIGURES PROVIDED BY THE FHWA.
3. TEST HOLE LOCATIONS ARE APPROXIMATE AND WERE LOCATED BY TAPE MEASUREMENT ON OR BESIDE THE EXISTING ROAD AT DESIGNATED STATIONS. MEASUREMENTS WERE REFERENCED TO MARKINGS AND STAKES PLACED ALONG THE EXISTING ROAD BY THE FHWA.
4. TEST HOLE LOGS IN THIS REPORT ARE SUBJECT TO LIMITATIONS, EXPLANATIONS, AND CONCLUSIONS OF THIS REPORT. SEE INDIVIDUAL LOGS IN APPENDICES FOR MATERIAL DESCRIPTIONS AT EACH TEST HOLE.
5. THE TALUS WAS DISTINGUISHED FROM THE FILL BY THE FOLLOWING GENERAL CHARACTERISTICS. THE TALUS IS MORE GRAY THAN BROWN, HAS A LARGER MEAN SIZE, LESS MATRIX MATERIAL WAS RECOVERED BY SPT SAMPLER IN TALUS, AND SOME TALUS MATRIX MATERIAL HAD ORGANIC CHARACTERISTICS.
6. APPROXIMATE DISTANCE TO REFRACTOR IS SHOWN.

BRTH4-2.DGN

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/01	FH 4-1(10) BEARTOOTH HIGHWAY	4-3	
SITE 2 SECTIONS 41+800, 41+900				



STA. 41+800
(BEARTOOTH RAVINE 50KM/HR OPTION)



STA. 41+900
(PRIMARY STATIONING)

NOTES:

1. See Figure 4-2 for legend and notes.

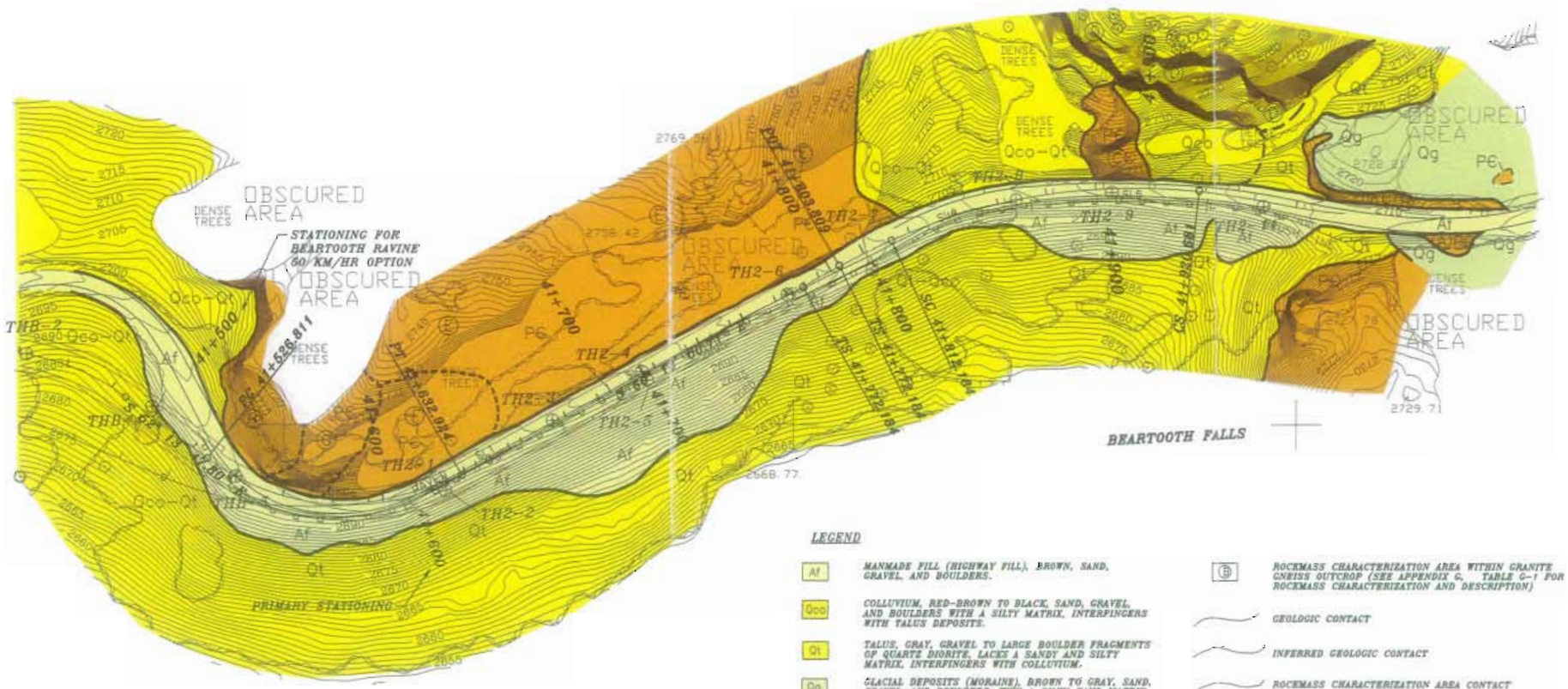
BRTH4-3.DGN

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/01	FH 4-1(0) BEARTOOTH HIGHWAY	4-4	
SITE 2 GEOLOGIC MAP				

"THE ALIGNMENT AND GRADE AS SHOWN HEREON ARE SUBJECT TO ADJUSTMENT"

N

20 0 20 40
SCALE IN METERS
(APPROXIMATE)



LEGEND

Af	MANMADE FILL (HIGHWAY FILL), BROWN, SAND, GRAVEL, AND BOULDERS.
Qco	COLLUVIUM, RED-BROWN TO BLACK, SAND, GRAVEL, AND BOULDERS WITH A SILTY MATRIX, INTERFINGERS WITH TALUS DEPOSITS.
Qt	TALUS, GRAY, GRAVEL TO LARGE BOULDER FRAGMENTS OF QUARTZ DIORITE, LACKS A SANDY AND SILTY MATRIX, INTERFINGERS WITH COLLUVIUM.
Qg	GLACIAL DEPOSITS (MORAINE), BROWN TO GRAY, SAND, GRAVEL, AND BOULDERS, WITH A SILTY SAND MATRIX, SOME ROUNDED FRAGMENTS.
PG	GRANITE GNEISS, GRAY, MEDIUM TO COARSE GRAINED, WITH SOME SCHIST.
B	ROCKMASS CHARACTERIZATION AREA WITHIN GRANITE GNEISS OUTCROP (SEE APPENDIX G, TABLE G-1 FOR ROCKMASS CHARACTERIZATION AND DESCRIPTION)
	GEOLOGIC CONTACT
	INFERRED GEOLOGIC CONTACT
	ROCKMASS CHARACTERIZATION AREA CONTACT
	APPROXIMATE SEISMIC REFRACTION LINE LOCATIONS (URS, 1999). SEE APPENDIX F.

NOTES:

1. FIGURES ADAPTED FROM (URS (1999).
2. SEE FIGURE 4-2 FOR ADDITIONAL LEGEND AND NOTES.

BRTH4-4.DGN

5.1 INVESTIGATION

Two test holes, THB-6 and THB-7, were drilled at the bridge on the right shoulder of the existing road. The holes were drilled using an H-size core barrel and SPT split spoon samples were generally collected at the end of each core run (1.5 m) unless the run ended in a boulder or bedrock. The test hole locations and results are summarized in Table 5-1. The test hole locations are shown on Figures 5-1 and 5-2, summary logs are shown on Figure 5-2, and individual logs and photos are provided in Appendix B.

Table 5-1

TEST HOLE SUMMARY, BEARTOOTH LAKE OUTLET BRIDGE

Test Hole Number	Stationing	Offset (m)	Elevation (m)	Depth to Bedrock (m)	Depth to Water[1] (m)	Test Hole Depth (m)
THB-6	42+238	0	2715.1	N.E.	2.3	22.9
THB-7	42+278	0.6 R	2716.1	N.E.	3.0	32.0

Notes: 1) Encountered in test hole at time of drilling. N.E. = Not Encountered

5.1.1 Site Characteristics

The Beartooth Lake outlet bridge will replace the existing single span bridge with a new 31.5 meter single span bridge. The bridge will be 14 meters wide and will include two travel lanes and a pedestrian walkway. The bridge is in a very visible location, at the outlet of the largest lake along the alignment, and near a popular campground. The replacement bridge will have an appearance similar to the existing bridge.

5.1.2 Subsurface Characteristics

The upper 10.5 to 11.5 meters consists of loose to medium dense sand, with some gravel, cobbles and boulders (THB-6 and THB-7), or loose to medium dense sand interbedded with gravel, cobbles and boulders with loose to medium dense sand matrix (THB-7). The longest length of boulder cored was 0.3 meters. Larger boulders are evident on the ground surface below the road. Standard Penetration Tests (SPT's) were generally conducted at the end of each 1.5 meter core run. N-values range from 6 to 26, with most values being less than 13. The presence of gravel, cobbles, and boulders may have impacted some SPT results, causing them to be higher than the density would indicate. The summary logs show recovery for core runs in the overburden and the SPT results. Core intervals include overcoring (reaming) the interval of the hole sampled with the SPT split spoon sampler. Recovery (based on the full length of the run) varied from 0 to 40 percent in these coarse grained deposits. Most runs had recovery of less than 20 percent. This may be indicative of the particle sizes encountered in the overburden.

Clay and silt are present below the coarse grained deposits. The clay/silt layer continues to at least 23.3 meters in THB-6 (the bottom of the hole), and to 27.7 meters depth in THB-7. The clay/silt is stiff to very stiff, clayey – with some sand, and with N-values of 10 to 26. Recovery (based on the full length of the run) varied from 0 to 77 percent in the fine grained deposits.

Medium dense sand, gravel, cobbles and boulders exist below the clay/silt layer in THB-7 to a depth of at least 32.5 meters, the bottom of the hole.

Water was encountered in the holes at depths of 2.3 to 3.0 meters. These levels correspond to the water elevation in Beartooth Creek. Drilling notes include observations of heaving sands at several locations, primarily in the upper, coarse grained soils.

Laboratory tests on selected samples of the fine grained soils indicated the soils is a CL-ML or ML, with 80 to 89 percent fines. One sample of CL tested had a dry density of 1539 kg/m^3 and a water content of 27.7 percent. The laboratory test results are summarized in Table 5-2.

Table 5-2

SUMMARY OF LABORATORY TEST RESULTS
BEARTOOTH LAKE OUTLET BRIDGE

Test Hole	Depth (m)	Moisture Content (%)[1]	Classification[2]	% Gravel[3]	% Sand[4]	% Fines[5]	Plasticity Index[6]	Liquid Limit[6]
THB-6	12.50		CL-ML		20	80		
THB-6	17.00		CL-ML	3	8	89	5	23
THB-6	23.10	27.7	CL					
THB-7	11.00		ML		13	87		
THB-7	14.00		ML		13	87	3	21

(1) Dry unit weight test performed in general accordance with ASTM D 2216.
(2) USCS = Unified Soil Classification System, based on visual observation when grain size data and plasticity data not available. Grain size distribution performed in general accordance with ASTM D422 Visual classification performed in general accordance with ASTM D 2488.
(3) Percent greater than the No. 4 sieve size.
(4) Percent finer than the No. 4 sieve size and coarser than the No. 200 sieve size.
(5) Percent finer than the No. 200 sieve size. Includes silt and clay sizes.
(6) Atterberg limits performed in general accordance with ASTM D 4318.

5.2 INTERPRETATION AND ANALYSIS

Observations and data on soil and bedrock have been interpreted and analyzed to develop geotechnical recommendations. At this site the overburden soil has been evaluated for foundation support and lateral earth pressure.

5.3 RECOMMENDATIONS

5.3.1 Foundations

The Beartooth Lake Outlet Bridge site appears suitable for the proposed construction from a geotechnical engineering point of view.

The following foundation systems were evaluated for bridge support at the site:

- driven friction-bearing pile (H-pile) or (closed-end pipe pile) foundation systems;
- micropiles; and
- straight shaft piers drilled into the underlying clay-silt layer.

Driven H-piles or closed-end pipe piles and micropiles are the more suitable foundation systems for this site due to the presence of shallow ground water and granular soils. Driven piles and micropiles should be installed in accordance with the Project Specifications.

Driven Piles

Steel H-piles (HP 10 x 57) or 300 mm diameter closed end pipe piles driven into the underlying cohesive soil layer at least 6 meters will have an allowable capacity of 110 to 130 kN. Actual capacity is defined in Section 551.06 of the Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects FP-96 (1996).

Bearing elevations and pile penetration into the cohesive layer to attain capacity have been estimated and are presented below. Note that the cohesive layer surface varies, so the presented values should be considered as typical.

Location	Typical Elevation (meters)	Estimated Layer Penetration (meters)
Beartooth Outlet Bridge		
Abutment 1	2703	6
Abutment 2	2705	6

Individual pile settlement should be 15 mm or less when designed according to the criteria presented herein.

Groups of piles required to support concentrated loads will require appropriate reductions of the axial, uplift and lateral capacities based on the effective envelope of the pile group. This reduction can be avoided by spacing piles at a minimum distance of at least three diameters center to center. Piles spaced less than three diameters center to center should be evaluated on an individual basis to determine appropriate reductions in axial, uplift and lateral capacities.

The contractor should select a driving hammer and cushion combination which is capable of installing the selected piling without overstressing the pile material. Pile capacities should be evaluated during pile installation using the Pile Driving Analyzer (PDA).

The pile hammer should be operated at the manufacturer's recommended stroke when measuring penetration resistance. Piles should be provided with driving shoes to protect the pile tips from damage when penetrating cobbles. Pile driving operations should be observed under the direction of a qualified geotechnical engineer. Each pile should be observed and checked for buckling, crimping and alignment, in addition to recording penetration resistance, depth of embedment, and general pile driving operations. No specific minimum loading is required, and it is not necessary to consider downdrag in the embankment.

Lateral resistance can be provided by the bending resistance of the piles, or by battered piles. Recommendations provided in the following section for micropiles are applicable to driven piles, as well.

Micropiles

The proposed bridge may be founded on micropiles bearing in the underlying cohesive soil layer. Micropiles should be designed and constructed in accordance with FHWA report FHWA-SA-97-070, Micropile Design and Construction Guidelines Implementation Manual, 2000, and the recommendations presented herein. Type A micropoiles are recommended (FHWA, 2000).

The geotechnical design for micropiles should be based on an allowable bond stress of the grout/soil bond of 19 kPa. Micropiles in the range of 254 to 305 mm will have capacities of 15 to 18 kN/m. Micropiles should be constructed with permanent steel casing extending at least 2 meters into the underlying cohesive soil. The piles should extend at least 6 meters into the clay/silt layer.

The structural design of micropiles should be based on an allowable load of 0.47 times the lower yield stress of the steel casing or reinforcement, and 0.4 times the unconfined compressive strength of the grout. The structural design should also provide for sufficient length for the grout to steel bond.

The casing should meet the physical properties of ASTM A-252 Grade 3, except that the minimum yield strength should be 552 MPa. Reinforcing steel may be Grade 60, 75 or 80 reinforcing rod (ASTM A615, A616 or A617). The grout should consist of neat cement and water mixed with a high shear colloidal type mixer. The grout should have a fluid consistency, a water/ cement ratio of about 0.45 and a minimum unconfined compressive strength of 27.6 MPa at 28 days.

Micropiles provide lateral resistance from the horizontal response of the adjacent soils and can sustain significant lateral deflection within the available structural pile capacity. Increased bending capacity can be provided by increasing the diameter of the pile or the thickness of the pipe. Lateral resistance can also be provided by battered piles. The vertical and horizontal components of the load will depend on the batter inclinations. Pile batters typically should be limited not to exceed 4:1 (vertical:horizontal).

We understand that resistance to lateral load will be evaluated by the L-Pile or COM 624 computer programs. Soil properties for lateral capacity analysis are presented as follows:

Material	Soil Type*	Subgrade Modulus (GPa/m)	ϕ (degrees)	c (kPa)	ε_{50}	Effective Unit Weight (kN/m^3)
Compacted Embankment Fill	4	41	35	0	---	21.2
Silty Clay and Silt	2	22	0	72	0.015	18.8
Silty Sand with Gravel	4	54	35	0	---	11.8

*Soil Type 4 is sand
*Soil Type 2 is stiff clay below the water table

Abutment Walls and Retaining Walls

The use of free draining granular soils for backfilling behind bridge abutments and retaining walls is recommended. Granular soils result in lower lateral earth pressures and provide better subsurface drainage. Abutments and retaining walls should be provided with an underdrain system consisting of a front drainage zone and an underdrain pipe at the base of the wall. The

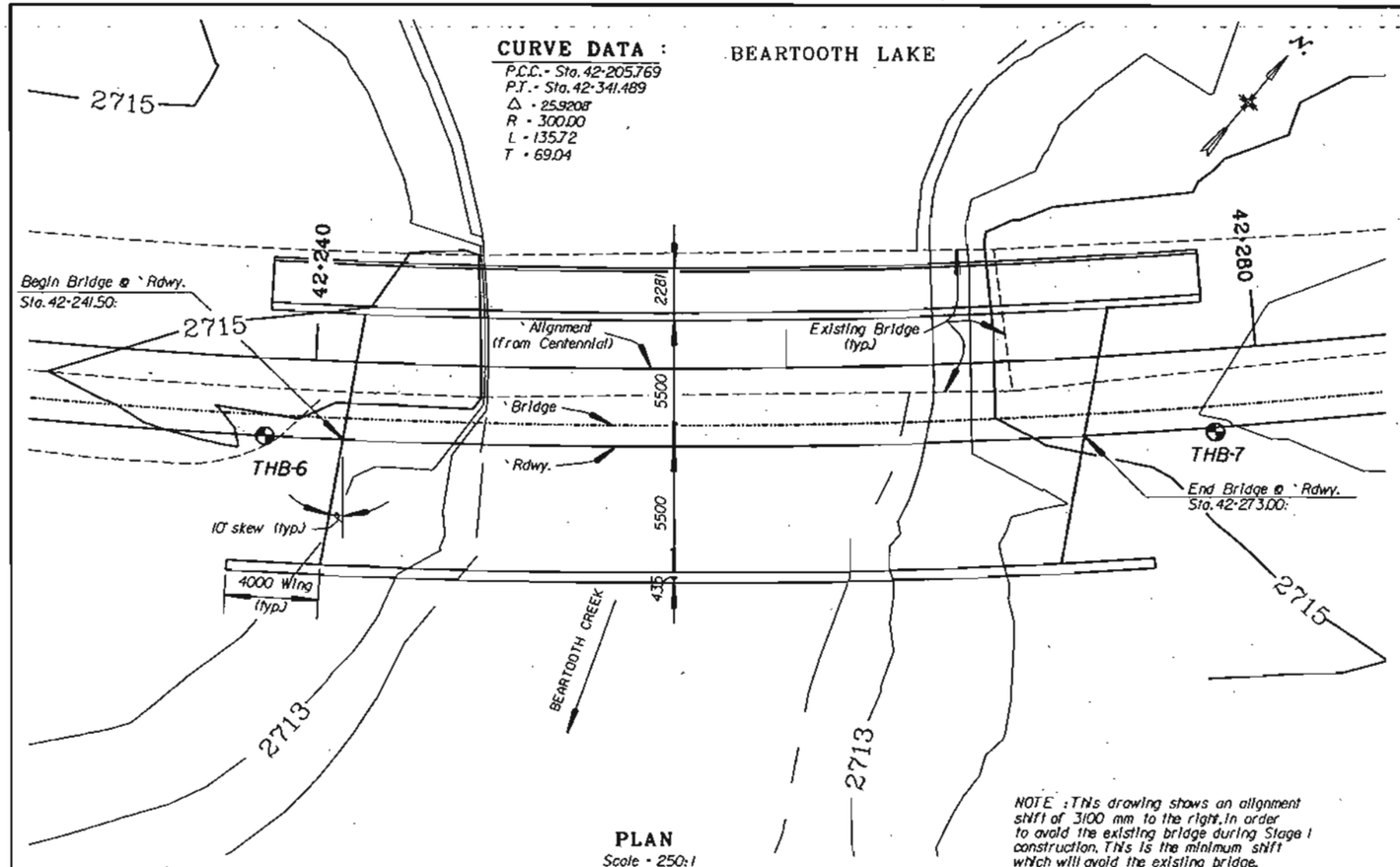
front drainage zone may consist of a vertical layer of either a granular soil layer meeting FP-96 Section 210 or a geocomposite drain meeting FP-96 Section 605.05. The underdrain pipe should consist of a minimum 100 mm PVC pipe encased in a geotextile wrapped granular layer or a geocomposite sheet drain. The pipe should be sloped to daylight at a minimum slope of 2 percent.

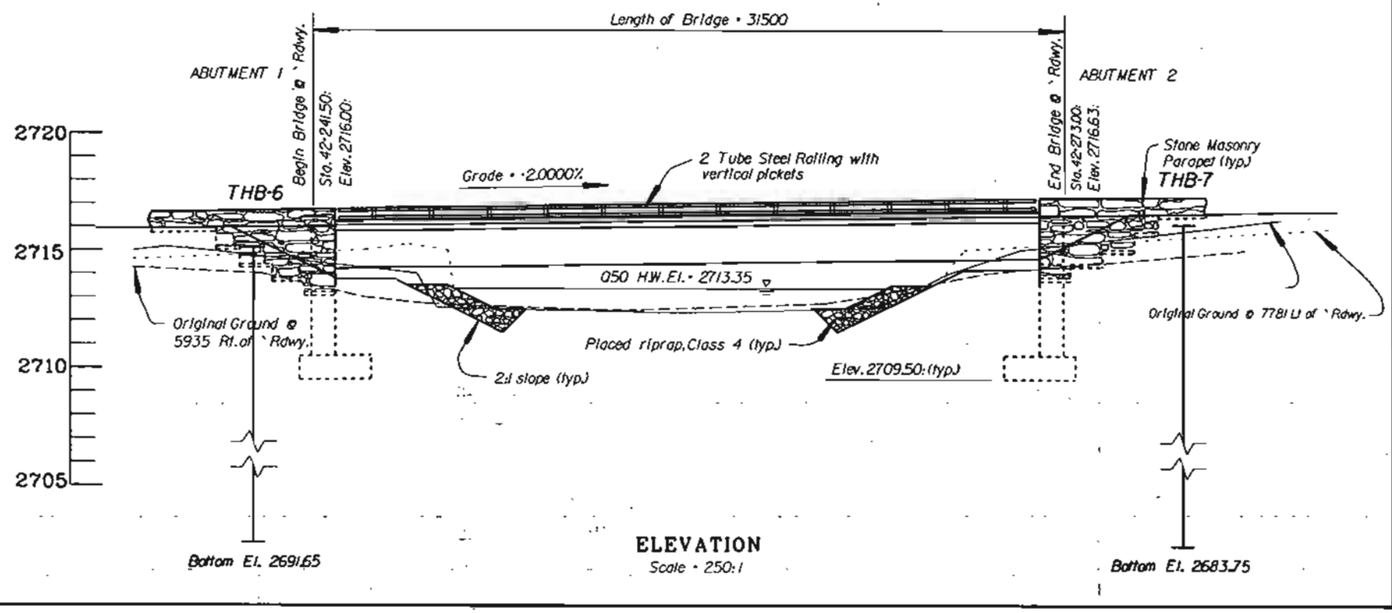
Abutment walls and retaining walls which can be expected to deflect sufficiently to mobilize the full active earth pressure condition should be designed for a lateral earth pressure computed on the basis of an equivalent fluid unit weight of 5.9 kN/m^3. This pressure assumes a free draining backfill. Retaining structures should be designed for appropriate surcharge pressures such as traffic, construction equipment and upward sloping backfill.

5.3.2 Other Considerations

- **Further Investigation.** An additional test hole is recommended at least at one abutment to collect tube samples of the fine grained soils. The soils should be tested for grain size, plasticity, density, water content, and undrained strength. Tube samples are needed for direct measurement of density and strength. The values will be used to verify foundation capacities currently based on blowcounts.

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/01	FH 4-1(0) BEARTOOTH HIGHWAY	5-1	
BEARTOOTH LAKE OUTLET BRIDGE PLAN AND ELEVATION				





BRTH5-1.DGN

NOTES:

1. Summary test hole logs are shown on Figure 5-2
2. See Figure 5-2 for legend and notes.

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION, DENVER, COLORADO

PRELIMINARY LAYOUT

BRIDGE NAME Beartooth Lake Outlet Bridge STREAM NAME Beartooth Creek

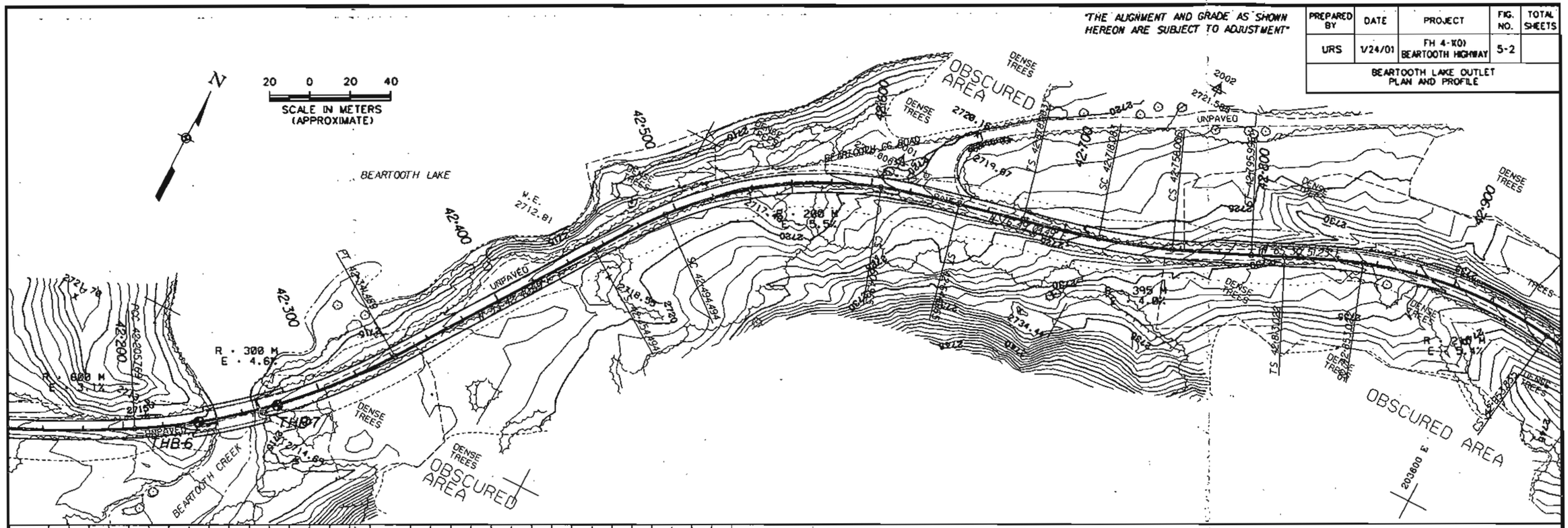
ROUTE NAME AND NUMBER Beartooth Highway, WY FS 4-1(0)

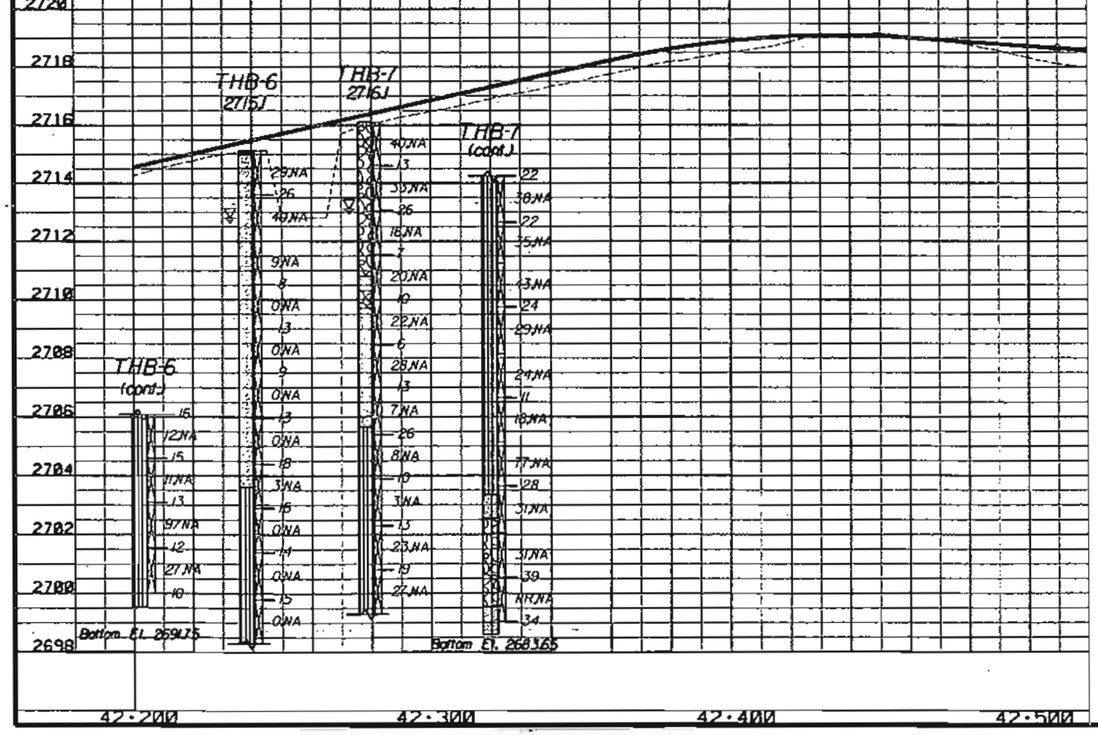
PREPARED BY FHWA (Base Drawing Only)

SCALE: As Shown DATE: Octoberer 16, 2000

"THE ALIGNMENT AND GRADE AS SHOWN HEREON ARE SUBJECT TO ADJUSTMENT"

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/01	FH 4-1(10) BEARTOOTH HIGHWAY	5-2	
BEARTOOTH LAKE OUTLET PLAN AND PROFILE				





LEGEND FOR BEARTOOTH LAKE OUTET BRIDGE

- ASPHALT PAVEMENT
- GRAVEL, COBBLE, AND BOULDERS, MEDIUM DENSE (BASED ON BLOWCOUNTS), BROWN, UNWEATHERED TO SLIGHTLY WEATHERED CLASTS
- SAND, SW, SP, OCCASIONALLY SILTY, SM, LOOSE TO MEDIUM DENSE, FINE TO COARSE GRAINED, BROWN, GRAY, SOME GRAVEL AND COBBLES, OCCASIONAL BOULDERS, MOIST TO SATURATED
- SILT, ML, SILTY CLAY, CL-ML, STIFF TO VERY STIFF, BROWN, GRAY, CLAYEY, SOME SAND, MOIST TO SATURATED
- THB-6 — TEST HOLE LOCATION
- 12 — INDICATES TOP OF INTERVAL FOR STANDARD PENETRATION TEST (SPT). INDICATES THE BLOWCOUNT (N-VALUE) FOR THE FINAL 300 MM OF 450 MM TOTAL PENETRATION IS 12. THE SPT CONSISTS OF RECORDING BLOWS OF A 63.5 KG HAMMER FALLING 760 MM TO DRIVE A 50 MM O.D. SPLIT SPOON SAMPLER. WHERE TWO VALUES ARE SHOWN SEPARATED BY A SLASH, REFUSAL WAS MET BEFORE 450 MM PENETRATION. THE FIRST NUMBER IS THE BLOWCOUNT. THE SECOND NUMBER IS THE PENETRATION DEPTH.
- 90,30 — INDICATES SPLIT TRIPLE-TUBE CORE BARREL WAS USED TO ADVANCE HOLE. 90,30 INDICATES 90% RECOVERY FOR INTERVAL INDICATED AND A ROCK QUALITY DESIGNATION (RQD) OF 30. RQD IS 100 TIMES THE TOTAL LENGTH OF SOUND CORE OVER 100 MM (40 IN.) LONG DIVIDED BY THE LENGTH OF RUN INDICATED. WHERE SPT INTERVALS PRECEDE CORE INTERVALS, THE SPT INTERVAL IS SUBSEQUENTLY OVER-CORED. RECOVERY AND RUN ARE BASED ON A CORE RUN LENGTH THAT INCLUDES THE OVER-CORED LENGTH.
- NA — NOT APPLICABLE
- NR — NOT RECORDED
- INDICATES WATER LEVEL AT TIME OF DRILLING

NOTES:

1. TEST HOLES WERE DRILLED ON 8/17/00 THROUGH 9/15/00 BY HAZTECH DRILLING, INC., USING A BK-81 (TRUCK MOUNT) RIG AND AN HQ SPLIT TRIPLE-TUBE CORE BARREL.
2. BASE MAPS OBTAINED FROM DESIGN FIGURES PROVIDED BY THE FHWA.
3. TEST HOLE LOCATIONS ARE APPROXIMATE AND WERE LOCATED BY TAPE MEASUREMENT ON OR BESIDE THE EXISTING ROAD AT DESIGNATED STATIONS. MEASUREMENTS WERE REFERENCED TO MARKINGS AND STAKES PLACED ALONG THE EXISTING ROAD BY THE FHWA.
4. TEST HOLE LOGS IN THIS REPORT ARE SUBJECT TO LIMITATIONS, EXPLANATIONS, AND CONCLUSIONS OF THIS REPORT. SEE INDIVIDUAL LOGS IN APPENDICES FOR MATERIAL DESCRIPTIONS AT EACH TEST HOLE.

3RTH5-2.DGN

6.1 INVESTIGATION

One test hole was drilled at each of two bridges crossing Little Bear Creek (THB-8 and THB-11). The holes were drilled on the right side of the existing road and advanced using an H-size core barrel. SPT split spoon samples were generally collected at the end of each core run (1.5 m) unless the run ended in a boulder or bedrock. The test hole locations and results are summarized in Table 6-1. The test hole locations are shown on Figures 6-1 through 6-4, summary logs are shown on Figures 6-2 and 6-4, and individual logs and photos are provided in Appendix B.

Table 6-1

TEST HOLE SUMMARY, LITTLE BEAR CREEK BRIDGE

Test Hole Number	Stationing	Offset (m)	Elevation (m)	Depth to Bedrock (m)	Depth to Water(1) (m)	Test Hole Depth (m)
THB-8	45+300	2.7 R	2838.8	8.6	2.9	11.7
THB-11	46+601	5.8 R	2858.7	3.0	3.4	6.1

Notes: (1) Encountered in test hole at time of drilling. N.E. = Not Encountered

6.1.1 Site Characteristics

The two Little Bear Creek bridges are in a broad valley near the Top of the World store. The road and creek are approximately parallel except where the creek meanders beneath the road. The valley is probably in its existing location in part because of an ancestral fault that places younger sedimentary bedrock adjacent to Precambrian igneous bedrock. The fault trace appears to be within a few meters of Little Bear Creek Bridge No. 2. Both bridges are replacements for the existing bridges. Little Bear Creek Bridge No. 1 will have a span of 25.5 meters and Little Bear Creek Bridge No. 2 will have a span of 25.0 meters.

6.1.2 Subsurface Characteristics

Little Bear Creek Bridge Number 1

Based on THB-8, the subsurface conditions at Little Bear Creek Bridge No. 1 consist of 8.6 meters of loose to medium dense gravel, cobbles and boulders. The longest length of cobble cored was 0.25 meters. Larger boulders are evident on the ground surface and in the stream channel. N-values range from 7 to 54, increasing consistently with depth. The presence of gravel, cobbles, and boulders has probably impacted some SPT results, causing them to be higher than the density would indicate.

The summary logs show recovery for core runs in the overburden, as well as the SPT results. Core intervals include overcoring (reaming) the interval of the hole sampled with the SPT split spoon sampler. Despite coring through this disturbed material, recovery (based on the full length of the run) generally varied from 31 to 66 percent in these deposits. One run, where sandier materials were encountered, had recovery of only 18 percent. This suggests percent recovery may correlate with the particle sizes encountered in the overburden.

Water was encountered in the hole at a 2.9 depth of meters. This appears to be about 0.5 meters below the channel bottom, based on the centerline elevation shown on Figure 6-2.

Bedrock consists of strong, slightly weathered gneiss. Recovery was 70 to 80 percent and the RQD is 10 to 20. The unconfined compressive strength of a sample from 11.0 meter depth is 47.0 MPa. The unconfined compressive strength estimated from point load testing varies from 127 to 174 MPa. The laboratory test results are summarized in Table 6-2.

Table 6-2

SUMMARY OF LABORATORY TEST RESULTS

Test Hole	Depth (m)	Bulk Density (kg/cu. m)	Classification	UCS (MPa)[1][2]	UCS (MPa)[2]
THB-8	9.60		Gneiss	174	
THB-8	11.00	2945	Gneiss		47
THB-8	11.60		Gneiss	127	
THB-11	3.10		Sandstone	27	
THB-11	4.00		Sandstone	20	
THB-11	5.20	2568	Sandstone		31

[1] Values determined from point load testing of core (ASTM D571), as described on data sheets in Appendix D.
[2] UCS = Unconfined Compressive Strength (ASTM D2938), as described on data sheets in Appendix D.

Little Bear Creek Bridge Number 2

Based on THB-11, the subsurface conditions at Little Bear Creek Bridge No. 2 consist of 3.0 meters of medium dense sand with few gravels and cobbles. Boulders were not encountered, but some boulders are evident on the nearby ground surface and in the stream channel. An SPT was conducted at 1.5 m depth, yielding an N-value of 29, and at the top of bedrock, where refusal was met. The presence of gravel and cobbles may have impacted the SPT results, causing them to be higher than the density would indicate. Recovery in the core barrel was only 5 to 15 percent in the overburden.

Water was encountered in the hole at a depth of 3.4 meters. This appears to be 1 to 2 meters below the channel bottom, based on the centerline elevation shown on Figure 6-3.

Bedrock consists of moderately strong, iron stained, medium to coarse grained sandstone, with some thin clay seams. Recovery was 73 to 100 percent and the RQD is 25 to 56. The unconfined compressive strength of a sample from 5.2 meter depth is 31.1 MPa. The unconfined compressive strength estimated from point load testing varies from 20 to 27 MPa.

6.2 INTERPRETATION AND ANALYSIS

Observations and data on soil and bedrock have been interpreted and analyzed to develop geotechnical recommendations. At this site the soil overburden has been evaluated for lateral earth pressure and the bedrock has been evaluated for the bridge foundation.

The bedrock quality has been evaluated based on the unconfined compressive strength of intact rock, the RQD, and the Rock Mass Rating (RMR). Table 6-3 presents the interpretation of bedrock quality and estimated ranges of engineering parameters based on these methods. In general, these characterizations show that the bedrock quality is very poor to fair. The rock mass Young's modulus (E_m), and the cohesive (c) and frictional (ϕ) strength parameters, are based on published correlations to the RMR. They are presented here to assist in characterizing the relative quality of the rock mass. These correlated values are part of the information used to develop the geotechnical recommendations for design presented in Section 6.3.

Table 6-3

INTERPRETATION OF BEDROCK QUALITY
LITTLE BEAR CREEK BRIDGES

Location	Typical RQD	RQD Based Rock Quality[1]	RMR[2] Range	RMR Class [2]	Em (GPa)[3]	C (kPa)[2]	ϕ (deg)[2]
Bridge No. 1 (West Abutment)	15	Very Poor	50-58	III (Fair)	10-16	250-300	30-35
Bridge No. 2 (East Abutment)	35	Poor	40-60	III (Fair)	6-18	200-300	25-35

Notes:

[1] Based on RQD, Deere (1964). Classification for RQD <25 = very poor, 25 < 50 = poor, 50 < 75 = fair, 75 < 90 = good, 90 < 100 = excellent
[2] Rock Mass Rating, Bieniawski (1989), classification for RMR < 20 = V (very poor), 21 to 40 = IV (poor), 41 to 60 = III (fair), 61 to 80 = II (good), 81 to 100 = I (very good). Cohesion and friction ranges are based on RMR class. For Class V: ϕ < 15°, c < 100 kPa; for Class IV: ϕ = 15-25°, c = 100-200 kPa; for Class III: ϕ = 25-35°, c = 200-300 kPa, for Class II: ϕ = 35-45°, c = 300-400 kPa; for Class I: ϕ > 45, c > 400 kPa.
[3] Young's Modulus of Rock Mass based on RMR, $E_m = 10^{(RMR-10)/40}$, Serafin and Pereira (1983)

6.3 RECOMMENDATIONS

6.3.1 Foundations

The Little Bear Creek Bridge Sites No. 1 and No. 2 appear suitable for the proposed construction from a geotechnical engineering point of view.

The following foundation systems were evaluated for bridge support at the site:

- driven end-bearing pile (H-pile) foundation systems; and
- straight shaft piers drilled into the bedrock.

Driven H-piles are a more suitable foundation system for these sites due to the presence of gravel and cobbles. Driven piles should be installed in accordance with the Project Specifications.

Driven Piles

Steel H-piles driven into the bedrock formation and to virtual refusal may be designed using a working stress of 82.8 MPa. Virtual refusal is defined in Section 551.06 of the Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects FP-96 (1996).

Bedrock elevations and pile penetration into bedrock to attain virtual refusal have been estimated and are presented below. Note that the bedrock surface varies, so the presented values should be considered as typical.

Location	Typical Bedrock Elevation (meters)	Estimated Bedrock Penetration (meters)
Little Bear Creek Bridge No. 1		
Abutment 1	2,830	2
Abutment 2	2,830	2
Little Bear Creek Bridge No. 2		
Abutment 1	2,854	1.5
Abutment 2	2,855	1.5

Individual pile settlement should be 15 mm or less when designed according to the criteria presented herein.

Groups of piles required to support concentrated loads will require appropriate reductions of the axial, uplift and lateral capacities based on the effective envelope of the pile group. This reduction can be avoided by spacing piles at a minimum distance of at least three diameters center to center. Piles spaced less than three diameters center to center should be evaluated on an individual basis to determine appropriate reductions in axial, uplift and lateral capacities.

The contractor should select a driving hammer and cushion combination which is capable of installing the selected piling without overstressing the pile material. We recommend that pile capacities be evaluated during pile installation using the Pile Driving Analyzer (PDA).

The pile hammer should be operated at the manufacturer's recommended stroke when measuring penetration resistance. Piles should be provided with driving shoes to protect the pile tips from damage when penetrating dense granular soils or cobbles. Pile driving operations should be observed under the direction of a qualified geotechnical engineer. Each pile should be observed and checked for buckling, crimping and alignment in addition to recording penetration resistance, depth of embedment, and general pile driving operations. No specific minimum loading is required, and it is not necessary to consider downdrag in the embankment.

Lateral resistance to horizontal forces can be provided by the bending resistance of piles. Lateral resistance can also be provided by battered piles. The vertical and horizontal components of the load will depend on the batter inclinations. Pile batters typically should be limited not to exceed 4:1 (vertical:horizontal).

We understand that resistance to lateral load will be evaluated by the L-Pile or COM 624 computer programs. Soil properties for lateral capacity analysis are presented as follows:

Material	Soil Type*	Subgrade Modulus (GPa/m)	ϕ (degrees)	c (kPa)	ε_{50}	Effective Unit Weight (kN/m^3)
Compacted Embankment Fill	4	41	35	0	---	21.2
Silty Sand with Gravel	4	27	35	0	---	11.8
Bedrock	2	543	0	239	0.007	11.8

*Soil Type 4 is sand
*Soil Type 2 is stiff clay below the water table

Abutment and Retaining Walls

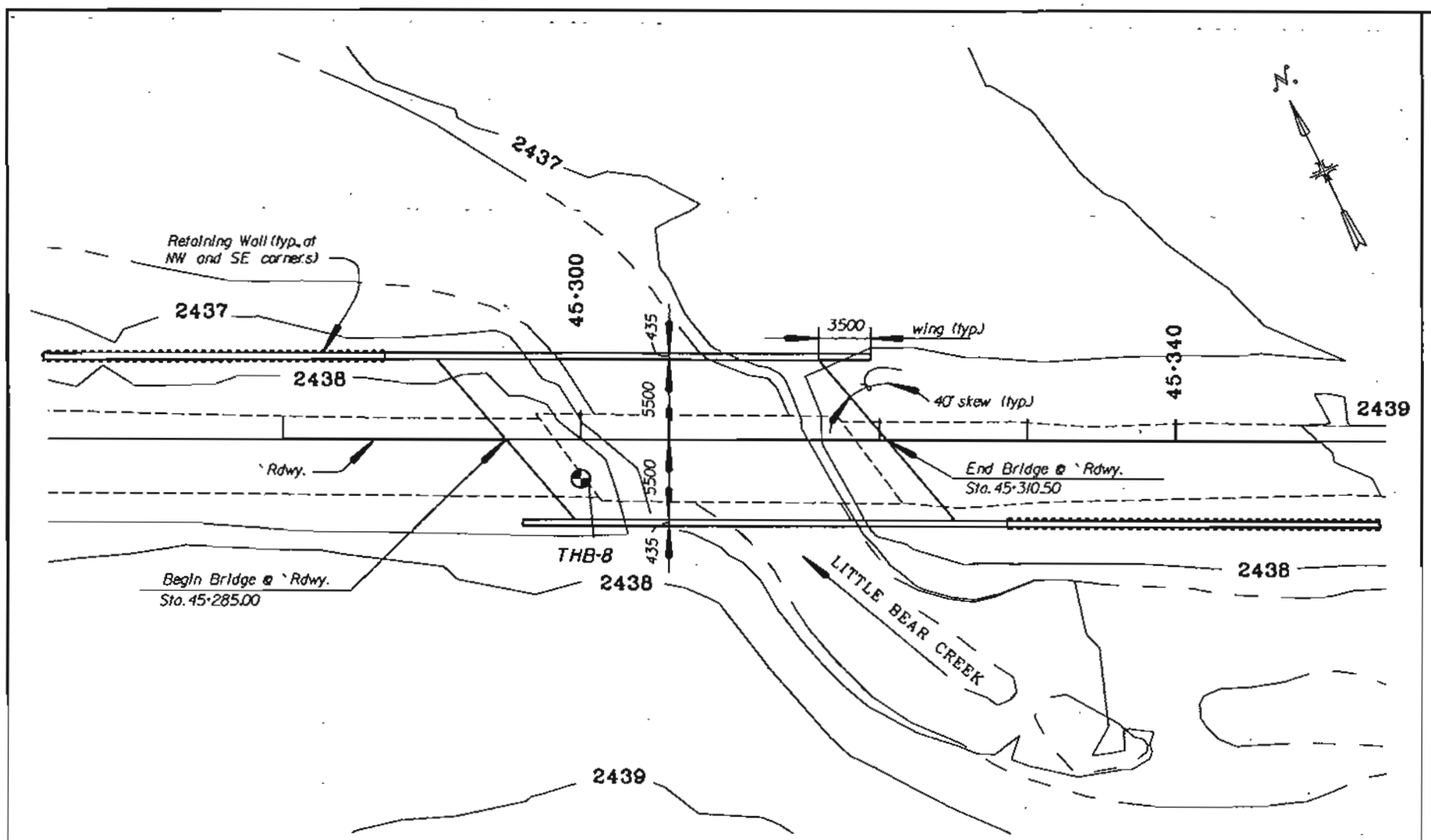
The use of free draining granular soils for backfilling behind bridge abutments and retaining walls is recommended. Granular soils result in lower lateral earth pressures and provide better subsurface drainage. Abutments and retaining walls should be provided with an underdrain system consisting of a front drainage zone and an underdrain pipe at the base of the wall. The front drainage zone may consist of a vertical layer of either a granular soil layer meeting FP-96 Section 210 or a geocomposite drain meeting FP-96 Section 605.05. The underdrain pipe should consist of a minimum 100 mm PVC pipe encased in a geotextile wrapped granular layer or a geocomposite sheet drain. The pipe should be sloped to daylight at a minimum slope of 2 percent.

Abutment walls and retaining walls which can be expected to deflect sufficiently to mobilize the full active earth pressure condition should be designed for a lateral earth pressure computed on the basis of an equivalent fluid unit weight of 5.9 kN/m^3. This pressure assumes a free draining backfill. Retaining structures should be designed for appropriate surcharge pressures such as traffic, construction equipment and upward sloping backfill.

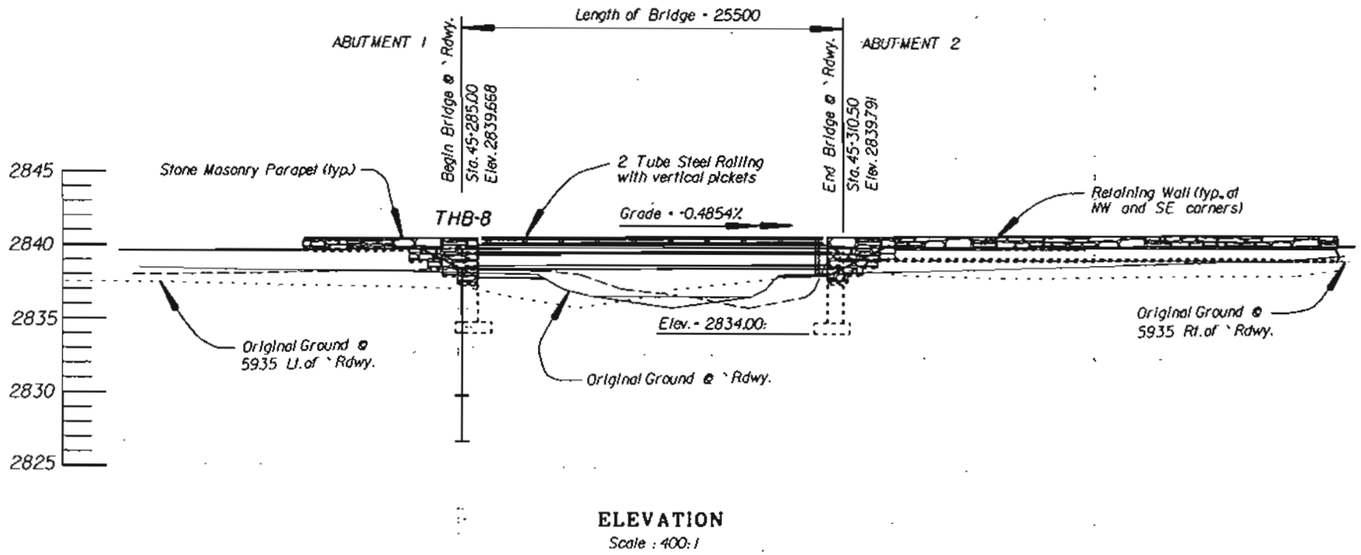
6.3.2 Other Considerations

Further Investigation. Only one abutment of each bridge was drilled. The other abutment should be drilled, sampled, and tested in a similar manner when the bridge location is finalized.

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/01	FH 4-1(0) BEARTOOTH HIGHWAY	6-1	
LITTLE BEAR CREEK BRIDGE NO. 1				



PLAN
Scale : 400: 1



ELEVATION
Scale : 400: 1

BRT H6-1.DGN

NOTES:

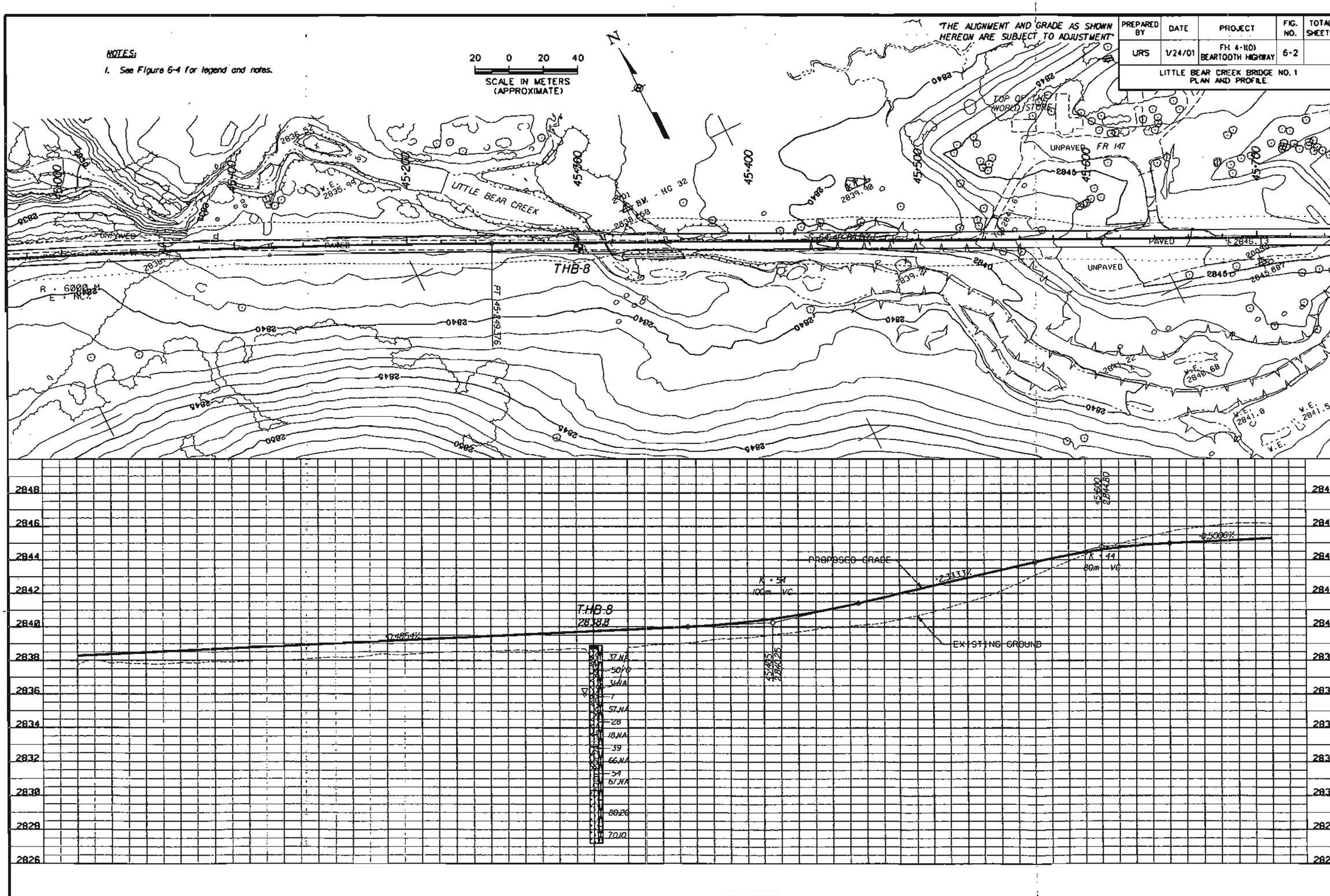
1. *Summary test hole log is shown on Figure 6-2.*
2. *See Figure 6-4 for legend and notes.*
3. *Intermediate tick on test hole stick logs indicates interpreted top of bedrock.*

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION, DENVER, COLORADO

PRELIMINARY LAYOUT

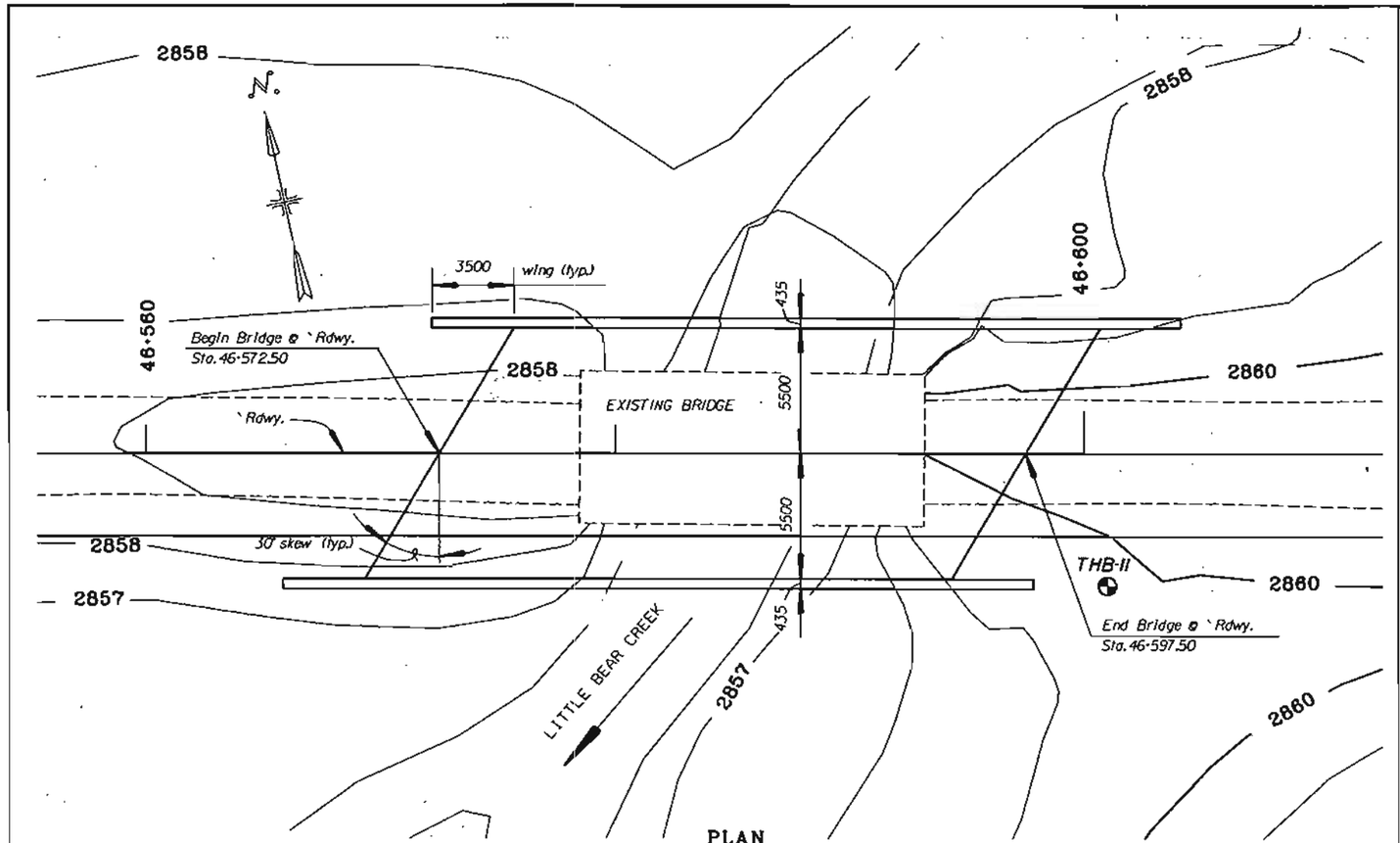
BRIDGE NAME *Little Bear Creek Bridge No.1* STREAM NAME *Little Bear Creek*
ROUTE NAME AND NUMBER *Beartooth Highway, WY FS 4-1(0)*
PREPARED BY *FHWA (Base Drawing) Only)*
SCALE: *As Shown* DATE: *October 16, 2000*

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/01	FH 4-1(0) BEARTOOTH HIGHWAY	6-2	
LITTLE BEAR CREEK BRIDGE NO. 1 PLAN AND PROFILE.				

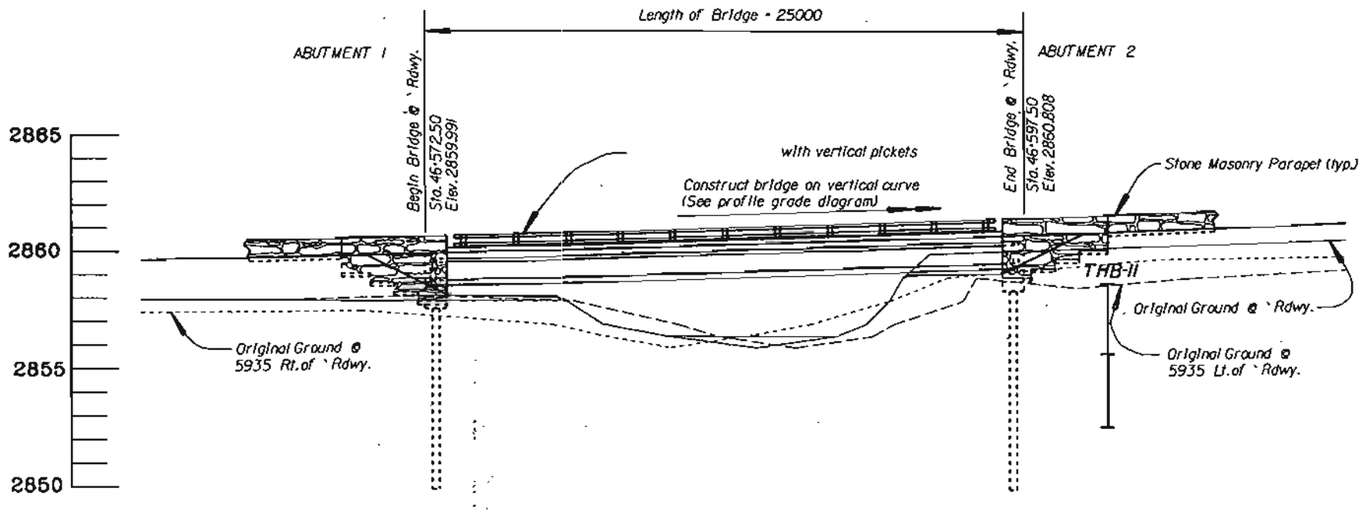


BRTH6-2.DGN

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/01	FH 4-K03 BEARTOOTH HIGHWAY	6-3	
LITTLE BEAR CREEK BRIDGE NO. 2				



PLAN
Scale : 250:1



ELEVATION
Scale : 250:1

BRTH6-3.DGN

NOTES:

1. Summary test hole log is shown on Figure 6-4.
2. See Figure 6-4 for legend and notes.
3. Intermediate tick on test hole stick logs indicates interpreted top of bedrock.

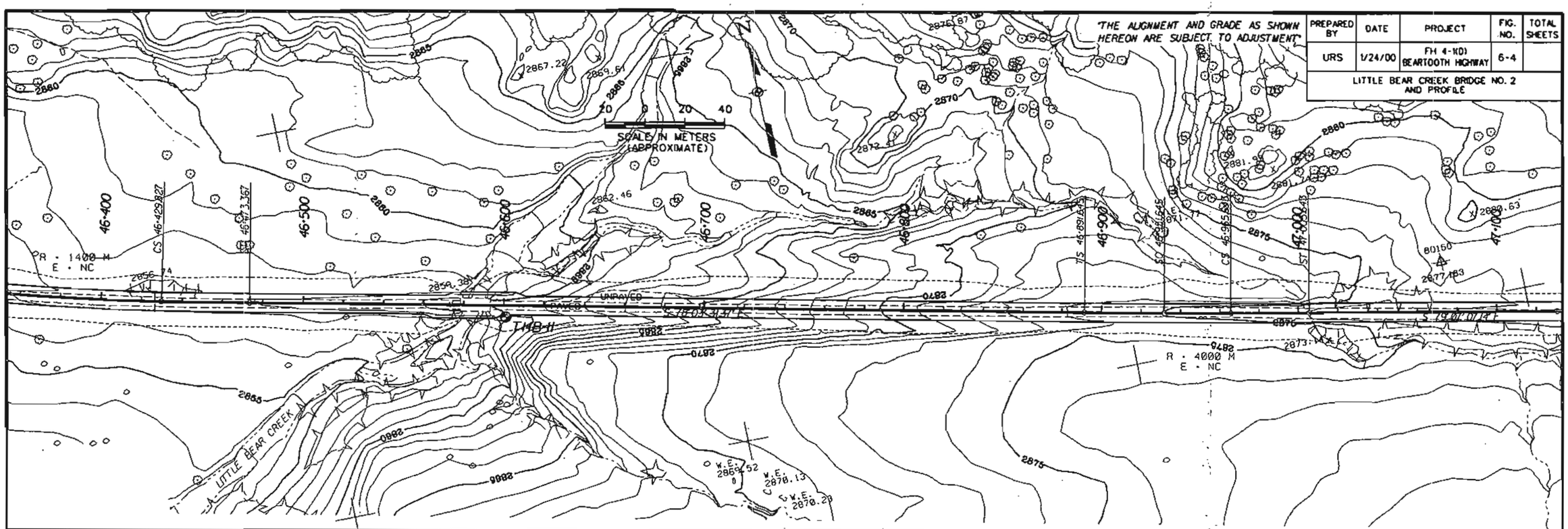
U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION, DENVER, COLORADO

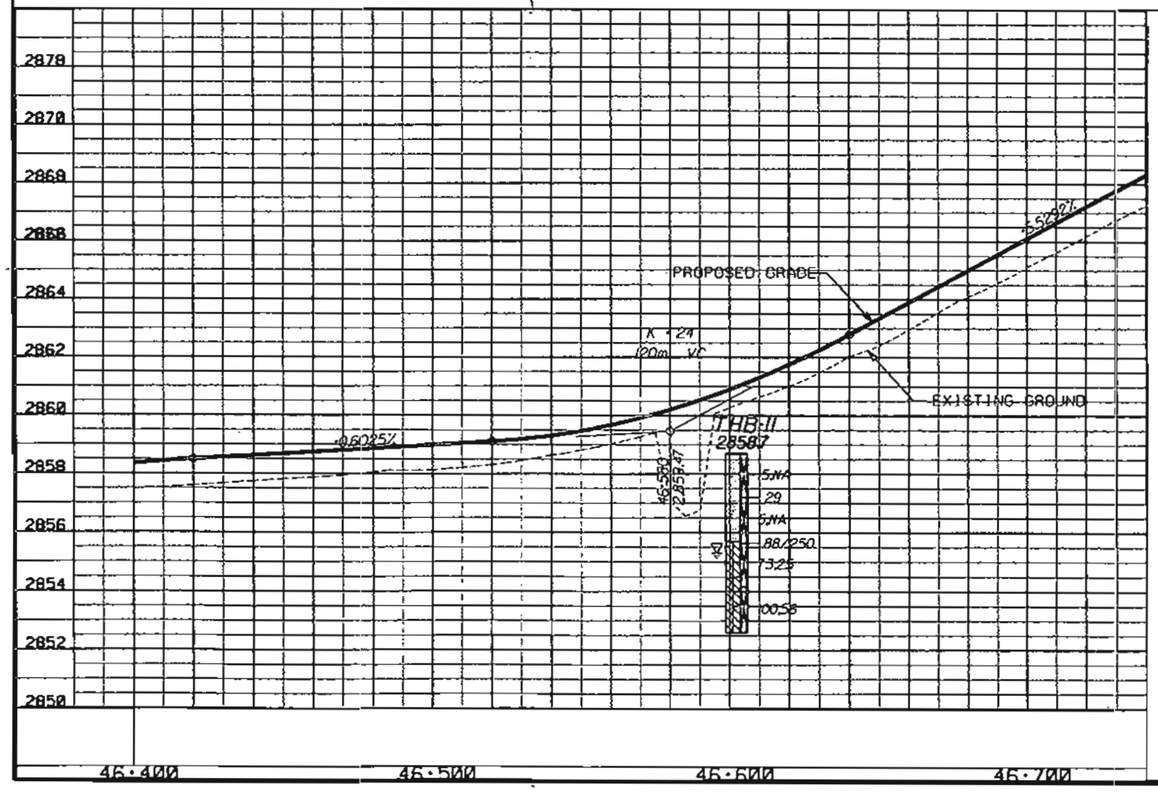
PRELIMINARY LAYOUT

BRIDGE NAME Little Bear Creek Bridge No.2 STREAM NAME Little Bear Creek
ROUTE NAME AND NUMBER Beartooth Highway, WY FS 4-(10)
PREPARED BY FHWA (Base Drawing Only)
SCALE: As Shown DATE: October 16, 2000

"THE ALIGNMENT AND GRADE AS SHOWN HEREON ARE SUBJECT TO ADJUSTMENT"

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/00	FH 4-1(10) BEARTOOTH HIGHWAY	6-4	
LITTLE BEAR CREEK BRIDGE NO. 2 AND PROFILE				





LEGEND FOR UTTLE BEAR CREEK BRIDGE NO. 1 AND NO. 2

- ASPHALT AND BASE COURSE PAVEMENT SECTION
- SAND, SW, MEDIUM DENSE, BROWN, SOME SILTY AND CLAYEY, SM, SC, GRAY, SOME GRAVEL AND COBBLES
- GRAVEL, COBBLE AND BOULDERS, LOOSE TO VERY DENSE (BASED ON BLOWCOUNTS), BROWN, REDDISH-BROWN, SILTY TO CLAYEY SAND MATRIX
- SANDSTONE, MODERATELY STRONG, GRAY, RED-BROWN, FINE TO COARSE GRAINED IRON STAINED, THIN LENSES OF CLAY
- GNEISS, MODERATELY STRONG TO VERY STRONG, GRAY, SLIGHTLY WEATHERED
- THB-II — TEST HOLE LOCATION
- — 12 — INDICATES TOP OF INTERVAL FOR STANDARD PENETRATION TEST (SPT). INDICATES THE BLOWCOUNT (N-VALUE) FOR THE FINAL 300 MM OF 450 MM TOTAL PENETRATION IS 12. THE SPT CONSISTS OF RECORDING BLOWS OF A 63.5 KG HAMMER FALLING 760 MM TO DRIVE A 50 MM O.D. SPLIT SPOON SAMPLER. WHERE TWO VALUES ARE SHOWN SEPARATED BY A SLASH, REFUSAL WAS MET BEFORE 450 MM PENETRATION. THE FIRST NUMBER IS THE BLOWCOUNT, THE SECOND NUMBER IS THE PENETRATION DEPTH.
- 90,30 — INDICATES SPLIT TRIPLE-TUBE CORE BARREL WAS USED TO ADVANCE HOLE. 90,30 INDICATES 90% RECOVERY FOR INTERVAL INDICATED AND A ROCK QUALITY DESIGNATION (ROD) OF 30. ROD IS 100 TIMES THE TOTAL LENGTH OF SOUND CORE OVER 100 MM (4 IN.) LONG DIVIDED BY THE LENGTH OF RUN INDICATED. WHERE SPT INTERVALS PRECEDE CORE INTERVALS, THE SPT INTERVAL IS SUBSEQUENTLY OVER-CORED. RECOVERY AND RUN ARE BASED ON A CORE RUN LENGTH THAT INCLUDES THE OVER-CORED LENGTH.
- NA — NOT APPLICABLE
- INDICATES WATER LEVEL AT TIME OF DRILLING

NOTES:

1. TEST HOLES WERE DRILLED ON 8/17/00 THROUGH 9/15/00 BY HAZTECH DRILLING, INC., USING A BK-81 (TRUCK MOUNT) RIG AND AN HQ SPLIT TRIPLE-TUBE CORE BARREL.
2. BASE MAPS OBTAINED FROM DESIGN FIGURES PROVIDED BY THE FHWA.
3. TEST HOLE LOCATIONS ARE APPROXIMATE AND WERE LOCATED BY TAPE MEASUREMENT ON OR BESIDE THE EXISTING ROAD AT DESIGNATED STATIONS. MEASUREMENTS WERE REFERENCED TO MARKINGS AND STAKES PLACED ALONG THE EXISTING ROAD BY THE FHWA.
4. TEST HOLE LOGS IN THIS REPORT ARE SUBJECT TO LIMITATIONS, EXPLANATIONS, AND CONCLUSIONS OF THIS REPORT. SEE INDIVIDUAL LOGS IN APPENDICES FOR MATERIAL DESCRIPTIONS AT EACH TEST HOLE.

BRTHB-4.DGN

7.1 INVESTIGATION

Three test holes, THB-12, THB-14, and THB-16a, were drilled at the bridge. The holes were drilled using an H-size core barrel and SPT split spoon samples were generally collected at the end of each 1.5 meter run, unless the run ended on a boulder or in bedrock. The test hole locations and results are summarized in Table 7-1. The test hole locations are shown on Figures 7-1 and 7-2. Summary logs are shown on Figure 7-2 and individual logs and photos are provided in Appendix B.

Table 7-1

TEST HOLE SUMMARY, FEN MITIGATION BRIDGE

Test Hole Number	Stationing	Offset (m)	Elevation (m)	Depth to Bedrock (m)	Depth to Water(1) (m)	Test Hole Depth (m)
THB-12	49+149	4.3 L	2913.6	5.6	2.5	10.7
THB-14	49+200	3.4 L	2913.4	8.6	1.5	13.7
THB-16a	49+276	1.07 L	2915.6	2.4	1.5	7.6

Notes: (1) Encountered in test hole at time of drilling. N.E. = Not Encountered

7.1.1 Site Characteristics

The Fen Mitigation bridge has five spans of 25.5 meters each, as shown on Figure 7-1. The bridge deck will be about 3.5 to 4.5 meters above the ground surface. The bridge spans a low alpine marsh area located above timberline.

7.1.2 Subsurface Characteristics

This section summarizes the abutment and pier foundation subsurface investigation results. The individual test hole logs are presented in Appendix B and the laboratory test results are presented in Appendix D.

THB-12 was drilled at the west abutment of the bridge. Slightly weathered granite bedrock was encountered approximately 5.6 meters below the ground surface. The overburden consists of gravel, cobbles and boulders in a matrix of sand, clay and silt. The longest length of boulder cored was 0.2 meters. Standard Penetration Tests (SPT's) were generally conducted at the end of each 1.5 meter core run, except at 4.5 meters, where an SPT was not performed because the end of the run was in a boulder. N-values range from 14 to 15. The summary logs show recovery for core runs in the overburden, as well as the SPT results. Core intervals include overcoring (reaming) the interval of the hole sampled with the SPT split spoon sampler. Recovery (based on the full length of the run) varied generally from 5 to 17 percent.

The bedrock was cored with an H-size core barrel and recovery was 100 percent. The rock quality designation (RQD) varies from 9 to 80. Groundwater was encountered at approximately 2.5 meter depth. The unconfined compressive strength of a sample from 6.1 meter depth is 84 MPa. The unconfined compressive strength estimated from point load testing varies from 93 to 100 MPa. Laboratory test results are summarized in Table 7-2.

Table 7-2

SUMMARY OF LABORATORY TEST RESULTS
FEN MITIGATION BRIDGE

Test Hole	Depth (m)	Bulk Density (kg/cu. m)	Classification	UCS (MPa)[1][2]	UCS (MPa)[2]
THB-12	6.10	2630	Granite		84
THB-12	9.30		Granite	93	
THB-12	10.60		Granite	100	
THB-14	9.30	2400	Gabbro		27
THB-14	10.35		Gabbro	87	
THB-16a	4.40	2621	Granite		80
THB-16a	6.30		Gneiss	93	
THB-16a	7.65		Granite	147	

[1] Values determined from point load testing of core (ASTM D5731), as described on data sheets in Appendix D.
[2] UCS = Unconfined Compressive Strength (ASTM D2938), as described on data sheets in Appendix D.

THB-14 was drilled at pier No. 2, at Station 49+199. Slightly to moderately weathered gabbro bedrock was encountered approximately 8.6 meters below the ground surface. The overburden consists of approximately 1.2 meters of gravel, cobbles and boulders in a sandy matrix, underlain by approximately 7.4 meters of medium stiff to very stiff clay. The longest length of boulder cored in the gravel material was 0.22 meters. N-values in the clay overburden ranged from 4 to 20.

The bedrock at THB-14 was cored with an H-size core barrel; recovery was 100 percent. The RQD varies from 0 to 58. Groundwater was encountered at approximately 1.5 meters. The unconfined compressive strength of a sample from 9.3 meters depth is 27 MPa. The unconfined compressive strength estimated from point load testing of a sample from 10.4 meters depth is 87 MPa.

THB-16a was drilled at the east abutment of the bridge. Slightly weathered granite bedrock was encountered approximately 2.4 meters below the ground surface. The overburden is gravel, cobbles and boulders. The longest length of boulder cored was 0.2 meters. An SPT was conducted at the end of the first 1.5 meter core run. The N-value was 23. The presence of gravel, cobbles, and boulders has probably impacted the SPT result.

Bedrock was cored with an H-size core barrel and recovery was 100 percent. The rock quality designation (RQD) varies from 0 to 13. Groundwater was encountered at approximately 1.5 meters below the ground surface. The unconfined compressive strength of a sample from 4.4 meters depth is 80 MPa. The unconfined compressive strength estimated from point load testing varies from 93 to 147 MPa.

7.2 INTERPRETATION AND ANALYSIS

Observations and data on soil and bedrock have been interpreted and analyzed to develop geotechnical recommendations. At this site the soil overburden has been evaluated for lateral earth pressure and the bedrock has been evaluated for the bridge foundation.

The bedrock quality has been evaluated based on the unconfined compressive strength of intact rock, the RQD, and the Rock Mass Rating (RMR). Table 7-3 presents the interpretation of bedrock quality and estimated ranges of engineering parameters based on these methods. In general, these characterizations show that the bedrock quality is very poor to fair. The bedrock quality appears to be better at the west abutment than elsewhere, but this difference is not significant enough to impact foundation recommendations.

The values of Young's modulus for the rock mass (E_m), and for the cohesive (c) and frictional (ϕ) strength parameters, are based on published correlations to the RMR. They are presented here to assist in characterizing the relative quality of the rock mass. These correlated values are part of the information used to develop the geotechnical recommendations for design presented in Section 7.3.

Table 7-3

INTERPRETATION OF BEDROCK QUALITY
FEN MITIGATION BRIDGE

Location	Typical RQD	RQD Based Rock Quality[1]	RMR[2] Range	RMR Class [2]	Em (GPa)[3]	C (kPa)[2]	ϕ (deg)[2]
West Abutment (No. 1)	40	Poor	52-53	III (Fair)	11-12	250-300	30-35
Pier 2 (Sta. 49+199)	20	Very Poor	40-44	III (Fair)	6-7	200-250	25-30
East Abutment (No. 2)	10	Very Poor	43-49	III (Fair)	7-9	200-250	25-30

Notes:

[1] Based on RQD, Deere (1964). Classification for RQD <25 = very poor, 25 < 50 = poor, 50 < 75 = fair, 75 < 90 = good, 90 < 100 = excellent
[2] Rock Mass Rating, Bieniawski (1989), classification for RMR < 20 = V (very poor), 21 to 40 = IV (poor), 41 to 60 = III (fair), 61 to 80 = II (good), 81 to 100 = I (very good). Cohesion and friction ranges are based on RMR class. For Class V: ϕ < 15°, c < 100 kPa; for Class IV: ϕ = 15-25°, c = 100-200 kPa; for Class III: ϕ = 25-35°, c = 200-300 kPa, for Class II: ϕ = 35-45°, c = 300-400 kPa; for Class I: ϕ > 45, c > 400 kPa.
[3] Young's Modulus of Rock Mass based on RMR, $E_m = 10^{(RMR-10)/40}$, Serafin and Pereira (1983)

7.3 RECOMMENDATIONS

7.3.1 Foundations

The following foundation systems were evaluated for bridge support at the site:

- straight shaft piers drilled into the bedrock;
- micropiles drilled into the bedrock
- driven end-bearing pile (H-pile) foundation systems; and

- spread footings bearing on bedrock at abutment No. 2.

Drilled shafts, driven piles, and spread footings should be installed in accordance with the Project Specifications.

Drilled Piers

Straight shaft piers should be drilled a minimum of 1.5 meters into bedrock. Design bedrock penetration should be measured below the bottom of temporary casing. The depth and quality of the bedrock should be evaluated under the direction of a qualified geotechnical engineer.

Piers should be designed for axial loads using a combination of end bearing and skin friction in the bedrock. Recommended allowable end bearing pressures and skin friction values for downward, compressive loads are as follows:

Location	Embedment in Bedrock (meters)	Allowable End Bearing Pressure (MPa)	Allowable Skin Friction, (Side Shear) (MPa)
Pier 1	All Depths	2.5	0.25
Piers 2 through 4	All Depths	1.5	0.15

Settlement of drilled pier foundations designed according to the above criteria should be 15 mm or less.

Piers should be considered to work in group action if the horizontal spacing is less than three pier diameters. A minimum practical horizontal spacing between piers of at least three diameters should be maintained, and adjacent piers should bear at about the same elevation. The capacity of individual piers must be reduced when considering the effects of group action. Capacity reduction is a function of pier spacing and the number of piers within a group. If group action analyses are necessary, capacity reduction factors can be provided for the analyses.

All piers should be reinforced full depth for the applied axial, lateral and uplift stresses imposed. A minimum reinforcement of at least one-half percent of the cross-sectional area of each pier is recommended.

Drilling to design depths should be possible with high capacity augers and core barrels. Groundwater and granular soil conditions indicate that temporary steel casing will be required to properly drill and clean piers prior to concrete placement. Groundwater should be removed from each pier hole prior to concrete placement. Pier concrete should be placed immediately after completion of drilling and cleaning. If pier concrete cannot be placed in dry conditions, a tremie should be used for concrete placement. Due to potential sloughing and raveling, foundation concrete quantities may exceed calculated geometric volumes.

Casing should be withdrawn in a slow continuous manner maintaining a head of at least 5 feet of concrete to prevent infiltration of water or the creation of voids in pier concrete. Pier concrete should have a relatively high fluidity when placed in cased pier holes or through a tremie. Concrete with slump in the range of 125 to 200 mm is recommended.

Free-fall concrete placement in piers is acceptable if provisions are taken to avoid striking the concrete on the sides of the hole or reinforcing steel. Use of hoppers with an elephant trunk or other approved device is recommended.

Pier bearing surfaces must be cleaned prior to concrete placement. The top of the piers should be smooth and free of protrusions. Drilled shaft installation should be observed under the direction of a qualified geotechnical engineer. The bearing surface and pier configuration should be observed.

Lateral resistance to horizontal forces can be provided by the bending resistance of piers and piles. Lateral resistance can also be provided by battered piles. The vertical and horizontal components of the load will depend on the batter inclinations. We recommend that pile batters typically be limited not to exceed 4:1 (vertical:horizontal).

We understand that resistance to lateral load will be evaluated by the L-Pile or COM 624 computer programs. Soil properties for lateral capacity analysis are presented as follows:

Material	Soil Type*	Subgrade Modulus (GPa/m)	ϕ (degrees)	c (kPa)	ε_{50}	Effective Unit Weight (kN/m^3)
Compacted Embankment Fill	4	41	35	0	---	21.2
Silty Clay (Piers 1 through 4)	3	22	0	72	0.015	18.8
Silty Sand with Gravel	4	27	35	0	---	11.8
Bedrock	2	543	0	239	0.007	11.8

*Soil Type 4 is sand
*Soil Type 3 is stiff clay above the water table
*Soil Type 2 is stiff clay below the water table

Micropiles

Micropiles should be designed and constructed in accordance with FHWA report FHWA-SA-97-070, Micropile Design and Construction Guidelines Implementation Manual, 2000, and the recommendations presented herein. Type A micropoiles are recommended (FHWA, 2000).

The geotechnical design for micropiles should be based on an allowable bond stress of the soil/rock bond of 0.25 MPa at pier and 0.15 MPa elsewhere. This recommendation is based on reported nominal bond stress values (FHWA, 2000), consideration of acceptable settlement in jointed rock mass using methodology for drilled piers, and a factor of safety of 2.5. Micropiles in the range of 229 to 254 mm diameter will have capacities of 108 to 120 kN/m using 0.15 MPa of bond strength.

The structural design of micropiles should be based on an allowable load of 0.47 times the lower yield stress of the steel casing or reinforcement, and 0.4 times the unconfined compressive strength of the grout. The structural design should also provide for sufficient length for the grout to steel bond.

Micropiles should be constructed with permanent steel casing, with a casing plunge extending at least 0.5 m into the underlying bedrock. The casing should meet the physical properties of

ASTM A-252 Grade 3, except that the minimum yield strength should be 552 MPa. Reinforcing steel may be Grade 60, 75 or 80 reinforcing rod (ASTM A615, A616 or A617). The grout should consist of neat cement and water mixed with a high shear colloidal type mixer. The grout should have a fluid consistency, a water/ cement ratio of about 0.45 and a minimum unconfined compressive strength of 27.6 MPa at 28 days.

Micropiles provide lateral resistance from the horizontal response of the adjacent soils and can sustain significant lateral deflection within the available structural pile capacity. Increased bending capacity can be provided by increasing the diameter of the pile or the thickness of the pipe. Lateral resistance can also be provided by battered piles. The lateral design parameters presented in the Drilled Piers section may be used for the design of micropiles.

Driven Piles

Steel H-piles driven into the bedrock formation and to virtual refusal may be designed using a working stress of 82.8 MPa. Virtual refusal is defined in Section 551.06 of the Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects FP-96 (1996).

Bedrock elevations and pile penetration into bedrock to attain virtual refusal have been estimated and are presented below. Bedrock surface varies, so the presented values should be considered as typical.

Location	Typical Bedrock Elevation (meters)	Estimated Bedrock Penetration (meters)
Abutment 1	2,908	1
Pier 1	2,906	1.5
Pier 2	2,905	1.5
Pier 3	2,905	1.5
Pier 4	2,906	1.5

Individual pile settlement should be 15 mm or less when designed according to the criteria presented herein.

Groups of piles required to support concentrated loads will require appropriate reductions of the axial, uplift and lateral capacities based on the effective envelope of the pile group. This reduction can be avoided by spacing piles at a minimum distance of at least three diameters center to center. Piles spaced less than three diameters center to center should be evaluated on an individual basis to determine appropriate reductions in axial, uplift and lateral capacities.

The contractor should select a driving hammer and cushion combination which is capable of installing the selected piling without overstressing the pile material. Pile capacities should be evaluated during pile installation using the Pile Driving Analyzer (PDA).

The pile hammer should be operated at the manufacturer's recommended stroke when measuring penetration resistance. Piles should be provided with driving shoes to protect the pile tips from damage when penetrating dense granular soils or cobbles. Pile driving operations should be observed under the direction of a qualified geotechnical engineer. Each pile should be observed

and checked for buckling, crimping and alignment in addition to recording penetration resistance, depth of embedment, and general pile driving operations.

With respect to driven pile foundations, no specific minimum loading is required, and it is not necessary to consider downdrag in the embankment. The lateral design parameters presented in the drilled pier section may be used for the design of driven piles.

Spread Footings

Abutment No. 2 may be founded on spread footings bearing on the underlying bedrock. Footings placed on undisturbed bedrock may be designed for an allowable bearing pressure of 0.5 MPa. We estimate total settlement of footings designed and constructed as discussed in this section will be less than 25 mm. Footings should be placed at least 1.5 m below the surrounding exterior grade to provide adequate frost protection. The lateral resistance of footings placed on the natural bedrock will be a combination of the sliding resistance on the bottom of the footing and the passive earth pressure against the side of the footing. Resistance on the bottom of the footing can be calculated based on a coefficient of friction of 0.4. Passive pressure against the side of the footing can be calculated using an equivalent fluid unit weight of 9 $kN/m^2/m$ assuming properly compacted granular backfill.

Abutment and Retaining Walls

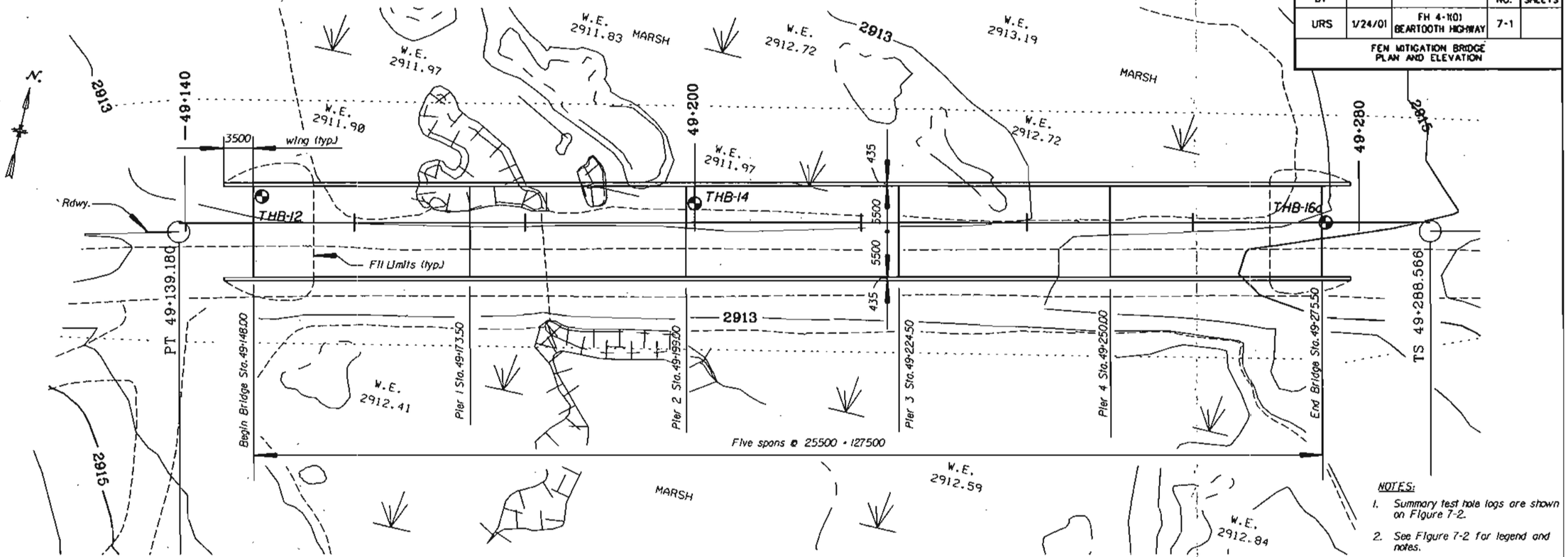
The use of free draining granular soils for backfilling behind bridge abutments and retaining walls is recommended. Granular soils result in lower lateral earth pressures and provide better subsurface drainage. Abutments and retaining walls should be provided with an underdrain system consisting of a front drainage zone and an underdrain pipe at the base of the wall. The front drainage zone may consist of a vertical layer of either a granular soil layer meeting FP-96 Section 210 or a geocomposite drain meeting FP-96 Section 605.05. The underdrain pipe should consist of a minimum 100 mm PVC pipe encased in a geotextile wrapped granular layer or a geocomposite sheet drain. The pipe should be sloped to daylight at a minimum slope of 2 percent.

Abutment walls and retaining walls which can be expected to deflect sufficiently to mobilize the full active earth pressure condition should be designed for a lateral earth pressure computed on the basis of an equivalent fluid unit weight of 5.9 kN/m^3. This pressure assumes a free draining backfill. Retaining structures should be designed for appropriate surcharge pressures such as traffic, construction equipment and upward sloping backfill.

7.3.2 Other Considerations

Further Investigation. Piers 1, 3, and 4 were not drilled as part of this investigation. These pier locations are recommended to be drilled, sampled, and tested in a manner similar to that done at Pier 2 and the abutments. This investigation should be conducted after the bridge pier locations have been finalized.

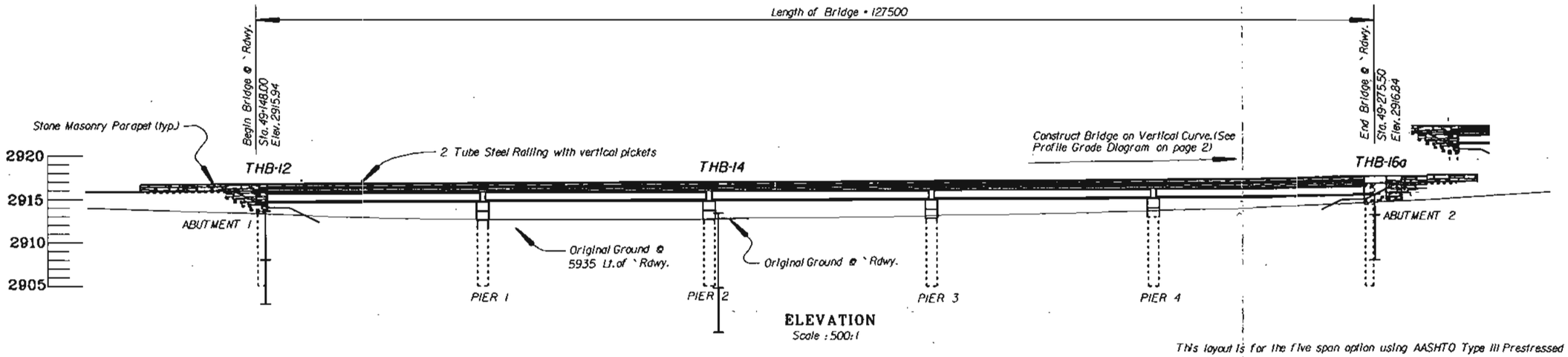
PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/01	FH 4-1(10) BEARTOOTH HIGHWAY	7-1	
FEN MITIGATION BRIDGE PLAN AND ELEVATION				



PLAN
Scale : 500:1

NOTES:

1. Summary test hole logs are shown on Figure 7-2.
2. See Figure 7-2 for legend and notes.
3. Intermediate tick on test hole stick logs indicates interpreted top of bedrock.



ELEVATION
Scale : 500:1

This layout is for the five span option using AASHTO Type III Prestressed Girders.

PREPARED BY FHWA (Base Drawing Only) SCALE: As Shown DATE: July 17, 2000

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION, DENVER, COLORADO

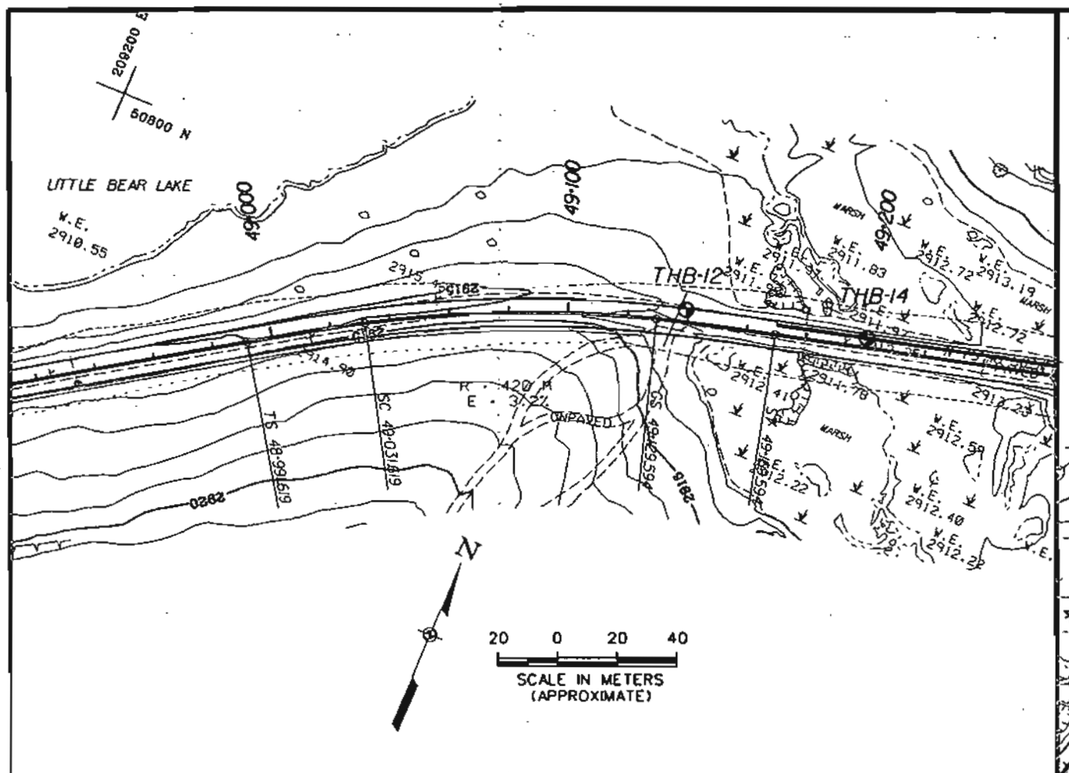
BRIDGE NAME Fen Mitigation Bridge

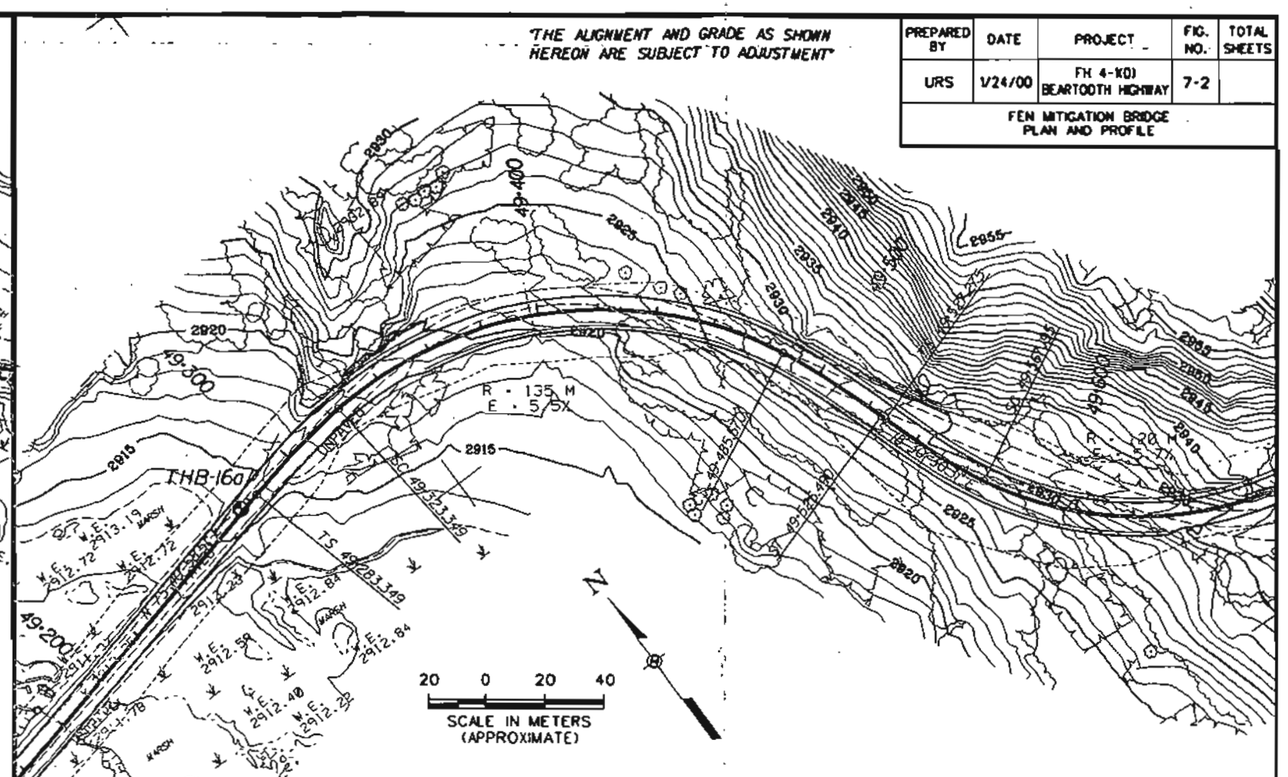
PRELIMINARY LAYOUT

BRTH7-1.DGN

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/00	FH 4-1(0) BEARTOOTH HIGHWAY	7-2	
FEN MITIGATION BRIDGE PLAN AND PROFILE				

"THE ALIGNMENT AND GRADE AS SHOWN HEREON ARE SUBJECT TO ADJUSTMENT"





LEGEND FOR FEN MITIGATION BRIDGE

- CLAY, CL, MEDIUM STIFF TO VERY STIFF, GRAY, SILTY, SOME GRAVEL
- GRAVEL, COBBLE, AND BOULDERS, MEDIUM DENSE, (BASED ON BLOWCOUNTS), BROWN, SANDY TO GRAVELLY MATRIX,
- GRANITE, MODERATELY STRONG TO VERY STRONG, RED-GRAY, RED, GRAY, PINK, BLACK, BLACK TO PINK, COARSE GRAINED, WEAKLY FOLIATED, SLIGHTLY WEATHERED
- GNEISS TO GRANITE, STRONG TO VERY STRONG, PINK TO BLACK, MEDIUM TO COARSE GRAINED, SLIGHTLY WEATHERED
- GRABBO, WEAK TO STRONG, GRAY TO BLACK, SLIGHTLY TO MODERATELY WEATHERED

THB-12 — TEST HOLE LOCATION

— 12 — INDICATES TOP OF INTERVAL FOR STANDARD PENETRATION TEST (SPT). INDICATES THE BLOWCOUNT (N-VALUE) FOR THE FINAL 300 MM OF 450 MM TOTAL PENETRATION IS 12. THE SPT CONSISTS OF RECORDING BLOWS OF A 63.5 KG HAMMER FALLING 760 MM TO DRIVE A 50 MM O.D. SPLIT SPOON SAMPLER. WHERE TWO VALUES ARE SHOWN SEPARATED BY A SLASH, REFUSAL WAS MET BEFORE 450 MM PENETRATION. THE FIRST NUMBER IS THE BLOWCOUNT, THE SECOND NUMBER IS THE PENETRATION DEPTH.

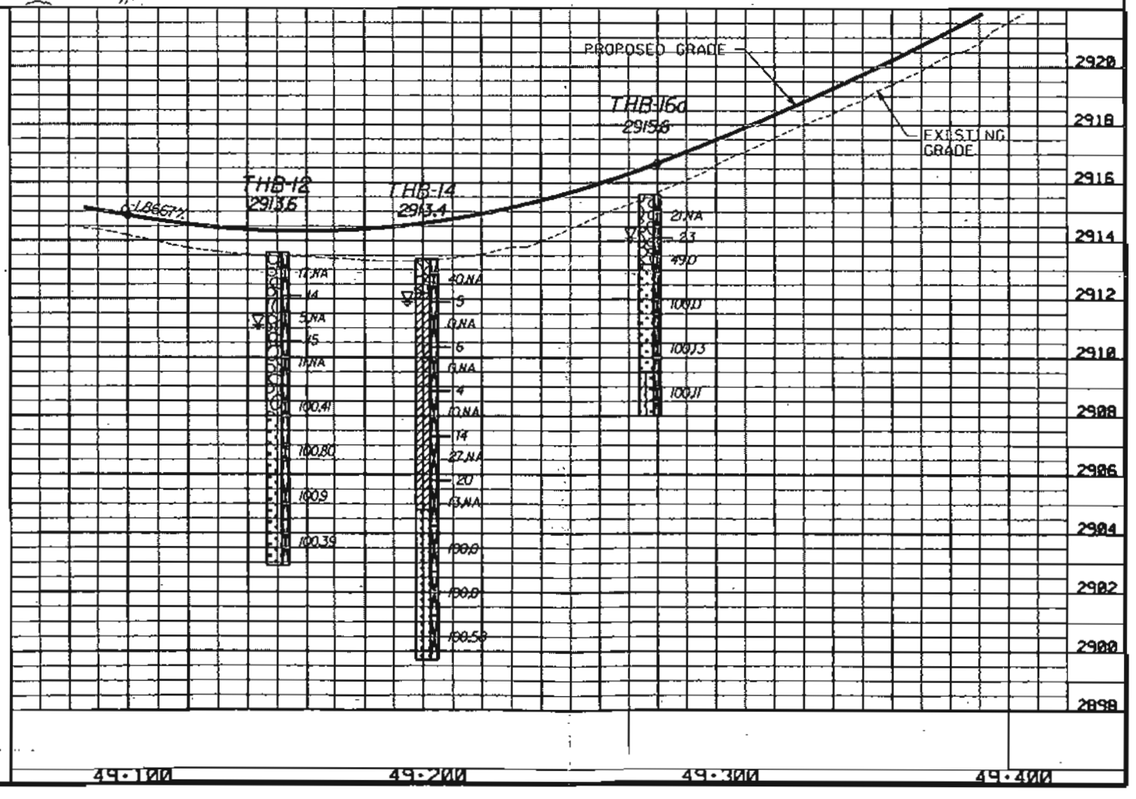
90,30 — INDICATES SPLIT TRIPLE-TUBE CORE BARREL WAS USED TO ADVANCE HOLE. 90,30 INDICATES 90% RECOVERY FOR INTERVAL INDICATED AND A ROCK QUALITY DESIGNATION (RQD) OF 30. RQD IS 100 TIMES THE TOTAL LENGTH OF SOUND CORE OVER 100 MM (40 IN.) LONG DIVIDED BY THE LENGTH OF RUN INDICATED. WHERE SPT INTERVALS PRECEDE CORE INTERVALS, THE SPT INTERVAL IS SUBSEQUENTLY OVER-CORED. RECOVERY AND RUN ARE BASED ON A CORE RUN LENGTH THAT INCLUDES THE OVER-CORED LENGTH.

NA — NOT APPLICABLE

▽ — INDICATES WATER LEVEL AT TIME OF DRILLING

NOTES:

1. TEST HOLES WERE DRILLED ON 8/17/00 THROUGH 9/15/00 BY HAZTECH DRILLING, INC., USING A BK-81 (TRUCK MOUNT) RIG AND AN HQ SPLIT TRIPLE-TUBE CORE BARREL.
2. BASE MAPS OBTAINED FROM DESIGN FIGURES PROVIDED BY THE FHWA.
3. TEST HOLE LOCATIONS ARE APPROXIMATE AND WERE LOCATED BY TAPE MEASUREMENT ON OR BESIDE THE EXISTING ROAD AT DESIGNATED STATIONS. MEASUREMENTS WERE REFERENCED TO MARKINGS AND STAKES PLACED ALONG THE EXISTING ROAD BY THE FHWA.
4. TEST HOLE LOGS IN THIS REPORT ARE SUBJECT TO LIMITATIONS, EXPLANATIONS, AND CONCLUSIONS OF THIS REPORT. SEE INDIVIDUAL LOGS IN APPENDICES FOR MATERIAL DESCRIPTIONS AT EACH TEST HOLE.



8RTH7-2.DGN

8.1 INVESTIGATION

Two test holes, THB-19 and THB-20 were drilled at the bridge. The holes were drilled using an H-size core barrel and SPT split spoon samples were generally collected at the end of each 1.5 meter core run unless the run ended on a boulder or in bedrock. The test hole locations and results are summarized in Table 8-1. The test hole locations are shown on Figures 8-1 and 8-2, summary logs are shown on Figure 8-2 and individual logs and photos are provided in Appendix B.

Table 8-1

TEST HOLE SUMMARY, LONG LAKE OUTLET BRIDGE

Test Hole Number	Stationing	Offset (m)	Elevation (m)	Depth to Bedrock (m)	Depth to Water(1) (m)	Test Hole Depth (m)
THB-19	50+262	7.6 L	2940.9	1.7	1.3	6.0
THB-20	50+285	6.7 L	2941.0	3.9	1.3	7.0

Notes: (1) Encountered in test hole at time of drilling. N.E. = Not Encountered

8.1.1 Site Characteristics

The Long Lake Outlet Bridge has one span of 19.5 meters, as shown on Figure 8-1. The bridge deck will be about 1.2 to 2.2 meters above the ground surface. The bridge spans Little Bear Creek, which is the outlet for Long Lake, located in a high alpine environment. We understand a retaining wall of less than approximately 2.5 meter height and 18 meter length is planned on the left side of the road, adjacent to Long Lake, east of the bridge.

8.1.2 Subsurface Characteristics

This section summarizes the abutment and pier foundation subsurface investigation results. The individual test hole logs are presented in Appendix B and the laboratory test results are presented in Appendix D.

THB-19 was drilled at the west abutment of the bridge. Slightly weathered granite bedrock was encountered approximately 1.7 meters below the road surface. The overburden consists of gravel, cobbles and boulders in a sand matrix. The longest length of cobble cored was 0.23 meters. Percent recovery in the core barrel of this overburden material was 40 percent. The bedrock was cored with an H-size core barrel and core recovery was generally 100 percent below 3 meters depth, and less at shallower depth. The rock quality designation (RQD) varies from 0 to 20. Groundwater was encountered at approximately 1.3 meter depth. The unconfined compressive strength of a sample from 3.5 meter depth is 42 MPa. The unconfined compressive strength estimated from point load testing varies from 119 to 125 MPa. The laboratory test results are summarized in Table 8-2.

Table 8-2

SUMMARY OF LABORATORY TEST RESULTS
LONG LAKE OUTLET BRIDGE

Test Hole	Depth (m)	Bulk Density (kg/cu. m)	Classification	UCS[1][2] (MPa)	UCS[2] (MPa)
THB-19	3.40	2765	Granite		42
THB-19	4.90		Granite	125	
THB-19	5.80		Granite	119	
THB-20	5.60	2623	Granite	200	118
THB-20	6.10		Granite	143	
THB-20	6.90		Granite	160	

[1] Values determined from point load testing of core (ASTM D5731), as described on data sheets in Appendix D.
[2] UCS = Unconfined Compressive Strength (ASTM D2938), as described on data sheets in Appendix D.

THB-20 was drilled at the east abutment of the bridge. Slightly weathered granite bedrock was encountered approximately 3.9 meters below the road surface. The overburden is gravel, cobbles and boulders in a sand matrix. The longest length of cobble cored was 0.2 meters. Standard Penetration Tests (SPT's) were generally conducted at the end of each 1.5 meter core run. N-values range from 17 to 24. The considerable presence of gravel, cobbles, and boulders has probably impacted some SPT results, especially the higher values. The summary logs show recovery for core runs in the overburden, as well as the SPT results. Core intervals include overcoring (reaming) the interval of the hole sampled with the SPT split spoon sampler. Despite coring through this disturbed material, recovery varied from 32 to 35.

Bedrock was cored with an H-size core barrel and recovery was 100 percent. The rock quality designation (RQD) varies from 20 to 78. Groundwater was encountered at approximately 1.3 meters below the ground surface. The unconfined compressive strength of a sample from 5.6 meters depth is 118 MPa. The unconfined compressive strength estimated from point load testing varies from 143 to 200 MPa.

8.2 INTERPRETATION AND ANALYSIS

Observations and data on soil and bedrock have been interpreted and analyzed to develop geotechnical recommendations. At this site the soil overburden has been evaluated primarily for temporary excavation slopes and lateral earth pressure. The bedrock has been evaluated for the bridge foundation.

The bedrock quality has been evaluated based on the unconfined compressive strength of intact rock, the Rock Quality Designation (RQD), and the Rock Mass Rating (RMR). Table 8-3 presents the interpretation of bedrock quality and estimated ranges of engineering parameters based on these methods. In general, these characterizations show that the bedrock quality varies from very poor to good. The bedrock quality appears to be slightly better at the east abutment than at the west abutment, but this difference is not significant enough to impact foundation recommendations.

The values of Young's modulus for the rock mass (E_m), and for the cohesive (c) and frictional (ϕ) strength parameters, are based on published correlations to the RMR. They are presented here to assist in characterizing the relative quality of the rock mass. These correlated values are part of the information used to develop the geotechnical recommendations for design presented in Section 8.3.

Table 8-3

INTERPRETATION OF BEDROCK QUALITY
LONG LAKE OUTLET BRIDGE

Location	Typical RQD	RQD Based Rock Quality[1]	RMR[2] Range	RMR Class[2]	Em (GPa)[3]	c (kPa)[2]	ϕ (deg)[2]
West Abutment (No. 1)	10	Very Poor	40-48	III (Fair)	6-9	200-250	25-30
East Abutment (No. 2)	40	Poor	48-66	III (Fair) II (Good)	9-25	250-350	30-40

Notes:

[1] Based on RQD, Deere (1964). Classification for RQD <25 = very poor, 25 < 50 = poor, 50 < 75 = fair, 75 < 90 = good, 90 < 100 = excellent

[2] Rock Mass Rating, Bieniawski (1989), classification for RMR < 20 = V (very poor), 21 to 40 = IV (poor), 41 to 60 = III (fair), 61 to 80 = II (good), 81 to 100 = I (very good). Cohesion and friction ranges are based on RMR class. For Class V: ϕ < 15°, c < 100 kPa; for Class IV: ϕ = 15-25°, c = 100-200 kPa; for Class III: ϕ = 25-35°, c = 200-300 kPa, for Class II: ϕ = 35-45°, c = 300-400 kPa; for Class I: ϕ > 45, c > 400 kPa.

[3] Young's Modulus of Rock Mass based on RMR, $E_m = {}^{10\,(RMR-10)/40}$, Serafin and Pereira (1983)

8.3 RECOMMENDATIONS

8.3.1 Foundations

The Long Lake Outlet Bridge site appears suitable for the proposed construction from a geotechnical engineering point of view.

Driven end-bearing pile (H-pile) foundation systems were evaluated for bridge support at the site. Driven H-piles are a suitable foundation system for this site due to the presence of gravel and cobbles. Driven piles should be installed in accordance with the Project Specifications.

Driven Piles

Steel H-piles driven into the bedrock formation and to virtual refusal may be designed using a working stress of 82.8 MPa. Virtual refusal is defined in Section 551.06 of the Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects FP-96 (1996).

Bedrock elevations and pile penetration into bedrock to attain virtual refusal have been estimated and are presented below. Note that the bedrock surface varies, so the presented values should be considered as typical.

Location	Typical Bedrock Elevation (meters)	Estimated Bedrock Penetration (meters)
Abutment 1	2939	1.5
Abutment 2	2937	1

Individual pile settlement should be 15 mm or less when designed according to the criteria presented herein.

Groups of piles required to support concentrated loads will require appropriate reductions of the axial, uplift and lateral capacities based on the effective envelope of the pile group. This reduction can be avoided by spacing piles at a minimum distance of at least three diameters center to center. Piles spaced less than three diameters center to center should be evaluated on an individual basis to determine appropriate reductions in axial, uplift and lateral capacities.

The contractor should select a driving hammer and cushion combination which is capable of installing the selected piling without overstressing the pile material. Pile capacities should be evaluated during pile installation using the Pile Driving Analyzer (PDA).

The pile hammer should be operated at the manufacturer's recommended stroke when measuring penetration resistance. Pile should be provided with driving shoes to protect the pile tips from damage when penetrating dense granular soils or cobbles. Pile driving operations should be observed under the direction of a qualified geotechnical engineer. Each pile should be observed and checked for buckling, crimping and alignment in addition to recording penetration resistance, depth of embedment, and general pile driving operations. No specific minimum loading is required, and it is not necessary to consider downdrag in the embankment.

Lateral resistance to horizontal forces can be provided by the bending resistance of piles. Lateral resistance can also be provided by battered piles. The vertical and horizontal components of the load will depend on the batter inclinations. Pile batters typically should be limited not to exceed 4:1 (vertical: horizontal).

We understand that resistance to lateral load will be evaluated by the L-Pile or COM 624 computer programs. Soil properties for lateral capacity analysis are presented as follows:

Material	Soil Type*	Subgrade Modulus (GPa/M)	ϕ (degrees)	c (kPa)	ε_{50}	Effective Unit Weight (kN/m^3)
Compacted Embankment Fill	4	41	35	0	---	21.2
Silty Sand with Gravel	4	27	35	0	---	11.8
Bedrock	2	543	0	239	0.007	11.8

*Soil Type 4 is sand
*Soil Type 2 is stiff clay below the water table

Abutment and Retaining Walls

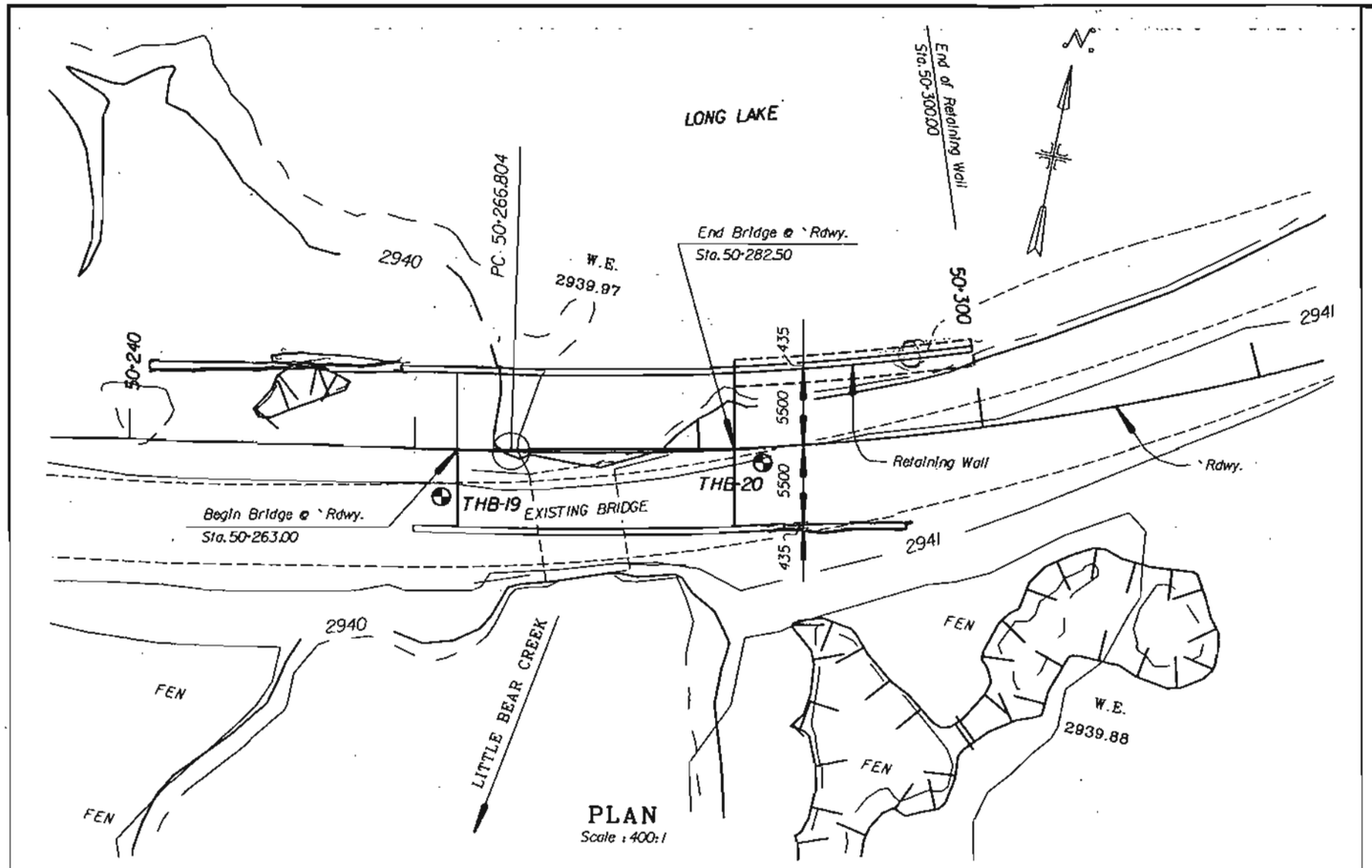
The use of free draining granular soils for backfilling behind bridge abutments and retaining walls is recommended. Granular soils result in lower lateral earth pressures and provide better subsurface drainage. Abutments and retaining walls should be provided with an underdrain system consisting of a front drainage zone and an underdrain pipe at the base of the wall. The front drainage zone may consist of a vertical layer of either a granular soil layer meeting FP-96 Section 210 or a geocomposite drain meeting FP-96 Section 605.05. The underdrain pipe should consist of a minimum 100 mm PVC pipe encased in a geotextile wrapped granular layer or a geocomposite sheet drain. The pipe should be sloped to daylight at a minimum slope of 2 percent.

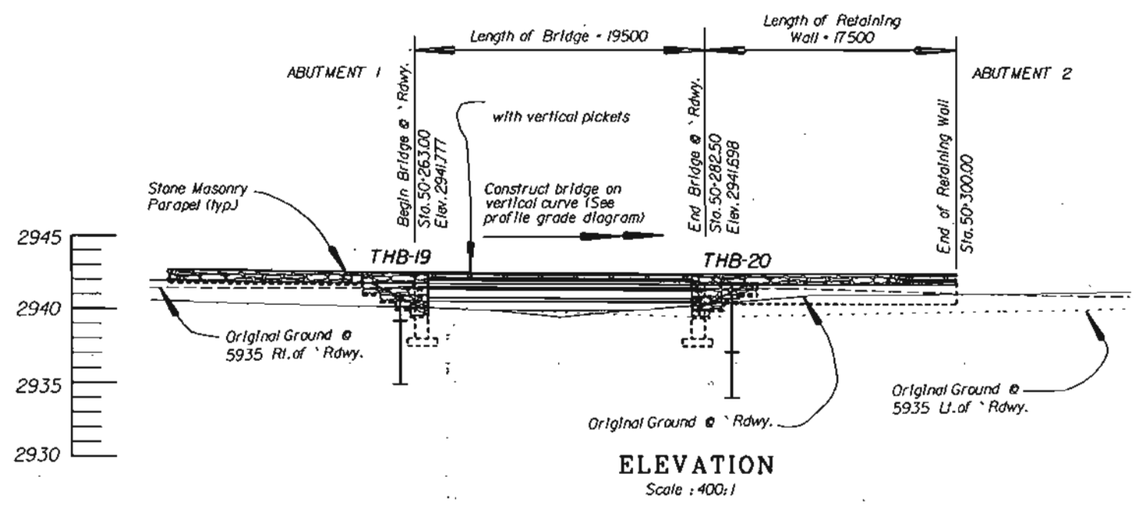
Abutment walls and retaining walls which can be expected to deflect sufficiently to mobilize the full active earth pressure condition should be designed for a lateral earth pressure computed on the basis of an equivalent fluid unit weight of 5.9 kN/m^3. This pressure assumes a free draining backfill. Retaining structures should be designed for appropriate surcharge pressures such as traffic, construction equipment and upward sloping backfill.

8.3.2 Other Considerations

Further Investigation. No further investigation is recommended unless the structure is significantly modified or moved as the design is finalized.

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/01	FH 4-1(0) BEARTOOTH HIGHWAY	8-1	
LONG LAKE OUTLET BRIDGE PLAN AND ELEVATION				





BRTH8-1.DGN

NOTES:

1. Summary test hole logs are shown on Figure 8-2.
2. See Figure 8-2 for legend and notes.
3. Intermediate ticks on test hole stick logs indicate interpreted top of bedrock.
4. Stationing and alignment on this figure are different than primary stationing and alignment shown on Figure 8-2.

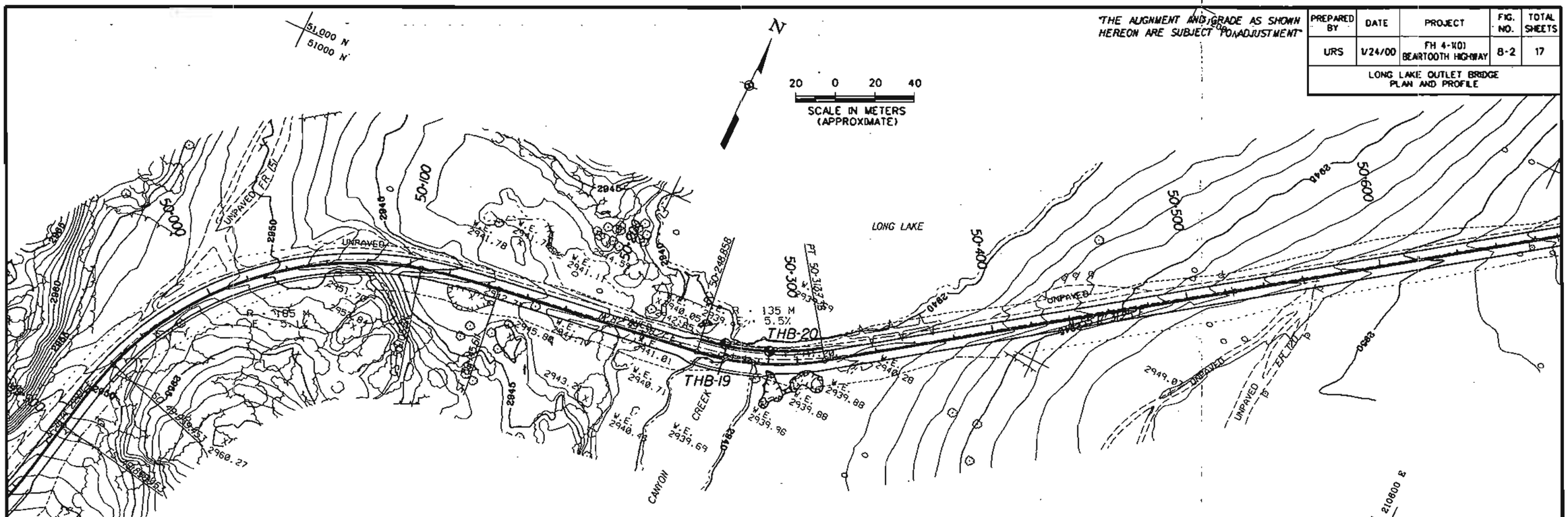
U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION, DENVER, COLORADO

PRELIMINARY LAYOUT

BRIDGE NAME Long Lake Outlet Bridge STREAM NAME Little Bear Creek
ROUTE NAME AND NUMBER Beartooth Highway, WY FS 4-1(0)
PREPARED BY FHWA (Base Drawing Only)
SCALE: As Shown DATE: October 16, 2000 (includes wall)

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/00	FH 4-1(0) BEARTOOTH HIGHWAY	8-2	17
LONG LAKE OUTLET BRIDGE PLAN AND PROFILE				

"THE ALIGNMENT AND GRADE AS SHOWN HEREON ARE SUBJECT TO ADJUSTMENT"

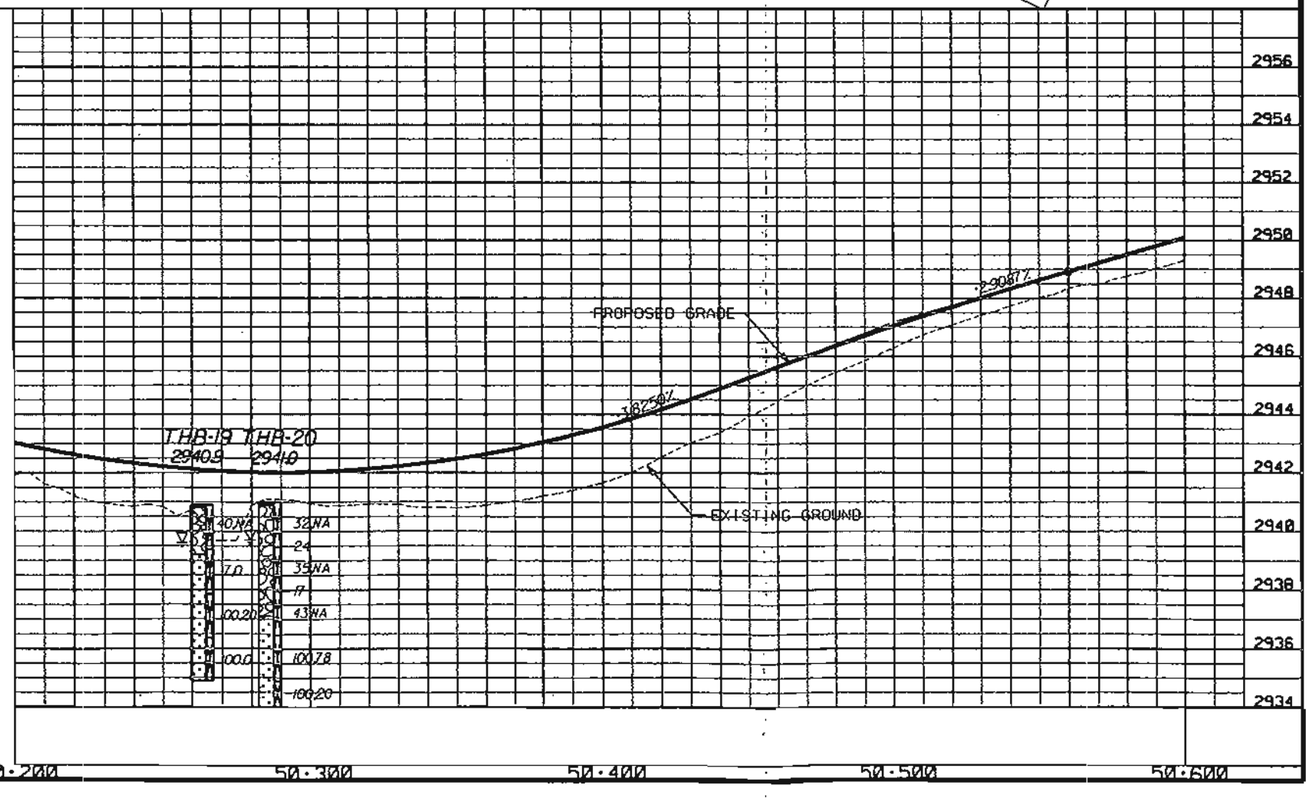


LEGEND FOR LONG LAKE OUTLET BRIDGE

- ASPHALT AND BASE COARSE PAVEMENT SECTION
- GRAVEL, COBBLE, AND BOULDERS, MEDIUM DENSE, (BASED ON BLOWCOUNTS), BROWN, SANDY TO GRAVELLY MATRIX.
- GRANITE, MODERATELY STRONG TO VERY STRONG, PINK, BLACK TO PINK, COARSE GRAINED, SLIGHTLY WEATHERED
- THB-19 ⊗ TEST HOLE LOCATION
- — 12 INDICATES TOP OF INTERVAL FOR STANDARD PENETRATION TEST (SPT). INDICATES THE BLOWCOUNT (N-VALUE) FOR THE FINAL 300 MM OF 450 MM TOTAL PENETRATION IS 12. THE SPT CONSISTS OF RECORDING BLOWS OF A 63.5 KG HAMMER FALLING 760 MM TO DRIVE A 50 MM O.D. SPLIT SPOON SAMPLER. WHERE TWO VALUES ARE SHOWN SEPARATED BY A SLASH, REFUSAL WAS MET BEFORE 450 MM PENETRATION. THE FIRST NUMBER IS THE BLOWCOUNT, THE SECOND NUMBER IS THE PENETRATION DEPTH.
- ☒ 90,30 INDICATES SPLIT TRIPLE-TUBE CORE BARREL WAS USED TO ADVANCE HOLE. 90,30 INDICATES 90% RECOVERY FOR INTERVAL INDICATED AND A ROCK QUALITY DESIGNATION (RQD) OF 30. RQD IS 100 TIMES THE TOTAL LENGTH OF SOUND CORE OVER 100 MM (40 IN.) LONG DIVIDED BY THE LENGTH OF RUN INDICATED. WHERE SPT INTERVALS PRECEDE CORE INTERVALS, THE SPT INTERVAL IS SUBSEQUENTLY OVER-CORED. RECOVERY AND RUN ARE BASED ON A CORE RUN LENGTH THAT INCLUDES THE OVER-CORED LENGTH.
- NA NOT APPLICABLE
- ▽ INDICATES WATER LEVEL AT TIME OF DRILLING

NOTES:

1. TEST HOLES WERE DRILLED ON 8/17/00 THROUGH 9/15/00 BY HAZTECH DRILLING, INC., USING A BK-81 (TRUCK MOUNT) RIG AND AN HQ SPLIT TRIPLE-TUBE CORE BARREL.
2. BASE MAPS OBTAINED FROM DESIGN FIGURES PROVIDED BY THE FHWA.
3. TEST HOLE LOCATIONS ARE APPROXIMATE AND WERE LOCATED BY TAPE MEASUREMENT ON OR BESIDE THE EXISTING ROAD AT DESIGNATED STATIONS. MEASUREMENTS WERE REFERENCED TO MARKINGS AND STAKES PLACED ALONG THE EXISTING ROAD BY THE FHWA.
4. TEST HOLE LOGS IN THIS REPORT ARE SUBJECT TO LIMITATIONS, EXPLANATIONS, AND CONCLUSIONS OF THIS REPORT. SEE INDIVIDUAL LOGS IN APPENDICES FOR MATERIAL DESCRIPTIONS AT EACH TEST HOLE.



BRTHB-2.DGN

9.1 INVESTIGATION

Investigation consisted of surface mapping of the rock outcrop. Face maps were prepared on photographs for significant cut areas. The rock and discontinuity characteristics are described on each map. Maps are provided in Appendix G. Additionally, line surveys were conducted at three areas.

9.1.1 Site Characteristics

This 1.6 kilometer long stretch of road is above timberline and traverses a slope which gently slopes to the west and south. Granitic bedrock, which appears to have been scoured by glaciers, outcrops above and below the road (Figure 9-1). The existing road has several cut and fill sections of low to moderate height, and several small radius curves within the bedrock outcrop. Most of the rock is fairly massive granite, with joint spacing of 0.5 to 3.0 meters. The planned improvements will create some significantly higher cuts, up to approximately 14 meters high (Stations 53+460, 53+480, 53+880, 54+380, and 54+440). Because of its location, the site is visible from many areas and from far away.

9.1.2 Rock Slope Characteristics

Face maps of existing rock slopes between stations 53+485 and 54+720 were prepared to provide information at proposed rock cut slopes. The maps were prepared using photographs of the existing rock slopes to locate and describe discontinuities (joint sets) in the rock mass. Results of the mapping are presented on Figures G-4 to G-14, included in Appendix G. Descriptions of the joint sets are summarized on the figures, including joint set orientation, width, infilling type and amount, surface shape, roughness, spacing, an RQD estimation, and estimated rock strength.

Results of the face mapping indicate many of the joint sets are oriented nearly vertically and dip into the slopes. The rock mass in the existing rock cut slopes and natural slopes typically consists of moderately to widely fractured strong to extremely strong rock.

Additionally, line surveys were conducted at three locations along the proposed cut. The results are presented in Table G-2, in Appendix G. In general, three to five joints sets were identified in each of the line surveys and there were multiple occurrences of joints in each set. Joints identified as part of a set generally have strikes and dips within about 10° to 20° of other joints in the set. Many joints dip very steeply, 70° to 90°, and some dip approximately 40° to 50°. There were few joints that appear to be random, and not associated with a joint set.

Multiple other fractures were observed in the line surveys. These fractures were generally fresh, randomly oriented, and were not persistent; they terminated at other joints. These fractures are believed to be a result of the blasting and excavation of the existing rock slope.

9.2 INTERPRETATION AND ANALYSIS

An analysis of the joint set data shown on the photo maps and Table G-1 (Appendix G) was conducted to interpret possible modes of failure for the rock mass to be exposed in the proposed rock cut slopes. The analysis included a survey of wedges that might form, bounded by joints in

the rock mass, that could slide out of the proposed cut slopes. Stereographic projections and a computer program called SWEDGE were used to conduct these kinematic analyses. The analyses included blocks of rock that could slide on 1 or 2 joint surfaces and separate from the rock mass along a hypothetical nearby vertical tension crack located behind a cut slope face. Results of the wedge analysis are summarized in Table G-3.

Not all discontinuities have been mapped or analyzed. The nature of the mapping conducted is such that the most prominent features were identified. These features have a dominating influence on the existing rock slope and it is believed they will have a similar influence on new rock slopes at the site. Discontinuities of differing characteristics and orientation will likely be encountered during excavation and these discontinuities may form failure wedges or modes that have not been identified herein. For these reasons, rock slope design should be based on the prominent features identified here and should have the flexibility to be adapted to additional modes of failure that may be identified during construction. The contractor should expect the identified discontinuities to have a large impact on the excavation. However, the contractor should also expect to encounter some discontinuities and failure modes not identified here.

Results of the wedge analysis indicate unstable blocks of rock do not exist at 10 of 14 locations of proposed rock cut slopes. The other 4 locations have what are interpreted to be stable wedges because the wedges would have to slide into the slope or slide out of the slope along a surface dipping only 20 to 25°, and this is considerably less than the friction angle for these joints. The friction angle is estimated to be 40°, based on joint descriptions.

The rock slope mapping was also used to evaluate the rock mass as a whole. Estimates of RQD and unconfined compressive strength were used with discontinuity data to evaluate RMR for the slope. The RMR and rock mass quality is sufficiently high that slope stability will be governed by discontinuities, not overall rock mass quality.

9.3 RECOMMENDATIONS

Rock slope recommendations are based on observations of existing slopes and the analyses presented in Section 4.2 and Appendix G. We recommend that slopes be cut to a ratio of 4:1 (vertical:horizontal) using controlled blasting methods. Existing discontinuities should be used as much as practical to form the back slope and limits of the cuts. Scaling and spot bolting should be performed as the cut is made and, as needed, when the cut is completed. Scaling by hand methods using bars and occasional light blasting is expected to be sufficient. Bolting of blocks should be used in lieu of scaling when block removal is judged to be undesirable because scaling may destabilize other blocks, or for aesthetic reasons. It is estimated that very few spot bolts will be required and bolts will be approximately 4 meters in length.

A rockfall collection ditch should be provided to catch and store as much of the rockfall as practical. Where practical, the Ritchie criteria should be followed, or design should be based on a criteria related to probability of rockfall retention. For reference, the Ritchie criteria, as presented in Federal Lands Highway, Project Development and Design Manual, 1996, for a ditch beyond the useable shoulder width, and slopes of 4:1 (vertical:horizontal) is as follows:

Slope Height (m)	Ditch Width (m)	Ditch Depth (m)
4.5 to 9.0	2.4	0.9
9.0 to 18.0	3.6	1.2
18 to 30	5.5	1.8

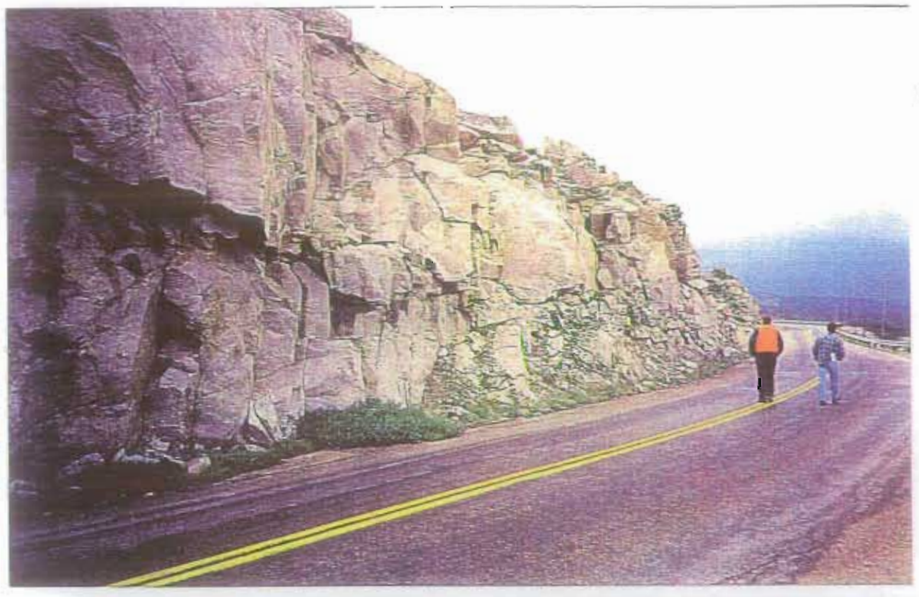
If the Ritchie criteria are not attainable, we recommend a minimum collection ditch width of 2 meters.

Other Considerations

Construction Observations. Excavation of rock slopes should be observed by a qualified geotechnical engineer. The engineer should identify joint-bounded blocks and wedges exposed by the excavation and should make recommendations, as needed, for removal or bolting of blocks and wedges. No more than very occasional spot bolting is expected to be required at Site 3 if controlled blasting techniques are used. Nevertheless, provisions for bolting are recommended to be included in the contract.

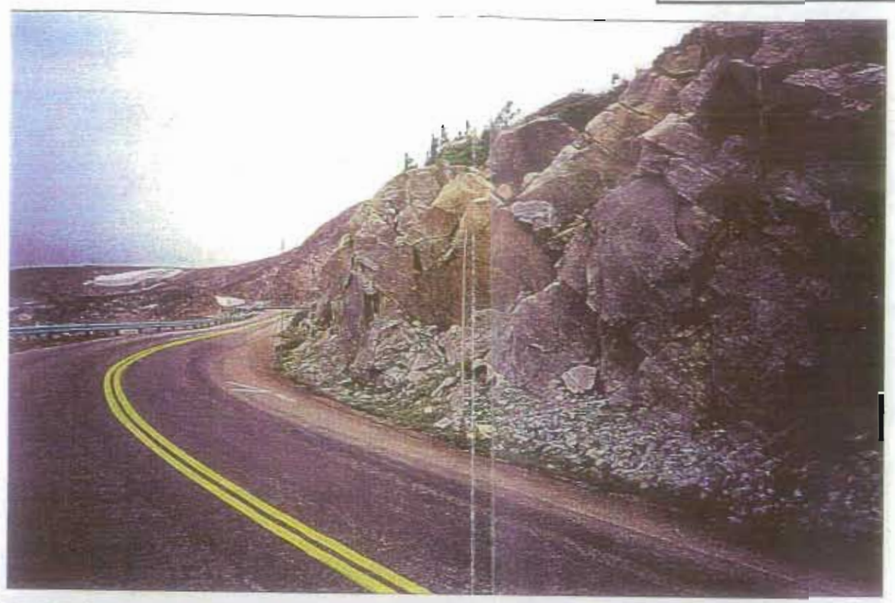
Further Investigation. No further investigation is recommended for Site 3.

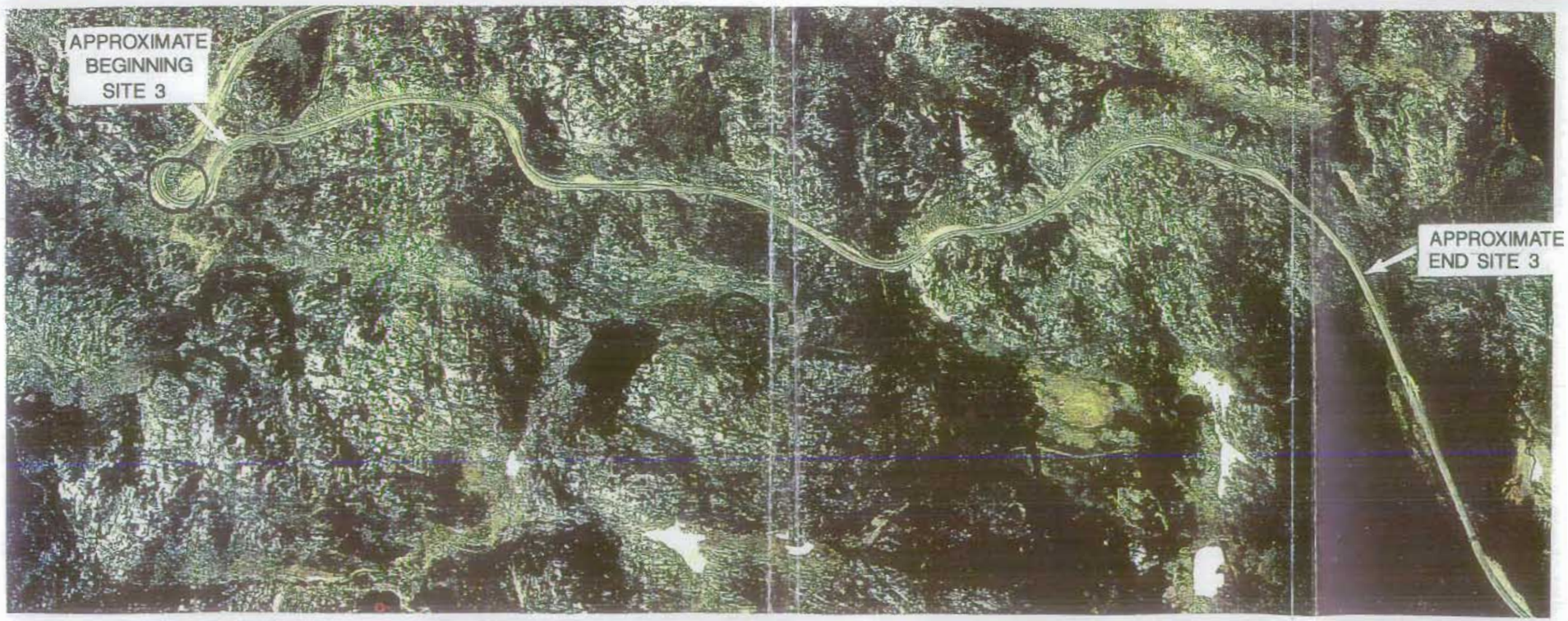
PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/8/01	FH 4-1(10) BEARTOOTH HIGHWAY	9-1	
SITE 3 STATION 53+200 TO 54+800				



(a) TYPICAL OUTCROP LOOKING BACK ON LINE

(b) TYPICAL OUTCROP LOOKING AHEAD ON LINE





(c) AERIAL VIEW

BRTH-TB.DGN

10.1 INVESTIGATION

Three (3) test holes were drilled using H-size core barrels to investigate conditions below the road and the foundation for the planned retaining wall. Test holes TH4-1, TH4-2, and TH4-3 were drilled near the outside edge of the existing road. SPT samples were taken every 1.5 meters except in bedrock or if the run ended in a boulder. The test hole locations and results are summarized in Table 10-1. The test hole locations are shown on Figure 10-1, summary logs are shown on Figure 10-2, and individual logs and photos are provided in Appendix B. A seismic refraction survey was conducted at the location shown on Figure 10-1. Limited bedrock outcrop mapping was also performed for the slope above the existing roadway at this site and is shown on Figure 10-1.

Table 10-1

TEST HOLE SUMMARY, SITE 4 WALL

Test Hole Number	Stationing	Offset (m)	Elevation (m)	Depth to Bedrock (m)	Depth to Water(1) (m)	Test Hole Depth (m)
TH4-1	56+970	0	3232.9	3.1	N.E.	6.9
TH4-2	57+095	1 R	3239.5	5.5	N.E.	8.7
TH4-3	57+040	1.7 R	3236.6	5.4	N.E.	9.0

Notes: (1) Encountered in test hole at time of drilling. N.E. = Not Encountered

10.1.1 Site Characteristics

The road climbs above a broad plateau with three hairpin turns, Site 4 is located near the lowest turn where the upper and lower roads are closest together, as shown on Figure 10-1. These switchbacks are highly visible from below. This is also a heavy snowdrift area and guardrail is not desired. The current plans for the site (dated July 2000) show a fill-side Mechanically Stabilized Earth (MSE) Wall. The wall has a free-face height of up to about 5 meters.

10.1.2 Subsurface Characteristics

This section summarizes the investigation results for the foundation for the proposed retaining wall, and the in situ materials to be retained by the wall. The individual test hole logs are presented in Appendix B and the laboratory test results are presented in Appendix D.

TH4-1, TH4-2, TH4-3 were drilled from the existing road, as far towards the outboard (right) shoulder as possible. Slightly to highly weathered granite and gneiss bedrock was encountered approximately 4 to 5 meters below the road surface. The bedrock was cored with an H-size core barrel and recovery ranged from 90 to 100 percent. The rock quality designation (RQD) varies from 0 to 56. Groundwater was not encountered and, based on the topography, it is anticipated that groundwater is below the planned wall elevation. It is anticipated that some water could be encountered at the bedrock contact. The unconfined compressive strength of bedrock estimated from point load testing varies from 26 to 199 MPa. Laboratory test results are summarized in Table 10-2.

Where test holes were drilled, the bedrock elevation is generally 3230 to 3234, which is generally just above or below the planned wall foundation elevation. At Station 56+980, where the bedrock was encountered at an elevation approximately 2 meters above the wall foundation, the RQD for the bedrock at this location is 0, which indicates that the bedrock is highly fractured and could likely be removed with earth moving equipment. In addition, given the slope of the bedrock contact, the bedrock is expected to be several meters below the planned foundation of the wall. The seismic refraction results (discussed below and shown on Figures 10-1 and 10-2) also indicate that the bedrock contact is several meters below the excavation limits for construction of the wall.

The overburden is talus and colluvium with particle sizes from sand through boulders. The longest length of boulder cored was up to 0.2 meters. Standard Penetration Tests (SPT's) were generally conducted at the end of each 1.5 meter core run. N-values range from 5 to 59. The considerable presence of gravel, cobbles, and boulders has probably impacted some SPT results, especially the higher values. The presence of some relatively low N-values indicates there are intervals of loose to medium dense soils, or possibly voids. The summary logs show recovery for core runs in the overburden, as well as the SPT results. Core intervals include overcoring (reaming) the interval of the hole sampled with the SPT split spoon sampler. Despite coring through this disturbed material, recovery (based on the full length of the run) varied generally from 25 to 43.

Table 10-2

SUMMARY OF LABORATORY TEST RESULTS
SITE 4 WALL

Test Hole	Depth (m)	Classification	UCS[1][2] (MPa)
TH4-1	5.50	Granite	87
TH4-2	7.10	Granite/Gneiss	199
TH4-2	7.90	Granite/Gneiss	93
TH4-3	7.70	Granite	26
TH4-3	8.90	Granite	99

[1] Values determined from point load testing of core (ASTM D5731), as described on data sheets in Appendix D.
[2] UCS = Unconfined Compressive Strength (ASTM D2938), as described on data sheets in Appendix D.

Seismic Refraction

The seismic refraction method was used to help characterize the subsurface materials, and specifically to help identify the bedrock surface, obtain seismic velocity information to facilitate estimation of subsurface excavatability, and provide subsurface information to augment findings obtained from drilling activities. A description of the seismic refraction survey is provided in Appendix F.

Three surface seismic lines (totaling approximately 300 linear meters) were surveyed between stations 56+920 and 57+219. The final location of each seismic line was chosen based on the survey objectives, and considered the site conditions and constraints. Final line locations are shown on Figure 10-1.

Seismic refraction data were collected using a signal-enhancing Geometrics Strataview seismograph and geophone lines. The geophone spacing ranged from 4 to 5 meters for spread lengths of 92 m and 115 m, respectively. A sledgehammer impacting an aluminum plate was generally used as the seismic source. At several locations, seismic energy was produced by using a small explosive charge, consisting of approximately 1/4 to 1/2 pound of Kinepak initiated by electric blasting cap to improve the data quality. Although the sledgehammer source appeared to be sufficient for most of the data sets, some of the hammer data were noisy, and of moderate quality. Data quality was significantly improved when using the explosive charges. Seismic sources were usually used at both ends of each spread for forward and reverse travel times and in the middle of the line to increase near-surface velocity control. Additional offset sources were also used to obtain better coverage.

Final interpretations are displayed for the seismic lines in cross-section form in Figures F-2 through F-5 of Appendix F, and a summary table of the seismic refraction results is shown in Table F-1. The depth and velocity sections in the figures are plotted in terms of seismic layers, which represent the interface(s) at which a distinct velocity contrast is interpreted to exist. These seismic layer interfaces often relate to geologic boundaries, depending on the seismic velocity contrast observed at the interface or boundary.

A two-layer model is representative of the subsurface along each seismic line. The near-surface material has a seismic velocity that varies from 370 to 740 m/s, and has a thickness of 4 to 12 meters. Based on correlation with drilling information, seismic "layer 1" likely consists of soil and colluvial materials, boulders and weathered bedrock.

The second seismic layer has a seismic velocity of 3,820 m/s to 4,570 m/s. Based on the seismic velocities and drilling information, seismic "layer 2" likely consists of slightly weathered to unweathered bedrock materials.

10.2 INTERPRETATION AND ANALYSIS

Observations and data on soil and bedrock have been interpreted and analyzed to develop geotechnical recommendations. At this site the soil overburden has been evaluated for MSE wall foundation, temporary excavation slopes, and lateral earth pressure.

The material overlying the bedrock is talus and colluvium. Test holes and observations indicate a wide range of particle sizes is present, from sand through large boulders. Due to the nature of the deposit (talus), we expect the finer material to generally fill the voids between the coarser materials. Some voids between larger materials may be filled very loosely, or only partially. Boulders and loose soil may cause difficulty in excavation and may locally present hazardous conditions in excavation slopes. The boulders and cobbles are granitic, being derived from the outcrop near the road, and probably have unconfined compressive strength similar to or less than the bedrock cores tested in this area.

The bedrock quality has been evaluated based on the unconfined compressive strength of intact rock, the Rock Quality Designation (RQD), and the Rock Mass Rating (RMR). Table 10-2 presents the interpretation of bedrock quality and estimated ranges of engineering parameters based on these methods. In general, these characterizations show that the bedrock quality is very poor to fair.

The rock mass Young's modulus (E_m), and the cohesive (c) and frictional (ϕ) strength parameters, are based on published correlations to the RMR. They are presented here to assist in characterizing the relative quality of the rock mass. These correlated values are part of the information used to develop the geotechnical recommendations for design presented in Section 10.3.

Table 10-3

INTERPRETATION OF BEDROCK QUALITY
SITE 4 WALL

Location	Characteristic RQD	Characteristic Rock Quality[1]	RMR[2] Range	Characteristic RMR Class[2]	E_m (GPa)[3]	c (kPa)[2]	ϕ (deg)[2]
56+800 to 57+200	20	Very Poor	47-70	III (Fair)	8-32	250-350	30-40

Notes:

[1] Based on RQD, Deere (1964). Classification for RQD <25 = very poor, 25 < 50 = poor, 50 < 75 = fair, 75 < 90 = good, 90 < 100 = excellent

[2] Rock Mass Rating, Bieniawski (1989), classification for RMR < 20 = V (very poor), 21 to 40 = IV (poor), 41 to 60 = III (fair), 61 to 80 = II (good), 81 to 100 = I (very good). Cohesion and friction ranges are based on RMR class. For Class V: ϕ < 15°, c < 100 kPa; for Class IV: ϕ = 15-25°, c = 100-200 kPa; for Class III: ϕ = 25-35°, c = 200-300 kPa, for Class II: ϕ = 35-45°, c = 300-400 kPa; for Class I: ϕ > 45, c > 400 kPa.

[3] Young's Modulus of Rock Mass based on RMR, $E_m = 10^{(RMR-10)/40}$, Serafin and Pereira (1983)

10.3 RECOMMENDATIONS

The proposed embankment wall at this site is about 400 meters in length. The proposed excavation depth is approximately 6 meters below the existing roadway grade and the maximum height of the proposed retaining wall is 5.5 meters. Based on the test holes, the excavated material will consist primarily of gravel with some sand, cobbles, and boulders, and fractured granitic bedrock.

Water was not encountered during drilling and was not evident in the seismic refraction survey.

Materials in which the embankment wall will be excavated and founded upon include road fill and underlying colluvial materials consisting primarily of gravel with some sand, cobbles and boulders, and fractured bedrock. Recommended soil parameters were developed from site observations, results of subsurface investigations, laboratory test results, published values and correlations, and our experience with similar materials. Recommendations are provided for the retained soil located behind the wall (indicated by subscript s), the reinforced soil comprising the wall (indicated by subscript r), and the foundation soil beneath the wall (indicated by subscript f), as shown in Table 10-4.

Table 10-4

RECOMMENDED DESIGN PARAMETERS FOR MSE WALL

Retained Soil		Foundation Soil		Reinforced Soil	
γ_s (kN/m^3)	ϕ'_s (deg)	γ_f (kN/m^3)	ϕ'_f (deg)	γ_s (kN/m^3)	ϕ'_r (deg)
20.4	34	20.4	34	19.6	34

The following recommendations pertain to the MSE wall:

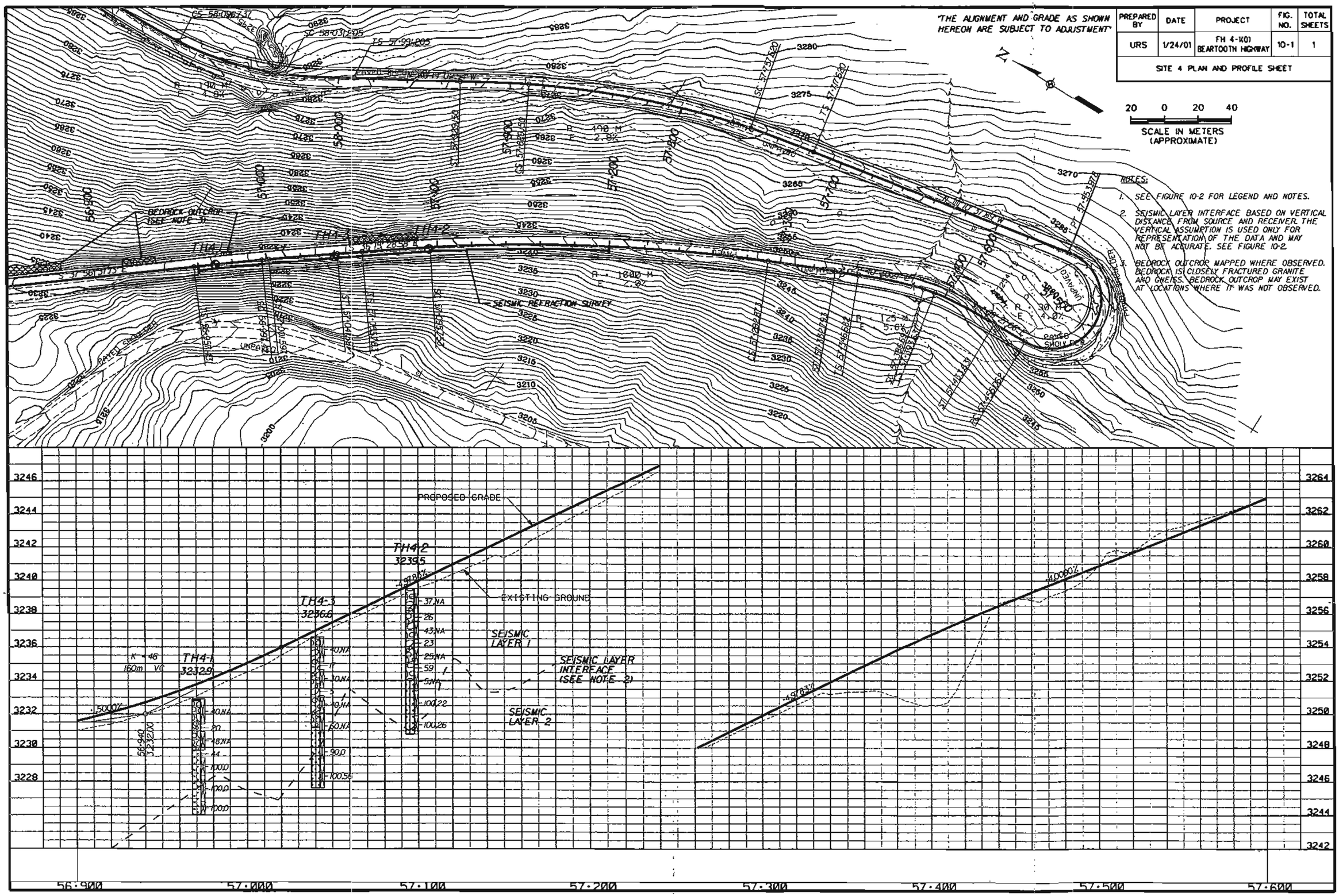
1. Wall design should follow the procedures in Mechanically Stabilized Earth Walls and Reinforced Soil Slopes: Design and Construction Guidelines (FHWA Publication SA-96-071, Oct. 1996) and meet the requirements in the current edition of AASHTO Standard Specifications for Highway Bridges).
2. A maximum temporary excavation slope ratio of 1:1 (vertical:horizontal) is recommended for unsaturated excavation slopes of up to 4 meters height, in overburden soils. A maximum temporary slope ratio of 1:1.5 (vertical:horizontal) is recommended for higher slopes. The slopes may be locally steeper (and flatter) because of boulder inclusions. Precarious boulders and cobbles on the excavation slope should be removed. Large boulders should be split so that depressions (and local oversteepening) are not caused by removal of the entire boulder. Shallow sloughing along the excavated cut slope should be expected and controlled to prevent hazards to construction workers and highway traffic. Recommended measures for controlling sloughing include the following:
   - Loosened boulders and other loose materials should be removed from the slopes. The slopes should be observed routinely so that these materials can be quickly identified and removed.
   - Temporary drainage ditches should be constructed at the top of the slope to minimize runoff down the slopes. Runoff down the slope should not be allowed.
   - The contract specifications should limit the amount of time a temporary excavation slope is left open. A maximum of 30 days is recommended. No more than 50 meters of continuous excavation should be open at one time.
   - If signs of slope instabilities still persist after these measures have been implemented, the slope should be excavated to a flatter, more stable inclination.
3. Temporary slopes excavated in bedrock should be no steeper than 1.33:1.0 (vertical:horizontal). Based on the test hole explorations, bedrock in the excavation area is expected to be jointed. Stabilization of potentially unstable rock wedges, which may daylight into the excavation may be required.
4. Temporary shoring using soil nails, a pile wall, or other means of face support may be used to develop steeper excavation slopes.
5. Select granular fill should be used for backfill in the reinforced portion of the wall. This material should conform to material and handling requirements for structural fill presented in Section 255 and Subsection 704.10 of the Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, FP-96, dated 1996. On-site material from the roadway excavations may not meet material requirements for granular fill (Subsection 704.10, FP-96) without screening and/or crushing. Granular fill may be obtained from other borrow sources to meet material requirements.

6. Loose zones of colluvium and talus, and voids may be encountered at subgrade level. Loose zones should be compacted with a vibratory compactor. Voids should be filled with clean granular material prior to wall construction. An allowable bearing capacity of 0.25 MPa is recommended for design.
7. Global stability evaluations for the wall should consider the slope below the wall in analysis.

A reinforced slope may be used to reduce the size or eliminate the planned embankment wall. A reinforced slope, revegetated on the face may have improved aesthetics, compared to an MSE wall. Reinforced slopes should be no steeper than 1:1 (vertical:horizontal) and should be designed according to FHWA Publication No. FHWA-SA-96-071 (FHWA, 1998). The facing should include wrapped geosynthetics and a permanent or long-lasting erosion control mat, with sufficient growth medium to facilitate establishment of vegetation. Given the high elevation of the wall, and the severity of the climate, special design features may be required to vegetate and stabilize the slope face.

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/01	FH 4-1(0) BEARTOOTH HIGHWAY	10-1	1
SITE 4 PLAN AND PROFILE SHEET				

"THE ALIGNMENT AND GRADE AS SHOWN HEREON ARE SUBJECT TO ADJUSTMENT"



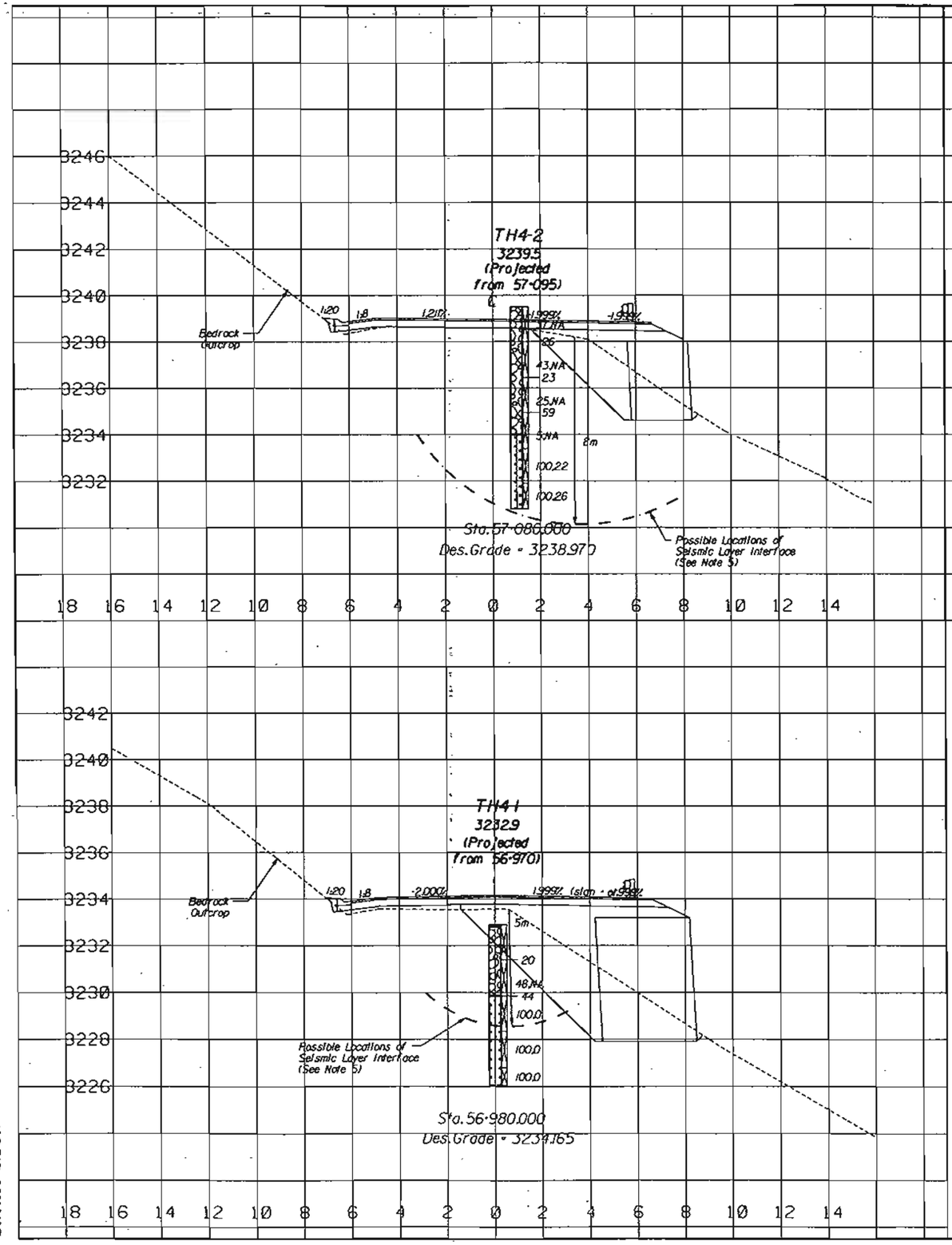
NOTES:

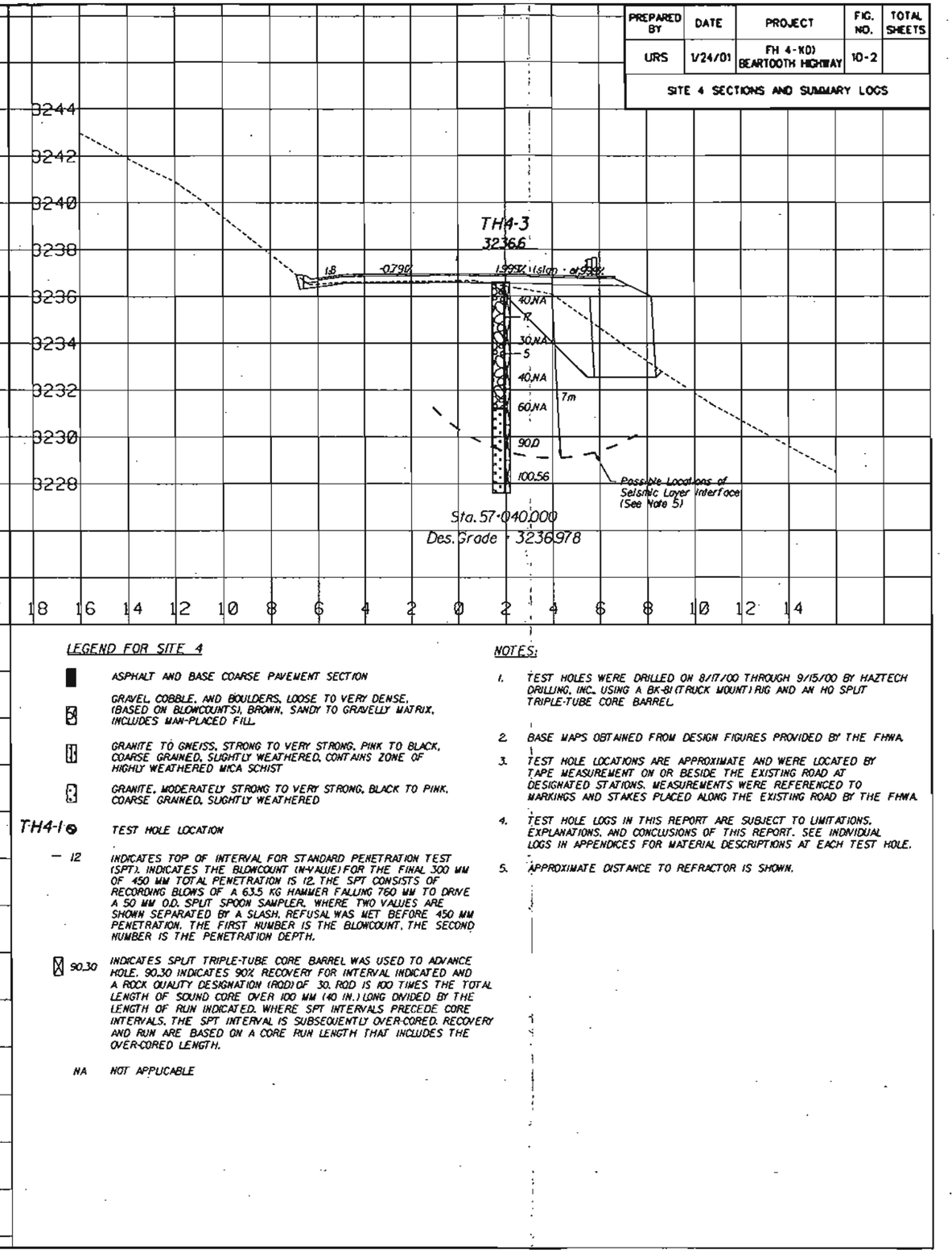
1. SEE FIGURE 10-2 FOR LEGEND AND NOTES.
2. SEISMIC LAYER INTERFACE BASED ON VERTICAL DISTANCE FROM SOURCE AND RECEIVER. THE VERTICAL ASSUMPTION IS USED ONLY FOR REPRESENTATION OF THE DATA AND MAY NOT BE ACCURATE. SEE FIGURE 10-2.
3. BEDROCK OUTCROP MAPPED WHERE OBSERVED. BEDROCK IS CLOSELY FRACTURED GRANITE AND GNEISS. BEDROCK OUTCROP MAY EXIST AT LOCATIONS WHERE IT WAS NOT OBSERVED.

BRTH10-1.DGN

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/01	FH 4-10) BEARTOOTH HIGHWAY	10-2	

SITE 4 SECTIONS AND SUMMARY LOGS





LEGEND FOR SITE 4

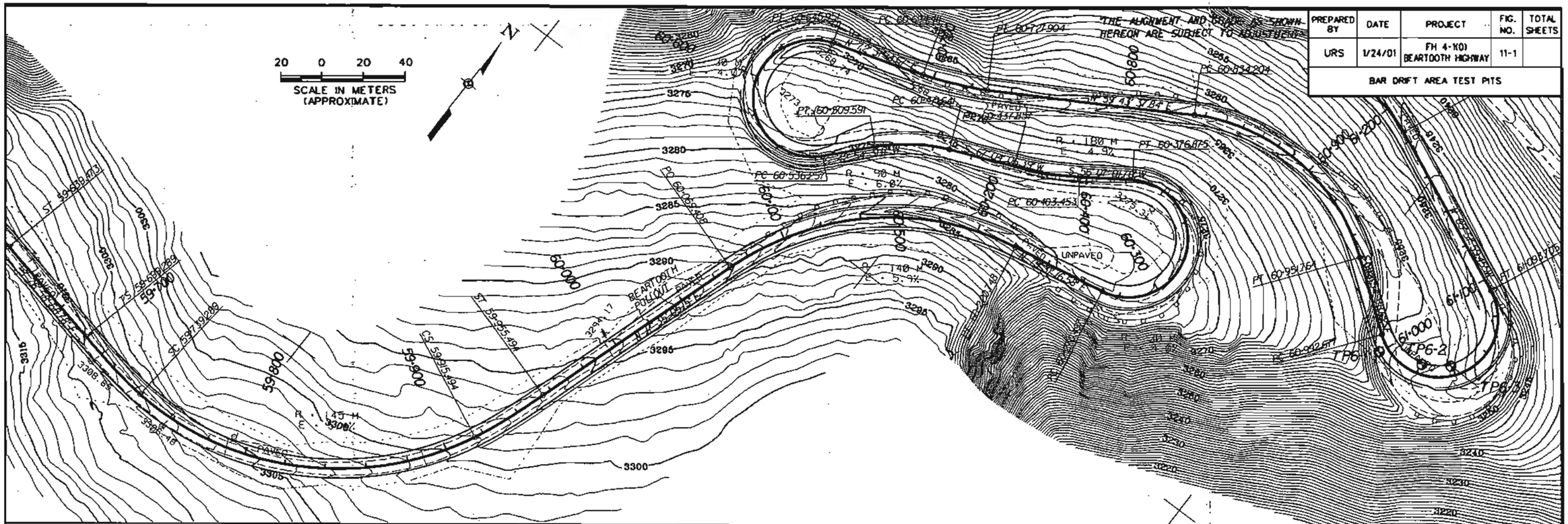
- ASPHALT AND BASE COARSE PAVEMENT SECTION
- GRAVEL, COBBLE, AND BOULDERS, LOOSE TO VERY DENSE, (BASED ON BLOWCOUNTS), BROWN, SANDY TO GRAVELLY MATRIX, INCLUDES MAN-PLACED FILL.
- GRANITE TO GNEISS, STRONG TO VERY STRONG, PINK TO BLACK, COARSE GRAINED, SLIGHTLY WEATHERED, CONTAINS ZONE OF HIGHLY WEATHERED MICA SCHIST
- GRANITE, MODERATELY STRONG TO VERY STRONG, BLACK TO PINK, COARSE GRAINED, SLIGHTLY WEATHERED
- TH4-1 ⊕ TEST HOLE LOCATION
- — 12 INDICATES TOP OF INTERVAL FOR STANDARD PENETRATION TEST (SPT). INDICATES THE BLOWCOUNT (N-VALUE) FOR THE FINAL 300 MM OF 450 MM TOTAL PENETRATION IS 12. THE SPT CONSISTS OF RECORDING BLOWS OF A 63.5 KG HAMMER FALLING 760 MM TO DRIVE A 50 MM O.D. SPLIT SPOON SAMPLER. WHERE TWO VALUES ARE SHOWN SEPARATED BY A SLASH, REFUSAL WAS MET BEFORE 450 MM PENETRATION. THE FIRST NUMBER IS THE BLOWCOUNT, THE SECOND NUMBER IS THE PENETRATION DEPTH.
- ⊠ 90.30 INDICATES SPLIT TRIPLE-TUBE CORE BARREL WAS USED TO ADVANCE HOLE. 90.30 INDICATES 90% RECOVERY FOR INTERVAL INDICATED AND A ROCK QUALITY DESIGNATION (RQD) OF 30. RQD IS 100 TIMES THE TOTAL LENGTH OF SOUND CORE OVER 100 MM (40 IN.) LONG DIVIDED BY THE LENGTH OF RUN INDICATED. WHERE SPT INTERVALS PRECEDE CORE INTERVALS, THE SPT INTERVAL IS SUBSEQUENTLY OVER-CORED. RECOVERY AND RUN ARE BASED ON A CORE RUN LENGTH THAT INCLUDES THE OVER-CORED LENGTH.
- NA NOT APPLICABLE

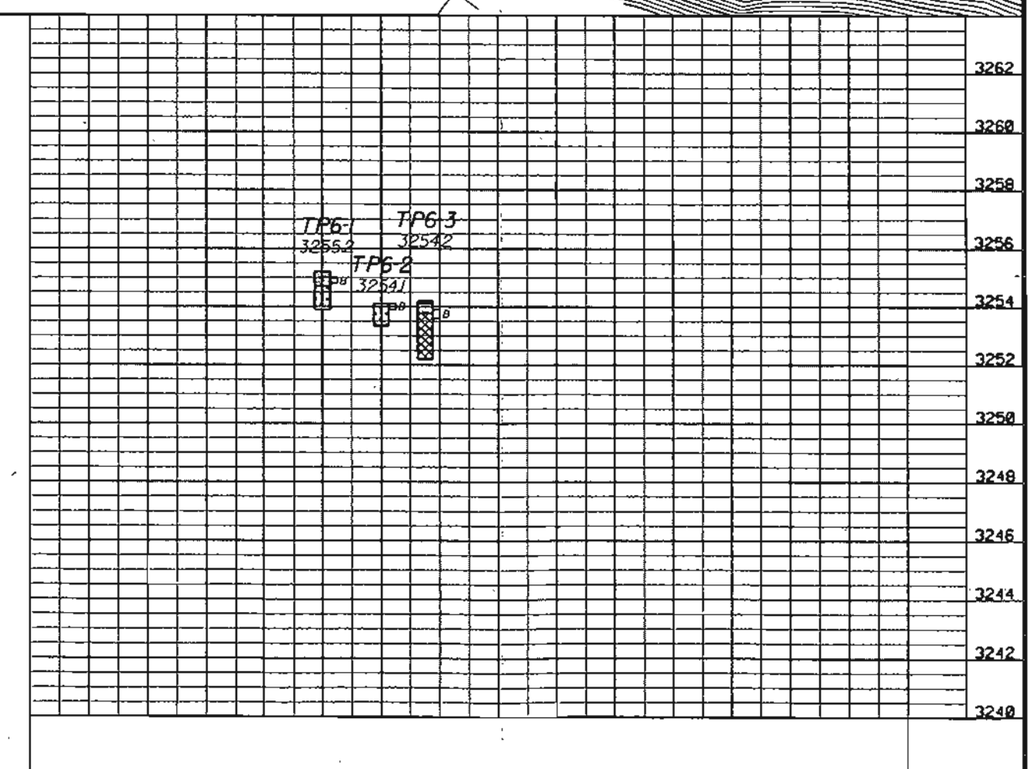
NOTES:

1. TEST HOLES WERE DRILLED ON 8/17/00 THROUGH 9/15/00 BY HAZTECH DRILLING, INC., USING A BK-81 (TRUCK MOUNT) RIG AND AN HQ SPLIT TRIPLE-TUBE CORE BARREL.
2. BASE MAPS OBTAINED FROM DESIGN FIGURES PROVIDED BY THE FHWA.
3. TEST HOLE LOCATIONS ARE APPROXIMATE AND WERE LOCATED BY TAPE MEASUREMENT ON OR BESIDE THE EXISTING ROAD AT DESIGNATED STATIONS. MEASUREMENTS WERE REFERENCED TO MARKINGS AND STAKES PLACED ALONG THE EXISTING ROAD BY THE FHWA.
4. TEST HOLE LOGS IN THIS REPORT ARE SUBJECT TO LIMITATIONS, EXPLANATIONS, AND CONCLUSIONS OF THIS REPORT. SEE INDIVIDUAL LOGS IN APPENDICES FOR MATERIAL DESCRIPTIONS AT EACH TEST HOLE.
5. APPROXIMATE DISTANCE TO REFRACTOR IS SHOWN.

BRTH10-2.DGN

PREPARED BY	DATE	PROJECT	FIG. NO.	TOTAL SHEETS
URS	1/24/01	FH 4-1(0) BEARTOOTH HIGHWAY	11-1	
BAR DRIFT AREA TEST PITS				





LEGEND FOR BAR DRIFT AREA TEST PITS

- ASPHALT AND BASE COARSE PAVEMENT SECTION
- SAND SM, GRAVELLY, SILTY, YELLOW-BROWN, INCLUDES FILL
- MAN-PLACED FILL, GRAVEL, COBBLE, AND BOULDERS, SANDY MATRIX
- GRANITE, STRONG, IRON OXIDE STAINED, CLOSE JOINTING, HIGHLY TO VERY HIGHLY WEATHERED
- TP6-1 TEST PIT LOCATION
- B INDICATES BAG SAMPLE

NOTES:

1. TEST PITS WERE EXCAVATED ON 9/15/00 BY DAN ESTES USING A 580K BACKHOE.
2. BASE MAPS OBTAINED FROM DESIGN FIGURES PROVIDED BY THE FHWA.
3. TEST PIT LOCATIONS ARE APPROXIMATE AND WERE LOCATED BY TAPE MEASUREMENT ON OR BESIDE THE EXISTING ROAD AT DESIGNATED STATIONS. MEASUREMENTS WERE REFERENCED TO MARKINGS AND STAKES PLACED ALONG THE EXISTING ROAD BY THE FHWA.
4. TEST PIT LOGS IN THIS REPORT ARE SUBJECT TO LIMITATIONS, EXPLANATIONS, AND CONCLUSIONS OF THIS REPORT. SEE INDIVIDUAL LOGS IN APPENDICES FOR MATERIAL DESCRIPTIONS AT EACH TEST PIT.

BRTH11-1.DGN

11.1 INVESTIGATION

Test pits TP6-1, TP6-2, and TP6-3 were excavated to a depth of approximately 1 to 2 meters in the Bar Drift area, near Station 61+000. The test pits were excavated in the shoulder of the existing road and in a turnout, where remnants of old asphalt pavement are present. The test pit locations and summary logs are shown on Figure 11-1. Individual logs and photos are provided in Appendix C.

11.1.1 Site Characteristics

The site is on a northward sloping ridge at an elevation of over 3200 meters. Vegetation is characterized by sparse alpine flowers and shrubs. The site is known as the Bar Drift area because of a large snow drift which typically forms. This is the last place along the alignment where snow typically melts off in the summer. We understand there has been a history of pavement distress in the area that may be related to frost heave or permafrost.

11.1.2 Subsurface Characteristics

The conditions encountered in TP6-1 consisted of silty, gravelly sand from the ground surface to 0.5 meter depth. Man-placed fill comprised of boulders, cobbles, gravel and sand was encountered beneath the silty, gravelly sand in TP6-3 to the bottom of the pit, at a depth of 2 meters. Strong granite bedrock was encountered beneath the silty, gravelly sand at TP6-1 and from essentially the ground surface at TP6-2. The close joint spacing and weathering permitted excavation of this bedrock with the backhoe. Silty gravelly sand was encountered in TP6-3 to the maximum depth investigation, 0.4 m.

Laboratory testing results are summarized in Table 11-1. Note that testing of the fractured bedrock from TP6-2 indicates the joint spacing was close enough that the sampled material classifies as a gravel (GW-GM).

Table 11-1

SUMMARY OF LABORATORY TEST RESULTS
BAR DRIFT AREA

Test Hole	Depth (m)	Classification[1]	% Gravel[2]	% Sand[3]	% Fines[4]
TP6-1	0.61	GW-GM	49	43	8
TP6-2	0.61	GW-GM	47	46	7
TP6-3	0.61	SM	23	59	18

[1] USCS = Unified Soil Classification System, based on visual observation when grain size data and plasticity data not available. Grain size distribution performed in general accordance with ASTM D422.
Visual classification performed in general accordance with ASTM D 2488.
[2] Percent greater than the No. 4 sieve size.
[3] Percent finer than the No. 4 sieve size and coarser than the No. 200 sieve size.
[4] Percent finer than the No. 200 sieve size. Includes silt and clay sizes.

11.2 INTERPRETATION AND ANALYSIS

The test pits show that a range of soil materials, either fill or colluvial, exist in the subsurface at the site. The silty sand materials are subject to heave during freezing, and may be responsible for the past pavement distress in the area. There was no evidence of water or ice in the soils when the pits were excavated, so it does not appear they are frozen or wet at all times.

The gravel and coarser grained soils appear to be sufficiently free-draining to avoid significant impact by freezing and thawing. However, the bedrock joints may contain sufficient silt and sand infilling to impede the flow of water and cause the bedrock to be impacted by freezing and thawing. Freezing and thawing may have contributed significantly to the weathering of bedrock at this site.

11.3 RECOMMENDATIONS

Good drainage and the use of coarse grained granular fill are recommended to mitigate frost damage. Surface and subsurface drainage should be designed to divert water from direct precipitation, and from melting of snowdrifts, away from the road subgrade. A paved ditch with an underdrain system may be needed beneath the primary Bar Drift area, from approximately Station 60+800 to 61+000.

Silty sand fill should not be used in new embankment construction. Existing silty sand fill should be excavated and replaced with clean sand, gravel and cobbles. Bedrock showing evidence of silt and sand filled joints should also be excavated. The depth of freezing at this site is unknown, and probably exceeds the practical limit of excavation. A minimum 1 meter excavation and replacement depth is recommended for these materials.

Geosynthetics in the pavement section and as a separator in the subgrade can be used to facilitate material compatibility and limit pavement distress from heaving.

12.1 LIMITATIONS

The analyses, interpretations and recommendations included in this report were prepared based on data obtained partly by URS and partly by others. The data obtained by others includes: topographic map, bridge drawings, stationing and seismic tomography, including a GPS survey.

The accuracy of the information provided by or obtained from reports or documents by others, relied on for the preparation of this report, could not be verified. URS assumes no responsibility for the accuracy of the data contained in those reports and documents.

URS warrants that our services are performed within the limits prescribed by our Clients, with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended in our proposals, contracts or reports.

12.2 CREDITS

This report was prepared by Aaron T. Burns, Dale M. Baures, P.E. and Tiffany E. Adams, P.E. under the direction of Scott A. Anderson, P.E., and was peer reviewed by Richard J. Tocher, P.E. Seismic refraction surveys and interpretation were preformed by John J. Nicholl.

A draft of this report was reviewed by Mr. Khamis Haramy, P.E., the FWHA-CFLHD Contracting Officer's Technical Representative; Mr. Gary Strike, P.E., the FHWA-CFLHD Design Manager; Mr. Pat Flynn, the FHWA-CFLHD Bridge Engineer; and Mr. Mark Taylor, the FHWA-CFLHD Project Manager. Their comments have been addressed in this final report.

Bieniawski, Z.T. 1989. Engineering Rock Mass Classifications: a Complete Manual for Engineers and Geologists in Mining, Civil, and Petroleum Engineering

Deere, D.U. 1964. "Technical Description of Rock Cores for Engineering Purposes." Rock Mechanics and Engineering Geology, Vol. 1, No. 1, pp. 17-22.

FHWA. 1996. MSE Walls and Reinforced Soil Slopes: Design and Construction Guidelines. (FHWA Publication SA-96-071)

FHWA. 1996. Federal Lands Highway: Project Development and Design Manual, June 1996 Metric Revision. (FHWA Publication DF-88003)

FHWA. 1996. Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, FP-96.

FHWA. 1998. Wyoming Beartooth Highway, Mile Post 24.4 to Mile Post 43.1, Preliminary Materials Report. (Report 98-16)

FHWA. 1999. Wyoming Beartooth Highway, Materials Source Investigation. (Report 99-14)

FHWA. 2000. Micropile Design and Construction Guidelines Implementation Manual. (FHWA Publication SA-97-070).

Hoek, E., and Bray, J. 1974. Rock Slope Engineering, The Institution of Mining and Metallurgy, London.

Romana, M. 1985. "New Adjustment Ratings for Application of Bieniawski Classification to Slopes." Proc. Int. Symp. Rock Mech. Excav. Min. Civ. Works, ISRM, Mexico City, pp. 59-68.

Serafim, J.L., and Pereira, J.P. 1983. "Considerations of the Geomechanics Classification of Bieniawski." Proc. Int. Symp. Eng. Geol. Underground Constr., A. A. Balkema, Rotterdam, vol. 1, pp. II.33- II.42.

URS. 1998. Initial Geohazards Evaluation and Geological Study: Beartooth Highway, U.S. 212, Wyoming. (WCFS Project No. FHAT0011)

URS. 1999. Initial Geotechnical Investigation: Beartooth Highway, U.S. 212, Wyoming. (URSGWCFS Project No. 68FHAT002200)

Appendix A

Appendix A
Soil Survey

Table A-1
SUMMARY OF SOIL SURVEY

Station to Station		Left Side Observations[1,2]		Right Side Observations[1,2]		Probable Water Problem Area[4]
		Material	Shrink-Swell Factor[3]	Material	Shrink-Swell Factor[3]	
39 + 450	39 + 480	Fill	-5	Fill	-5	No
39 + 480	39 + 600	Cut, granite, gneiss, WF	35	Fill	-5	No
39 + 600	39 + 740	Cut, CL-CH silty, sandy, granite gneiss boulders	-4	Fill	-5	No
39 + 740	39 + 820	Fill	-5	Fill	-5	No
39 + 820	40 + 000	Cut, CL-CH silty, shale & limestone clasts	-4		--	No
40 + 000	40 + 050	Fill	-5	Fill	-5	No
40 + 050	40 + 180	Cut, CL-CH, shale, limestone clasts	-4	Fill	-5	No
40 + 180	40 + 300	CL-CH, shale, limestone clasts	-4	Cut, gneiss, MF-WF	35	No
40 + 300	40 + 340	Fill, SM-SP	-5	Fill	-5	No
40 + 340	40 + 500	Cut, granite – gneiss, MF-WF	35	Fill	-5	No
40 + 500	40 + 700	Fill	-5	Fill	-5	No
40 + 700	40 + 740	Fill	-5	Cut, gneiss, MF-WF	35	No
40 + 740	40 + 760	Fill, river crossing	-5	Fill	-5	No
40 + 760	41 + 000	Cut, SM-SP w/G&B	-4	Fill	-5	No
41 + 000	41 + 080	Fill	-5	Fill	-5	No
41 + 080	41 + 300	Cut, SM-SP w/G&B	-4	Fill	-5	No
41 + 300	41 + 440	Granite, MF-WF w/thin SP cover	35		--	Unknown
41 + 440	41 + 560	Cut, SM-SP w/G&B	-4	Fill	-5	Unknown
41 + 560	41 + 600	Granite, MF	35	Fill	-5	No

Table A-1
SUMMARY OF SOIL SURVEY

Station to Station		Left Side Observations[1,2]		Right Side Observations[1,2]		Probable Water Problem Area[4]
		Material	Shrink-Swell Factor[3]	Material	Shrink-Swell Factor[3]	
41 + 600	41 + 780	Granite, WF	35	Fill	-5	No
41 + 780	41 + 820	Granite, CF-MF	35	Fill	-5	No
41 + 820	41 + 900	Cut, SM-SP w/G&B	-4	Fill	-5	No
41 + 900	41 + 920	Granite, MF	35	Fill	-5	No
41 + 920	41 + 980	Cut, SM-SP w/boulders	-4	Fill	-5	No
41 + 980	42 + 000	Granite, MF	35	Fill	-5	No
42 + 000	42 + 060	Granite, MF	35	Granite, CF-MF	35	No
42 + 060	42 + 180	Fill	-5	Fill	-5	Unknown
42 + 180	42 + 220	Cut, SM-SP w/G&B	-5		--	Yes
42 + 220	42 + 400	Fill (river)	-4	Fill	-4	Yes
42 + 400	44 + 740	Fill (woods)	-4	Cut, SM-SP w/G&B, thin organics (woods)	-5	42+400 → 42+580 Portion by Beartooth Lake – yes 42+580 → 44+740 No
44 + 740	44 + 780	Cut, granite, WF	35		--	No
44 + 780	44 + 840	Fill	-5	Fill	-5	No
44 + 840	44 + 900	SM-SP cover	-4	Granite, MF-WF, w/ SM-SP cover	35	Unknown
44 + 900	45 + 000	Fill	-5	Fill	-5	Unknown
45 + 000	45 + 080	Fill	-4	Cut, SM-SP, organics	-5	Yes
45 + 080	45 + 740	Fill, SM-SP w/50-150 mm organics	-5	Fill	-4	Yes
45 + 740	45 + 820	Cut, SM-SP w/G&B	-5	Fill	-4	Yes
45 + 820	46 + 540	Fill, some organic soils	-4	Fill	-4	Yes

Table A-1
SUMMARY OF SOIL SURVEY

Station to Station		Left Side Observations[1,2]		Right Side Observations[1,2]		Probable Water Problem Area[4]
		Material	Shrink-Swell Factor[3]	Material	Shrink-Swell Factor[3]	
46 + 540	46 + 560	Cut, granite, WF	35	Fill	-4	Yes
46 + 560	46 + 600	Fill	-4	Fill	-4	Yes
46 + 600	46 + 920		---	Cut, SM-SP w/G&B	-5	Yes
46 + 920	47 + 900	Fill, small cuts	-4	Fill – small cuts	-4	Yes
47 + 900	47 + 980	Cut, SM-SP w/G&B	-4	Fill	-5	Unknown
47 + 980	48 + 140	Fill	-5	Fill	-5	Unknown
48 + 140	48 + 320	Cut, SM-SP w/G&B	-4	Fill	-5	Unknown
48 + 320	48 + 620	Fill	-4	Fill	-4	Yes
48 + 620	48 + 690	Cut, SM-SP w/G&B	-5	Fill	-4	Yes
48 + 690	48 + 710	Cut, granite, WF	35	Fill	-4	Yes
48 + 710	48 + 820	Cut, SM-SP w/G&B	-4	Fill	-4	Yes
48 + 820	49 + 140	Fill	-4	Cut, SM-SP w/G&B	-5	Yes
49 + 140	49 + 300	Fill, Wetlands organic soils	0	Fill	-4	Yes
49 + 300	49 + 360	Cut, granite, MF-WF	35	Fill	-5	Unknown
49 + 360	49 + 480	Fill	-5	Fill	-5	No
49 + 480	49 + 840	Cut, granite, WF	35	Fill	-5	No
49 + 840	49 + 980	Fill	-5	Fill	-5	No
49 + 980	50 + 000	Fill	-5	Cut, granite, WF	35	No
50 + 000	50 + 020	Fill	-5	Cut, SM-SP	-5	No
50 + 020	50 + 040	Fill	-5	Cut, granite, WF	35	No
50 + 040	50 + 080	Fill, SM-SP w/large boulders	-5	Fill	-5	No
50 + 080	50 + 400	Fill, wetlands	0	Fill	-4	Yes
50 + 400	50 + 540	Fill	-5	Cut, SM-SP w/G&B	-5	Yes

Table A-1
SUMMARY OF SOIL SURVEY

Station to Station		Left Side Observations[1,2]		Right Side Observations[1,2]		Probable Water Problem Area[4]
		Material	Shrink-Swell Factor[3]	Material	Shrink-Swell Factor[3]	
50 + 540	51 + 340	Fill on SM-SP soils w/50 – 100 mm organics	-5	Fill	-5	Unknown
51 + 340	51 + 700	Fill	-5	Cut, SM-SP w/G&B, large boulders	-5	Unknown
51 + 700	51 + 740	Fill	-5	Cut, granite, MF-WF	35	Unknown
51 + 740	51 + 920	Fill	-5	Fill	-5	Unknown
51 + 920	51 + 970	Cut	--	Cut, granite, MF-WF	35	No
51 + 970	52 + 200	Fill	-5	Cut, granite, MF-WF	35	No
52 + 200	52 + 260	Fill	-5	Fill	-5	No
52 + 260	52 + 320	Fill	-5	Cut, granite - frost shattered	35	No
52 + 320	53 + 340	Cuts, granite, WF, some gneiss	35	Fill	-4	Yes
53 + 340	53 + 650	Fill	-5	Cut, granite, MF-WF	35	No
53 + 650	54 + 120	Fill	-5	Cut, gneiss, MF-WF	35	No
54 + 120	54 + 840	Fill	-5	Cut, granite, MF-MF w/some SM-SP w/G&B cover	35	No
54 + 840	55 + 180		--	Cut, SM-SP w/G&B	-5	Unknown
55 + 180	55 + 280	Fill	-5	Fill	-5	Unknown
55 + 280	55 + 340	Cut, SM-SP w/G&B some MF gneiss-granite	-10	Fill	-5	No
55 + 340	55 + 640	Cut, SM-SP w/G&B	-10	Fill	-5	No
55 + 640	55 + 740	Fill	-5	Fill	-5	No
55 + 740	55 + 960	Fill	-5	Cut, SM-SP w/G&B and granite MF-WF, frost shattered	-4	No

Table A-1
SUMMARY OF SOIL SURVEY

Station to Station		Left Side Observations[1,2]		Right Side Observations[1,2]		Probable Water Problem Area[4]
		Material	Shrink-Swell Factor[3]	Material	Shrink-Swell Factor[3]	
55 + 960	55 + 980	Fill	-5	Cut, granite, MF-WF	35	No
55 + 980	56 + 290	Fill	-5	Cut, SM-SP w/G&B, some fill	-4	No
56 + 290	56 + 320	Fill	-5	Cut, granite, MF	35	No
56 + 320	56 + 380	Fill	-5	Cut, SM-SP w/G&B	-4	No
56 + 380	56 + 410	Fill	-5	Cut, granite-gneiss	35	No
56 + 410	56 + 520	Fill	-5	Cut, SM-SP w/G&B	-4	No
56 + 520	56 + 570	Fill	-5	Cut, granite, MF	35	No
56 + 570	56 + 780	Small fills	-4	No cut	--	No
56 + 780	57 + 400	Cut, SM-SP w/G&B 5% bedrock, some granite	-4		--	No
57 + 400	57 + 500	Fill	-5	No cut small	--	No
57 + 500	57 + 540	Fill	-5	Cut, granite, MF	35	No
57 + 540	58 + 020	Fill	-5	Cut, SM-SP w/G&B	-4	No
58 + 020	58 + 060	Fill	-5	Cut, granite, MF, frost	35	No
58 + 060	58 + 300	Fill	-5	Cut, SM-SP w/G&B	-4	No
58 + 300	58 + 380	Fill	-5	Fill	-5	No
58 + 380	58 + 400	Cut, MF gneiss	35	Fill	-5	No
58 + 400	58 + 420	Fill	-5	Fill	-5	No
58 + 420	58 + 680	Cut, SM-SP w/G&B	-4	Fill	-5	No
58 + 680	58 + 740	No cut	--	Fill	-5	No
58 + 740	59 + 100	Cut, SM-SP w/G&B	-4	Fill	-5	No
59 + 100	59 + 280		--	No cut or fill	--	No
59 + 280	59 + 700	Fill	-5	Cut, SM-SP w/G&B	-4	No

Table A-1
SUMMARY OF SOIL SURVEY

Station to Station		Left Side Observations[1,2]		Right Side Observations[1,2]		Probable Water Problem Area[4]
		Material	Shrink-Swell Factor[3]	Material	Shrink-Swell Factor[3]	
59 + 700	59 + 780	Fill	-5	No cut	--	No
59 + 780	59 + 960		--	No cut or fill	--	No
59 + 960	60 + 240	Fill	-5	Cut, SM-SP w/G&B	-4	No
60 + 240	60 + 300	Cut, granite, CF-MF	35		--	No
60 + 300	60 + 440	Small cut	--	Fill	-5	No
60 + 440	60 + 580	Cut, Granite-Gneiss, CF-MF w/ SM-SP, w/G&B	35	Fill	-5	No
60 + 580	60 + 600	Cut, SM-SP w/G&B	-4	Fill	-5	No
60 + 600	60 + 660	Fill	-5	Fill	-5	No
60 + 660	60 + 980	Fill	-5	Cut, SM-SP w/G&B	-4	No
60 + 980	61 + 040	Fill	-5	Cut, CF-MF, granite w/<0.5m SM cover	35	No
61 + 040	61 + 240	Cut, SM-SP w/G&B	-4	Fill	-5	No
61 + 240	61 + 340	Cut, CF-MF, granite and gneiss w/thin cover of SM-SP soils, w/G&B	35	Fill	-5	No
61 + 340	61 + 660	Fill	-5	Cut, SM-SP w/G&B	-4	No
61 + 660	61 + 720	Fill	-5	Cut granite, CF, w/SM soil cover	35	No
61 + 720	61 + 900	Some fills on SM-SP, w/G&B	-4	No cut	--	No
61 + 900	61 + 920	Cut, bedrock, frost shattered boulders w/ SM-SP, w/G&B	35		--	No
61 + 920	62 + 020	Fill	-5	Fill	-5	No
62 + 020	63 + 140	Boulder field, SM soils w/G&B	-4	No cut of fill, some small cuts in SM	--	No

Table A-1
SUMMARY OF SOIL SURVEY

Station to Station		Left Side Observations[1,2]		Right Side Observations[1,2]		Probable Water Problem Area[4]
		Material	Shrink-Swell Factor[3]	Material	Shrink-Swell Factor[3]	
63 + 140	63 + 320	Cut – SM-SP w/G&B	-4	Fill	-5	No
63 + 320	63 + 500	Cut, SM-SP w/gravel to large boulders over MF-CF bedrock	-4	Fill	-5	No
63 + 500	63 + 620	Boulder field	-4	No cut or Fill	--	No
63 + 620	63 + 900	Fill	-5	Cut, SM-SP w/gravel to large boulders	-4	No
63 + 900	64 + 100	Boulder field	-4	No cut or fill	--	No
64 + 100	64 + 280	Fill	-5	Cut, Granite, MF w/SM + boulder cover	35	No
64 + 280	64 + 800	Fill or boulder field	-4	Cut, SM w/boulder field large boulders	-5	No
64 + 800	64 + 960	Cut, SM w/gravel to large boulders	-4	Fill	-5	No
64 + 960	65 + 000	Cut	--	No fill or	-5	No
65 + 000	65 + 200		--	Cut, SM-SP w/gravel to large boulders	-4	No
65 + 200	66 + 100	Fill, SM-SP w/gravel to boulders	-4	No cut or fill	--	No
66 + 100	66 + 600	Fill	-5	Cut, SM-SP w/ gravel to large boulders	-4	No
66 + 600	66 + 960		--	Cut, granite, CF-MF frost shattered w/thin SM-SP cover	35	No
66 + 960	67 + 010	Fill	-5	Cut, SM-SP w/G&B	-4	No
67 + 010	67 + 340	Fill	-5	Cut, SM-SP w/ large boulders	-4	No

Table A-1
SUMMARY OF SOIL SURVEY

Station to Station		Left Side Observations[1,2]		Right Side Observations[1,2]		Probable Water Problem Area[4]
		Material	Shrink-Swell Factor[3]	Material	Shrink-Swell Factor[3]	
67 + 340	Montana Border	Fill	-5	Cut, SM-SP w/G&B	-4	Unknown

Notes:

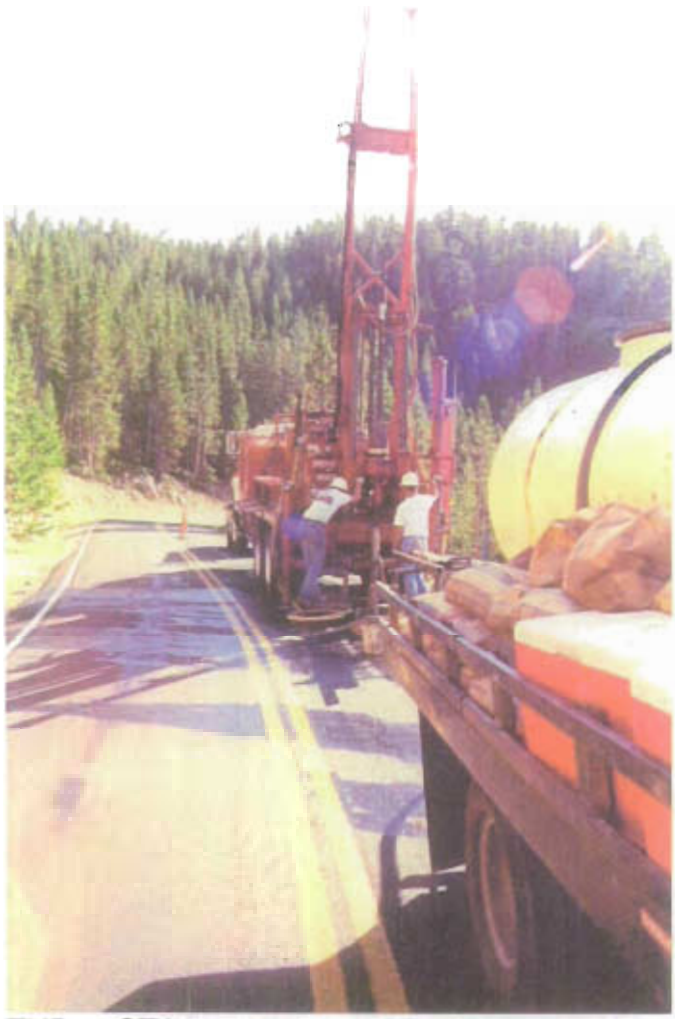
1. CF, MF, WF indicate characteristic fracture spacing of outcrop or cut. CF indicates 0.07 to 0.15 m spacing, MF indicates 0.15 to 0.30 m spacing, and WF indicates 0.3 to 1.0 m spacing.
2. Visual classification of soil (and shale) is provided using the Unified Soil Classification System. G&B indicates the presence of gravel (<75 mm), cobbles (<300 mm), and boulders (>300 mm) was observed.
3. Shrink and Swell Factors are based largely on the Federal Lands Highway Project Development and Design Manual, June 1996. Negative numbers represent shrinkage. Factors are subject to 33% ± variation.
4. Areas where groundwater or surface water control is likely to impact near surface excavations or the construction of embankments or structures are identified as Probable Water Problem Areas. Areas are identified as unknown where the probability of water impact could not be estimated.

Appendix B

TH1-4, STA 39+870

TH1-1, STA 39+980

TH1-3, STA 39+980



THB-1, STA 41+400

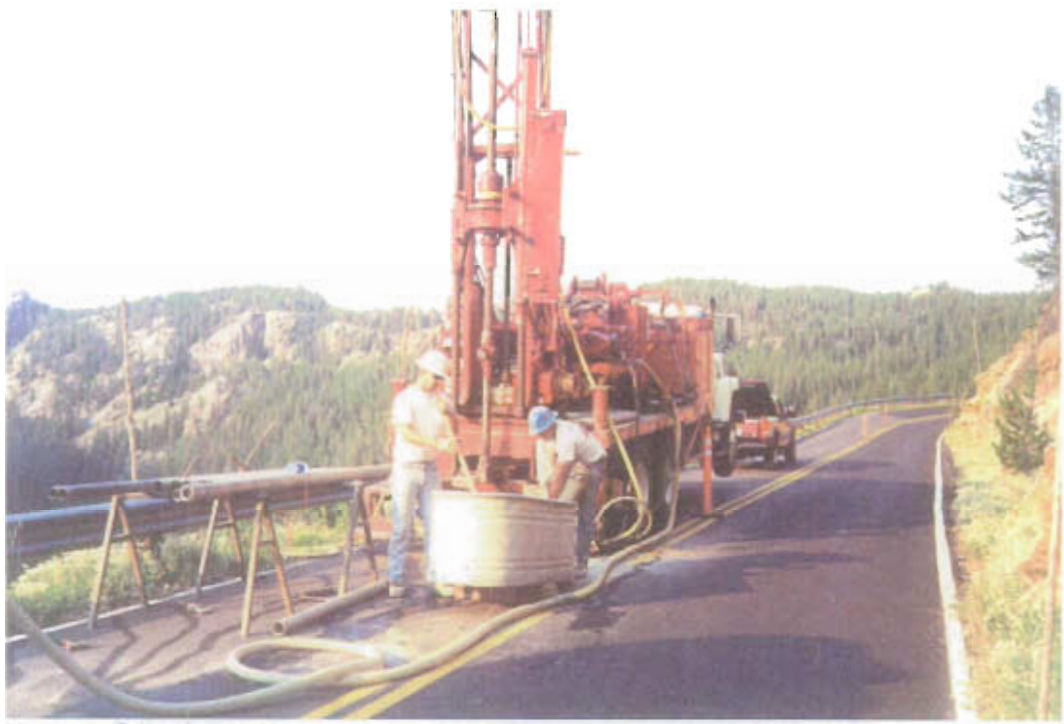
THB-2, STA 41+443

THB-4, STA 41+497

THB-5, STA 41+540

TH2-1, STA 41+610

TH2-2, STA 41+610



TH2-3, STA 41+660

TH2-4, STA 41+700

TH2-5, STA 41+700

TH2-6, STA 41+765

TH2-7, STA 41+800

TH2-8, STA 41+865

TH2-9, STA 41+903

TH2-11, STA 41+964

THB-6, STA 42+238

THB-7, STA 42+378

THB-8, STA 45+300

THB-11, STA 46+601

THB-12, STA 49+149

THB-14, STA 49+200

THB-16a, STA 49+276

THB-19, STA 50+262

THB-20, STA 50+285

TH4-1, STA 56+970

TH4-3, STA 57+040

TH4-2, STA 57+095

(a) THB-1, 0 - 3.35 m

(b) THB-1, 3.35 – 6.2 m

(c) THB-1, 6.2 – 8.6 m

(d) THB-2, 0 - 5.35 m

(e) THB-2, 5.35 – 8.4 m

(f) THB-2, 8.4 – 10.7 m

(g) THB-2, 10.7 – 11.0 m

(h) THB-4, 0 – 5.35 m

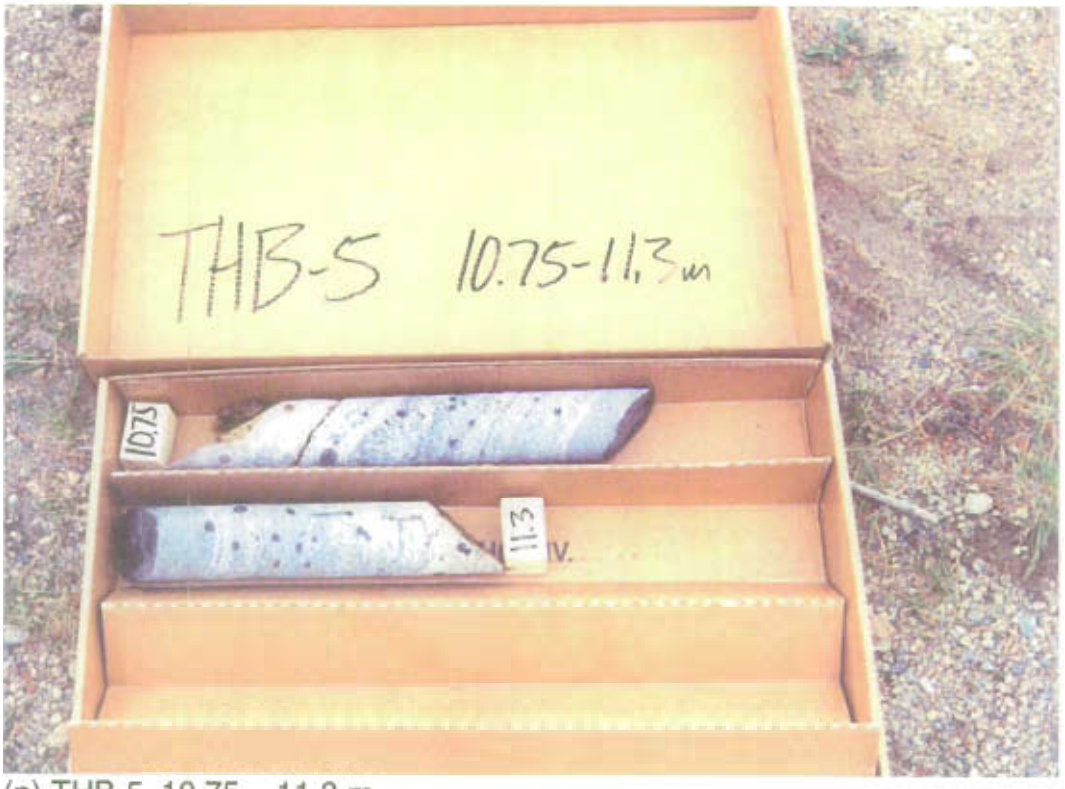
(i) THB-4, 5.35 – 7.6 m

(j) THB-4, 7.6 – 10.0 m

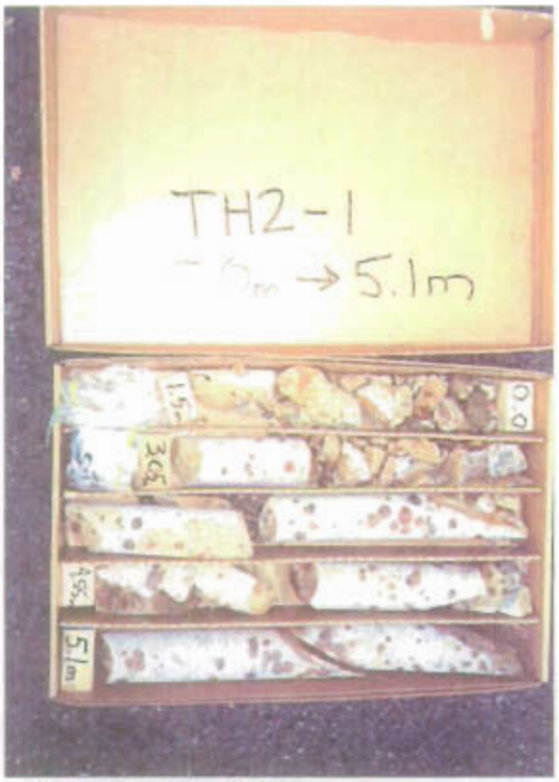
(k) THB-5, 0 – 6.1 m

(l) THB-5, 6.1 – 8.4 m

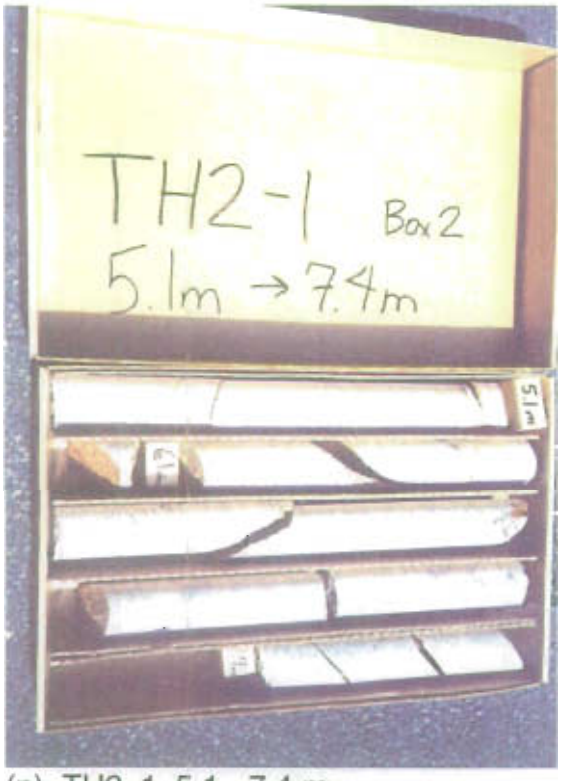
(m) THB-5, 8.4 – 10.75 m



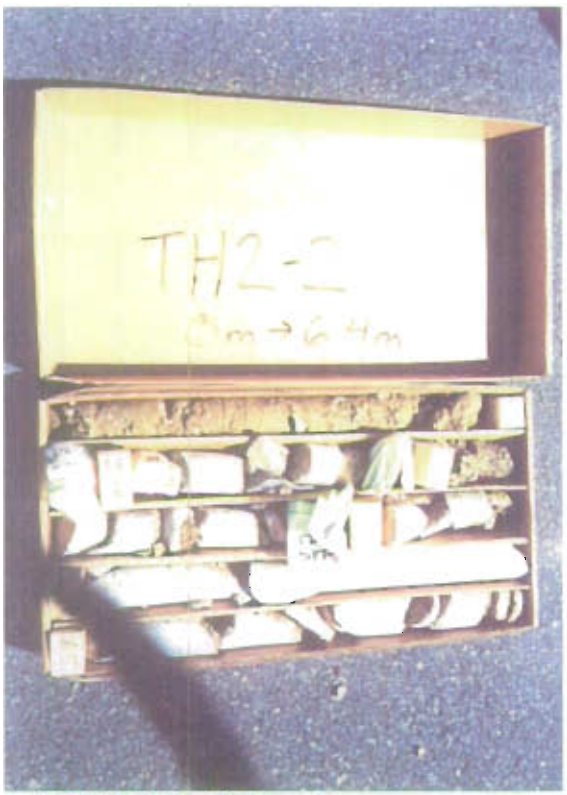
(n) THB-5, 10.75 – 11.3 m



(o) TH2–1, 0 - 5.1 m

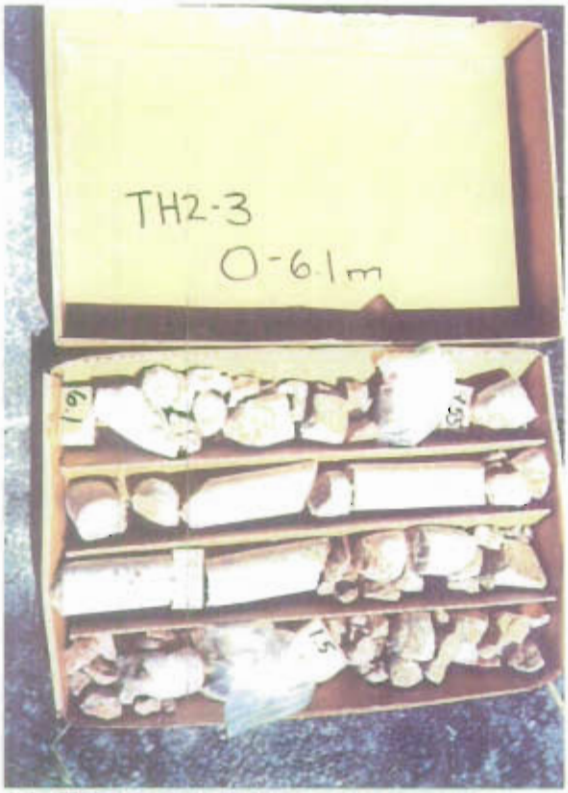


(p) TH2–1, 5.1 - 7.4 m



(q) TH2-2, 0 - 6.4 m

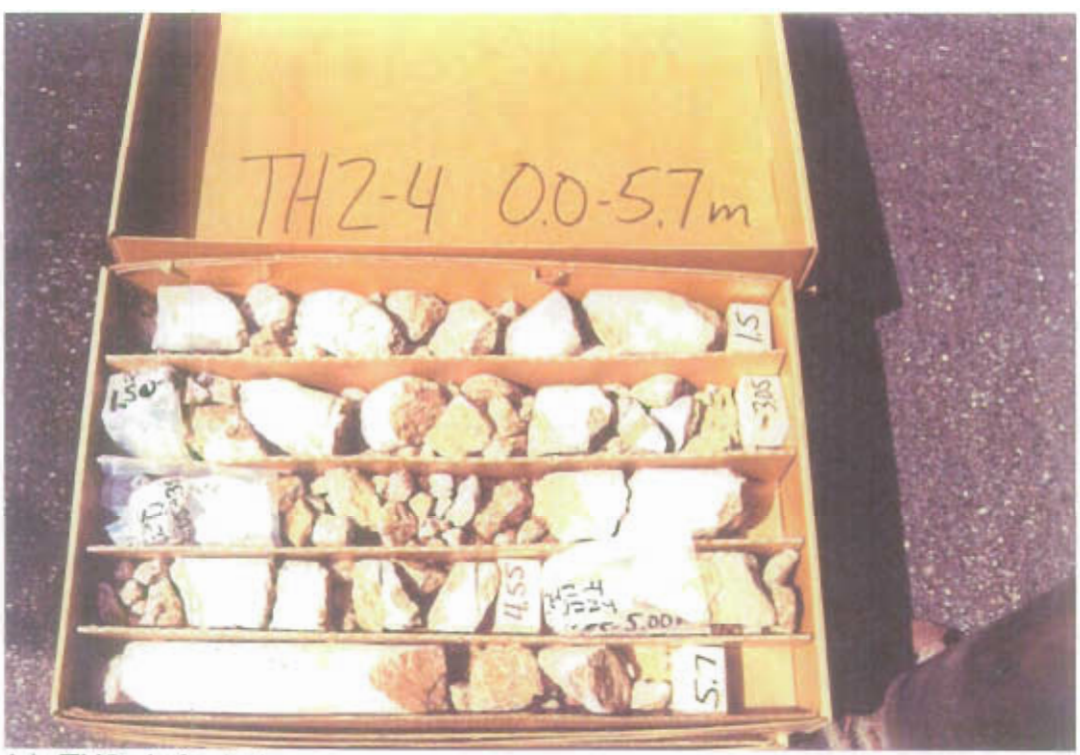
(r) TH2–2, 6.4 - 8.55 m



(s) TH2-3, 0 – 6.1 m

(t) TH2-3, 6.1 - 9.7 m

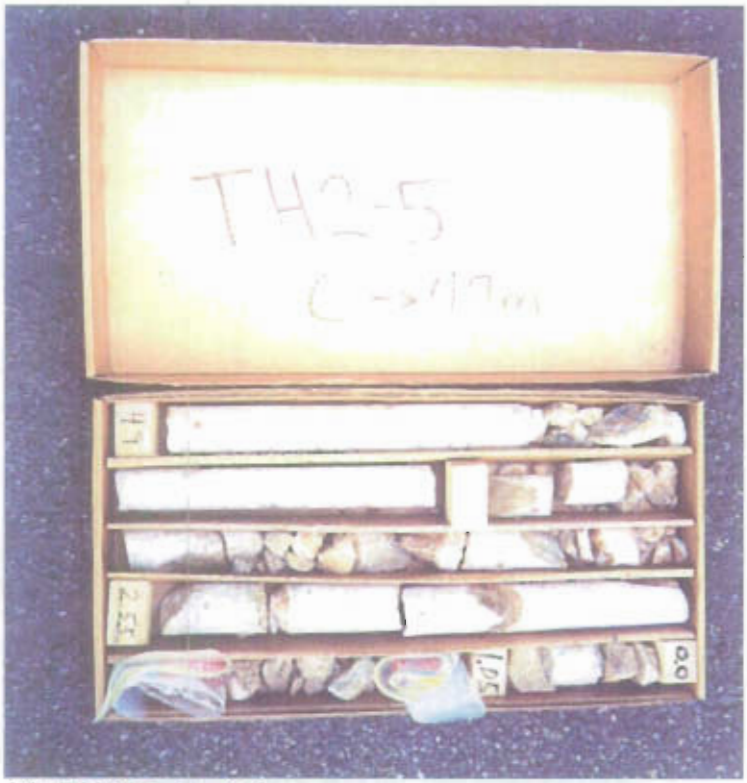
(u) TH2–3, 9.7 - 10.7 m



(v) TH2–4, 0 - 5.7 m

(w) TH2–4, 5.7 - 10.4 m

(x) TH2–4, 10.4 - 12.45

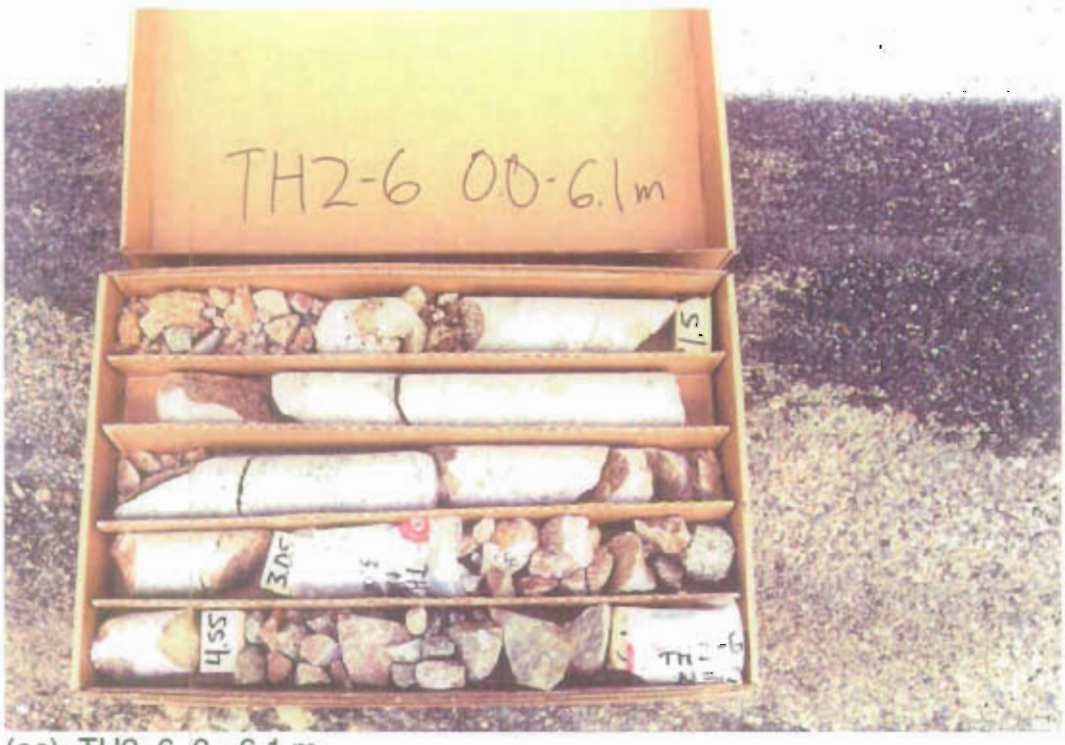


(y) TH2–5, 0 - 4.9

(z) TH2–5, 4.9 - 10.0 m

(aa) TH2–5, 10.0 - 13.2 m

(ab) TH2–5, 13.2 – 14 m



(ac) TH2–6, 0 - 6.1 m

(ad) TH2–6, 6.1 - 9.45 m

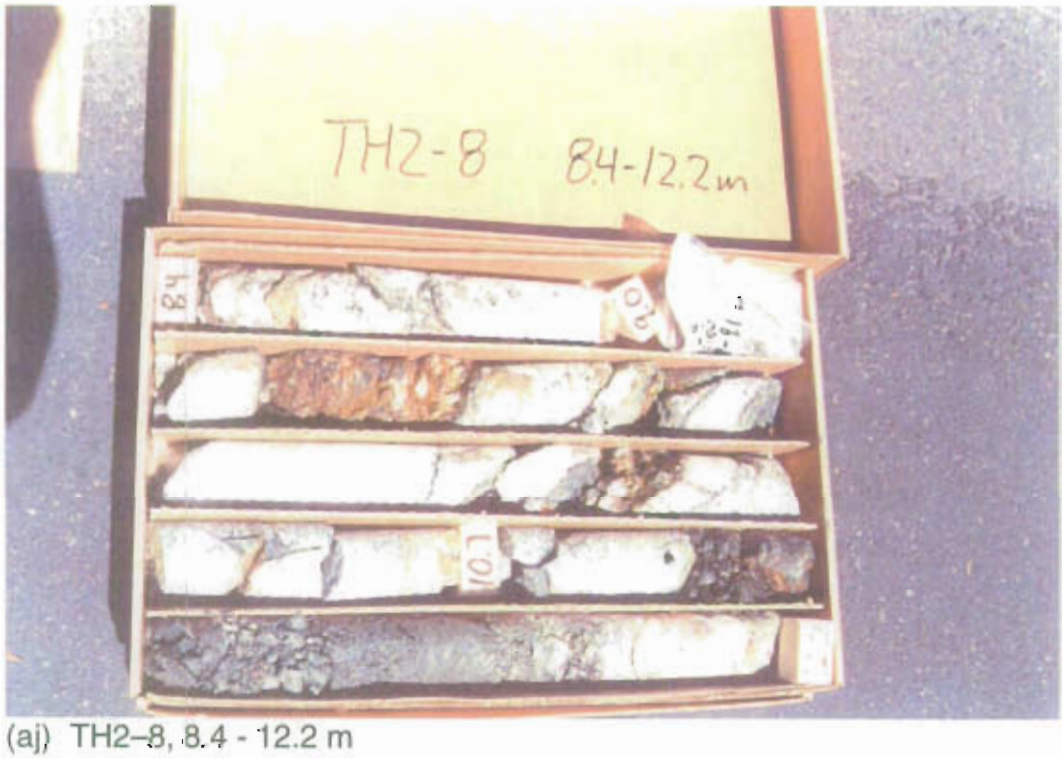
(ae) TH2–6, 9.45 - 10.65 m

(af) TH2–7, 0 - 3.8 m

(ag) TH2–7, 3.8 - 8.4 m

(ah) TH2–7, 8.4 - 11.0 m

(ai) TH2–8, 0 - 8.4 m



(aj) TH2–8, 8.4 - 12.2 m

(ak) TH2–8, 12.2 - 14.8 m

(al) TH2–8, 14.8 - 15.6

(am) TH2–9, 0 - 5.2 m

(an) TH2–9, 5.2 - 7.6 m

(ao) TH2–11, 0 - 4.7 m

(ap) TH2–11, 4.7 - 7.8 m

(aq) TH2–11, 7.8 - 10.7 m

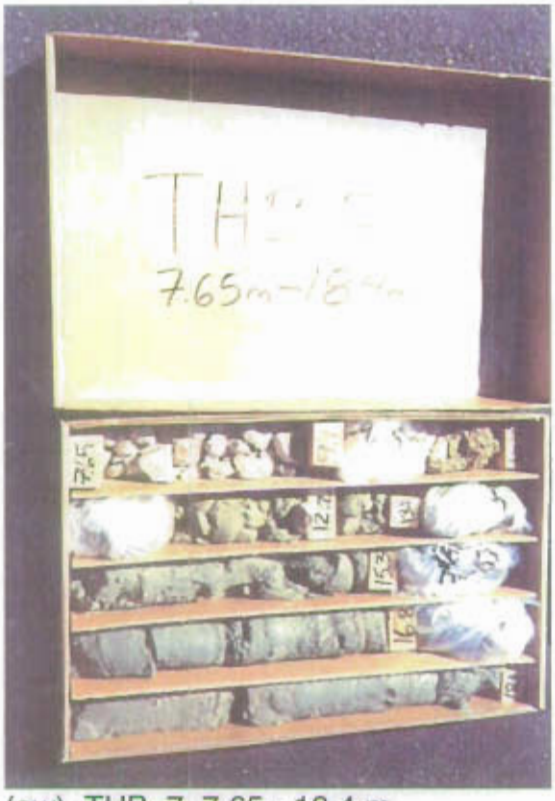
(ar) TH2–11, 10.7 - 13.4 m

(as) TH2–11, 13.4 - 15.8 m

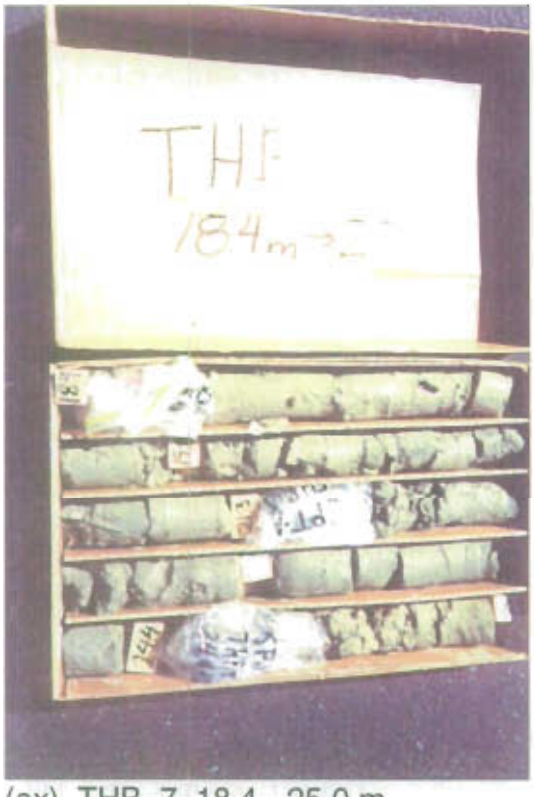
(at) TH2–11, 15.8 - 16.9 m

(au) THB–6, 0 - 23.35 m

(av) THB–7, 0 - 7.65 m



(aw) THB–7, 7.65 - 18.4 m



(ax) THB–7, 18.4 - 25.0 m

(ay) THB–7, 25.0 – 32.0 m

(az) THB–8, 6.1 - 9.7 m

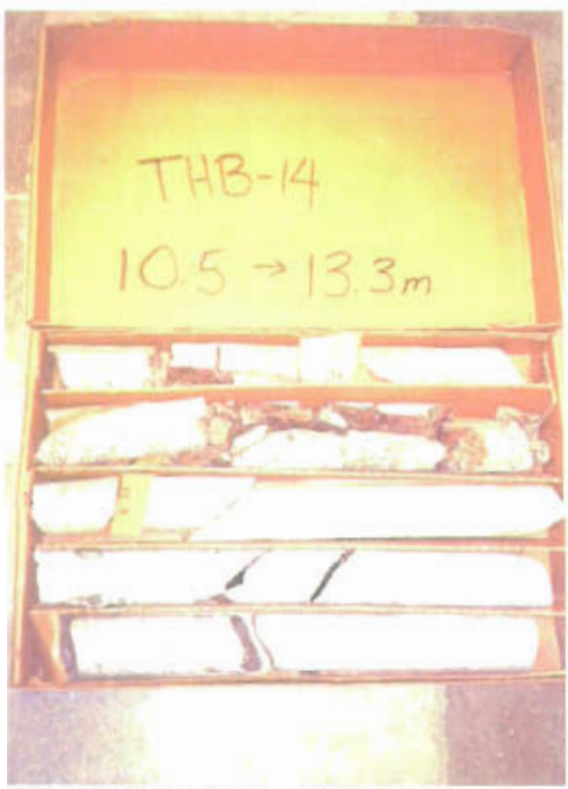
(ba) THB–8, 9.7 – 11.7 m

(bb) THB–12, 0 - 7.3 m

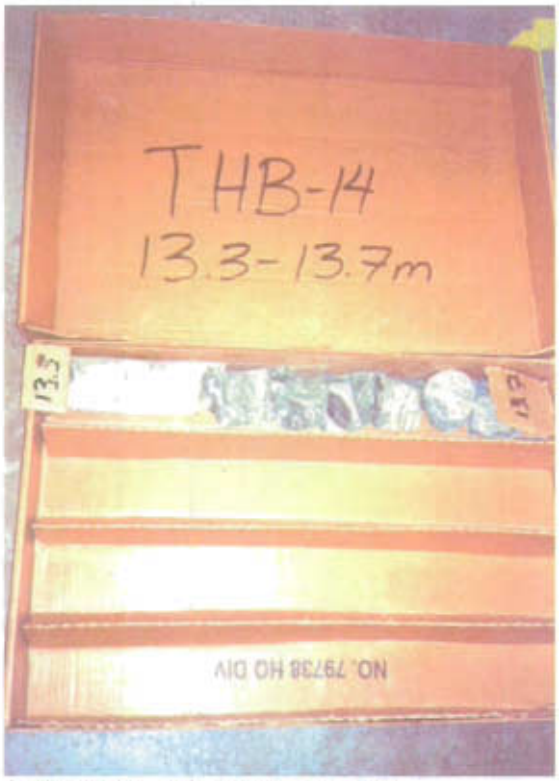
(bc) THB–12, 7.3 - 10.4 m

(bd) THB–12, 10.4 - 10.7 m

(be) THB–14, 0 - 10.5 m



(bf) THB–14, 10.5 - 13.3 m



(bg) THB–14, 13.3 - 13.7 m

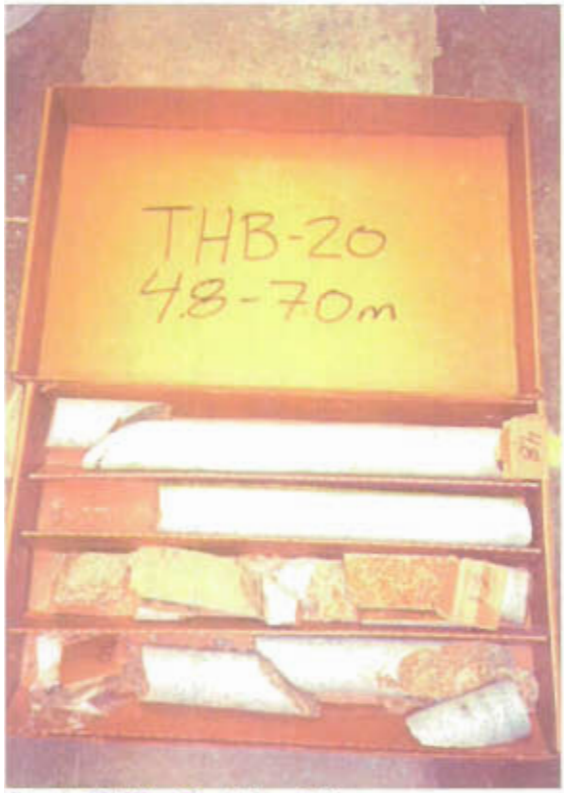
(bh) THB–16a, 0 - 4.6 m

(bi) THB–16a, 4.6 – 7.6 m

(bj) THB–19, 0 - 5.0 m

(bk) THB–19, 5.0 - 6.0 m

(bl) THB–20, 0 - 4.8 m



(bm) THB–20, 4.8 - 7.0 m

(bn) TH4–1, 0 - 4.55 m

(bo) TH4–1, 4.55 - 6.85 m

(bp) TH4-3, 0 - 6.1 m

(bq) TH4–3, 6.1 - 9.0

(br) TH4–2, 0 - 6.7 m

(bs) TH4–2, 6.7 - 8.7 m

Project: Beartooth Highway
Project Location: Site 1
Project Number: 68FHAT0027.00

Key to Log of Test Hole

Elevation, meters	Depth, meters	SAMPLES Type	SAMPLES Number	Blows per 150 mm	Recovery, %	Graphic Log	MATERIAL DESCRIPTION	Well Construction Schematic	Water Content, %	Dry Unit Weight, kg/m³	REMARKS AND OTHER TESTS
1	2	3	4	5	6	7	8	9	10	11	12

COLUMN DESCRIPTIONS

1 **Elevation:** Elevation in meters referenced to mean sea level (MSL) or site datum.

2 **Depth:** Depth in meters below the ground surface.

3 **Sample Type:** Type of soil sample collected at depth interval shown; sampler symbols are explained below.

4 **Sample Number:** Sample identification number.

5 **Sampling Resistance:** Number of blows required to advance driven sampler 300 mm beyond first 150-mm interval, or distance noted, using a 63.5-kg hammer with a 762-mm drop.

6 **Recovery:** Percentage of driven or pushed sample recovered relative to sampled interval; "NR" indicates data not recorded.

7 **Graphic Log:** Graphic depiction of subsurface material encountered; typical symbols are explained below.

8 **Material Description:** Description of material encountered; may include relative density / consistency, moisture, color, and grain size.

9 **Well Construction Schematic:** Graphic depiction of piezometer or well installation; materials are listed in header block; graphic symbols are explained below.

10 **Water Content:** Water content of soil sample measured in laboratory, expressed as percentage of dry weight of specimen.

11 **Dry Unit Weight:** Dry density of soil sample measured in laboratory, expressed in kilograms per cubic meter (kg/m³).

12 **Remarks and Other Tests:** Comments and observations regarding drilling or sampling made by driller or field personnel. Other field and laboratory test results, using the following abbreviations:

LL	Liquid Limit
PI	Plasticity Index
Cc	Compression Index

TYPICAL SOIL GRAPHIC SYMBOLS

SAND (SP/SW)
SILT (ML)
CLAY (CL)
GRAVEL (GP/GW)
Silty SAND (SM)
Silty to Clayey SAND (SM/SC)
Clayey SAND (SC)
COBBLES and BOULDERS
SHALE
CLAY with rock inclusions (CL)
Asphalt
Road Base

TYPICAL SAMPLER GRAPHIC SYMBOLS

50-mm-OD split spoon sampler (SPT)
76-mm-OD split spoon sampler (California)
Shelby tube sampler (thin-wall, fixed head)
Bulk or grab sample

TYPICAL WELL GRAPHIC SYMBOLS

Blank casing in cement
Blank casing in filter sand
Blank casing in bentonite and cement grout
Slotted casing in filter sand
Blank casing in bentonite chips or pellets
Native backfill

OTHER GRAPHIC SYMBOLS

First water encountered at time of drilling and sampling (ATD)
Change in material properties within a stratum
Inferred contact or gradational lithologic change

Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive; field descriptions may have been modified to reflect lab test results. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced; they are not warranted to be representative of subsurface conditions at other locations or times.

BLOW COUNT DENSITY AND CONSISTENCY RELATIONSHIPS

COARSE-GRAINED SOILS:

Apparent Density	SPT Blow Count
Very loose	0 to 4
Loose	>4 to 10
Medium dense	>10 to 30
Dense	>30 to 50
Very dense	>50

FINE-GRAINED SOILS:

Consistency	SPT Blow Count
Very soft	0 to 2
Soft	>2 to 4
Medium stiff	>4 to 8
Stiff	>8 to 15
Very stiff	>15 to 30
Hard	>30

Report: GEO_12W_OAKDENV_KEY; File: BTOOTHWY.GPJ; 1/14/2001 keysoil

URS

Project: Beartooth Highway
Project Location: Site 2, Site 4, and Bridges
Project Number: 68FHAT0027.00

Key to Log of Test Hole

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17

COLUMN DESCRIPTIONS

1 **Elevation:** Elevation in meters referenced to mean sea level (MSL).

2 **Depth:** Distance in meters below the collar of the borehole.

3 **Run No.:** Number of the individual coring interval.

4 **Box No.:** Number of the core box which contains core from the corresponding run.

5 **Recovery:** Amount (in percent) of core recovered from the coring interval; calculated as length of core recovered divided by length of run.

6 **Fractures per 300 mm:** (Fracture Frequency) The number of naturally occurring fractures in each 300 mm of core; does not include mechanical breaks (induced by drilling) or healed fractures. "NA" indicates not applicable due to lack of core recovery.

7 **R.Q.D:** (Rock Quality Designation) Percentage of intact core (pieces of sound core longer than 102 mm) in each coring interval; calculated as the sum of lengths of intact core divided by length of core run.

8 **Fracture Drawing:** Sketch of the naturally occurring fractures and mechanical breaks, showing the angle of the fractures relative to the cross-sectional axis of the core. "NR" indicates no recovery.

9 **Fracture Number:** Location of each naturally occurring fracture (numbered) and mechanical break (labeled "M"). Naturally occurring fractures are described in Column 11 (keyed by number) using descriptive terms defined on Sheet 2 (items a through g).

10 **Lithology:** A graphic log of material encountered using symbols to represent differing soil and types; graphic symbols are explained below.

11 **Description:** Lithologic description of soil (may include USCS, density/consistency, color, grain size) or rock (may include strength, color, grain size, weathering, other features). Selected descriptive terms are defined on Sheet 2. Also, abbreviated description of fractures numbered in Column 9 using terms defined on Sheet 2.

12 **Sample Type:** Type of soil sample collected at depth interval shown; sampler symbols are explained below.

13 **Sample Number:** Sample identification number.

14 **Blows per 150 mm:** Number of blows to advance driven sampler each 150-mm drive interval, or distance noted, using a 63.5-kg hammer with a 762-mm drop.

15 **Recovery:** Actual soil recovery in driven sampler as a percentage of the sampler penetration.

16 **Drill Time:** Time in 24-hour clock at start and finish of each sampling interval.

17 **Field Notes and Lab Tests:** Comments and observations regarding drilling or sampling made by driller or field personnel. Lab tests are indicated using abbreviations explained below.

TYPICAL MATERIAL GRAPHIC SYMBOLS

SAND (SP/SW)
SILT (ML)
CLAY (CL)
GRAVEL (GP/GW)
Gravelly SAND (SP/SW)
Silty SAND (SM)
Clayey SAND (SC)
COBBLES and BOULDERS
GRANITE
GNEISS
GABBRO
SANDSTONE

TYPICAL SAMPLER GRAPHIC SYMBOLS

50-mm-OD split spoon sampler (SPT)
76-mm-OD split spoon sampler (California)

OTHER GRAPHIC SYMBOLS

▽ First water encountered at time of drilling (ATD)
Change in material properties within a stratum

Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive; field descriptions may have been modified to reflect lab test results. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced; they are not warranted to be representative of subsurface conditions at other locations or times.

LABORATORY TEST ABBREVIATIONS

Cc	Compression Index
LL	Liquid Limit
PI	Plastic Limit
PLI	Point Load Index
CS	Compression Strength [MPa] evaluated from point load test
UCS	Unconfined Compressive Strength [MPa]
*	Indicates L/D<2.0 correction factor from ASTM D2938 used

BLOW COUNT DENSITY AND CONSISTENCY RELATIONSHIPS

COARSE-GRAINED SOILS:

Apparent Density	SPT Blow Count
Very loose	0 to 4
Loose	>4 to 10
Medium dense	>10 to 30
Dense	>30 to 50
Very dense	>50

FINE-GRAINED SOILS:

Consistency	SPT Blow Count
Very soft	0 to 2
Soft	>2 to 4
Medium stiff	>4 to 8
Stiff	>8 to 15
Very stiff	>15 to 30
Hard	>30

Report: GEO_CORE+SOIL_17_METDENV_KEY, File: BTOOTHWY.GPJ, 1/14/2001 keycore

URS

Project: Beartooth Highway Project Location: Site 2, Site 4, and Bridges Project Number: 68FHAT0027.00	Key to Log of Test Hole Sheet 2 of 2

KEY TO DESCRIPTIVE TERMS USED ON CORE LOGS

DISCONTINUITY DESCRIPTORS

a Dip of discontinuity, measured relative to a plane normal to the core axis.

b **Discontinuity Type:**

- F - Fault
- J - Joint
- Sh - Shear
- Fo - Foliation
- V - Vein
- B - Bedding

c **Aperture (mm):**

- W - Wide (13-51)
- MW - Moderately Wide (2.5-13)
- N - Narrow (1.3-2.5)
- VN - Very Narrow (<1.3)
- T - Tight (0)

d **Type of Infilling:**

- Cl - Clay
- Ca - Calcite
- Ch - Chlorite
- Fe - Iron Oxide
- Gy - Gypsum
- H - Healed
- Mn - Manganese Oxide
- No - None
- Py - Pyrite
- Qz - Quartz
- Sd - Sand

e **Amount of Infilling:**

- Su - Surface Stain
- Sp - Spotty
- Pa - Partially Filled
- Fi - Filled
- No - None

f **Surface Shape of Joint:**

- Pl - Planar
- Wa - Wavy
- St - Stepped
- Ir - Irregular

g **Roughness of Surface:**

- Slk - Slickensided [surface has smooth, glassy finish with visual evidence of striations]
- S - Smooth [surface appears smooth and feels so to the touch]
- SR - Slightly Rough [asperities on the discontinuity surfaces are distinguishable and can be felt]
- R - Rough [some ridges and side-angle steps are evident; asperities are clearly visible, and discontinuity surface feels very abrasive]
- VR - Very Rough [near-vertical steps and ridges occur on the discontinuity surface]

ROCK WEATHERING / ALTERATION

Description	Recognition
Residual Soil	Original minerals of rock have been entirely decomposed to secondary minerals, and original rock fabric is not apparent; material can be easily broken by hand
Completely Weathered/Altered	Original minerals of rock have been almost entirely decomposed to secondary minerals, although original fabric may be intact; material can be granulated by hand
Highly Weathered/Altered	More than half of the rock is decomposed; rock is weakened so that a minimum 50-mm-diameter sample can be broken readily by hand across rock fabric
Moderately Weathered/Altered	Rock is discolored and noticeably weakened, but less than half is decomposed; a minimum 50-mm-diameter sample cannot be broken readily by hand across rock fabric
Slightly Weathered/Altered	Rock is slightly discolored, but not noticeably lower in strength than fresh rock
Fresh/Unweathered	Rock shows no discoloration, loss of strength, or other effect of weathering/alteration

ROCK STRENGTH

Description	Recognition
Extremely Weak Rock	Can be indented by thumbnail
Very Weak Rock	Can be peeled by pocket knife
Weak Rock	Can be peeled with difficulty by pocket knife
Moderately Strong Rock	Can be indented 5 mm with sharp end of pick
Strong Rock	Requires one hammer blow to fracture
Very Strong Rock	Requires many hammer blows to fracture
Extremely Strong Rock	Can only be chipped with hammer blows

Report: GEO_CORE+SOIL_17_METDBNV_KEY; File: BTOOTHWY.GPJ; 1/14/2001 keycore

URS

Project: Beartooth Highway Project Location: Site 1 Project Number: 68FHAT0027.00	**Log of Test Hole TH1-1** Sheet 1 of 3

Date(s) Drilled	8/23/00	Logged By	A. Burns	Checked By	D. Baures
Drilling Method	Hollow-Stem Auger	Drill Bit Size/Type	114-mm-ID auger bit	Total Depth of Borehole	15.50 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Surface Elevation	2733.2 m MSL
Groundwater Level and Date Measured	11.7 m bgs ATD	Sampling Method(s)	SPT, California, Shelby tube	Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop
Borehole Completion	Inclinometer casing grouted in place to 13.7 m bgs, flush cover			Location	Station 39+980, offset 1.8 R

Elevation, meters	Depth, meters	SAMPLES Type	SAMPLES Number	Blows per 150 mm	Recovery, %	Graphic Log	MATERIAL DESCRIPTION	Inclinometer Construction Schematic	Water Content, %	Dry Unit Weight, kg/m³	REMARKS AND OTHER TESTS
2733	0						Asphalt pavement Road Base CLAY (CL), medium stiff to stiff, brown, moist, silty, few fine gravel				Begin drilling at 1230.
2732	1	CAL	1	2 2 10	44						Sampled at 1234.
2731	2						CLAY (CL), stiff to stiff, gray-brown, silty, some inclusions of highly weathered to unweathered shale; includes silty sand (SM)				
2730	3	CAL	2	10 8 6	89						Sampled at 1240.
2729	4										
2728	5	ST	1		89						Sampled at 1250.
		SPT	1	3 5 6	100						Sampled at 1253. N = 11
	6										

Report: GED_12W_OAK_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 01TH1-01

URS

Project: Beartooth Highway
Project Location: Site 1
Project Number: 68FHAT0027.00

Log of Test Hole TH1-1

Sheet 2 of 3

Elevation, meters	Depth, meters	SAMPLES Type	SAMPLES Number	Blows per 150 mm	Recovery, %	Graphic Log	MATERIAL DESCRIPTION	Inclinometer Construction Schematic	Water Content, %	Dry Unit Weight, kg/m³	REMARKS AND OTHER TESTS
	6						CLAY (CL), medium stiff to stiff, gray-brown, silty, some inclusions of highly weathered to unweathered shale; includes silty sand (SM) (continued)				
2727		CAL	3	6 9 11	89		Becomes stiff to very stiff				Sampled at 1300.
	7										
2726											
		CAL	4	3 10 10	91				17.6	1823	Sampled at 1310. LL = 19, PI = 3
	8										
2725											
	9										
2724		CAL	5	6 8 11	22						Sampled at 1325.
	10										Increase in shale inclusions.
2723											
		CAL	6	8 11 13	89						Sampled at 1335.
	11							▽			
2722											Water encountered at approx. 11.7 m.
	12						SAND (SM), medium dense to dense, silty, gray to brown, saturated, fine- to coarse-grained				
2721		CAL	7	18 19 19	44				7.8	2204	Sampled at 1347. Nonplastic
	13										

Report GEO_12W_OAK_METRIC; File BTOOTHWY.GPJ; 1/14/2001 01TH1-01

URS

Project: Beartooth Highway
Project Location: Site 1
Project Number: 68FHAT0027.00

Log of Test Hole TH1-1

Sheet 3 of 3

Elevation, meters	Depth, meters	SAMPLES Type	SAMPLES Number	Blows per 150 mm	Recovery, %	Graphic Log	MATERIAL DESCRIPTION	Inclinometer Construction Schematic	Water Content, %	Dry Unit Weight, kg/m³	REMARKS AND OTHER TESTS
2720	13						SAND (SM), medium dense to dense, silty, gray to brown, saturated, fine- to coarse-grained (continued)				
		CAL	8	22 27 23	NR						Sampled at 1410.
2719	14										
	15						SHALE, weak to moderately strong, dark gray, highly weathered, mica-rich				
2718		CAL	9	32 50/10	40				14.8	1804	Sampled at 1445. Nonplastic
							Bottom of hole at 15.60 m				Completion at 1800.
2717	16										
2716	17										
2715	18										
2714	19										
	20										

Report: GEO_12W_OAK_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 01TH1-01

URS

Project: Beartooth Highway Project Location: Site 1 Project Number: 68FHAT0027.00	**Log of Test Hole TH1-3** Sheet 1 of 2

Date(s) Drilled	8/23/00	Logged By	A. Burns	Checked By	D. Baures
Drilling Method	Hollow-Stem Auger	Drill Bit Size/Type	114-mm-ID auger bit	Total Depth of Borehole	12.50 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Surface Elevation	2733.8 m MSL
Groundwater Level and Date Measured	11.95 m bgs ATD	Sampling Method(s)	SPT, California, Shelby tube	Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop
Borehole Completion	51-mm-dia. PVC screened 6.1-12.5 m in 10/20 sand, bentonite plug 2.44-5.5 m, flush-mount cover set in cement grout			Location	Station 39+980, offset 5.4 L

Elevation, meters	Depth, meters	SAMPLES Type	Number	Blows per 150 mm	Recovery, %	Graphic Log	MATERIAL DESCRIPTION	Well Construction Schematic	Water Content, %	Dry Unit Weight, kg/m³	REMARKS AND OTHER TESTS
	0						CLAY (CL), gray, some organics [Topsoil]				Begin drilling at 0817.
2733	1						CLAY (CL), stiff, gray, slightly moist, some highly weathered shale inclusions up to 0.1 m diameter				
		CAL	1	4 8 12	100						Sampled at 0823.
2732	2										
2731	3										
		ST	1		NR						Sampled at 0832.
2730	4	SPT	1	4 5 7	NR						Sampled at 0840. N = 12
		CAL	2	7 9 13	89		CLAY (CL), stiff, gray, slightly moist		10.0	1805	Sampled at 0846. LL = 35, PI = 11
2729	5										
2728	6						CLAY (CL), stiff to very stiff, gray-brown, silty, some inclusions of slightly to highly weathered shale				

Report GEO_12W_OAK_METRIC; File BTOOTHWY.GPJ; 1/14/2001; 02TH1-03

URS

Project: Beartooth Highway
Project Location: Site 1
Project Number: 68FHAT0027.00

Log of Test Hole TH1-3

Sheet 2 of 2

Elevation, meters	Depth, meters	SAMPLES Type	SAMPLES Number	Blows per 150 mm	Recovery, %	Graphic Log	MATERIAL DESCRIPTION	Well Construction Schematic	Water Content, %	Dry Unit Weight, kg/m³	REMARKS AND OTHER TESTS
	6						CLAY (CL), stiff to very stiff, brown-gray, silty, some inclusions of slightly to highly weathered shale (continued)				
		CAL	3	5 10 14	100						Sampled at 0855.
2727	7										
		CAL	4	7 16 12	100						Sampled at 0907.
2726	8										
2725	9						Becomes very stiff to hard, with few limestone inclusions up to 0.1 m diameter; includes silty, clayey sand with gravel (SC-SM)				
		CAL	5	13 24 14	89				9.3	2098	Sampled at 0915. Nonplastic
2724	10										
		CAL	6	6 23 45	84						Sampled at 0940.
2723	11										
2722	12						GRAVEL to SAND (GP/SP, SW-SM), very dense, gray, saturated, fine- to coarse-grained sand, trace silt				Water encountered at 11.95 m.
		CAL	7	50/70	22				17.0	1607	Sampled at 1000. Nonplastic
2721	13						Bottom of hole at 12.60 m; auger refusal on top of bedrock				Completion at 1200.

Report GEO_12W_OAK_METRIC; File BTOOTHWY.GPJ 1/14/2001 02TH1-03

URS

Project: Beartooth Highway Project Location: Site 1 Project Number: 68FHAT0027.00	**Log of Test Hole TH1-4** Sheet 1 of 2

Date(s) Drilled	8/27/00	Logged By	A. Burns	Checked By	D. Baures
Drilling Method	Hollow-Stem Auger	Drill Bit Size/Type	114-mm-ID auger bit	Total Depth of Borehole	10.80 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Surface Elevation	2727.9 m MSL
Groundwater Level and Date Measured	Not encountered	Sampling Method(s)	SPT, California, Shelby tube	Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop
Borehole Completion	51-mm-dia. PVC screened 4.9-10.8 m in 10/20 sand, bentonite plug 1-4 m, flush-mount cover set in cement grout			Location	Station 39+870, offset 2.5 R

Elevation, meters	Depth, meters	SAMPLES Type	SAMPLES Number	Blows per 150 mm	Recovery, %	Graphic Log	MATERIAL DESCRIPTION	Well Construction Schematic	Water Content, %	Dry Unit Weight, kg/m³	REMARKS AND OTHER TESTS
	0						SAND (SC), clayey, fine- to coarse-grained				Begin drilling at 0955.
							CLAY (CL), stiff, brown, slightly moist, silty, some shale inclusions				
2727	1										
		CAL	1	3 6 5	98						Sampled at 1005.
2726	2										
2725	3	CAL	2	7 6 7	93						Sampled at 1040.
2724	4										
		ST	1		89						Sampled at 1100.
2723	5	SPT	1	50/20	0						Sampled at 1105; spoon refusal.
							CLAY (CL), stiff to very stiff, brown, slightly moist, silty, some shale, limestone, and granite gravel and cobbles				
2722	6										

Report GEO_12W_OAK_METRIC, File: BTOOTHWY.GPJ, 1/14/2001 03TH1-04

URS

Project: Beartooth Highway
Project Location: Site 1
Project Number: 68FHAT0027.00

Log of Test Hole TH1-4

Sheet 2 of 2

Elevation, meters	Depth, meters	SAMPLES Type	Number	Blows per 150 mm	Recovery, %	Graphic Log	MATERIAL DESCRIPTION	Well Construction Schematic	Water Content, %	Dry Unit Weight, kg/m³	REMARKS AND OTHER TESTS
	6						CLAY (CL), brown, very stiff, slightly moist, silty, some shale, limestone, and granite gravel and cobbles; includes clayey gravel with sand (GC)				Sampled at 1111.
			CAL-3	6 10 14	100				7.0	2031	LL = 30, PI = 13
2721	7						SAND (SM/SC), brown, dense, slightly moist, silty and clayey, some inclusions of unweathered limestone, shale, and granite; includes clayey gravel with sand (GC)				
			CAL-4	14 18 23	87				4.8	2129	Sampled at 1125. LL = 35, PI = 14
2720	8										
2719	9										Auger problem; drilling delayed 45 min.
			CAL-5	22 34 39	NR		BOULDERS in matrix of poorly graded sand with silt and gravel (SP-SM) (Possible Bedrock)		4.5	1783	Nonplastic Sampled at 1215.
2718	10										
			CAL-6	50/0	0						Attempted sample at 1250; no penetration.
2717	11						**Bottom of hole at 10.80 m**				Completion at 1420.
2716	12										
2715	13										

Report GEO_12W_OAK_METRIC, File: BTOOTHWY.GPJ; 1/14/2001 03TH1-04

URS

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-1

Sheet 1 of 2

Date(s) Drilled	8/26/00	Logged By	D. Baures	Reviewed By	A. Burns
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	8.60 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2702.6 m MSL
Groundwater Level	Not encountered	Location	Station 41+400, centerline	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	51-mm-dia. PVC with cement grout backfill, flush cover			Hammer Data	Not applicable

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D., %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
	0		1						Asphalt pavement					0840	
									GRAVEL, sandy [Road Base]						
									GRAVEL, COBBLES, and BOULDERS, brown, sandy matrix, maximum 0.2 m cored length						
2702		1		52		NA									
	1				1		1		GRANITE and GNEISS, moderately strong to very strong, pink to black, coarse-grained, slightly weathered [Bedrock]						
					1				1: 25°, J, N, Fe, Sp, Pl, SR					0849	
														0854	
					2		1		1: 50°, J, N, Fe, Su, Ir, R-VR						
2701					1										
	2	2		100	2	68	2		2: 20°, J, VN, Fe, Su, Ir, VR						
					3		3		3: 70°, J, N, Fe, Su-Sp, Wa-Ir, VR						
							4		4: 40°, J, N, Fe, Fi, Wa-Ir, VR						
2700					1										
	3						M		Possible weathered joint; no recovery					0903	
					3									0906	No recovery in upper 0.12 m; possibly a boulder or weathered joint.
							1		1: 40°, J, N, Fe, Su, Wa, R						
		3	2	92	1	79									
							2		2: 75°, J, N, Fe, Su, Ir, VR						
2699					1										
	4						M								

Report: GEO CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 04THB-01

URS

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-1

Sheet 2 of 2

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4		2		0				GRANITE and GNEISS, moderately strong to very strong, pink to black, coarse-grained, slightly weathered [Bedrock] (continued)					
	3		92	0	79								PLI = 1064 CS = 200 MPa
						M						0920 0925	
2698				0									
5				0									
	4		70	0	70								
				1		1		1: 25°, J, N, Fe, Su, Wa-Ir, VR					Lower 0.37 m of Run 4 left in hole.
2697												0944 0951	Polished bit.
6				2		1		1: 40°, J, N, Fe, Su, Wa, R					
		3		1									
	5		100	1	80	2		2: 45°, J, VN, Qz, Fi, Wa, SR					PLI = 999 CS = 187 MPa
2696				1		3		3: 45°, J, VN, Fe, Su-Sp, Wa, SR					Recovered 0.32 m from Run 4 with Run 5.
7				3		4		4: 50°, J, N, Fe+Mn, Fi, Wa, SR-R				1010 1129	
						1		1: 20°, J, N, Fe, Su, Wa, R					
				5		1							
						2		2: 45°, J, VN, Fe, Su-Sp, Ir, VR					
						3		3: 20°, J, N, Fe, Su, Wa, R					
				3		3							
2695	6		100		72								UCS = 57.2 MPa PLI = 179 CS = 34 MPa
8				3									
				1									
						4		4: 40°, J, N, Fe, Su, Wa-Ir, R-VR					
				3								1144	
2694								Bottom of hole at 8.60 m					
9													

Report: GEO_CORE+SOIL_17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 04THB-01

URS

Project: Beartooth Highway Project Location: Beartooth Lake Bridge Project Number: 68FHAT0027.00	**Log of Test Hole THB-2** Sheet 1 of 3

Date(s) Drilled	8/27/00	Logged By	D. Baures	Reviewed By	A. Burns
Drilling Method	NQ Wireline Core	Drill Bit Size/Type	NQ2 diamond-impregnated	Total Depth of Test Hole	11.00 m
Drill Rig Type	Diedrich D-25 (skid-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2688.6 m MSL
Groundwater Level	Not encountered	Location	Station 41+443, centerline	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	51-mm-dia. PVC with cement grout backfill, 0.4-m stickup			Hammer Data	63.5 kg / 762-mm drop

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
0		1						GRAVEL, COBBLES, and BOULDERS, medium dense, brown, sandy matrix, maximum 0.3 m cored length [Talus / Colluvium]					1002	
2688														
1	1		21		NA									
2687													1027	
2													1035	
	2		10		NA								1042	
											19		1046	
	3		40		NA				SPT-1		12	29		N = 24
2686											12			Lost circulation at about 2.6 m.
													1058	Core barrel blocked.
3													1100	
	4		62		NA									50% water return.
2685													1118	
4	5		31		NA				SPT-2		14	24	1124	N = 29

Report: GEO CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 05THB-02

URS

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-2

Sheet 2 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4		1						GRAVEL, COBBLES, and BOULDERS, medium dense, brown, sandy matrix, maximum 0.3 m cored length [Talus / Colluvium]	SPT-2	16 13	24		N = 29
	5		31		NA							1136	
2684												1144	60% water return.
	6		82		NA								
5												1158	
		2							SPT-3	15 50/ 100	35	1205	
2683						1		GRANITE and GNEISS, moderately strong to very strong, pink to black, coarse-grained, slightly weathered [Bedrock]					PLI = 836 CS = 132 MPa
				2		2		1: 45°, J, N, Fe, Su, Ir, VR 2: 20°, J, VN, Fe, Su, Pl, SR					
6	7		85	1	60	3		3: 60°, J, VN, Fe, Su, Ir, R					
				1									30% water return.
2682				1		4		4: 55°, J, N, Fe, Su, Ir, R				1241	
						1		1: 60°, J, VN, Fe+Mn, Su, Pl, R				1245	
7				3		1							PLI = 1040 CS = 165 MPa
				3		2		2: 15°, J, VN, Qz, Fi, Pl, SR					
						3		3: 70°, J, N, Fe, Su, Ir, R					
2681	8		100	2	50								25% water return.
				2		4		4: 20°, J, VN, Fe+Qz, Su-Fi, Pl, SR				1317	
8				7		4						1342	Core barrel swelled.
		3				4							
				3		1		1: 20°, J, N, Fe, Su, Wa, R					
2680	9		100		72	1							
				3		2		2: 40°, J, VN, Fe, Su, Pl-Wa, R					
9													

Report GEO_CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 05THB-02

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-2

Sheet 3 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
9		3		1				GRANITE and GNEISS, moderately strong to very strong, pink to black, coarse-grained, slightly weathered [Bedrock]						
	9		100	2	72	3		3: 50°, J, VN, Fe, Su, Wa-Ir, R						50% water return.
2679				2		4		4: 25°, J, VN, Fe, Su, Wa, R					1413 1420	
10				2		M								
						1		1: 40°, J, VN, Fe, Su, Wa, R						
						2		2: 10°, J, VN, Fe, Su, Pl, SR						
	10		100	3	88	1								
2678				3		3		3: 65°, J, VN, Fe+Mn, Su-Sp, Ir, R						
		4				1								UCS = 37.0 MPa *
						2								PLI = 362 CS = 58 MPa
11				2									1451	
								Bottom of hole at 11.00 m						
2677														
12														
2676														
13														
2675														
14														

Report: GED CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 05THB-02

URS

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-4

Sheet 1 of 3

Date(s) Drilled	8/24/00	Logged By	D. Baures	Reviewed By	A. Burns
Drilling Method	NQ Wireline Core	Drill Bit Size/Type	NQ2 diamond-impregnated	Total Depth of Test Hole	10.00 m
Drill Rig Type	Diedrich D-25 (skid-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2687.5 m MSL
Groundwater Level	Not encountered	Location	Station 41+495, centerline	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	51-mm-dia. PVC with cement grout backfill, flush cover			Hammer Data	63.5 kg / 762-mm drop

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
0		1						GRAVEL, COBBLES, and BOULDERS, medium dense to dense, red-brown, sandy matrix, maximum 0.5 m cored length [Talus]				0942	
2687													Lost circulation at about 0.3 m.
1	1		43		NA								90% water return.
2686												1014	
2									SPT-1	4 8 50/50	29	1030	
2685													100% water return.
	2		35		NA								Core barrel blocked.
3												1047	
2684												1059	25% water return.
	3		31		NA				SPT-2	8 12 16	55		N = 28
4													

Report: GEO_CORE+SOIL_17_METRIC, File: BTOOTHWY.GPJ, 1/16/2001 06THB-04

URS

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-4

Sheet 2 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4		1						GRAVEL, COBBLES, and BOULDERS, medium dense to dense, red-brown, sandy matrix, maximum 0.5 m cored length [Talus] (continued)						
2683	3		31		NA									
				2				GRANITE, strong to very strong, pink, coarse-grained, slightly weathered [Bedrock]					1120	
5													1133	No SPT taken; core run ended in boulder.
				2		1		1: 30°, J, N, Fe+Cl, Fl, Pl-Wa, SR-R						
2682		2		2										
	4		100	3	84	1,1 2		2: 70°, J, N, Fe, Sp, Wa-Ir, R						
6				2		1								
				2										
2681						1							1210	
	5		100	1	100	1		1: 55°, J, N, Fe, Fl, Wa, R					1220	
													1234	Changed bit.
7				2		1		1: 25°, J, N, Fe, Sp, Wa, R					1251	
						2		2: 40°, J, N, Fe, Sp-Fi, Wa, VR						UCS = 86.3 MPa *
	6		100	3	63									
2680						3,3		3: 20°, J, VN, Fe, Su-Sp, Wa, R						
		3		4		3 3								
	7		100		100	1		1: 30°, J, N, Fe, Su, Wa, R					1328 1335	
8				4		1,1 1,1		1: 30°, J, N, Fe, Su-Sp, Pl-Wa, SR-R					1401	Changed bit.
				5		1								
	8		100		48	1,1								
2679				3		1								PLI = 468 CS = 74 MPa
				2		1								
9						2		2: 15°, J, N, Fe+Cl, Fl, Pl-Wa, SR						

Report: GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/16/2001 05THB-04

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-4

Sheet 3 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
9								GRANITE, strong to very strong, pink, coarse-grained, slightly weathered (Bedrock) (continued)					
	8	3	100	2	48	1						1450	
				3		1						1500	
2678													
	9		73	2	64								PLI = 879 CS = 108 MPa
						1		1: 25°, J, VN, Fe, Su, Pl, SR				1520	
10								Bottom of hole at 10.00 m					
2677													
11													
2676													
12													
2675													
13													
2674													
14													

Report: GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/16/2001 06THB-04

URS

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-5

Sheet 1 of 3

Date(s) Drilled	8/26/00	Logged By	D. Baures	Reviewed By	A. Burns
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	11.30 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2694.2 m MSL
Groundwater Level	Not encountered	Location	Station 41+540, centerline	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	51-mm-dia. PVC with cement grout backfill, flush cover			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
0 / 2694		1						GRAVEL, brown, sandy (Road Base)					0832	
								GRAVEL, COBBLES, and BOULDERS, medium dense, red-brown, sandy matrix, maximum 0.2 m cored length (Fill)						
1 / 2693	1		34		NA									
													0846	
											8		0851	Lost circulation at about 1.6 m.
									SPT-1		15	18		N = 24
											9			
2 / 2692	2		29		NA									
3 / 2691								Becomes dense					0859	
											5		0908	
									SPT-2		23	36		N = 46
											23			
	3		65		NA									
4														

Report GED_CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 07THB-05

URS

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-5

Sheet 2 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4		1						GRAVEL, COBBLES, and BOULDERS, dense, red-brown, sandy matrix, maximum 0.2 m cored length [Fill] (continued)					
2690	3		65		NA								
									SPT-3	50/0	0	0916 0922	SPT attempted; refusal.
5													
2689	4		59		NA								
6												0932 0939	No SPT; in bedrock. UCS = 70.5 MPa *
2688		2		6		1		GRANITE, strong to very strong, pink, coarse-grained, slightly weathered [Bedrock]					
				12		2		1: 80°, J, N, Fe, Su, Ir, R					
								2: 45°, J, T, Fe+Qz, Fi, Wa-Ir, R					
	5		98	5	38	3		3: 50°, J, VN, Fe, Fi, Ir, VR					
7				5		4,4 4		4: 10°, J, N, Fe, Su, Wa, R					
2687						5		5: 80°, J, N, Fe, Su, Wa, R					
				3		6 6		6: 45°, J, N, Fe, Su, Pl-Wa, SR					
												0953 1009	
				1		7		7: 55°, J, N, Fe, Sp, Wa-Ir, R					
8				1									
2686						1		1: 40°, J, N, Fe, Sp, Wa, R					
	6		100	2	100	2		2: 15°, J, N, Fe, Su-Sp, Wa, R					
		3		2									PLI = 1070 CS = 200 MPa
				2		3		3: 85°, J, VN, Fe, Su, Wa, SR					
9						M							

Report: GEO CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 07THB-05

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-5

Sheet 3 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
9	6	3	100	2	100	M		GRANITE, strong to very strong, pink, coarse-grained, slightly weathered [Bedrock] (continued)				1121	
2685												1130	
	7		100	3	100								
						M						1153	Bit polished.
	8		100		100							1159	
				2		M						1207	
												1246	Changed bit.
10				1		1		1: 45°, J, N, Fe, Su, Wa, R					
2684	9		100	0	100								
				1									PLI = 720
						1						1301	CS = 134 MPa
						1						1350	
				1		1		1: 40°, J, VN, Fe, Su, Wa, R					
11	10		100		91								
2683				1		M						1405	
								Bottom of hole at 11.30 m					
12													
2682													
13													
2681													
14													

Report: GEO_CORE+SOIL 17 METRIC; File: BTCOTHWY.GPJ; 1/14/2001 07THB-05

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-1

Sheet 1 of 2

Date(s) Drilled	8/24/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	7.40 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2693.1 m MSL
Groundwater Level	Not encountered	Location	Station 41+610, centerline	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	51-mm-dia. PVC with cement grout backfill, flush cover			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
0 / 2693		1						Asphalt pavement					0830	
								GRAVEL, gray, sandy (Road Base)						
								GRAVEL, COBBLE, and BOULDERS, medium dense, gray, sandy matrix, maximum 0.3 m cored length (Fill)						
	1		24		NA									
1 / 2692														
													0850	
									SPT	1	2 8 4	11		N = 12
													0855	
2 / 2691	2		25		NA									
3 / 2690				3		1		GRANITE, moderately strong to very strong, pink, gray, and black, coarse-grained, slightly weathered (Bedrock)			7		0905	
	3		87	3	51	M		1: 70°, J, N, Fe, Su, Wa, SR	SPT	2	7 7 4	24		N = 11
						2		2: 20-80°, J, MW, Fe, Sp, Wa, SR					0910	
				2										
4						3		3: 40°, J, W, Cl, Sp, Wa, R						

Report: GEO CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 08TH2-01

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-1

Sheet 2 of 2

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	RQD, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4 / 2689		1		5		4		GRANITE, moderately strong to very strong, pink, gray, and black, coarse-grained, slightly weathered [Bedrock] (continued)					
	3		87	4	51			4: 35°, J, N, Fe, Su, Pl, SR					
						1		1: 60°, J, N, Fe, Su, Wa, R				0920	
				3		2		2: 60°, J, N, Fe, Fi, Wa, R					
5 / 2688		2		3		3,3		3: 15°, J, N, Fe, Su, Pl, SR					
	4		100	0	90	3							
				1		4		4: 15°, J, N, Fe, Pa, Wa, R					
						M							
						5		5: 15°, J, N, Cl, Fi, Pl, SR					
						6		6: 50°, J, N, Fe, Pa, Wa, R					
6 / 2687				2		M						1020	
				2		1		1: 40°, J, N, Fe, Fi, Pl, SR					
				1									
						2		2: 50°, J, N, Fe, Pa, Wa, R					
	5		100	0	80	M							
						M							
7 / 2686				2		3		3: 15°, J, N, Fe, Su, Pl, SR					
				3		3							
				1		4		4: 50°, J, N, Fe, Su, Pl, SR				1050	
8 / 2685								Bottom of hole at 7.40 m					Casing completed at 1400.
9													

Report: GEO_CORE+SOIL_17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 08TH2-01

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-2

Sheet 1 of 2

Date(s) Drilled	8/25/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	NQ Wireline Core	Drill Bit Size/Type	NQ2 diamond-impregnated	Total Depth of Test Hole	8.55 m
Drill Rig Type	Diedrich D-25 (skid-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2691.7 m MSL
Groundwater Level	Not encountered	Location	Station 41+610, offset 3.5 R	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	51-mm-dia. PVC with cement grout backfill, flush cover			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
0 / 2691		1						GRAVEL, COBBLES, and BOULDERS, brown, sandy clay matrix, maximum 0.2 m cored length					0740	
1 / 2690	1		57		NA			GRAVEL, COBBLES, and BOULDERS, loose to medium dense, brown, sandy matrix, maximum 0.2 m cored length			7 3 2		0810 0815	N = 5
									SPT-1			11		
2 / 2689	2		20		NA									
3 / 2688	3		20		NA						3 7 5		0830 0835	N = 12
4									SPT-2			11		

Report GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 09TH2-02

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-2

Sheet 2 of 2

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4	3	1	20		NA			GRAVEL, COBBLES, and BOULDERS, loose, red-brown, sandy matrix, maximum 0.3 m cored length						
											5		0850	
										SPT-3	3	4		N = 6
2687											3		0855	
5	4		60	2	31	1		GRANITE, strong to very strong, pink, coarse-grained, slightly weathered [Bedrock] 1: 45°, J, N, Cl, Pa, Wa, SR						
				2		2		2: 60°, J, N, Fe, Su, Pl, SR						
2686				2		3		3: 20°, J, N, Fe, Su, Pl, SR						
				4		1		1: 45°, J, N, Cl, Pa, Pl, SR					0912	
6		2				1								
				3		1								
	5		90	2	34	1								
						2		2: 50°, J, N, Cl, Pa, Pl, SR						
2685				3		3		3: 20°, J, N, Fe, Su, Wa, SR						
7				4		4		4: 30°, J, N, Cl, Pa, Wa, R						
				2		1		1: 20°, J, MW, Cl, Fi, Wa, R					0950	PLI = 924 CS = 147 MPa
				2		2		2: 25°, J, MW, Cl, Pa, Wa, SR						
2684						3		3: 15°, J, W Cl, Pa, Pl, SR						
	6		100	1	88									UCS = 90.0 MPa
8						4		4: 40°, J, N, Fe, Su, Pl, SR						
				1										PLI = 1247 CS = 198 MPa
						5		5: 70°, J, T, Fe, Fi, Wa, R					1030	
2683								Bottom of hole at 8.65 m						
9														

Report: GEO_CORE+SOIL 17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 08TH2-02

URS

Project: Beartooth Highway Project Location: Site 2 Project Number: 68FHAT0027.00	**Log of Test Hole TH2-3** Sheet 1 of 3

Date(s) Drilled	8/22/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	10.67 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2694.6 m MSL
Groundwater Level	Not encountered	Location	Station 41+660, offset 1.7 L	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	51-mm-dia. PVC with cement grout backfill, flush cover			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
0		1						Asphalt pavement					0815	
								GRAVEL, gray, sandy [Road Base]					0905	Downtime to mix grout for other holes.
								GRAVEL, COBBLES, and BOULDERS, medium dense, red-brown, sandy matrix, maximum 0.3 m cored length [Fill]						
2694	1		17		NA									Lost circulation at about 0.9 m.
1													0920	
2693									SPT-1		15 6 5	33		N = 11
2													0931	
	2		47		NA									
2692														
3													0950	No SPT taken; core run ended in large boulder.
	3		60		NA									
2691														
4														

Report GEO CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 10TH2-03

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-3

Sheet 2 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	RQD, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4		1						GRAVEL, COBBLES, and BOULDERS, medium dense, red-brown, sandy matrix, maximum 0.3 m cored length (Fill) (continued)					
	3		60		NA								
2690										4		1000	
									SPT-2	9	27		N = 15
										6		1007	
5													
	4		27		NA								
2689													
6													
		2										1020	No SPT taken; core run ended in large boulder.
2688								GRAVEL, COBBLES, and BOULDERS, brown, sandy matrix to no matrix, maximum 0.3 m cored length (Talus)					
	5		50		NA								
7													
						M		GRANITE to GNEISS, strong to very strong, pink to black, coarse-grained, slightly to highly weathered (Bedrock)					
2687				8		1		1: 60°, J, N, Fe, Su, Wa, R	SPT-3	6	33	1028	
										50/100			
8				9									
	6		97	1	59	2		2: 50°, J, N, Fe, Su, Wa, R					UCS = 77.6 MPa
						M							
2686				1									PLI = 855
						3		3: 30°, J, N, Fe, Sp, Pl, SR					CS = 160 MPa
				1									
9													

Report GEO CORE+SOIL_17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 10TH2-03

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-3

Sheet 3 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	RQD, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
9		3		2		1		GRANITE to GNEISS, strong to very strong, pink to black, coarse-grained, slightly to highly weathered [Bedrock] (continued)				1055	
								1: 20°, J, N, Fe, Su, Wa, R					
				3		2							
2685								2: 25°, J, N, Fe, Sp, Pl, SR					PLI = 1002
						M							CS = 187 MPa
	7		97	0	93	M							
10						M							
				2									
						3		3: 60°, J, N, Fe, Sp, Wa, R					
				2									
2684						4		4: 70°, J, N, Fe, Su, Wa, SR				1140	
								Bottom of hole at 10.67 m					
11													
2683													
12													
2682													
13													
2681													
14													

Report GEO_CORE+SOIL_17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 10TH2-03

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-4

Sheet 1 of 3

Date(s) Drilled	8/21/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	12.45 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2696.5 m MSL
Groundwater Level	Not encountered	Location	Station 41+700, offset 1.5 L	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	51-mm-dia. PVC with cement grout backfill, flush cover			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
0		1						Asphalt pavement				1039	
								GRAVEL, brown, sandy [Road Base]					
2696								GRAVEL, COBBLES, and BOULDERS, medium dense, red-brown, sandy matrix, maximum 0.35 m cored length [Fill]					
	1		37		NA								
1													
2695									SPT-1	5 6 12	2	1045 1100	N = 18
2	2		32		NA								
2694													
3									SPT-2	4 11 6	20	1111 1118	N = 17
2693	3		48		NA								
4													

Report GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 11TH2-04

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-4

Sheet 2 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D. %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4		1						GRAVEL, COBBLES, and BOULDERS, medium dense, red-brown, sandy matrix, maximum 0.35 m cored length [Fill] (continued)						
	3		48		NA									
2692											8		1128	
									SPT	3	14	24		N = 24
											10		1138	
5	4		61		NA									
2691								GRAVEL, COBBLES, and BOULDERS, medium dense, red-brown to black, sandy matrix to no matrix, maximum 0.5 m cored length [Talus]						
		2												
6											4		1149	
									SPT	4	4	31		N = 11
2690											7		1158	
	5		41		NA									
7														
2689													1202	No SPT taken; core
													1209	run ended in boulder.
8														
	6		31		NA									
2688								Becomes predominantly boulders, minor sand and gravel						
9														

Report GEO CORE+SOIL 17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 11TH2-04

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-4

Sheet 3 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
9	6	2	31		NA			BOULDERS, red-brown to black, minor sand and gravel matrix, maximum 0.5 m cored length [Talus]	SPT-S		50/50	0	1216 1253	
2687				1				GRANITE, moderately strong to very strong, pink, coarse-grained, slightly weathered [Bedrock]						
10	7		94	4	37	1		1: 85°, J, VN, Fe, Su, Wa-St, R						
						2		2: 45°, J, N, Fe, Su, Wa, R						
				2		3		3: 60°, J, N, Fe, Su, Wa, R						
2686		3		1									1308 1340	
				2		1		1: 85°, J, N, Fe, Su-Sp, Wa, R						
11						2		2: 45°, J, N, Fe, Su, Ir, VR						
	8		100	4	87	2								
						M								
2685				1										
						3		3: 45°, J, VN, Fe, Fl, Wa, R					1358	
				1		M							1404	Core barrel blocked.
12	9		100		100	1		1: 70°, J, N, Fe, Fl, Wa-St, R-VR						
				1		M							1417 1420	
	10		100		100	1		1: 25°, J, N, Fe, Su, Wa, R						
						M							1438	
2684								Bottom of hole at 12.45 m						
13														
2683														
14														

Report: GEO_CORE+SOIL_17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 11TH2-04

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Te[illegible] TH2-5
Sh[illegible]

Date(s) Drilled	8/25/00 and 8/26/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	NQ Wireline Core	Drill Bit Size/Type	NQ2 diamond-impregnated	Total Depth of Test Hole	14.00 m
Drill Rig Type	Diedrich D-25 (skid-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2695.6 m MSL
Groundwater Level	Not encountered	Location	Station 41+700, offset 2.5 R	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	51-mm-dia. PVC with cement grout backfill, flush cover			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
0 / 2695		1						GRAVEL, COBBLES, and BOULDERS, loose to medium dense, brown, coarse-grained sand matrix, maximum 0.3 m cored length					1605	100% water return.
	1		10		NA									
1 / 2694											2		1615	
									SPT-1		1	2		N = 3
											2		1620	
	2		45		NA									50% water return.
2 / 2693								Becomes medium dense to dense			25		1700	Drillers cased hole to 2 m; ended drilling for 8/25/00.
									SPT-2		18	4		N = 35
3 / 2692											17		0830	Resumed drilling on 8/26/00.
	3		55		NA									100% water return.
4													0920	

Report GEO CORE+SOIL 17 METRIC, File: BTOOTHWY.GPJ; 1/14/2001 12TH2-05

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-5

Sheet 2 of 3

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
	4		1						GRAVEL, COBBLES, and BOULDERS, medium dense to dense, brown, coarse-grained sand matrix, maximum 0.3 m cored length (continued)					0950	No SPT taken; refusal on rock.
2691		4		93		NA									
	5		2												
2690														1030	No SPT taken; refusal on rock.
	6	5		65		NA									
2689															
	7											6		1105	
										SPT	3	11	NA		N = 27
2688												16		1125	
	8	6		35		NA									
2687										SPT	4	50/10	100	1150	
	9	7		51		NA								1155	

Report: GEO_CORE+SOIL_17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 12TH2-05

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-5

Sheet 3 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
9		2						GRAVEL, COBBLES, and BOULDERS, medium dense to dense, brown, coarse-grained sand matrix, maximum 0.3 m cored length (continued)					80% water return.
2686	7		51		NA								
10		3										1220	No SPT; core run ended in boulder.
2685								GRANITE, moderately strong to very strong, pink and black, coarse-grained, slightly weathered (Bedrock)					
	8		58	0	40								
11				1		1 M		1: 75°, J, N, Fe, Su, Wa, SR					UCS = 51.4 MPa
2684				1								1335	
				1		1		1: 80°, J, N, Fe, Pa, Pl, SR					
12						2		2: 45°, J, N, Fe, Su, Wa, SR					
	9		79	3	64	3		3: 15°, J, N, Fe, Su, Wa, R					
						4		4: 45°, J, N, Cl, Pa, Pl, R					
				1									PLI = 1250 CS = 198 MPa
2683						M						1515	
				1		1		1: 45°, J, W, Fe, Su, Wa, R					
13				0		M							
	10	4	100	2	68	2		2: 40°, J, N, Fe, Pa, Pl, SR					PLI = 1435 CS = 229 MPa
2682				1		M							
						M							
						3		3: 60°, J, N, Fe, Su, Pl, SR					
				2		4		4: 40°, J, N, Fe, Pa, Pl, SR					
14								Bottom of hole at 14.00 m				1615	

Report: GEO_CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 12TH2-05

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-6

Sheet 1 of 3

Date(s) Drilled	8/20/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	10.65 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2699.4 m MSL
Groundwater Level	Not encountered	Location	Station 41+765, centerline	Inclination from Horizontal/Bearing	88° (to right)
Borehole Completion	51-mm-dia. PVC with cement grout backfill, flush cover			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
0		1						Asphalt pavement					0819	
								GRAVEL, brown, sandy [Road Base]						
2699								GRAVEL, COBBLES, and BOULDERS (GP), medium dense, red-brown, sandy matrix, maximum 0.4 m cored length (*Fill*)						
	1		36		NA									Lost circulation at about 0.8 m.
1														
2698													0832	
									SPT-1		50/100	0	0843	Drove rock with SPT.
2	2		71		NA			Boulders 0.3 to 0.5 m cored length, little to no matrix						
2697														
3													0904	
									SPT-2		4 7 11	24		N = 18
2696	3		23		NA								0918	
4														

Report: GEO CORE+SOIL 17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 15:TH2-06

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-6

Sheet 2 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4		1						GRAVEL, COBBLES, and BOULDERS (GP), medium dense, red-brown, sandy matrix, maximum 0.4 m cored length [Fill] (continued)						
	3		23		NA									
2695											14		0929	
									SPT	3	14	0		N = 23
											9		0942	
5								GRAVEL, COBBLES, and BOULDERS, medium dense, gray, sandy matrix to no matrix, maximum 0.3 m cored length [Talus]						
	4		23		NA									
2694														
6														
		2									6		0954	
									SPT	4	4	7		N = 10
2693											6		1033	
	5		31		NA									
7														
													1044	
2692									SPT	5	50/70	2	1132	
				2				GRANITE and GNEISS, moderately strong to very strong, pink to black, coarse-grained, slightly to highly weathered [Bedrock]						
				6		1		1: 40°, J, N, Fe, Sp, Pl, SR						
8	6		100	3	62	2		2: 60°, J, VN, Fe, Fi, Wa, R						
2691				9		3		3: 30°, J, N, Fe, Su, Ir, R						
				6		4		4: 20°, J, VN, Fe, Su, Pl, SR					1144	PLI = 645 CS = 121 MPa
9														

Report GEO CORE+SOIL_17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 13TH2-06

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-6

Sheet 3 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D. %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
9		2		0				GRANITE and GNEISS, moderately strong to very strong, pink to black, coarse-grained, slightly to highly weathered (Bedrock) (continued)						PLI = 349 CS = 66 MPa
2690		3		0										
	7		98	1	88									
10				2		1		1: 40°, J, N, Fe, Fl, Pl-Wa, SR						
2689				2		1 1								UCS = 42.7 MPa
				2		1.1							1203	
								Bottom of hole at 10.65 m						
11														
2688														
12														
2687														
13														
2686														
14														

Report: GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 13TH2-06

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-7

Sheet 1 of 3

Date(s) Drilled	8/28/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	NQ Wireline Core	Drill Bit Size/Type	NQ2 diamond-impregnated	Total Depth of Test Hole	11.00 m
Drill Rig Type	Diedrich D-25 (skid-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2698.0 m MSL
Groundwater Level	Not encountered	Location	Station 41+800, offset 7.3 R	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	61-mm-dia. PVC with cement grout backfill, flush cover			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	RQD, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
2698	0		1						BOULDERS, red-brown, gravelly sand matrix to no matrix, maximum 0.5 m cored length (Fill)					0745	
															Lost circulation at about 0.5 m.
2697	1	1		60		NA									
2696	2													0833	
														0841	No SPT taken; core run ended in boulder.
2695	3	2		85		NA									
														0920	
		3		95		NA			GRAVEL, COBBLES, and BOULDERS, medium dense, red-brown, sandy matrix, maximum 0.35 m cored length (Talus / Colluvium)					0926	No SPT taken; core run ended in boulder. Core barrel blocked.
		4	2	40		NA								0932	
2694	4														

Report: GEO CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 14TH2-07

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-7

Sheet 2 of 3

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
2694	4		2						GRAVEL, COBBLES, and BOULDERS, medium dense, red-brown, sandy matrix, maximum 0.35 m cored length [Talus / Colluvium] (continued)						
		4		40		NA									
2693	5											5		0959	
		5		48		NA					SPT-1	5	27		N = 19
												14		1010	
2692	6														
		6		43		NA								1032	Core barrel blocked.
														1043	
											SPT-2	50/0	0	1050	SPT attempted;
2691	7													1100	refusal on boulder.
		7		90		NA									
														1119	
									GRANITE, moderately strong to very strong, pink, coarse-grained, slightly weathered [Bedrock]					1126	
2690	8				3		1		1: 45°, J, N, Fe, Sp, Wa, R						UCS = 98.0 MPa
		8		92		0	2		2: 85°, J, N, Fe+Sd, Fi, Wa, R						
					3		3		3: 60°, J, VN, Fe, Su, Wa, SR						
														1139	
			3		3										
		9		100		51	1		1: 85°, J, N, Fe+Sd, Fi, Wa-Ir, R-VR						
					3		2		2: 20°, J, VN, Fe, Su, Wa, R						
2689	9														

Report: GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 14TH2-07

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-7

Sheet 3 of 3

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	RQD, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
2689	9		3		2		3		GRANITE, moderately strong to very strong, pink, coarse-grained, slightly weathered (Bedrock) (continued)						
									3: 45°, J, N, Fe, Sp, Wa, R						
		9		100	2	51									
							4		4: 65°, J, N, Fe+Qz, Sp, Ir, SR						
					6		5		5: 80°, J, N, Fe, Fl, Ir, R					1218	
														1230	
2688	10				2		1		1: 65°, J, N, Fe, Fl, Ir, R						PLI = 517 CS = 82 MPa
							2		2: 40°, J, N, Fe, Sp, Wa, R						
					2		3		3: 15°, J, VN, Fe, Sp, Wa, R						
		10		100		95									
					0										PLI = 1220 CS = 196 MPa
					1									1308	
2687	11								Bottom of hole at 11.00 m						
2686	12														
2685	13														
2684	14														

Report: GEO CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 14TH2-07

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-8

Sheet 1 of 4

Date(s) Drilled	8/19/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	15.60 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2704.4 m MSL
Groundwater Level	Not encountered	Location	Station 41+865, centerline	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	51-mm-dia. PVC with cement grout backfill, flush cover			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D. %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
0		1						Asphalt pavement					0938	
								GRAVEL, gray, sandy (Road Base)						
2704								GRAVEL, COBBLES, and BOULDERS, loose to medium dense, gray, sandy matrix, maximum 0.25 m cored length (Fill)						
	1		0		NA									
1														
2703											4		0945	
									SPT	1	4	53		N = 8
2											4		0958	Lost circulation at about 1.9 m.
2702	2		26		NA									
3											2		1010	
									SPT	2	4	7		N = 12
2701											8		1016	
	3		17		NA									
4														

Report GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 15:TH2-08

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-8

Sheet 2 of 4

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4		1						GRAVEL, COBBLES, and BOULDERS, medium dense, gray, sandy matrix, maximum 0.25 m cored length (Fill)					
	3		17		NA								
2700													
									SPT-3	40 / 50/80	13	1026 / 1034	
5								GRAVEL, COBBLES, and BOULDERS, dense, red-brown, sandy to gravelly matrix, maximum 0.9 m cored length (Talus)					
	4		31		NA								
2699													
6									SPT-4	50/0	0	1043 / 1048	SPT attempted; refusal on boulder.
2698													
	5		33		NA								
7													
2697												1058 / 1103	No SPT taken; core run ended in boulder.
								← Becomes black due to gneiss boulders					
8	6		69		NA								
2696												1112 / 1117	Core barrel blocked.
		2											
	7		62		NA								
9								← Sandy clay lens 4 mm thick				1122	Core barrel blocked.

Report GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 15TH2-08

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-8

Sheet 3 of 4

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
9		2						GRAVEL, COBBLES, and BOULDERS, dense to very dense, black, sandy to gravelly matrix, occasional clay lenses, maximum 0.35 m cored length (Talus) (continued)	SPT-5		50/100	70	1136	
2695														
	8		74		NA									
10														
2694													1150	
													1156	
11														
2693	9		60		NA									
12														
													1208	
2692		3		6				GRANITE and GNEISS, moderately strong to very strong, gray to black, coarse-grained, slightly to highly weathered; gneiss is weaker and more weathered (Bedrock)					1239	
				5		1		1: 25°, J, N, Ch, Su, Wa, R						
13	10		100	2	68	2		2: 70°, J, N, Ch+Cl, Fl, Wa, S-SR						
						3		3: 45°, J, VN, Ch, Su, Wa, R						
2691				4										
				4		4		4: 40°, Fo, T, Ch, Fi, Pl, S					1252	
								Clayey, weathered	SPT-6		50/75	70		
14	11		100	2	100								1311	UCS = 30.5 MPa

Report GEO_CORE+SOIL_17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 15TH2-08

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Project: Beartooth Highway
Project Location: Site 2
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Log of Test Hole TH2-8

Sheet 4 of 4

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D., %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
14		4						GRANITE and GNEISS, moderately strong to very strong, gray to black, coarse-grained, slightly to highly weathered; gneiss is weaker and more weathered (Bedrock) (continued)						PLI = 289 CS = 54 MPa
2690				2		1								
						1		1: 45°, Fo, T, Cl, Fi, Pl, S						
	11		100	1	100									
		5		0		M								
15														
				0									1329	
						M							1336	Polished blt.
2689	12		100	1	100	1		1: 45°, J, VN, Qz, Fi, Pl-Wa, SR					1430	PLI = 1000 CS = 187 MPa
								Bottom of hole at 15.60 m						
16														
2688														
17														
2687														
18														
2686														
19														

Report: GEO_CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 15TH2.08

URS

Project: Beartooth Highway Project Location: Site 2 Project Number: 68FHAT0027.00	**Log of Test Hole TH2-9** Sheet 1 of 2

Date(s) Drilled	8/18/00	Logged By	D. Baures	Reviewed By	A. Burns
Drilling Method	Hollow-Stem Auger to 1.95 m; HQ Wireline Core 1.95-7.6 m	Drill Bit Size/Type	152.4-mm-OD auger bit; HQ diamond-impregnated core bit	Total Depth of Test Hole	7.60 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2706.2 m MSL
Groundwater Level	Not encountered	Location	Station 41+903, offset 1.6 R	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	51-mm-dia. PVC with cement grout backfill, flush cover			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D. %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
0 / 2706								Asphalt pavement						Advanced with hollow-stem auger to 1.95 m.
								GRAVEL, sandy (Road Base)						
								GRAVEL, COBBLES, and BOULDERS, loose to medium dense, brown, sandy matrix, maximum 0.2 m cored length [Fill]						
1 / 2705														
									SPT	1	3 14 10	33		N = 24
2 / 2704	1	1	34		NA								1305	Began coring at 1.95 m; no circulation.
3 / 2703	2		68	1	59				SPT	2	3 3 4	33	1323 1330	N = 7
4								GRANITE, moderately strong to very strong, red-gray, coarse-grained to pegmatitic, slightly weathered [Bedrock]						

Report GEO_CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 16TH2-09

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-9

Sheet 2 of 2

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D. %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4 / 2702	2	1	68	1 3	59	1,1		GRANITE, moderately strong to very strong, red-gray, coarse-grained to pegmatitic, slightly weathered (Bedrock) 1: 15°, J, N, Fe, Su, Pl-St, SR				1341	
				5		1 1,1 1		1: 20°, J, N, Fe, Sp, Pl, SR				1400	
5 / 2701	3	2	100	2 3	71	2 3 3		2: 30°, J, N, Fe, Su, Ir, R 3: 40°, J, VN, Fe, Su, Ir, R					
				1 1		4		4: 85°, J, N, Fe, Sp, Wa, SR					
6 / 2700				5								1415	
				5		1,1		1: 15°, J, VN, Fe, Su, Pl-Wa, SR				1507	
	4		100	2 1	50	2		2: 65°, J, VN, Fe, Fi, Wa, R					
7 / 2699				3		3		3: 15°, J, N, Fe, Sp, Pl, SR				1521	
								Bottom of hole at 7.60 m					
8 / 2698													
9													

Report: GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 16TH2-09

URS

Project: Beartooth Highway Project Location: Site 2 Project Number: 68FHAT0027.00	**Log of Test Hole TH2-11** Sheet 1 of 4

Date(s) Drilled	8/17/00 and 8/18/00	Logged By	D. Baures	Reviewed By	A. Burns
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impreginated	Total Depth of Test Hole	16.90 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2708.2 m MSL
Groundwater Level	~14 m bgs ATD	Location	Station 41+864, offset 2.05 R	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	51-mm-dia. PVC with cement grout backfill, flush cover			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
	0		1						Asphalt pavement					1022	
2708									GRAVEL (Road Base)						
									GRAVEL, COBBLES, and BOULDERS, medium dense to very dense, red-brown, sandy matrix to no matrix, maximum 1.1 m cored length (Fill)						
		1		70		NA									
	1													1036	
										SPT	1	5 7 7	\ 0		N = 14
2707														1056	
	2	2		29		NA									Lost circulation at about 2.7 m.
2706														1106	Core barrel blocked.
	3									SPT	2	50/50	0		
2705		3		20		NA								1140	
														1150	
		4		68		NA								1212	Broke sample head; changed to #6 bit.
	4														

Report GEO_CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 17TH2-11

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-11

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
	4								GRAVEL, COBBLES, and BOULDERS, medium dense to very dense, red-brown, sandy matrix to no matrix, maximum 1.1 m cored length [Fill] (continued)						
2704		4	1	68		NA								1232	
														1240	No SPT taken; core run ended in boulder.
		5	2	76		NA									
	5													1258	
2703									BOULDERS, gray, weathered surfaces, sand and gravel matrix to no matrix, maximum 1.2 m cored length [Talus]					1319	Changed to #8 bit.
		6		87		NA									
	6														
2702											SPT-3	50/0		1340	
														1406	
		7		87		NA									
	7														
2701															
			3											1420	
														1428	No SPT taken; core run ended in boulder.
	8														
2700		8		89		NA									
	9														

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-11

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
9								BOULDERS, gray, weathered surfaces, sand and gravel matrix to no matrix, maximum 1.2 m cored length (Talus) (continued)						
2699	8	3	89		NA				SPT-4		50/0		1438 1444	SPT attempted; refusal.
								BOULDERS, black to red-brown, sandy to clayey matrix, weathered clasts, maximum 1.6 m cored length [Glacial Moraine]						
10	9		89		NA									
2698								Rock becomes black, foliated					1451 1532	Core barrel blocked, bit burned. Tripped out to change bit.
		4												
11														
2697	10		81		NA									
12													1550 1558	
2696	11		87		NA									
													1606 1615	Core barrel blocked.
13	12		89	2	65	1		GRANITE, strong to very strong, gray, coarse-grained, weakly foliated, slightly weathered [Bedrock]						
2695													1621 1630	
		5		3		1		1: 10°, J, N, Fe, Sp, Wa, R						
	13		100		100	1		1: 75°, J, N, Fe, Sp, Wa, R						
				3		2		2: 15°, J, VN, Fe, Sp, St, R						
													1648 0810	Ended for 8/17/00. Resumed on 8/18/00.
14	14		100	1	100	1		1: 15°, J, VN, Fe+Mn, Su, Wa, R						

Report: GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 17TH2-11

URS

Project: Beartooth Highway
Project Location: Site 2
Project Number: 68FHAT0027.00

Log of Test Hole TH2-11

Sheet 4 of 4

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
14		5				M		GRANITE, strong to very strong, gray, coarse-grained, weakly foliated, slightly weathered [Bedrock]					
2694				1		2		2: 15°, J, VN, Fe+Cl, Sp, Ir, R					
				1									PLI = 1054 CS = 199 MPa
	14		100		100	3		3: 25°, J, N, Fe, Su, Wa, R					
				1									
15													
				0									
2693													PLI = 1061 CS = 200 MPa
				1		4		4: 45°, J, VN, Fe, Su, St, R				0840	
				1									
		6				1		1: 40°, J, VN, Fe, Sp, Wa, R					
16				2		2		2: 5°, J, N, Fe, Su, St, R					UCS = 64.5 MPa
	15		100		90	3		3: 10°, J, N, Ch, Sp, Wa, SR					
2692				2		4		4: 30°, J, N, Fe, Sp, Wa, R					
						5		5: 5°, J, VN, Fe+Ch, Su, Pl, SR					
				2									
						6		6: 20°, J, VN, Fe, Sp, Wa, R					
						7		7: 20°, J, N, Fe, Fl, Wa, R				0900	
17								**Bottom of hole at 16.90 m**					
2691													
18													
2690													
19													

Report: GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 17TH2-11

URS

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-6

Sheet 1 of 5

Date(s) Drilled	9/13/00 and 9/15/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	23.35 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2715.1 m MSL
Groundwater Level	~2.3 m bgs ATD	Location	Station 42+238, centerline	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	Holeplug capped with concrete			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
2715	0		1						Asphalt pavement					0850	
									SAND (SW), medium dense, brown to gray, well-graded, some gravel and cobbles, occasional boulder up to to 0.3 m cored length						
		1		29		NA									
2714	1											5		0902	
										SPT-1		9	94		N = 26
												17		0908	
2713	2								Boulder about 300 mm diameter						
		2		40		NA									Water encountered at approx. 2.3 m.
2712	3													0916	
														0925	SPT-2 not taken; in boulder.
		3		9		NA									
	4														

Report GEO_CORE+SOIL_17 METRIC File: BTOOTHWY.GPJ; 1/14/2001 18THB-06

URS

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-6

Sheet 2 of 5

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q.D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4 / 2711	3	1	9		NA			SAND (SW), medium dense, brown to gray, well-graded, some gravel and cobbles, occasional boulder up to to 0.3 m cored length (continued)					
								Becomes loose	SPT-3	6 3 5	0	0930 0944	N = 8
5 / 2710	4		0		NA			SAND (SP), loose to medium dense, medium- to coarse-grained, no fines, no cohesion					Heaving sand.
6 / 2709									SPT-4	6 6 7	100	0948 0957	N = 13
	5		0		NA								
7 / 2708									SPT-5	1 2 7	89	1004 1010	N = 9
8 / 2707	6		0		NA								
9													

Report GEO CORE+SOIL 17 METRIC; File BTOOTHWY.GPJ; 1/14/2001 16THB-06

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-6

Sheet 3 of 5

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
9 / 2706	6	1	0		NA			SAND (SW), medium dense, fine- to coarse-grained, few gravel or cobbles, no cohesion					
									SPT-6	7 7 6	22	1017 1045	Heaving sand. N = 13
10 / 2705	7		0		NA								
									SPT-7	8 7 11	18	1049 1101	N = 18
11 / 2704	8		3		NA							1121	Heaving sand.
								SILT (ML), medium dense, gray, clayey					
12 / 2703									SPT-8	6 6 10	78	1126 1138	N = 16
13 / 2702	9		0		NA								
									SPT-9	9 7	89	1144	N = 14
14	10		0		NA								

Report: GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 18THB-06

URS

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-6

Sheet 4 of 5

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
14		1						SILT (ML), medium dense, gray, some sand and clay	SPT-9	7	89		Rig broke down; stopped for 9/13/00.
2701												0925	Resumed drilling on 9/15/00 after 1-1/2 days of no drilling due to inoperable rig.
	10		0		NA								
15													
2700													
										4		0930	
									SPT-10	7	22		N = 15
										8		0938	
16	11		0		NA								
2699													
										5		0945	LL = 23, PI = 5
17									SPT-11	5	89		N = 16
2698										11		0954	
	12		12		NA								
18													
2697													
										4		0959	
									SPT-12	7	89		N = 15
	13		11		NA					8		1015	
19												1032	Heaving sand.

Report: GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 18THB-06

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-6

Sheet 5 of 5

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q.D, %-	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
19 / 2696		1						SILT (ML), medium dense, gray, some sand and clay (continued)						
	13		11		NA									
											5		1037	
20 / 2695										SPT-13	6	89		N = 13
											7		1050	
	14		9		NA									
21 / 2694														
											4		1058	
										SPT-14	5	89		N = 12
											7		1106	
22 / 2693	15		27		NA									
											3		1111	Cc = 38.0
23 / 2692										SPT-15	4	89		N = 10
											6		1121	
24								Bottom of hole at 23.36 m						

Report GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 18THB-06

URS

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-7

Sheet 1 of 7

Date(s) Drilled	8/28/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	32.45 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2716.1 m MSL
Groundwater Level	~3 m bgs ATD	Location	Station 42+378, offset 0.6 R	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	Holeplug and cuttings capped with concrete			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D. %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
2716	0		1						GRAVEL, COBBLES, and BOULDERS, medium dense, brown, slightly weathered to unweathered clasts up to 0.15 m cored length					0755	
		1		40		NA									
2715	1														
												6		0820	
										SPT	1	7	11		N = 13
												6		0830	
2714	2	2		33		NA									
															Water encountered at approx. 3.0 m.
2713	3											7		0845	
										SPT	2	14	13		N = 26
		3		16		NA						12		0850	
	4														

Report: GEO_CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 19THB-07

URS

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-7

Sheet 2 of 7

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
2712	4	3	1	16		NA			GRAVEL, COBBLES, and BOULDERS, medium dense, brown, slightly weathered to unweathered clasts up to 0.15 m cored length (continued)						
												4		0859	
										SPT	3	3	11		N = 7
												4		0904	
2711	5	4		20		NA			SAND (SM/SC), brown, medium-grained, silty to clayey						
									GRAVEL, COBBLES, and BOULDERS, medium dense, brown, slightly weathered to unweathered clasts						
2710	6											5		0918	
										SPT	4	5	2		N = 10
									SAND (SP), loose to medium dense, light brown, coarse-grained, some rounded gravel and cobbles			5		0930	
2709	7	5		22		NA									
			2									6		0940	
										SPT	5	3	0		N = 6
2708	8											3		0946	
		6		28		NA									
	9														

Report: GEO CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 19THB-07

URS

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-7

Sheet 3 of 7

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
9	6	2	28		NA			SAND (SP), medium dense, light brown, coarse-grained, some rounded gravel and cobbles					
2707										6		1002	
									SPT-6	6	11		N = 13
										7		1030	
	7		7		NA								
10													Heaving sands; had to flush hole repeatedly.
2706								SILT (ML), medium dense, brown and gray, clayey					
										7		1120	
									SPT-7	14	89		N = 26
11										12		1140	
2705	8		8		NA								
12													
2704										3		1205	
									SPT-8	5	0		N = 10
										5		1210	
13	9		3		NA								
2703													
	10		23		NA				SPT-9	5	100	1217	LL = 21, PI = 3
14													

Report: GED_CORE+SOIL_17_METRIC File: BTOOTHWY.GPJ; 1/14/2001 18THB-07

URS

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-7

Sheet 4 of 7

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q.D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
2702	14		2						SILT (ML), medium dense, gray, clayey	SPT-9	6 7	100	1225	N = 13
		10		23		NA								
2701	15												1230	
										SPT-10	6 10 9	100		N = 19
													1245	
2700	16	11		27		NA								
													1250	
2699	17									SPT-11	9 10 12	100	1303	N = 22
		12		38		NA								
2698	18												1350	
			3							SPT-12	6 11 11	100		N = 22
		13		35		NA							1400	
	19													

Report GEO CORE+SOIL 17_METRIC File: BTOOTHWY.GPJ 1/14/2001 19THB-07

URS

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-7

Sheet 5 of 7

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D, %-	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
2697	19		3						SILT (ML), medium dense, gray, clayey (continued)						
		13		35		NA								1520	SPT not taken.
2696	20	14		43		NA									
2695	21									SPT	13	9 11 13	100	1630 1535	N = 24
2694	22	15		29		NA									
2693	23													1545	SPT not taken.
		16		24		NA									
	24														

Report GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 19THB-07

URS

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-7

Sheet 6 of 7

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
	24								SILT (ML), medium dense, gray, clayey (continued)					
2692		16	3	24		NA								
											4		1555	
										SPT-14	4	100		N = 11
											7		1600	
	25													
2691		17	4	16		NA								
													1610	SPT not taken.
	26													
2690														
		18		77		NA								
	27													
2689														
													1620	
											4			
									SAND (SM), medium dense, brown, fine-grained, silty	SPT-15	10	100		N = 28
											18		1630	
	28													
2688		19		31		NA								
									GRAVEL, COBBLES, and BOULDERS, brown, fine-grained sand matrix, slightly weathered to unweathered clasts up to 0.13 m cored length					
	29													

Report GEO_CORE+SOIL_17_METRIC File: BTOOTHWY.GPJ; 1/14/2001 19THB-07

URS

Project: Beartooth Highway
Project Location: Beartooth Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-7

Sheet 7 of 7

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
29 / 2687		4						GRAVEL, COBBLES, and BOULDERS, brown, fine-grained sand matrix, slightly weathered to unweathered clasts up to 0.13 m cored length (continued)					1645	SPT not taken; in boulder.
	20		31		NA									
30 / 2686													1710	SPT not taken; in boulder.
31 / 2685	21		NR		NA									
32 / 2684								SAND (SM/SC), dense, gray, fine-grained, silty to clayey			7		1720	
										SPT-16	13	68		N = 34
											21		1750	
33 / 2683								Bottom of hole at 32.45 m						
34														

Report: GEO CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 19THB-07

URS

Project: Beartooth Highway
Project Location: Little Bear Creek Bridge 1
Project Number: 68FHAT0027.00

Log of Test Hole THB-8

Sheet 1 of 3

Date(s) Drilled	8/30/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	11.70 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2838.8 m MSL
Groundwater Level	~2.9 m bgs ATD	Location	Station 45+300, offset 2.7 R	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	Holeplug capped with concrete			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
	0		1						Asphalt pavement					0825	
									SAND, brown, medium-grained [Road Base]						
									GRAVEL, COBBLES, and BOULDERS, loose to medium dense, brown-red, silty to clayey well-graded sand matrix, maximum 0.25 m cored length						
2838		1		37		NA									
	1														
2837										SPT-1	50/0	0		0840 0845	Drove SPT on boulder.
	2	2		31		NA									
2836															Water encountered at approx. 2.9 m.
	3											3		0900	
										SPT-2		2	100		N = 7
												5		0905	
		3		57		NA									
2835															
	4														

Report: GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 20THB-08

URS

Project: Beartooth Highway
Project Location: Little Bear Creek Bridge 1
Project Number: 68FHAT0027.00

Log of Test Hole THB-8

Sheet 2 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q-D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4		1						GRAVEL, COBBLES, and BOULDERS, loose to medium dense, brown-red, silty to clayey well-graded sand matrix, maximum 0.2 m cored length (continued)					
	3		57		NA								
								← Becomes medium dense		10		0915	
2834									SPT-3	10	100		N = 28
5										18		0920	
	4		18		NA			SAND (SM/SC), gray, fine-grained, silty to clayey, some gravel and cobbles					
2833													
6								GRAVEL, COBBLES, and BOULDERS, dense to very dense, brown, silty to clayey well-graded sand matrix, maximum 0.2 m cored length		11		0930	
		2							SPT-4	15	100		N = 39
										24		0940	
2832	5		66		NA								
7													
										14		0950	
2831									SPT-5	24	67		
8										30		0959	
	6		67		NA								
								GNEISS, moderately strong to very strong, gray, medium-grained, slightly weathered [Bedrock]					
2830				2									
9						1		1: 75°, J, N, Cl, Pa, Pl, SR					

Report: GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 20THB-08

URS

Project: Beartooth Highway
Project Location: Little Bear Creek Bridge 1
Project Number: 68FHAT0027.00

Log of Test Hole THB-8

Sheet 3 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q-D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
9	6	2	67	0	NA	1		GNEISS, moderately strong to very strong, gray, medium-grained, slightly weathered [Bedrock] (continued)				1120	
				2		2		1: 60°, J, N, Cl, Su, Pl, R					
				2		2		2: 30°, J, N, Cl, Su, Pl, SR					PLI = 930 CS = 174 MPa
2829		3				2							
10	7		80	2	20	2							
				1		1							
				0								1142	
2828						M							
11				0									UCS = 47.0 MPa
	8		70	1	10	M		1: 45°, J, N, Cl, Su, Pl, SR					
				1		1							PLI = 681 CS = 127 MPa
												1215	
2827								Bottom of hole at 11.70 m					
12													
2826													
13													
2825													
14													

Report: GEO CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 20THB-08

URS

Project: Beartooth Highway
Project Location: Little Bear Creek Bridge 2
Project Number: 68FHAT0027.00

Log of Test Hole THB-11

Sheet 1 of 2

Date(s) Drilled	9/12/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	6.10 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2858.7 m MSL
Groundwater Level	~3.4 m bgs ATD	Location	Station 46+601, offset 5.8 R	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	Hofeplug capped with concrete			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
0		1						SAND (SW), medium dense, brown, well-graded, few gravel and cobbles					1510	
2858 / 1	1		15		NA									
1											8		1520	
2857									SPT	1	14	22		N = 29
2											15		1530	
2	2		5		NA									
2856 / 3								SANDSTONE, moderately strong, gray, medium-grained, thin clay layers up to 10 mm thick spaced about 0.2 m apart [Bedrock]					1540	PLI = 143 CS = 27 MPa
								1: 0°, J, MW, Cl, Pa, Wa, R	SPT	2	38 50/100	11	1545	
	3		73	2	25	1								Water encountered at approx. 3.4 m.
				2		1								
2855				2		1								
4						1		Grades fine- to medium-grained						

Report GEO CORE+SOIL_17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 21:THB-11

URS

Project: Beartooth Highway
Project Location: Little Bear Creek Bridge 2
Project Number: 68FHAT0027.00

Log of Test Hole THB-11

Sheet 2 of 2

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4		1		3		1		SANDSTONE, moderately strong, gray, fine- to medium-grained, thin clay layers up to 10 mm thick spaced about 0.2 m apart [Bedrock] (continued)					PLI = 106 CS = 20 MPa
	3		73	2	25	2 1 1		2: 80°, J, N, Fe, Su, Wa, R				1549 1559	
2854				7				Soft, red-brown, iron oxide-stained, coarse-grained, no clay layers, highly fractured					
5				2		1		1: 0°, J, MW, Cl, Pa, Wa, R					
	4		100	2	56	1							UCS = 31.1 MPa *
				6		1 1							
2853				3				SANDSTONE, moderately strong, red-brown, iron oxide-stained, coarse-grained [Bedrock]					
6												1615	
								Bottom of hole at 6.10 m					
2852													
7													
2851													
8													
2850													
9													

Report: GEO CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 21THB-11

URS

Project: Beartooth Highway Project Location: Fen Mitigation Bridge Project Number: 68FHAT0027.00	**Log of Test Hole THB-12** Sheet 1 of 3

Date(s) Drilled	9/8/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	10.70 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2913.6 m MSL
Groundwater Level	~2.5 m bgs ATD	Location	Station 49+149, offset 4.3 L	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	Holeplug capped with concrete			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
0		1						GRAVEL, COBBLES, and BOULDERS, medium dense, brown, matrix of sand, clay, and silt, maximum 0.2 m cored length					1317	
2913 / 1	1		17		NA									
2912 / 2									SPT-1		7 7 7	67	1323 1326	N = 14
2911 / 3	2		5		NA									Water encountered at approx. 2.5 m.
2910 / 4	3		11		NA				SPT-2		1 8 7	33	1334	N = 15

Report GEO_CORE+SOIL 17 METRIC, File: BTOOTHWY.GPJ; 1/14/2001 22THB-12

URS

Project: Beartooth Highway
Project Location: Fen Mitigation Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-12

Sheet 2 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	RQD, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4	3	1	11		NA			GRAVEL, COBBLES, and BOULDERS, medium dense, brown, matrix of sand, clay, and silt, maximum 0.2 m cored length (continued)				1342	
												1435	SPT not taken; in boulder.
2909													
												1440	Stopped drilling to get water; 40-min. delay due to core problem.
5					NA							1520	Resumed coring.
	4		100										
2908						1		GRANITE, strong to very strong, pink, coarse-grained, slightly weathered [Bedrock]					
				3	47	1							
6						1		1: 70°, J, N, Cl, Pa, Pl, SR					
				3								1530	UCS = 83.8 MPa *
						1		1: 60°, J, N, Cl, Su, Pl, R					
				1		1							
2907						1							
	5		100	0	80	M							
7				0									
		2		0		M							
2906						M							
				5				Highly fractured zone				1542	
						1,1		1: 45°, J, N, Cl, Su, Pl, SR					
				7		1,1							
8						1							
						1							
	6		100	8	9			Highly fractured zone					
2905				8		1							
				3									
9													

Report: GEO CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 22THB-12

URS

Project: Beartooth Highway
Project Location: Fen Mitigation Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-12

Sheet 3 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D. %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
9	6	2	100	1	9	1		GRANITE, strong to very strong, pink, coarse-grained, slightly weathered [Bedrock]					1	
								1: 45°, J, W, Cl, Fl, Pl, SR						PLI = 499 CS = 93 MPa
2904				1		M 2		2: 45°, J, N, Cl, Pa, Pl, SR						
				1		M 2								
	7		100		39	2 2								
10				2										
						3		3: 20°, J, W, Cl, Su, Wa, R						
				1		M M								
		3												
2903				2		2 2								PLI = 536 CS = 100 MPa
								Bottom of hole at 10.70 m						
11														
2902														
12														
2901														
13														
2900														
14														

Report: GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 22THB-12

URS

Project: Beartooth Highway Project Location: Fen Mitigation Bridge Project Number: 68FHAT0027.00	**Log of Test Hole THB-14** Sheet 1 of 3

Date(s) Drilled	9/8/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	13.70 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2913.4 m MSL
Groundwater Level	~1.5 m bgs ATD	Location	Station 49+200, offset 3.4 L	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	Holeplug capped with concrete			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
0 / 2913		1						GRAVEL, COBBLES, and BOULDERS, light brown, sandy matrix, maximum 0.22 m cored length				0945	
1 / 2912	1		40		NA			CLAY (CL), medium stiff, gray, silty, trace gravel					
								▽	SPT-1	5 3 2	22	1000 1010	Water encountered at approx. 1.5 m. N = 5
2 / 2911	2		0		NA								
3 / 2910	3		0		NA				SPT-2	1 1 5	0	1020 1024	N = 6
4													

Report: GEO_CORE+SOIL 17_METRIC, File: BTOOTHWY.GPJ; 1/14/2001 23THB-14

URS

Project: Beartooth Highway
Project Location: Fen Mitigation Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-14

Sheet 2 of 3

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	RQD, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4		1						CLAY (CL), medium stiff, gray, silty, trace gravel (continued)					
2909	3		0		NA								
									SPT-3	2 2 2	68	1030 1035	N = 4
5													
2908	4		10		NA								
6								Becomes stiff					
									SPT-4	5 6 8	0	1039 1045	N = 14
2907	5		27		NA								
7													
2906								Becomes very stiff					
									SPT-5	10 8 12	0	1053 1102	N = 20
8	6		13		NA								
2905													
				1		M		GABBRO, weak to strong, gray to black, slightly to moderately weathered [Bedrock]					
9													

Report GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 23THB-14

URS

Project: Beartooth Highway
Project Location: Fen Mitigation Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-14

Sheet 3 of 3

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
	9	6	1	13	1	NA	1		GABBRO, weak to strong, gray to black, slightly to moderately weathered (Bedrock) (continued)						
							1		1: 10°, J, W, Fe, Su, Wa, R					1109	
2904					4		1,1								UCS = 26.7 MPa *
							1,1								
							1		2: 60°, J, N, Fe, Su, Pl, SR						
					3		2		Becomes slightly weathered						
							2								
	10	7		100		0	M								
					3										
							M								
					1										
2903															PLI = 464 CS = 87 MPa
			2				3		3: 45°, J, N, Fe, Su, Wa, R						
					2									1121	
							1		1: 10°, J, W, Fe, Su, Wa, R						
	11				3										
					10				Highly fractured zone						
2902		8		100		0									
					2		2		2: 45°, J, N, Fe, Su, Wa, R						
					0										
	12														
					1		3		3: 45°, J, N, Fe, Su, Wa, R					1139	
2901					1		M								
							1		1: 45°, J, N, Fe, Su, Wa, R						
					2		1								
							M								
	13	9		100	0	58	M								
					1		1								
2900			3		8				Highly fractured zone					1152	
									Bottom of hole at 13.70 m						
	14														

Report: GEO CORE+SOIL 17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 23:THB-14

URS

Project: Beartooth Highway Project Location: Fen Mitigation Bridge Project Number: 68FHAT0027.00	**Log of Test Hole THB-16a** Sheet 1 of 2

Date(s) Drilled	9/8/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	7.60 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2916.6 m MSL
Groundwater Level	~1.5 m bgs ATD	Location	Station 49+276; offset 1.07 L	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	Holeplug capped with concrete			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
	0		1						GRAVEL, COBBLES, and BOULDERS, medium dense, brown, coarse-grained sand matrix, maximum 0.2 m cored length					0815	
2915		1		21		NA									
	1														
2914											SPT-1	8 7 16	0	0823 0832	Water encountered at approx. 1.5 m. N = 23
	2	2		49	2	NA	1		GRANITE, strong to very strong, black and pink, coarse-grained, slightly weathered [Bedrock] 1: 65°, J, N, Cl, Fi, Wa, SR						
2913					10		M 2.2		Highly fractured zone						
	3								2: 70°, J, N, Fe, Pa, Pl, SR					0840	
		3		100	2	0	1		1: 45°, J, N, Fe, Su, Pl, SR					0850	
2912					4		1								
	4				6		1 1 1 1 1								

Report: GEO_CORE+SOIL_17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 24THB-16a

URS

Project: Beartooth Highway
Project Location: Fen Mitigation Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-16a

Sheet 2 of 2

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q-D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4		1		5		1		GRANITE, strong to very strong, black and pink, coarse-grained, slightly weathered [Bedrock] (continued)					
						2		2: 45°, J, N, Fe, Su, Pl, SR					
	3		100	7	0	1							UCS = 80.4 MPa *
						1							
						1							
2911		2		10				Highly fractured zone				0905	
						M							
5				1									
						1		1: 45°, J, N, Fe, Su, Pl, SR					
						M							
	4		100	1	13	2		2: 50°, J, N, Fe, Su, Pl, SR					
2910				1									
						1							
						2							
6				2		1		GNEISS, strong to very strong, black to gray, medium- to coarse-grained, slightly weathered, mica-rich [Bedrock]				0920	
						1		1: 55°, J, N, CH+Fe, Pa, Wa, SR					PLI = 499 CS = 93 MPa
				1									
2909				2		2		2: 40°, J, N, Fe, Su, Wa, R					
	5		100		11	2							
				2		2							
7						1		GRANITE, strong to very strong, pink, coarse-grained, slightly weathered [Bedrock]					
				2		1							
2908												0935	PLI = 783 CS = 147 MPa
								Bottom of hole at 7.60 m					
8													
2907													
9													

Report GEO CORE+SOIL 17 METRIC File: BTOOTHWY.GPJ 1/14/2001 24THB-16a

URS

Project: Beartooth Highway Project Location: Long Lake Bridge Project Number: 68FHAT0027.00	**Log of Test Hole THB-19** Sheet 1 of 2

Date(s) Drilled	9/6/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	6.00 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2940.9 m MSL
Groundwater Level	~1.3 m bgs ATD	Location	Station 50+262, offset 7.6 L	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	Holeplug capped with concrete			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number / Blows per 150 mm / Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
0		1						Asphalt pavement		1348	
								GRAVEL, COBBLES, and BOULDERS, light brown, sandy matrix, maximum 0.23 m cored length			
	1		40		NA						
2940 / 1								▽			Water encountered at approx. 1.3 m.
										1400	SPT not taken; on rock.
								GRANITE, moderately strong to very strong, black and pink, coarse-grained, slightly weathered [Bedrock]			Top of bedrock based on depth at which drill rig began to core very slowly.
2939 / 2	2		7		0						Poor recovery due to core barrel problems.
2938 / 3				5				Highly fractured zone		1510	SPT not taken; on rock.
	3		100	7	20	1,1		1: 40°, J, N, Cl, Su, Pl, SR			UCS = 41.6 MPa *
				10		1		Highly fractured zone			
2937 / 4											

Report: GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 25THB-19

URS

Project: Beartooth Highway
Project Location: Long Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-19

Sheet 2 of 2

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4								GRANITE, moderately strong to very strong, black and pink, coarse-grained, slightly weathered [Bedrock] (continued0						
	3	2	100	3	20	M								
				0										
				3		1		Highly fractured zone; very high angle joints					1528	
2936				3		1								PLI = 684 CS = 125 MPa
5						M 2		1: 70°, J, N, Fe, Pa, Pl, SR						
	4		100	3	0	2		2: 65°, J, N, Fe, Su, Pl, SR						
				2		M 3		3: 50°, J, N, Fe, Su, Pl, SR						
2935				1										PLI = 635 CS = 119 MPa
6													1616	
								Bottom of hole at 6.00 m						
2934														
7														
2933														
8														
2932														
9														

Report: GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2003 25THB-19

URS

Project: Beartooth Highway Project Location: Long Lake Bridge Project Number: 68FHAT0027.00	**Log of Test Hole THB-20** Sheet 1 of 2

Date(s) Drilled	9/6/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	7.00 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	2941.0 m MSL
Groundwater Level	~1.3 m bgs ATD	Location	Station 50+285, offset 6.7 L	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	Holeplug capped with concrete			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
2941	0		1						Asphalt pavement					0915	
									GRAVEL, COBBLES, and BOULDERS, medium dense, light brown, sandy matrix, maximum 0.2 m cored length						
		1		32		NA									Lost circulation.
2940	1														Water encountered at approx. 1.3 m.
												5		0925	
										SPT	1	12	22		N = 24
												12		0930	
2939	2	2		35		NA									
2938	3											5		0942	
										SPT	2	8	44		N = 17
		3		43		NA						9		1015	
2937	4								GRANITE (Bedrock) (see description next page)						

Report: GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 28THB-20

Project: Beartooth Highway
Project Location: Long Lake Bridge
Project Number: 68FHAT0027.00

Log of Test Hole THB-20

Sheet 2 of 2

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q-D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
2937	4		1		5		1		GRANITE, strong to very strong, pink, coarse-grained, slightly weathered [Bedrock]						
		3		43	2	35	2 M		1: 45°, J, N, Fe, Su, Wa, R 2: 45°, J, N, Fe, Su, Pl, R						
							M 1		1: 75°, J, N, Cl, Pa, Wa, SR					1030	
			2		2		M								
2936	5				1		2		2: 45°, J, N, Fe, Su, Wa, R						
		4		100	0	78									
					2		3 3		3: 45°, J, N, Cl, Su, Pl, R						UCS = 117.6 MPa PLI = 1069 CS = 200 MPa
					0		4		4: 5°, J, N, Fe, Su, Pl, SR						
2935	6				2									1050	PLI = 761 CS = 143 MPa
		5		100	6	20									
					4		1 2		1: 80°, J, N, Cl, Fi, Wa, R 2: 50°, J, N, Cl, Su, Wa, SR						
2934	7						2							1105	PLI = 856 CS = 160 MPa
									Bottom of hole at 7.00 m						
2933	8														
2932	9														

Report: GEO_CORE+SOIL_17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 26THB-20

URS

Project: Beartooth Highway
Project Location: Site 4 - Wall
Project Number: 68FHAT0027.00

Log of Test Hole TH4-1

Sheet 1 of 2

Date(s) Drilled	9/7/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	6.85 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	3232.9 m MSL
Groundwater Level	Not encountered	Location	Station 56+970, centerline	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	Holeplug capped with cement grout			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
	0		1						Asphalt pavement					0845	
									GRAVEL, COBBLES, and BOULDERS, medium dense, brown, coarse-grained sand matrix, maximum 0.2 m cored length						
3232	1	1		40		NA									
												10		0852	
										SPT-1		10	11		N = 20
3231	2											10		0901	
		2		48		NA									
3230	3				2		1		GRANITE, moderately strong to very strong, black to pink, coarse-grained, slightly weathered [Bedrock]			17		0908	
							M			SPT-2		12	49		N = 44
							2		1: 85°, J, N, Fe, Pa, Wa, SR						
					5				2: 30°, J, N, Fe, Su, Wa, R			32		0922	
		3		100		0									
3229	4				4										

Report: GEO CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 27TH4-01

URS

Project: Beartooth Highway
Project Location: Site 4 - Wall
Project Number: 68FHAT0027.00

Log of Test Hole TH4-1

Sheet 2 of 2

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4		1		3				GRANITE, moderately strong to very strong, black to pink, coarse-grained, slightly weathered [Bedrock] (continued)					
	3		100	>10	0			Highly fractured zone					
						1		1: 50°, J, N, Cl, Fi, Pl, SR				0935	
3228		2		1									
5				8				Highly fractured zone					
	4		100	3	0	2		2: 30°, J, N, Fe, Su, Wa, R					PLI = 463 CS = 87 MPa
				2		M							
						M							
3227				1		M							
6												0950	
				2		1							
	5		100	4	0	M		1: 45°, J, N, Cl, Pa, Wa, R					
						M							
				5		2		2: 60°, J, N, Fe, Pa, Pl, SR					
3226						2						1015	
7								Bottom of hole at 6.85 m					
3225													
8													
3224													
9													

Report: GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 2TTH4-01

URS

Project: Beartooth Highway
Project Location: Site 4 - Wall
Project Number: 68FHAT0027.00

Log of Test Hole TH4-2

Sheet 1 of 2

Date(s) Drilled	9/15/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	8.70 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	3239.6 m MSL
Groundwater Level	Not encountered	Location	Station 57+095, offset 1.0 R	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	Holeplug capped with concrete			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
0		1						GRAVEL, COBBLES, and BOULDERS, medium dense, light brown, sandy matrix, maximum 0.2 m cored length					1219	
3239	1		37		NA									
1														
3238											8		1230	
									SPT	1	9	89		N = 26
											17		1249	
2	2		43		NA									
3237														
3											9		1301	
									SPT	2	14	91		N = 23
3236	3		25		NA						9		1311	
4														

Report: GEO_CORE+SOIL_17_METRIC; File: BTOOTHWY.GPJ; 1/14/2001 28T H4-02

URS

Project: Beartooth Highway
Project Location: Site 4 - Wall
Project Number: 68FHAT0027.00

Log of Test Hole TH4-2

Sheet 2 of 2

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
4		1						GRAVEL, COBBLES, and BOULDERS, medium dense, light brown, sandy matrix, maximum 0.2 m cored length (continued)					
	3		25		NA								
3236								← Becomes dense to very dense	SPT-3	10 23 36	33	1320 1358	
5													
	4		5		NA								
3234								GRANITE and GNEISS, strong to very strong, pink to black, coarse-grained, slightly weathered, contains highly weathered zones of mica schist [Bedrock]					
								} Highly fractured zone					
6						1		1: 50°, J, N, Fe, Su, Pl, SR					
				2		1 2		1: 50°, J, N, Fe, Su, Pl, SR 2: 40°, J, N, Fe, Su, Pl, SR				1415	
3233				4		1 1 3		3: 10°, J, N, Fe, Su, Pl, SR					
	5	2	100	4	22	3							
7				3		1 1 1 1							PLI = 1056 CS = 199 MPa
3232				3		1							
				4		1,1		1: 40°, J, N, Fe, Su-Pa, Pl, SR				–	
				3		1 1,1							PLI = 492 CS = 93 MPa
8	6		100		26	1 1 1,1		} Highly fractured zone					
				5		1 1							
3231				5		1,1 1,1 1		} Highly fractured zone					
9								Bottom of hole at 8.70 m					

Report: GEO_CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 28TH4-02

URS

Project: Beartooth Highway Project Location: Site 4 - Wall Project Number: 68FHAT0027.00	**Log of Test Hole TH4-3** Sheet 1 of 2

Date(s) Drilled	9/7/00	Logged By	A. Burns	Reviewed By	D. Baures
Drilling Method	HQ Wireline Core	Drill Bit Size/Type	HQ diamond-impregnated	Total Depth of Test Hole	9.00 m
Drill Rig Type	BK-81 (truck-mount)	Drilling Contractor	Haztech	Approximate Surface Elevation	3236.6 m MSL
Groundwater Level	Not encountered	Location	Station 57+040, offset 1.7 R	Inclination from Horizontal/Bearing	90° (vertical)
Borehole Completion	Holeplug capped with cement grout			Hammer Data	Pneumatic hammer; 63.5 kg / 762-mm drop

Elevation, meters / Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R.Q.D., %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type / Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
0		1						GRAVEL, COBBLES, and BOULDERS, loose to medium dense, light brown, coarse-grained sand matrix, maximum 0.2 m cored length				1131	
3236	1		40		NA								
1													
3235										13		1140	
									SPT-1	9	11		N = 17
										8		1146	
2	2		30		NA								
3234													
3										4		1151	
									SPT-2	3	0		N = 5
										2		1158	
3233	3		40		NA								
4													

Report: GEO CORE+SOIL 17 METRIC, File: BTOOTHWY.GPJ, 1/14/2001 29TH4-03

URS

Project: Beartooth Highway
Project Location: Site 4 - Wall
Project Number: 68FHAT0027.00

Log of Test Hole TH4-3

Sheet 2 of 2

Elevation, meters	Depth, meters	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per 300 mm	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows per 150 mm	Recovery, %	Drill Time, 24-hr clock	FIELD NOTES AND LAB TESTS
	4		1						GRAVEL, COBBLES, and BOULDERS, loose to medium dense, light brown, coarse-grained sand matrix, maximum 0.2 m cored length (continued)						
		3		40		NA									
3232														1215	No SPT taken; on rock.
	5														
		4		60		NA									
					4		1		GRANITE, moderately strong to very strong, black to pink, coarse-grained, slightly weathered [Bedrock]						
3231									1: 30°, J, N, Fe, Su, Pl, SR						
					5										
	6														
			2		2		1							1226	
							1		1: 30°, J, W, Fe, Pa, Wa, R						
					6		2		2: 50°, J, N, Fe, Su, Pl, SR						Stopped drilling 1235-1355 to refill water trucks.
3230							3		3: 45°, J, N, Fe, Su, Pl, SR						
		5		90	3	0									
	7				5		4		4: 45°, J, W, Fe, Fi, Pl, SR						
							5		5: 45°, J, W, Fe, Fi, Pl, SR						
					6										
3229														1356	
					2		1		1: 45°, J, MW, Cl, Fi, Pl, SR						PLI = 140 CS = 26 MPa
							1								
	8				2										
		6		100	0	56									
					2		2		2: 30°, J, N, Fe, Su, Pl, SR						
3228							2								
							3		3: 60°, J, N, Fe, Su, Pl, SR						
					4		2								
	9						M		Bottom of hole at 9.00 m					1403	PLI = 524 CS = 99 MPa

Report GEO CORE+SOIL 17 METRIC; File: BTOOTHWY.GPJ; 1/14/2001 29TH4-03

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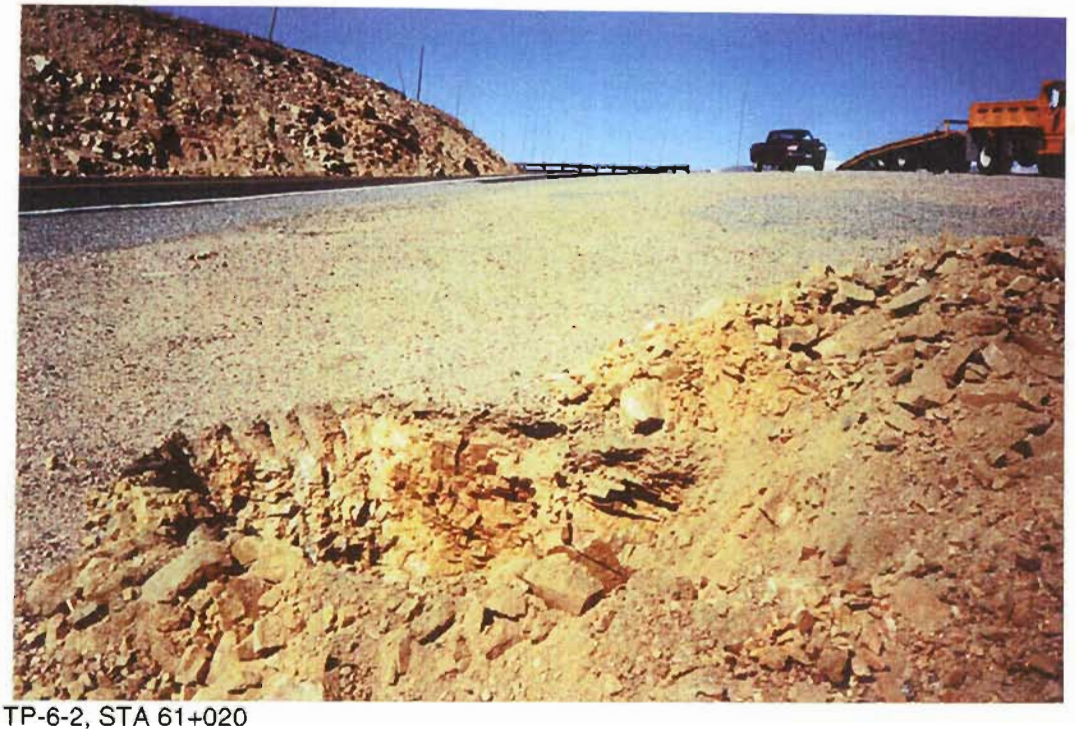
Appendix C

Appendix C

Test Pit Logs and Photographs

TP-6-1, STA 61+000

TP-6-1, STA 61+000



TP-6-2, STA 61+020

TP-6-2, STA 61+020

TP-6-3, STA 61+035

TP-6-3, STA 61+035

Project: Beartooth Highway
Project Location: Bar Drift Area
Project Number: 68FHAT0027.00

Log of Test Pit TP6-1

Date(s) Excavated	9/15/00	Logged By	A. Burns	Reviewed By	T. Adams
Length of Excavation	~1.5 m	Width of Excavation	~1 m	Depth of Excavation	1.30 m
Excavation Equipment	580K Backhoe	Excavation Contractor	Dan Estes	Approximate Surface Elevation	3255.2 m MSL
Water Observations	No water observed during excavation activities			Location	Station 61+000
Comments					

Elevation, meters	Depth, meters	Sample Type / Sample Number	Graphic Log	MATERIAL DESCRIPTION	REMARKS
3255	0.0	Bag 1		SAND (SM), silty, yellow-brown	
	0.5			GRANITE, strong, iron oxide-stained, highly weathered, high-angle close jointing [Bedrock]	
3254	1.0				
	1.3			Bottom of test pit at 1.30 m	Backhoe refusal.
	1.5				
3253	2.0				
	2.5				
	3.0				
3252					
	3.5				
	4.0				

Report: GEO_5D_TEST_PIT; File: BTOOTHWY.GPJ; 1/16/2001 TP6-01

URS

Project: Beartooth Highway Project Location: Bar Drift Area Project Number: 68FHAT0027.00	Log of Test Pit TP6-2

Date(s) Excavated	9/15/00	Logged By	A. Burns	Reviewed By	T. Adams
Length of Excavation	~1.5 m	Width of Excavation	~1 m	Depth of Excavation	0.76 m
Excavation Equipment	580K Backhoe	Excavation Contractor	Dan Estes	Approximate Surface Elevation	3254.1 m MSL
Water Observations	No water observed during excavation activities			Location	Station 61+020
Comments					

Elevation, meters	Depth, meters	Sample Type	Sample Number	Graphic Log	MATERIAL DESCRIPTION	REMARKS
3254	0.0		Bag 1		GRANITE, strong, iron oxide-stained, highly to very highly weathered, close jointing; with zones of completely weathered rock (Bedrock)	Sample collected from completely weathered zones.
	0.5					
					Bottom of test pit at 0.76 m.	Backhoe refusal.
3253	1.0					
	1.5					
3252	2.0					
	2.5					
3251	3.0					
	3.5					
	4.0					

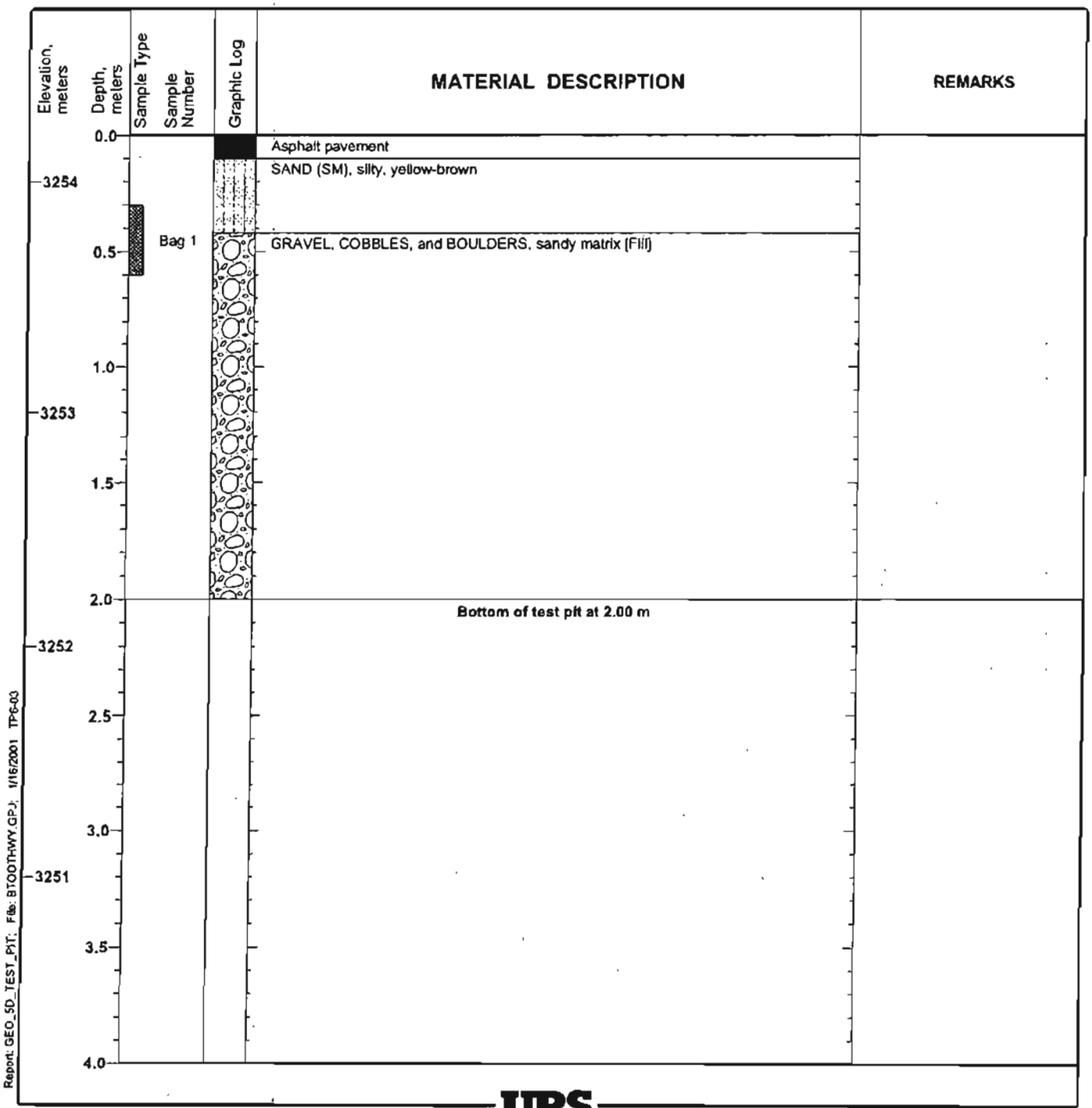
Report: GEO_SD_TEST_PIT; File: BTOOTHWY.GPJ; 1/16/2001 TP6-02

URS

Project: Beartooth Highway
Project Location: Bar Drift Area
Project Number: 68FHAT0027.00

Log of Test Pit TP6-3

Date(s) Excavated	9/15/00	Logged By	A. Burns	Reviewed By	T. Adams
Length of Excavation	~1.6 m	Width of Excavation	~1.2 m	Depth of Excavation	2.00 m
Excavation Equipment	580K Backhoe	Excavation Contractor	Dan Estes	Approximate Surface Elevation	3254.2 m MSL
Water Observations	No water observed during excavation activities			Location	Station 61+035
Comments					



URS

Appendix D

Appendix D
Laboratory Test Results

Table D-1
SUMMARY OF LABORATORY TEST RESULTS

Site	Test Hole	Depth (m)	Bulk Density (kg/cu. m)[1]	Dry Density (kg/cu. m)[1]	Moisture Content (%)[1]	Classification[2]	% Gravel[3]	% Sand[4]	% Fines[5]	Plasticity Index[6]	Liquid Limit[6]	UCS[7] (MPa)	UCS[8] (MPa)
Site 1	TH1-1	7.60			17.6	SM	22	29	49	3	19		
Site 1	TH1-1	12.20		2204	7.8	SM	40	45	15	NP			
Site 1	TH1-1	15.25		1804	14.8	SM	0	67	33	NP			
Site 1	TH1-3	4.55		1805	10.0	CL	3	37	60	11	35		
Site 1	TH1-3	9.15		2098	9.3	SM	31	40	29	NP			
Site 1	TH1-3	12.20		1607	17.0	SW-SM	3	88	9	NP			
Site 1	TH1-4	6.10		2031	7.0	GC	46	21	33	13	30		
Site 1	TH1-4	7.60		2129	4.8	GC	70	16	14	14	35		
Site 1	TH1-4	9.15		1783	4.5	SP-SM	20	69	11	NP			
Beartooth Ravine Bridge	THB-1	4.30				Granite/Gneiss						200	
Beartooth Ravine Bridge	THB-1	6.40				Granite/Gneiss						187	
Beartooth Ravine Bridge	THB-1	7.80	2658			Granite/Gneiss						34	57
Beartooth Ravine Bridge	THB-2	5.80				Granite/Gneiss						132	
Beartooth Ravine Bridge	THB-2	7.00				Granite/Gneiss						165	
Beartooth Ravine Bridge	THB-2	10.80	2598			Granite/Gneiss						58	37
Beartooth Ravine Bridge	THB-4	7.00	2595			Granite							86
Beartooth Ravine Bridge	THB-4	8.60				Granite						74	
Beartooth Ravine Bridge	THB-4	9.60				Granite						108	
Beartooth Ravine Bridge	THB-5	6.17	2557			Granite							70
Beartooth Ravine Bridge	THB-5	8.60				Granite						200	
Beartooth Ravine Bridge	THB-5	10.65				Granite						134	
Site 2	TH2-11	14.50				Granite						199	
Site 2	TH2-11	15.30				Granite						200	
Site 2	TH2-11	15.90	2656			Granite							65
Site 2	TH2-2	7.35				Granite						147	
Site 2	TH2-2	7.95	2615			Granite							90
Site 2	TH2-2	8.25				Granite						198	
Site 2	TH2-3	8.20	2639			Granite/Gneiss							78
Site 2	TH2-3	8.40				Granite/Gneiss						160	
Site 2	TH2-3	9.60				Granite/Gneiss						187	
Site 2	TH2-5	11.10	2506			Granite							51
Site 2	TH2-5	12.40				Granite						198	
Site 2	TH2-5	13.27				Granite						229	
Site 2	TH2-6	8.85				Granite/Gneiss						121	
Site 2	TH2-6	9.20				Granite/Gneiss						66	
Site 2	TH2-6	10.45	2800			Granite/Gneiss							43
Site 2	TH2-7	7.90	2588			Granite							98
Site 2	TH2-7	10.00				Granite						82	
Site 2	TH2-7	10.70				Granite						196	

Table D-1
SUMMARY OF LABORATORY TEST RESULTS

Site	Test Hole	Depth (m)	Bulk Density (kg/cu. m) [1]	Dry Density (kg/cu. m) [1]	Moisture Content (%) [1]	Classification [2]	% Gravel [3]	% Sand [4]	% Fines [5]	Plasticity Index [6]	Liquid Limit [6]	UCS [7] (MPa)	UCS [8] (MPa)
Site 2	TH2-8	13.80	2839			Granite/Gneiss							31
Site 2	TH2-8	14.10				Granite/Gneiss						54	
Site 2	TH2-8	15.45				Granite/Gneiss						187	
Beartooth Lake Bridge	THB-6	12.50				CL-ML		20	80				
Beartooth Lake Bridge	THB-6	17.00				CL-ML	3	8	89	5	23		
Beartooth Lake Bridge	THB-6	28.10		1539	27.7	CL							
Beartooth Lake Bridge	THB-7	11.00				ML		13	87				
Beartooth Lake Bridge	THB-7	14.00				ML		13	87	3	21		
Little Bear Creek Br. 1	THB-8	9.60				Gneiss						174	
Little Bear Creek Br. 1	THB-8	11.00	2945			Gneiss							47
Little Bear Creek Br. 1	THB-8	11.60				Gneiss						127	
Little Bear Creek Br. 2	THB-11	3.10				Sandstone						27	
Little Bear Creek Br. 2	THB-11	4.00				Sandstone						20	
Little Bear Creek Br. 2	THB-11	5.20	2568			Sandstone							31
Fen Mitigation Bridge	THB-12	6.10	2630			Granite							84
Fen Mitigation Bridge	THB-12	9.30				Granite						93	
Fen Mitigation Bridge	THB-12	10.60				Granite						100	
Fen Mitigation Bridge	THB-14	9.30	2400			Gabbro							27
Fen Mitigation Bridge	THB-14	10.35				Gabbro						87	
Fen Mitigation Bridge	THB-16a	4.40	2621			Granite							80
Fen Mitigation Bridge	THB-16a	6.30				Gneiss						93	
Fen Mitigation Bridge	THB-16a	7.65				Granite						147	
Long Lake Bridge	THB-19	3.40	2765			Granite							42
Long Lake Bridge	THB-19	4.90				Granite						125	
Long Lake Bridge	THB-19	5.80				Granite						119	
Long Lake Bridge	THB-20	5.60	2623			Granite						200	118
Long Lake Bridge	THB-20	6.10				Granite						143	
Long Lake Bridge	THB-20	6.90				Granite						160	
Site 4	TH4-1	5.50				Granite						87	
Site 4	TH4-2	7.10				Granite/Gneiss						199	
Site 4	TH4-2	7.90				Granite/Gneiss						93	
Site 4	TH4-3	7.70				Granite						26	
Site 4	TH4-3	8.90				Granite						99	

Table D-1
SUMMARY OF LABORATORY TEST RESULTS

Site	Test Hole	Depth (m)	Bulk Density (kg/cu. m)[1]	Dry Density (kg/cu. m)[1]	Moisture Content (%)[1]	Classification[2]	% Gravel[3]	% Sand[4]	% Fines[5]	Plasticity Index[6]	Liquid Limit[6]	UCS[7] (MPa)	UCS[8] (MPa)
Bar Drift Area	TP6-1	0.61				GW-GM	49	43	8				
Bar Drift Area	TP6-2	0.61				GW-GM	47	46	7				
Bar Drift Area	TP6-3	0.61				SM	23	59	18				

[1] Dry unit weight test performed in general accordance with ASTM D 2216

[2] USCS Unified Soil Classification System, based on visual observation when grain size data and plasticity data not available. Grain size distribution performed in general accordance with ASTM D422. Visual classification performed in general accordance with ASTM D 2488.

[3] Percent greater than the No. 4 sieve size.

[4] Percent finer than the No. 4 sieve size and coarser than the No 200 sieve size.

[5] Percent finer than the No. 200 sieve size. Includes silt and clay sizes.

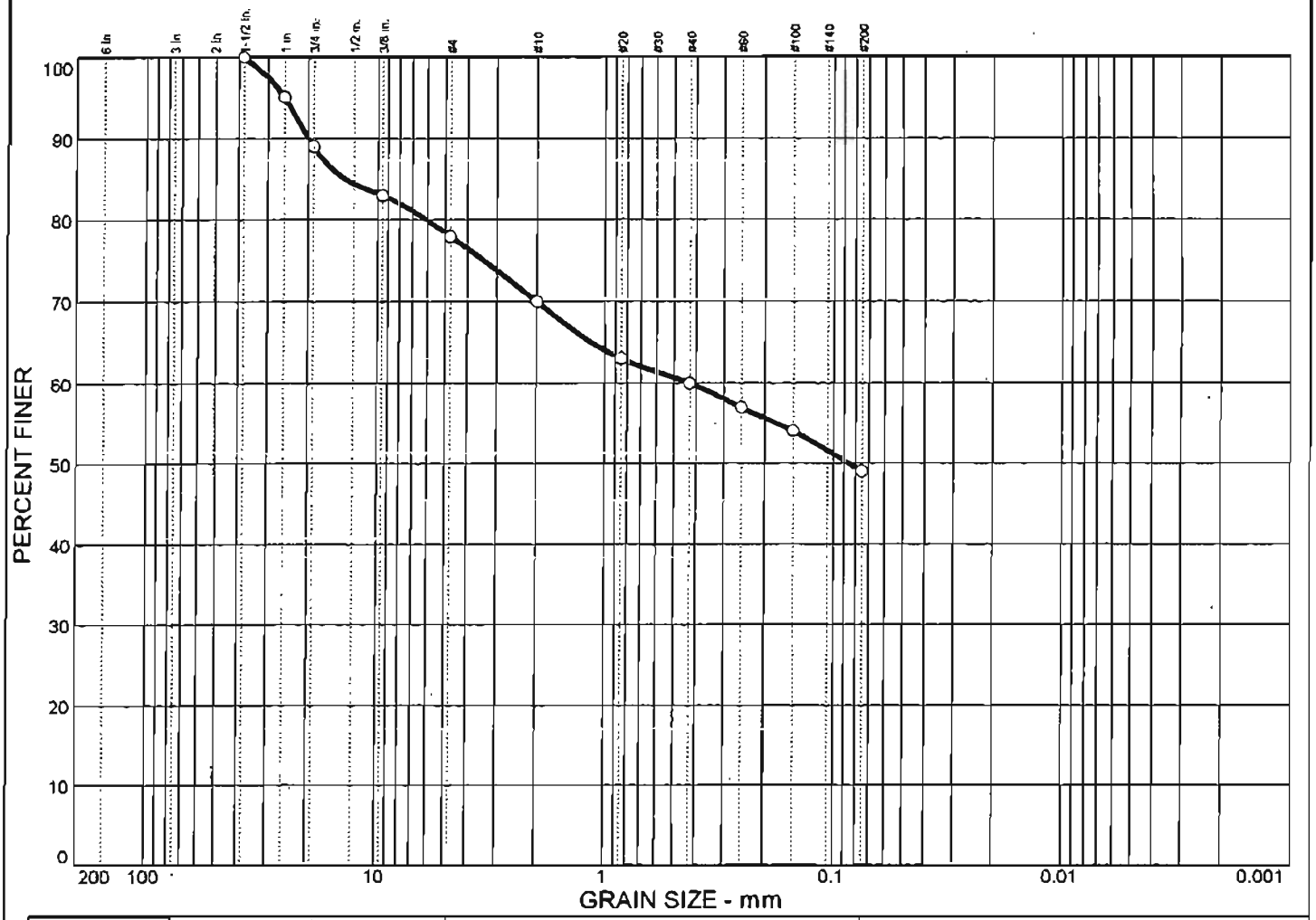
[6] Atterberg limits performed in general accordance with ASTM D 4318.

[7] Values determined from point load testing and the point load index.

[8] UCS Unconfined Compressive Strength from Unconfined Compressive Strength Test

[10] Indicates L/D < 2.0. Correction Factor from ASTM D 2938 used.

PARTICLE-SIZE DISTRIBUTION REPORT



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	11.0	11.0	8.0	10.0	11.0	49.0	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	19	3	13.8	0.425	0.0855					

MATERIAL DESCRIPTION	USCS	AASHTO
○ Silty sand with gravel	SM	

Project No. 68FHAT0027 **Client:**
Project: BEAR TOOTH HIGHWAY

○ **Source:** TH1-1 **Sample No.:** CAL-4 **Elev./Depth:** 7.6m

Remarks:
○

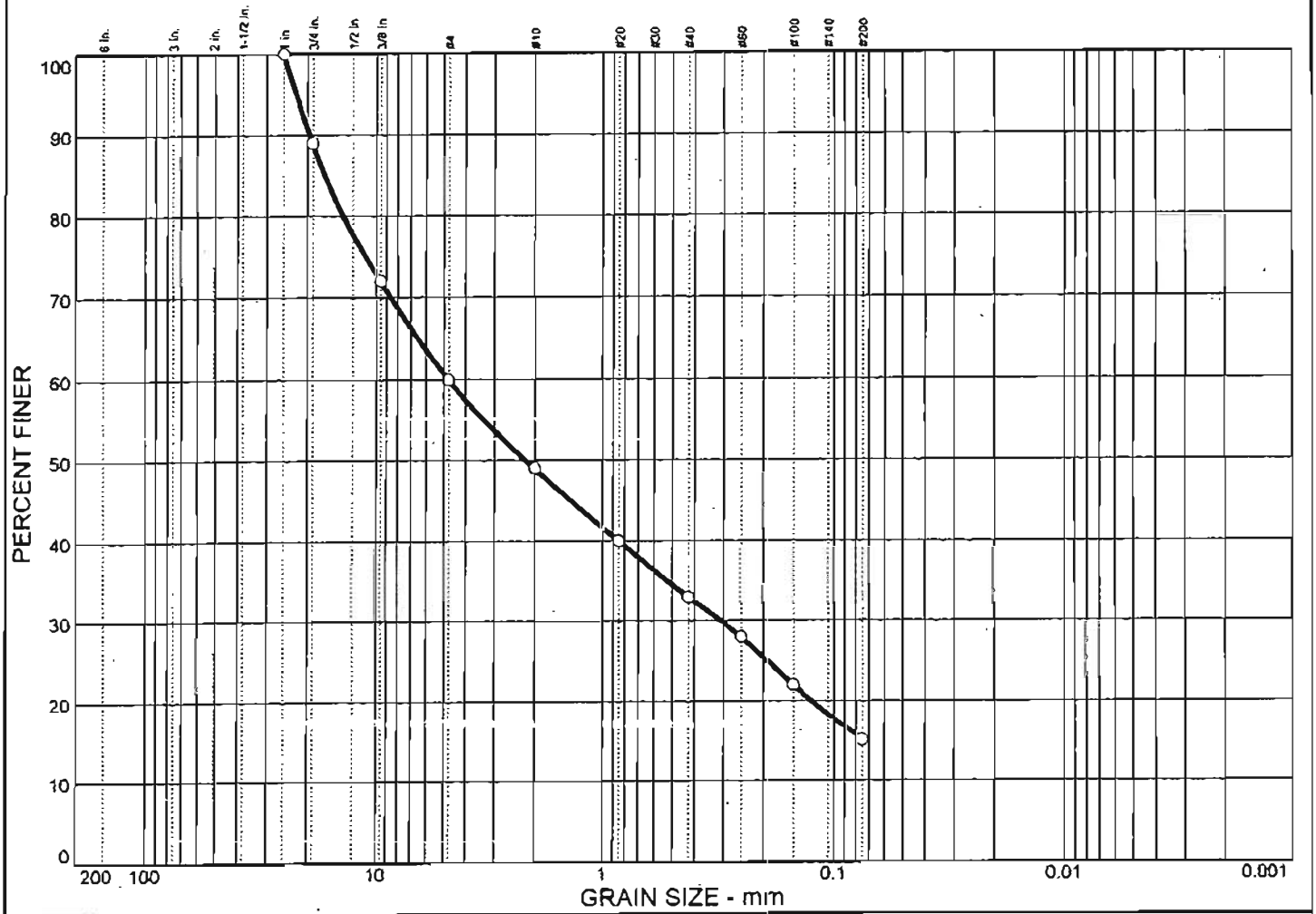
PARTICLE-SIZE DISTRIBUTION REPORT

URS GREINER WOODWARD CLYDE

D-1

Fig.

PARTICLE-SIZE DISTRIBUTION REPORT



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	11.0	29.0	11.0	16.0	18.0	15.0	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○		NONPLASTIC	16.8	4.75	2.19	0.306	0.0750			

MATERIAL DESCRIPTION	USCS	AASHTO
○ Silty sand with gravel	SM	

Project No. 68FHAT0027 **Client:**

Project: BEAR TOOTH HIGHWAY

○ **Source:** TH1-1 **Sample No.:** CAL-7 **Elev./Depth:** 12.2m

Remarks:

○

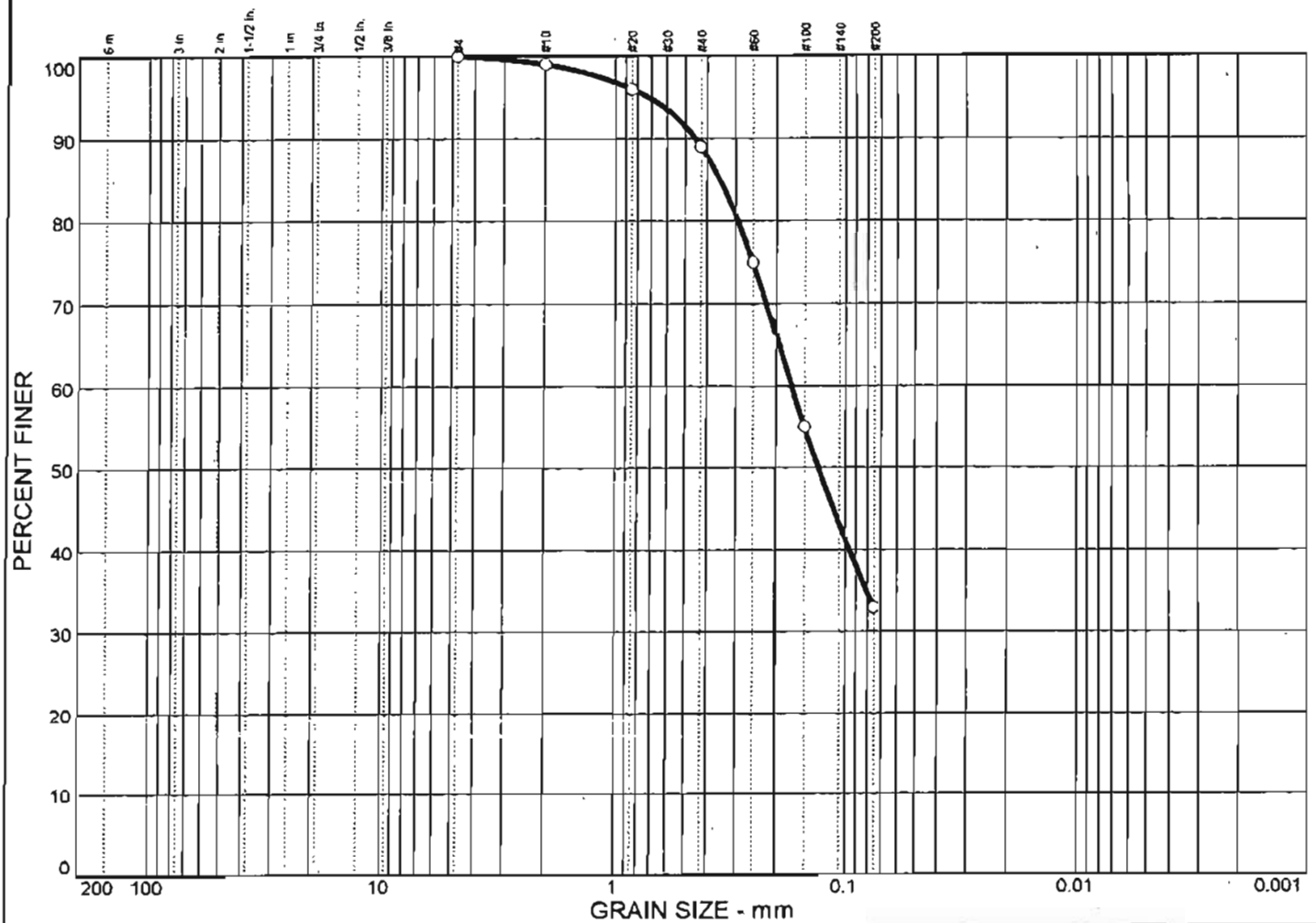
PARTICLE-SIZE DISTRIBUTION REPORT

URS GREINER WOODWARD CLYDE

D-2

Fig.

PARTICLE-SIZE DISTRIBUTION REPORT



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.0	1.0	10.0	56.0	33.0	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○		NONPLASTIC	0.351	0.170	0.131					

MATERIAL DESCRIPTION	USCS	AASHTO
○ Silty sand	SM	

Project No. 68FHAT0027 **Client:**
Project: BEAR TOOTH HIGHWAY

○ **Source:** TH1-1 **Sample No.:** CAL-9 **Elev./Depth:** 15.25m

Remarks:
○

PARTICLE-SIZE DISTRIBUTION REPORT

URS GREINER WOODWARD CLYDE

D-3

Fig.

PARTICLE-SIZE DISTRIBUTION REPORT

	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	3.0	8.0	17.0	12.0	60.0	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	35	11	1.55	0.0750						

MATERIAL DESCRIPTION	USCS	AASHTO
○ Sandy lean clay	CL	

Project No. 68FHAT0027 **Client:**
Project: BEAR TOOTH HIGHWAY

○ **Source:** TH1-3 **Sample No.:** CAL-2 **Elev./Depth:** 4.55m

Remarks:
○

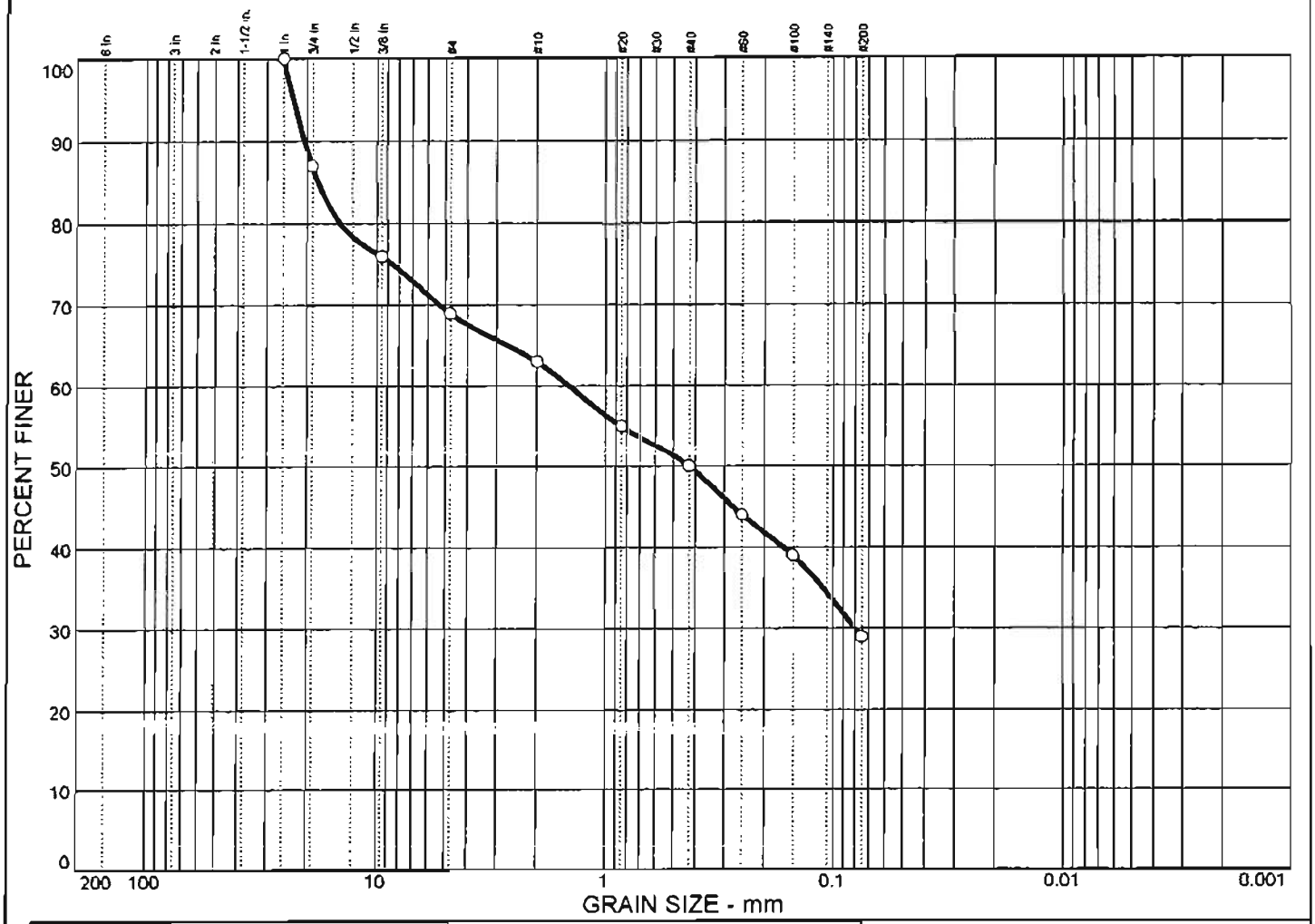
PARTICLE-SIZE DISTRIBUTION REPORT

URS GREINER WOODWARD CLYDE

D-4

Fig.

PARTICLE-SIZE DISTRIBUTION REPORT



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	13.0	18.0	6.0	13.0	21.0	29.0	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○		NONPLASTIC	17.9	1.44	0.425	0.0798				

MATERIAL DESCRIPTION	USCS	AASHTO
○ Silty sand with gravel	SM	

Project No. 68FHAT0027 **Client:**
Project: BEAR TOOTH HIGHWAY

○ **Source:** TH1-3 **Sample No.:** CAL-5 **Elev./Depth:** 9.15m

Remarks:
○

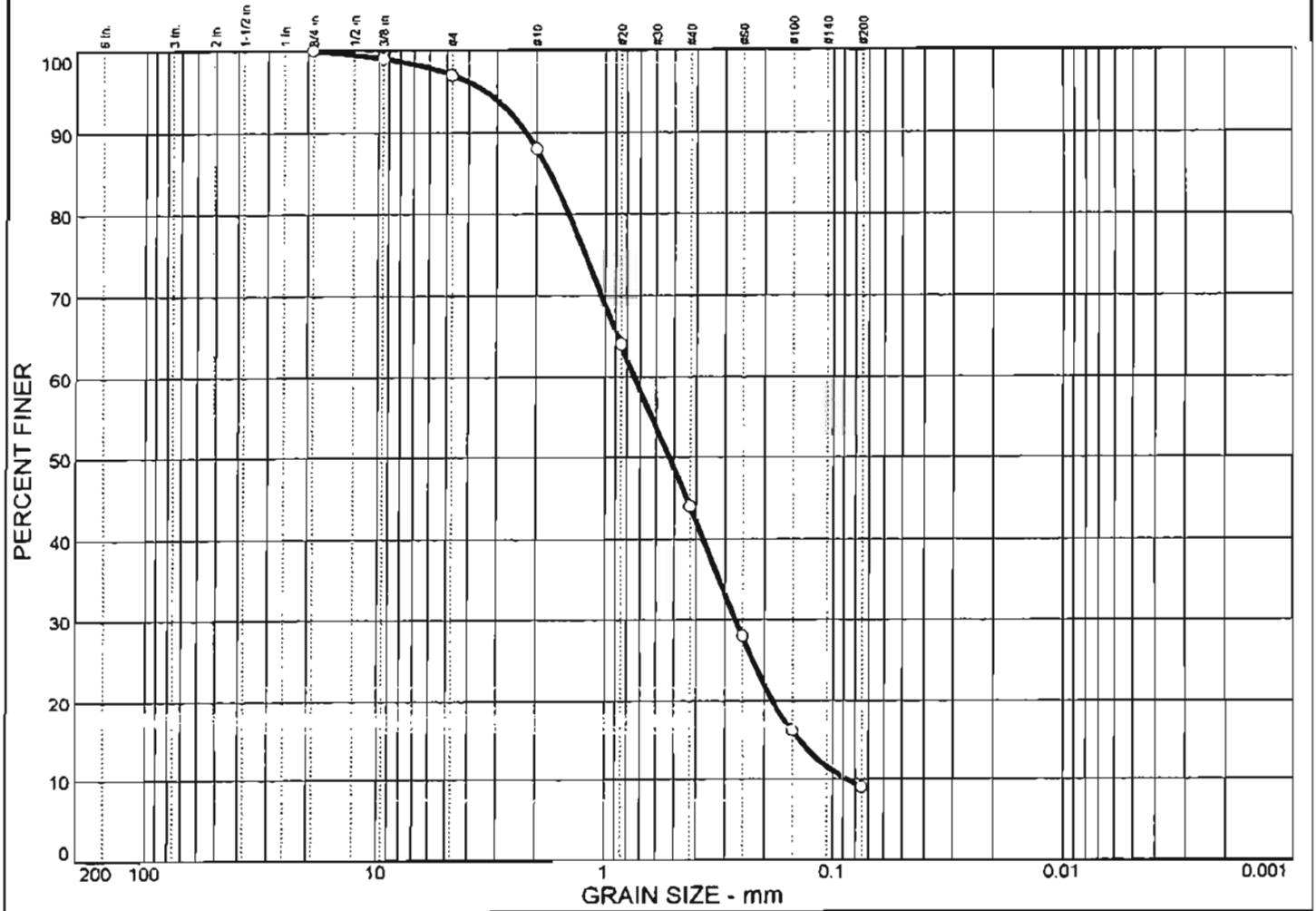
PARTICLE-SIZE DISTRIBUTION REPORT

URS GREINER WOODWARD CLYDE

D-5

Fig.

PARTICLE-SIZE DISTRIBUTION REPORT



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
o	0.0	0.0	3.0	9.0	44.0	35.0	9.0	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
o		NONPLASTIC	1.75	0.742	0.522	0.268	0.141	0.0870	1.11	8.53

MATERIAL DESCRIPTION	USCS	AASHTO
o Well-graded sand with silt	SW-SM	

Project No. 68FHAT0027 **Client:**
Project: BEAR TOOTH HIGHWAY

o **Source:** TH1-3 **Sample No.:** CAL-7 **Elev./Depth:** 12.2m

Remarks:
o

PARTICLE-SIZE DISTRIBUTION REPORT

URS GREINER WOODWARD CLYDE

D-6

Fig.

PARTICLE-SIZE DISTRIBUTION REPORT

	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
o	0.0	27.0	19.0	5.0	6.0	10.0	33.0	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
o	30	13	29.3	10.7	2.37					

MATERIAL DESCRIPTION	USCS	AASHTO
o Clayey gravel with sand	GC	

Project No. 68FHAT0027 **Client:**
Project: BEAR TOOTH HIGHWAY

o **Source:** TH1-4 **Sample No.:** CAL-3 **Elev./Depth:** 6.1m

Remarks:
o

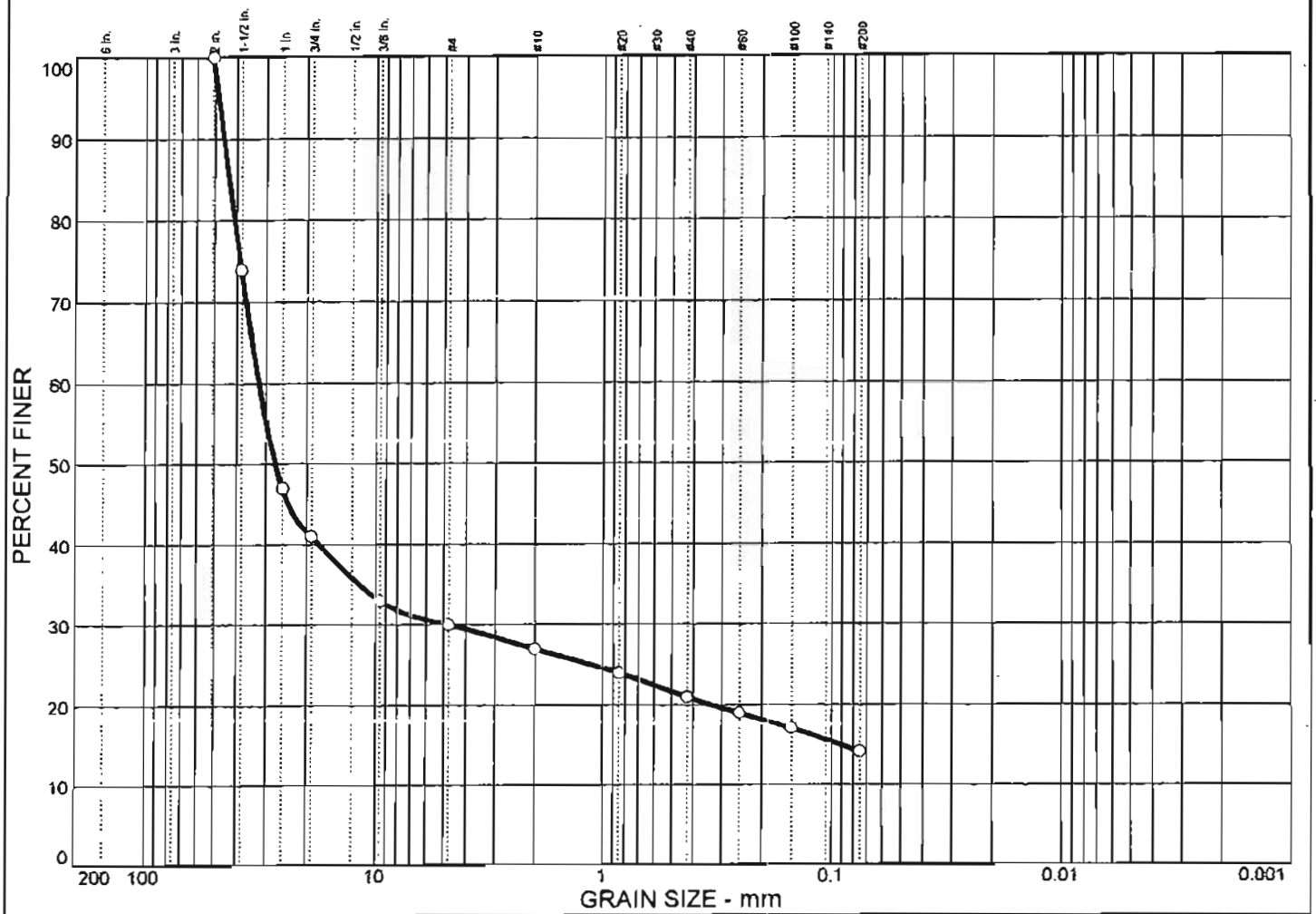
PARTICLE-SIZE DISTRIBUTION REPORT

URS GREINER WOODWARD CLYDE

D-7

Fig.

PARTICLE-SIZE DISTRIBUTION REPORT



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
o	0.0	59.0	11.0	3.0	6.0	7.0	14.0	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
o	35	14	43.2	32.0	27.2	4.75	0.0941			

MATERIAL DESCRIPTION	USCS	AASHTO
o Clayey gravel with sand	GC	

Project No. 68FHAT0027 **Client:**
Project: BEAR TOOTH HIGHWAY

o **Source:** TH1-4 **Sample No.:** CAL-4 **Elev./Depth:** 7.6m

Remarks:
o

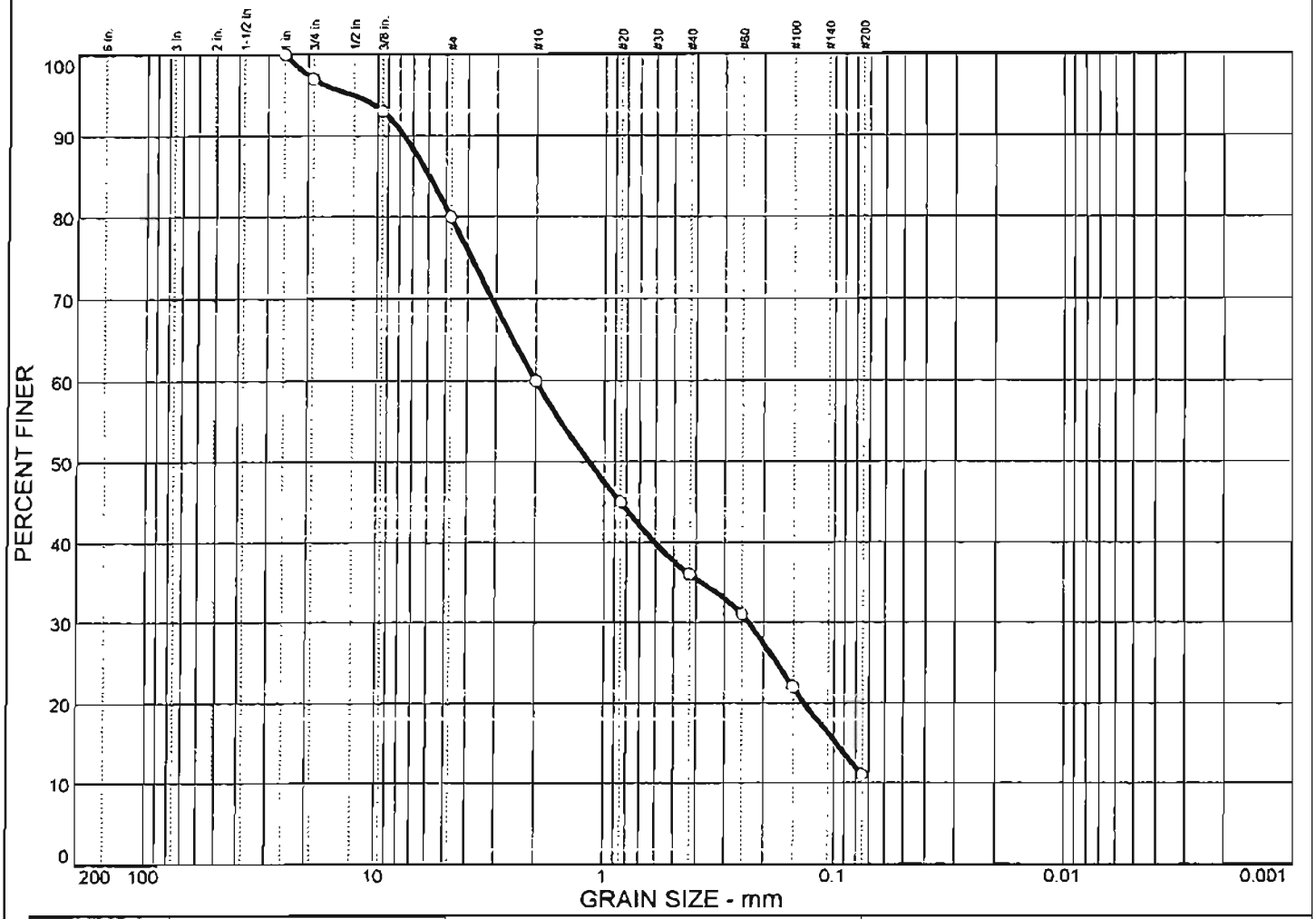
PARTICLE-SIZE DISTRIBUTION REPORT

URS GREINER WOODWARD CLYDE

D-8

Fig.

PARTICLE-SIZE DISTRIBUTION REPORT



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	3.0	17.0	20.0	24.0	25.0	11.0	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○		NONPLASTIC	5.92	2.00	1.16	0.233	0.0982			

MATERIAL DESCRIPTION	USCS	AASHTO
○ Poorly graded sand with silt and gravel	SP-SM	

Project No. 68FHAT00270 **Client:**
Project: BEAR TOOTH HIGHWAY

○ **Source:** TH1-4 **Sample No.:** CAL-5 **Elev./Depth:** 9.15m

Remarks:
○

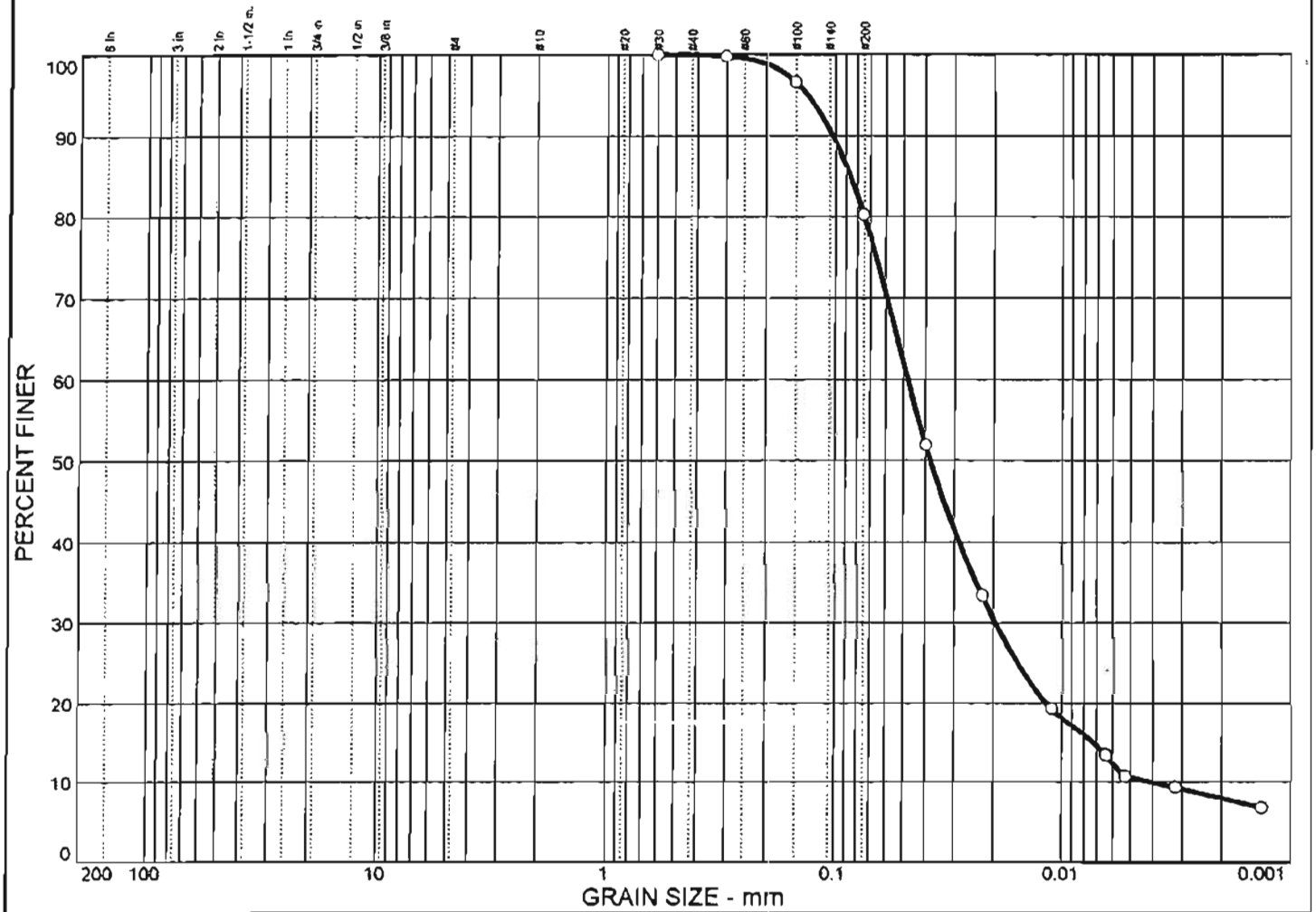
PARTICLE-SIZE DISTRIBUTION REPORT

URS GREINER WOODWARD CLYDE

D-9

Fig.

PARTICLE-SIZE DISTRIBUTION REPORT



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.0	0.0	0.0	19.7	69.8	10.5

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.0858	0.0477	0.0378	0.0195	0.0073	0.0041	1.95	11.64

MATERIAL DESCRIPTION	USCS	AASHTO
○ Silty clay with sand	CL-ML	

Project No. 68FHAT0027 **Client:**
Project: BEAR TOOTH HIGHWAY

○ **Source:** THB-6 **Sample No.:** SPT-8 **Elev./Depth:** 12.5m

Remarks:
○

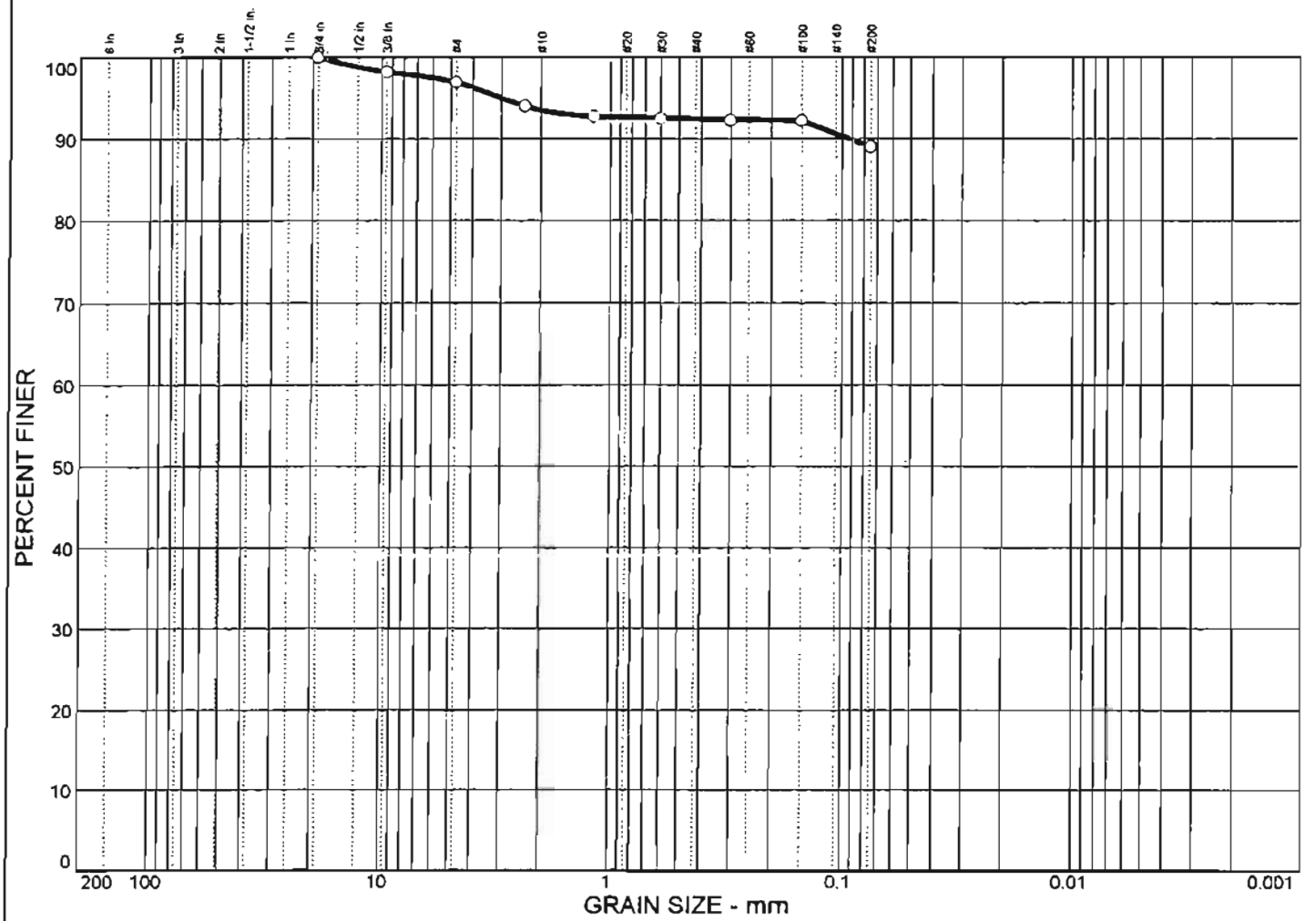
PARTICLE-SIZE DISTRIBUTION REPORT

URS GREINER WOODWARD CLYDE

D-10

Fig.

PARTICLE-SIZE DISTRIBUTION REPORT



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	3.1	3.4	1.1	3.4	89.0	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	23	5								

MATERIAL DESCRIPTION	USCS	AASHTO
○ Silty clay	CL-ML	

Project No. 68FHAT00270 **Client:**
Project: BEAR TOOTH HIGHWAY

○ **Source:** THB-6 **Sample No.:** SPT-11 **Elev./Depth:** 17.0m

Remarks:
○

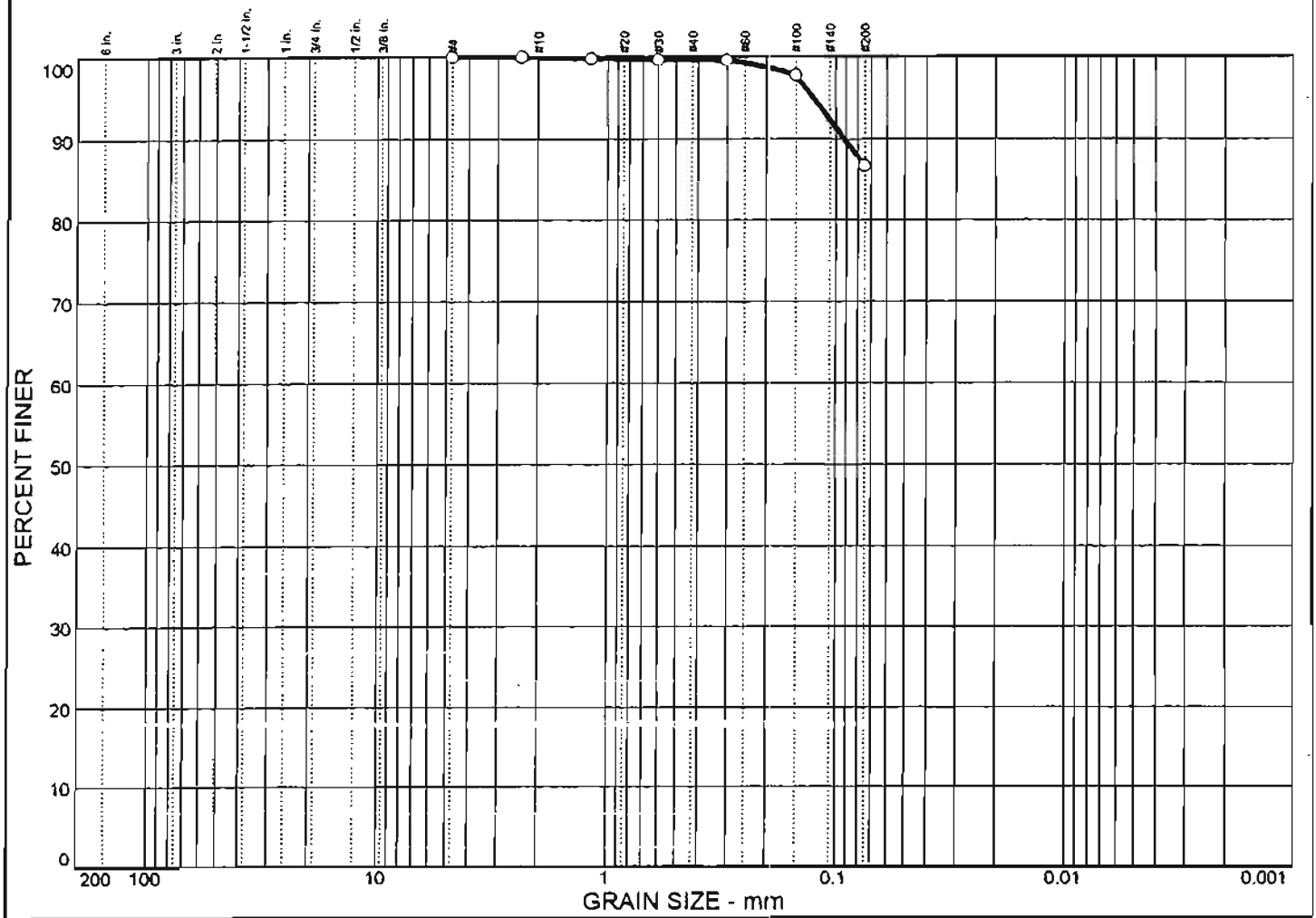
PARTICLE-SIZE DISTRIBUTION REPORT

URS GREINER WOODWARD CLYDE

D-11

Fig.

PARTICLE-SIZE DISTRIBUTION REPORT



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
o	0.0	0.0	0.0	0.1	0.3	12.9	86.7	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
o		NONPLASTIC								

MATERIAL DESCRIPTION	USCS	AASHTO
o Silt	ML	

Project No. 68FHAT0027 **Client:**
Project: BEAR TOOTH HIGHWAY

o **Source:** THB-7 **Sample No.:** STP-7 **Elev./Depth:** 11.0m

Remarks:
o

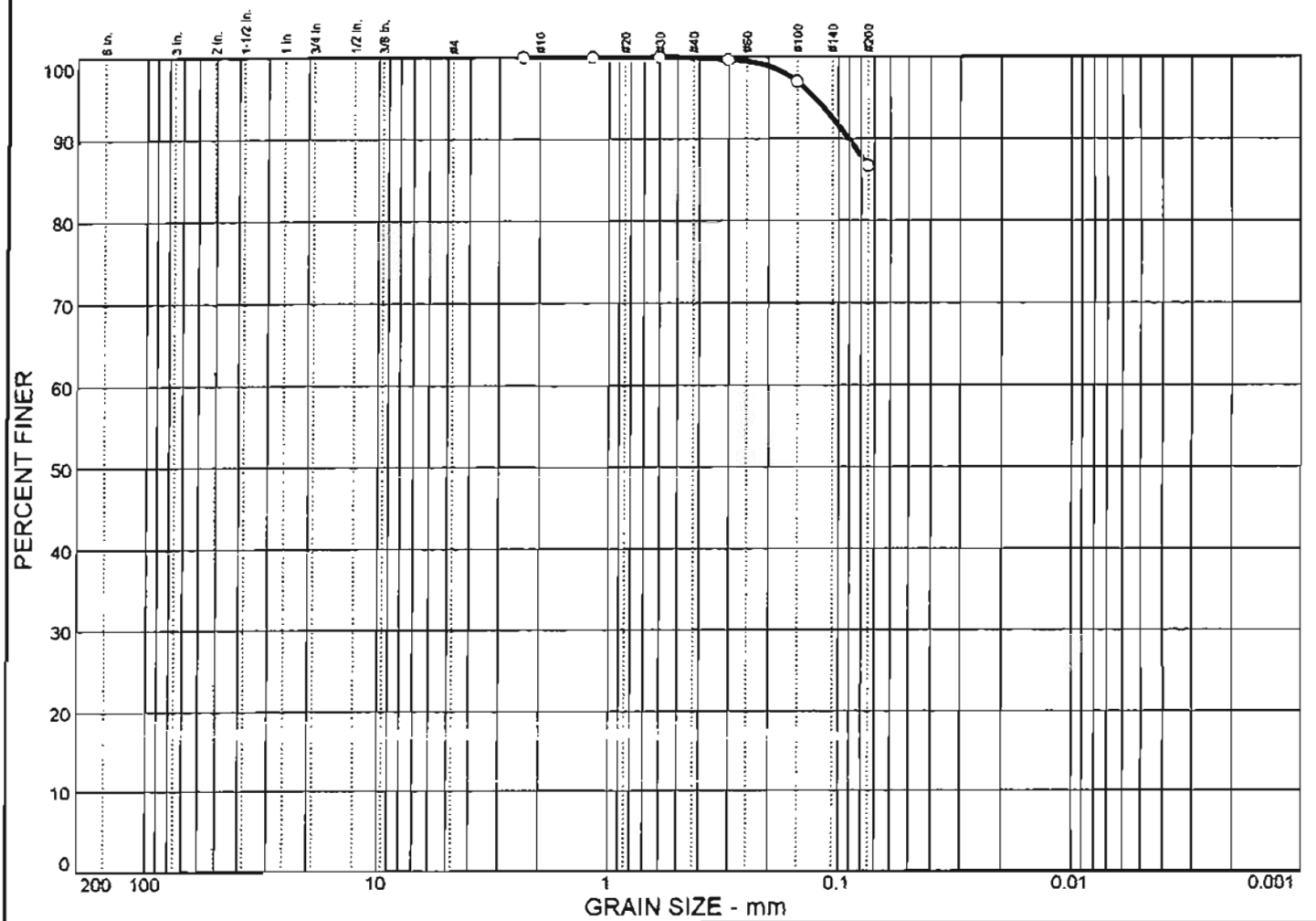
PARTICLE-SIZE DISTRIBUTION REPORT

URS GREINER WOODWARD CLYDE

D-12

Fig.

PARTICLE-SIZE DISTRIBUTION REPORT



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.0	0.0	0.1	13.2	86.7	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	21	3								

MATERIAL DESCRIPTION	USCS	AASHTO
○ Silt	ML	

Project No. 68FHAT0027 **Client:**
Project: BEAR TOOTH HIGHWAY

○ **Source:** THB-7 **Sample No.:** SPT-9 **Elev./Depth:** 14.0m

Remarks:
○

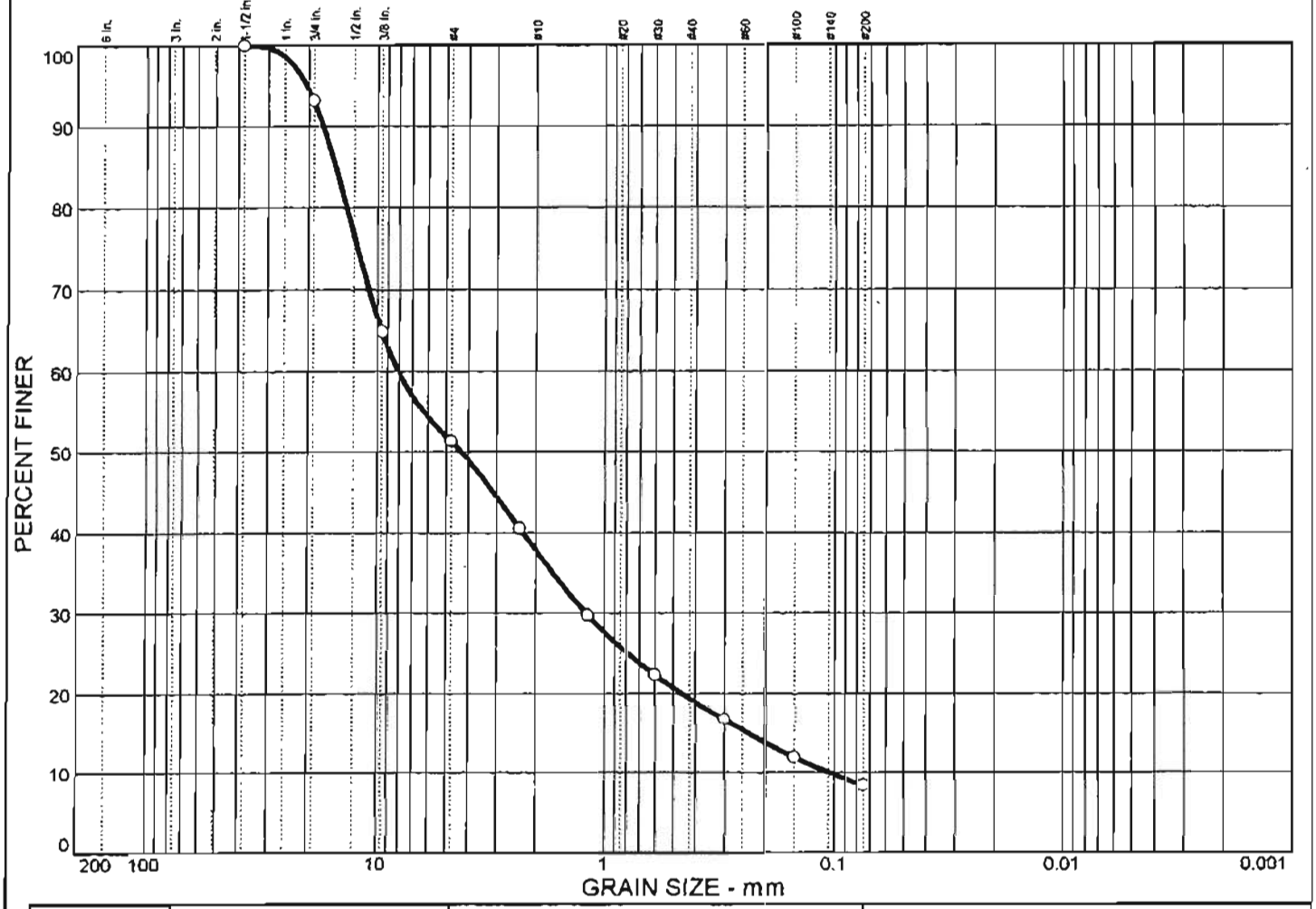
PARTICLE-SIZE DISTRIBUTION REPORT

URS GREINER WOODWARD CLYDE

D-13

Fig.

PARTICLE-SIZE DISTRIBUTION REPORT



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	6.8	41.8	13.5	18.4	11.1	8.4	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			15.3	8.05	4.29	1.20	0.239	0.105	1.69	76.49

MATERIAL DESCRIPTION	USCS	AASHTO
○ Well-graded gravel with silt and sand	GW-GM	

Project No. 68FHAT0027 **Client:**
Project: BEAR TOOTH HIGHWAY

○ **Source:** TP6-1 **Sample No.:** BAG-1 **Elev./Depth:** 0.2-0.4m

Remarks:
○

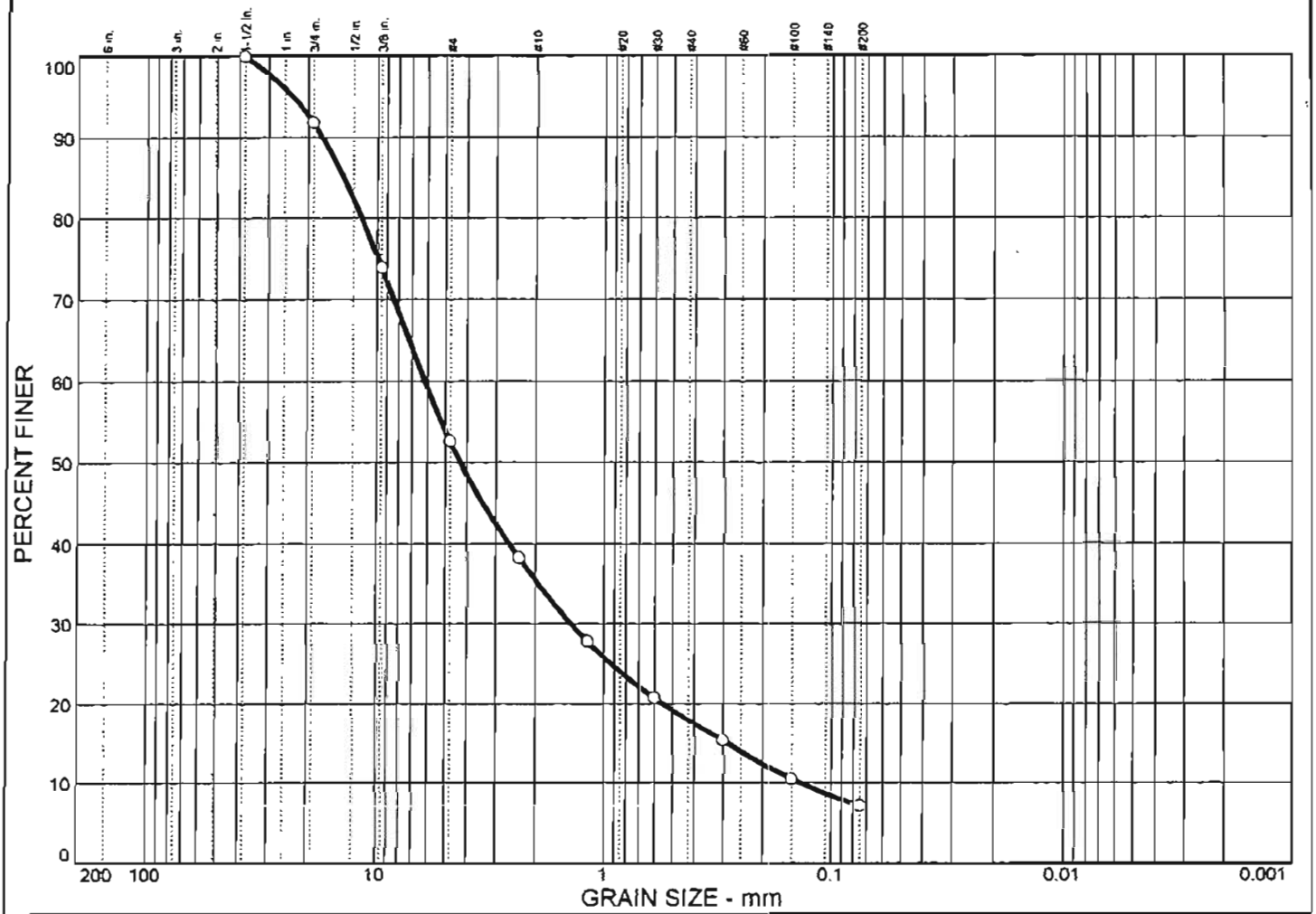
PARTICLE-SIZE DISTRIBUTION REPORT

URS GREINER WOODWARD CLYDE

D-14

Fig.

PARTICLE-SIZE DISTRIBUTION REPORT



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	8.2	39.1	17.2	17.6	10.8	7.1	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			14.0	6.12	4.27	1.39	0.292	0.137	2.31	44.61

MATERIAL DESCRIPTION	USCS	AASHTO
○ Well-graded gravel with silt and sand	GW-GM	

Project No. 68FHAT0027 **Client:**
Project: BEAR TOOTH HIGHWAY

○ **Source:** TP6-2 **Sample No.:** BAG-1 **Elev./Depth:** 0-0.2m

Remarks:
○

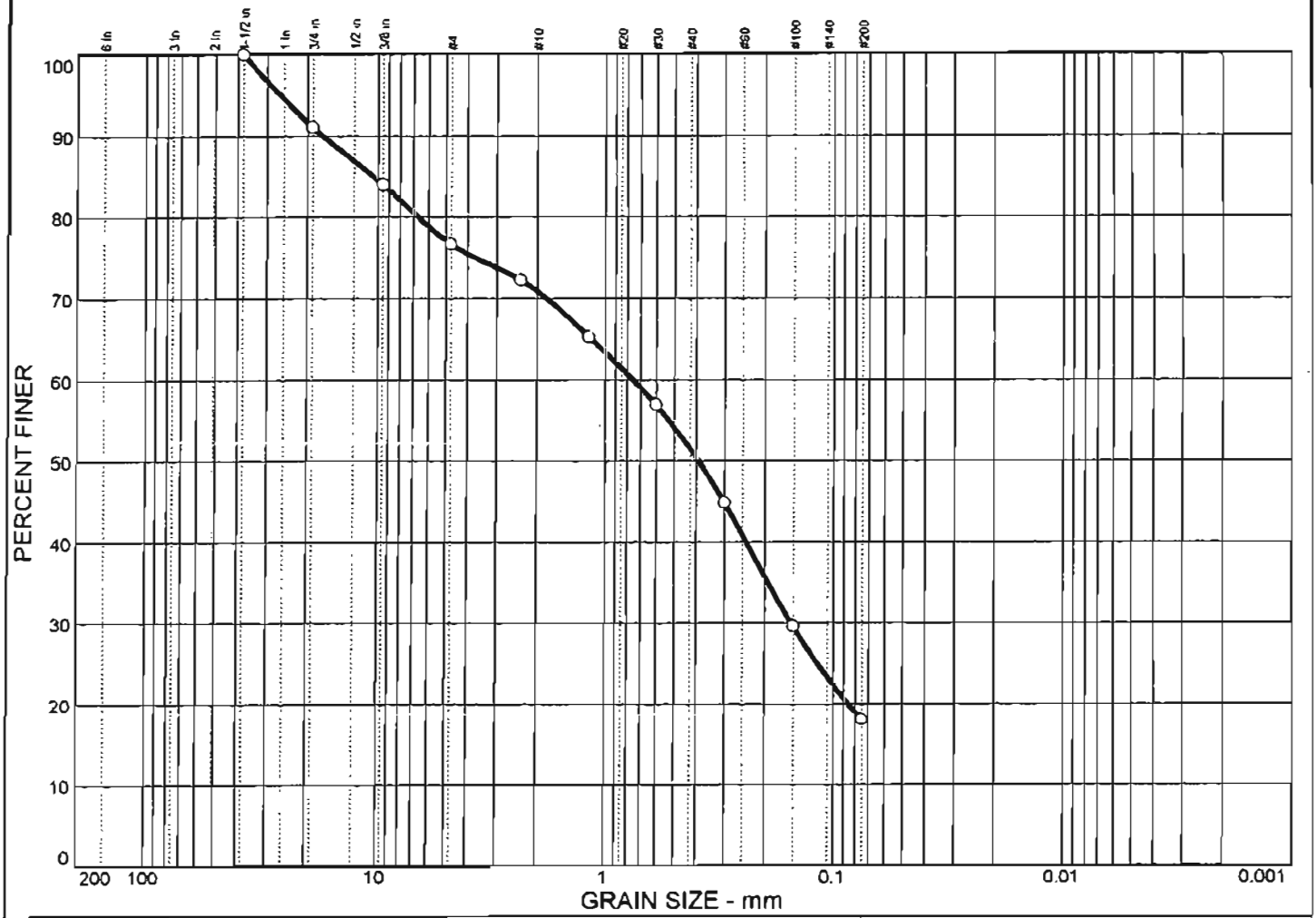
PARTICLE-SIZE DISTRIBUTION REPORT

URS GREINER WOODWARD CLYDE

D-15

Fig.

PARTICLE-SIZE DISTRIBUTION REPORT



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	8.9	14.4	5.8	19.3	33.6	18.0	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			10.5	0.752	0.389	0.153				

MATERIAL DESCRIPTION	USCS	AASHTO
○ Silty sand with gravel	SM	

Project No. 68FHAT0027 **Client:**
Project: BEAR TOOTH HIGHWAY

○ **Source:** TP6-3 **Sample No.:** BAG-1 **Elev./Depth:** 0.3-0.6m

Remarks:
○

PARTICLE-SIZE DISTRIBUTION REPORT

URS GREINER WOODWARD CLYDE

D-16

Fig.

Table D-2

UNCONFINED COMPRESSIVE STRENGTH
ASTM D 2938

CLIENT: URS Greiner Woowward Clyde JOB NO.: 2093-92

LOCATION: Bear tooth Highway DATE TESTED: 1/11-1/12/00 HN

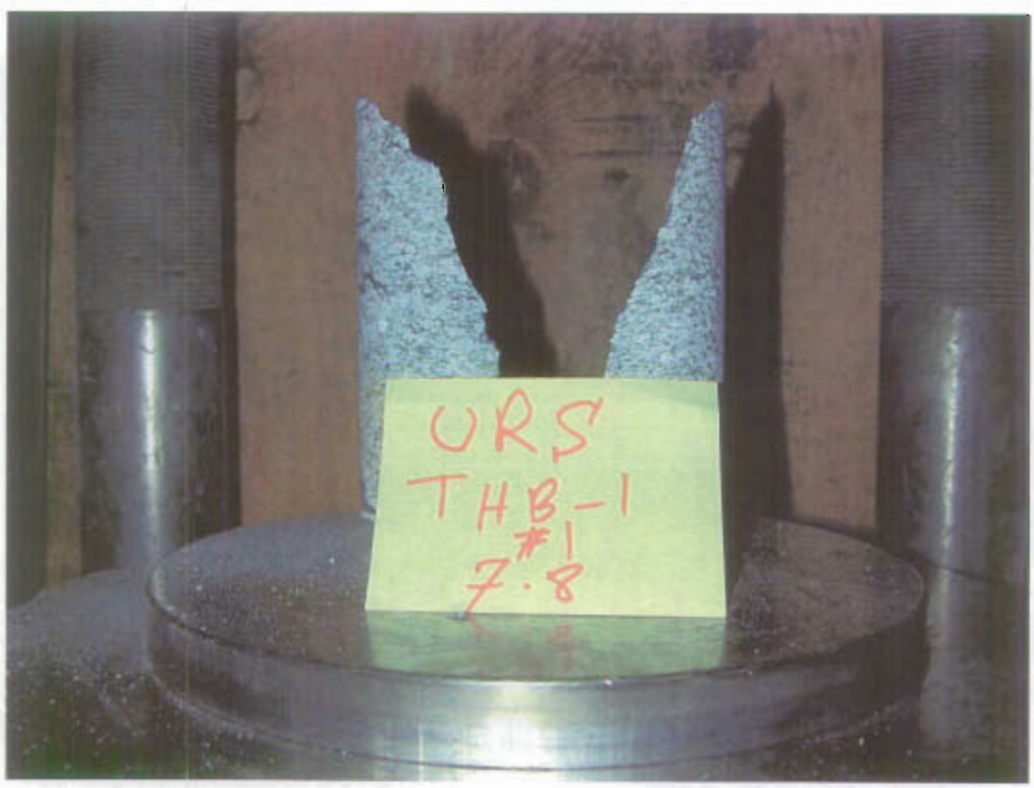
Specimen ID			Diameter	Length	Mass	Moisture	Wet	Dry	Failure	Compressive
Boring	Depth (m)	Sample No.	(in.)	(in.)	(gms)	Content (%)	Density (pcf)	Density (pcf)	Load (lb)	Strength (psi)
THB-1	7.8	1	2.414	5.422	1081.58		166.0		38,000	8,300
THB-2	10.8	1	2.002	3.078	412.77		162.3		17,500	*5370
THB-4	7	1	1.997	3.585	477.77		162.1		39,750	*12520
THB-5	6.17	1	2.402	4.106	780.01		159.7		47,250	*10220
THB-8	11	1	2.405	5.108	1120.40		183.9		31,000	6,820
THB-11	5.2	1	2.391	4.333	819.24		160.4		20,500	*4510
THB-12	6.1	1	2.404	4.583	897.00		164.3		55,500	*12160
THB-14	9.3	1	2.411	4.602	826.46		149.9		17,750	*3870
THB-16a	4.4	1	2.411	3.774	740.34		163.7		55,000	*11660
THB-19	3.4	1	2.406	4.134	851.98		172.7		28,000	*6040
THB-20	5.6	1	2.413	5.185	1019.46		163.8		78,000	17,060
TH2-2	7.95	1	1.988	4.076	542.18		163.3		40,500	13,050
TH2-3	8.2	1	2.403	5.170	1014.10		164.8		51,000	11,250
TH2-5	11.1	1	1.993	4.102	525.86		156.5		23,250	7,450
TH2-6	10.45	1	2.409	5.463	1143.43		174.9		28,250	6,200
TH2-7	7.9	1	1.997	4.064	540.05		161.6		44,500	14,210
TH2-8	13.8	1	2.398	5.256	1105.00		177.3		20,000	4,430
TH2-11	15.9	1	2.405	5.152	1019.00		165.9		42,500	9,360

Notes and Comments:

* Indicates L/D < 2.0. Correction Factor from ASTM D 2938 used.
C=Ca/[0.88+0.24b/h]
Ca = Failure Load / Surface Area
b = Sample Diameter
h = Sample Length

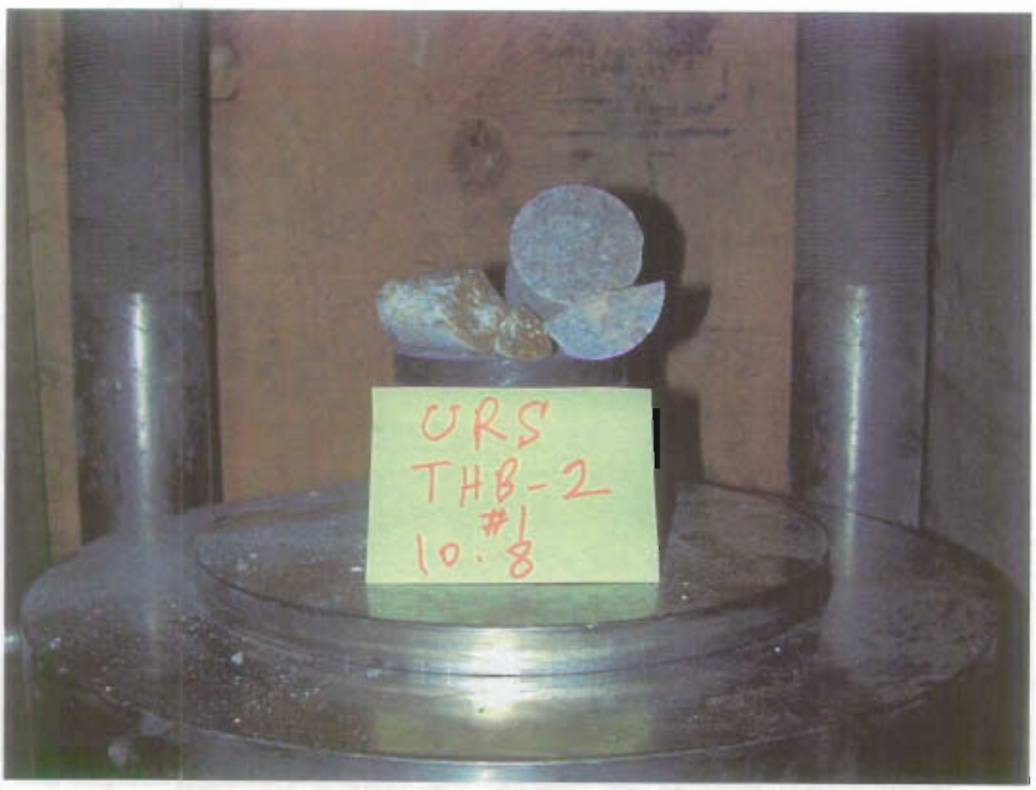
Data Entered By: HN Date: 11/12/2000
Data Checked By: CJ Date: 11/13/00
Filename: WCUCSR4

ADVANCED TERRA TESTING, inc.



Filename: WCDPTH78

ADVANCED TERRA TESTING INC.



Filename: WCOPTH12

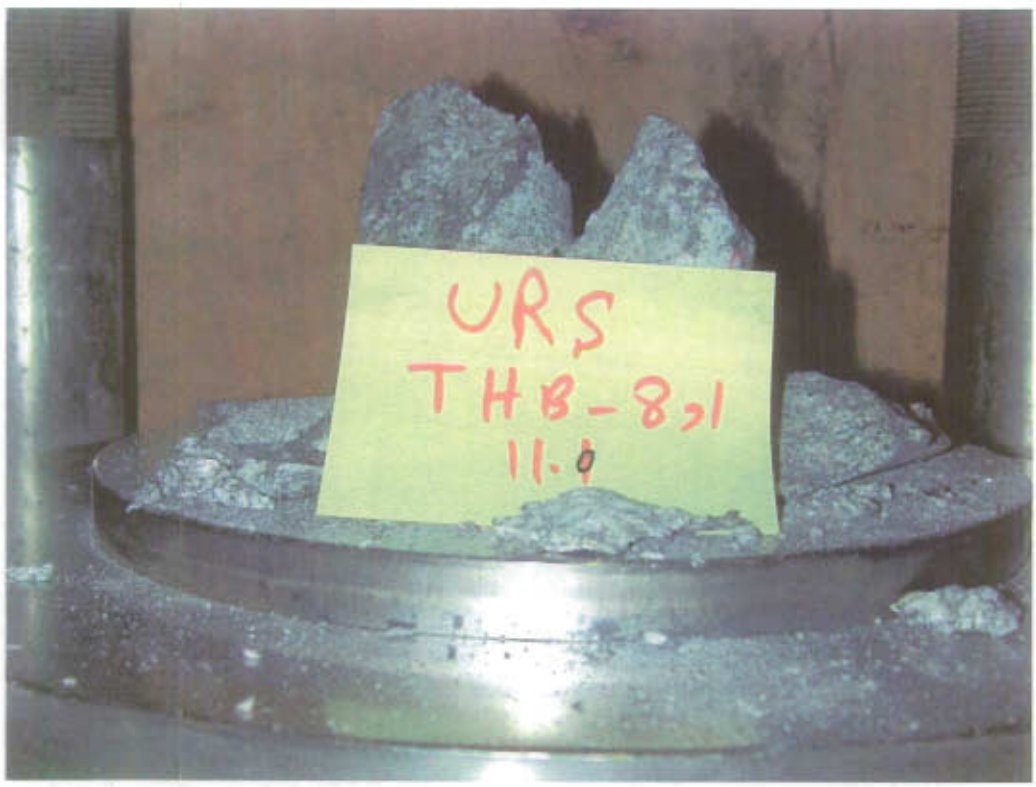
ADVANCED TERRA TESTING INC.

Filename: WCDPTH78

ADVANCED TERRA TESTING INC.

Filename: WCODTH61

ADVANCED TERRA TESTING INC.



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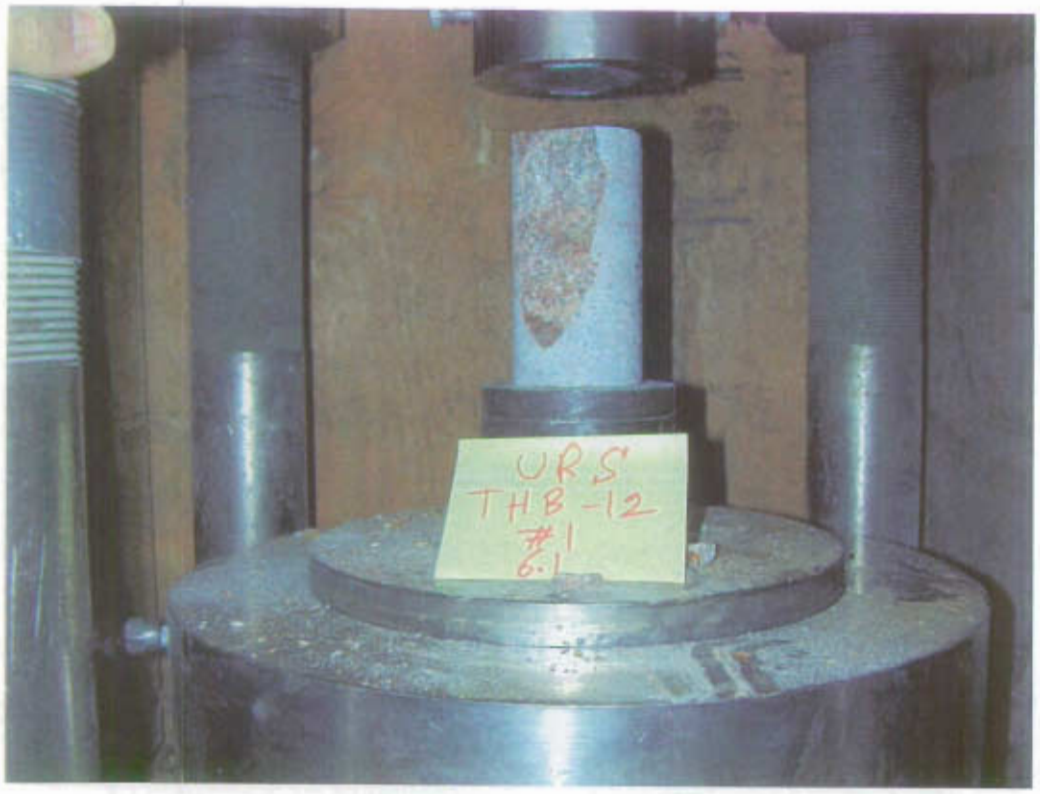
ADVANCED TERRA TESTING INC.

Filename: WCDPTH52

ADVANCED TERRA TESTING INC.



Filename: WCDPTH61

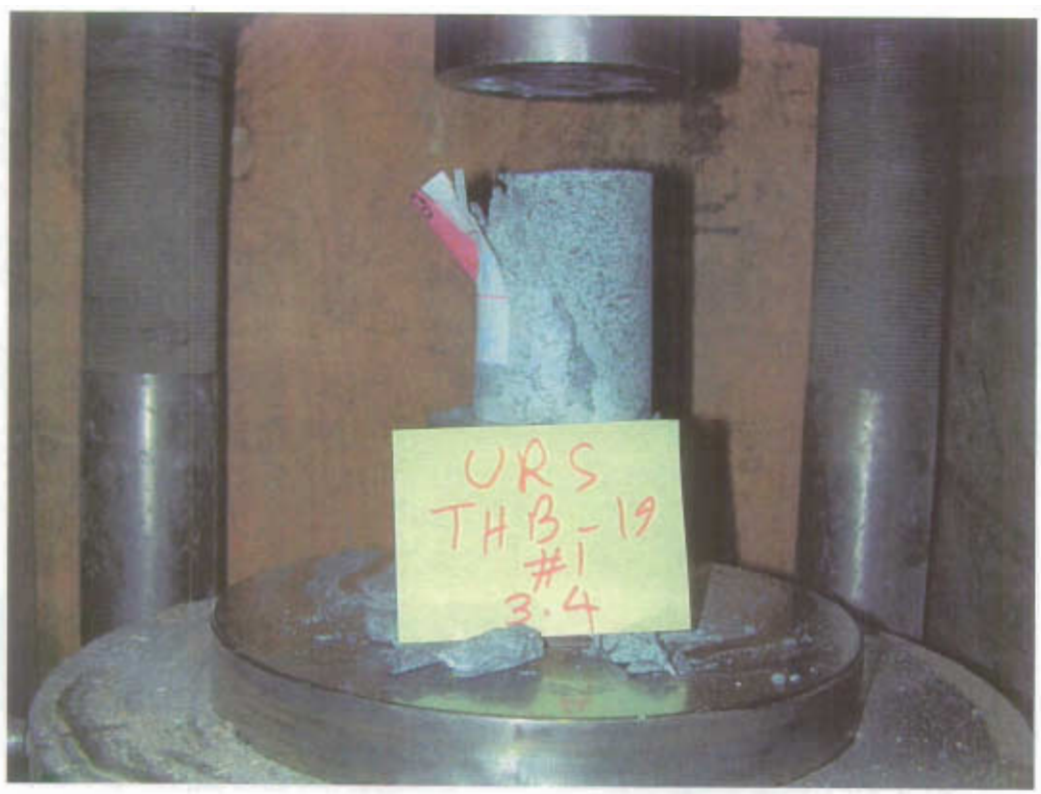
ADVANCED TERRA TESTING INC.

Filename: WC DPT H93

ADVANCED TERRA TESTING INC.

Filename: WCDPTH44

ADVANCED TERRA TESTING INC.



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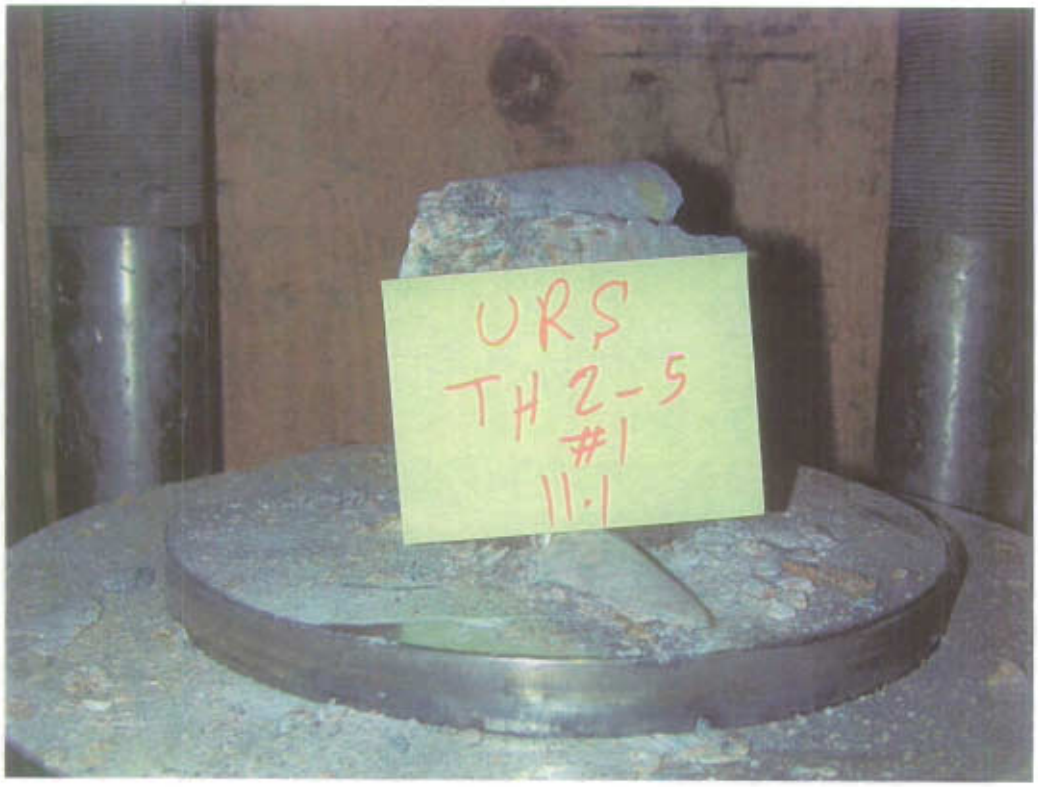

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Filename: WCDPTH15
ADVANCED TERRA TESTING INC.

Table D-3

DIAMETRAL POINT LOAD TEST
ASTM D 5731

CLIENT: URS Greiner Woowward Clyde

JOB NO.: 2093-92

LOCATION: Bear tooth Highway

DATE TESTED: 11/8/00 HN

PAGE: 1 of 4

Specimen ID Boring, Depth(m)	Length (in.)	Diameter (in.)	De^2 (in^2)	Gauge Failure Load (psig)	P (lb)	Is	F	Is(50)	C	Compressive Strength (psi)	Loading with respect to Fracture	Failure Mode
THB-1, 1, 7.8	3.084	2.404	5.779	500	1035.0	179.1	1.1	194.5	25.0	4,860	Parallel	C
THB-1, 2, 4.3	3.154	2.416	5.837	3000	6210.0	1063.9	1.1	1158.3	25.0	28,980	N/A	S
THB-1, 3, 6.4	2.860	2.409	5.803	2800	5796.0	998.7	1.1	1086.0	25.0	27,130	N/A	S
THB-2, 1, 10.8	2.727	2.001	4.004	700	1449.0	361.9	1.0	362.0	23.1	8,350	Parallel	C
THB-2, 2, 7.0	2.109	1.995	3.980	2000	4140.0	1040.2	1.0	1039.0	23.0	23,940	N/A	S
THB-2, 3, 5.8	1.983	1.991	3.964	1600	3312.0	835.5	1.0	833.8	23.0	19,200	Perpendicular	S
THB-4, 2, 8.6	2.924	1.995	3.980	900	1863.0	468.1	1.0	467.6	23.0	10,770	Perpendicular	C
THB-4, 3, 9.6	2.432	1.991	3.964	1300	2691.0	678.8	1.0	677.5	23.0	15,600	Perpendicular	C
THB-5, 2, 8.6	3.293	2.409	5.803	3000	6210.0	1070.1	1.1	1163.5	25.0	29,070	N/A	S
THB-5, 3, 10.65	2.993	2.398	5.750	2000	4140.0	719.9	1.1	781.2	24.9	19,480	Parallel	C
THB-8, 2, 11.6	2.844	2.404	5.779	1900	3933.0	680.5	1.1	739.3	25.0	18,450	N/A	S
THB-8, 3, 9.6	2.736	2.406	5.789	2600	5382.0	929.7	1.1	1010.4	25.0	25,230	N/A	S

Notes:

L: Sample Length
D: Sample Diameter
De^2: Exquivalent Diameter = D^2
P: Gauge Failure Load * Piston area (2.07 in^2)
Is: Point Load Index Strength = P/De^2
F: Size Correction Factor to 2.0 in = (De/2.0)^0.45
Is(50): Size Corrected Index Strength = F* Is
C: Factor to Estimate Compressive Strength related to Core Diameter
Compressive Strength in psi = C * Is(50)

Failure Modes:

F: Fracture Controlled
S: Substance Controlled
C: Combination Substance & Fracture

Data Entered By: HN Date: 11/12/2000

Data Checked By: CV Date: 11/19/00

Filename: WCPTLOA1

ADVANCED TERRA TESTING, inc.

Table D-3
(continued)
DIAMETRAL POINT LOAD TEST
ASTM D 5731

CLIENT: URS Greiner Woowward Clyde JOB NO.: 2093-92

LOCATION: Bear tooth Highway DATE TESTED: 11/8/00 HN
PAGE: 2 of 4

Specimen ID Boring, Depth(m)	Length (in.)	Diameter (in.)	De^2 (in^2)	Gauge Failure Load (psig)	P (lb)	Is	F	Is(50)	C	Compressive Strength (psi)	Loading with respect to Fracture	Failure Mode
THB-11, 2, 3.1	2.674	2.409	5.803	400	828.0	142.7	1.1	155.1	25.0	3,880	N/A	S
THB-11, 3, 4.0	2.698	2.422	5.866	300	621.0	105.9	1.1	115.4	25.0	2,890	Perpendicular	S
THB-12, 2, 10.6	2.611	2.408	5.798	1500	3105.0	535.5	1.1	582.1	25.0	14,540	Perpendicular	S
THB-12, 3, 9.3	2.810	2.411	5.813	1400	2898.0	498.5	1.1	542.3	25.0	13,550	N/A	S
THB-14, 2, 10.35	3.068	2.409	5.803	1300	2691.0	463.7	1.1	504.2	25.0	12,600	Parallel	C
THB-16a, 2, 6.3	2.730	2.411	5.813	1400	2898.0	498.5	1.1	542.3	25.0	13,550	Perpendicular	C
THB-16a, 3, 7.65	2.701	2.412	5.818	2200	4554.0	782.8	1.1	851.6	25.0	21,290	Parallel	C
THB-19, 2, 4.9	2.789	2.334	5.448	1800	3726.0	684.0	1.1	733.2	24.6	18,060	Perpendicular	S
THB-19, 3, 5.8	2.628	2.423	5.871	1800	3726.0	634.7	1.1	691.9	25.0	17,330	Perpendicular	C
THB-20, 1, 5.6	3.006	2.410	5.808	3000	6210.0	1069.2	1.1	1162.8	25.0	29,050	N/A	S
THB-20, 2, 6.9	2.938	2.409	5.803	2400	4968.0	856.1	1.1	930.8	25.0	23,250	N/A	S
THB-20, 3, 6.1	2.191	2.419	5.852	2150	4450.5	760.6	1.1	828.5	25.0	20,740	N/A	S

Notes:
L: Sample Length
D: Sample Diameter
De^2: Exquivalent Diameter = D^2
P: Gauge Failure Load * Piston area (2.07 in^2)
Is: Point Load Index Strength = P/De^2
F: Size Correction Factor to 2.0 in = (De/2.0)^0.45
Is(50): Size Corrected Index Strength = F* Is
C: Factor to Estimate Compressive Strength related to Core Diameter
Compressive Strength in psi = C * Is(50)

Failure Modes:
F: Fracture Controlled
S: Substance Controlled
C: Combination Substance & Fracture

Data Entered By: HN Date: 11/12/2000
Data Checked By: W Date: 11/19/00
Filename: WCPTLOA2

ADVANCED TERRA TESTING, inc.

Table D-3
(continued)

DIAMETRAL POINT LOAD TEST
ASTM D 5731

CLIENT: URS Greiner Woodward Clyde JOB NO.: 2093-92

LOCATION: Bear tooth Highway DATE TESTED: 11/8/00 HN
PAGE: 3 of 4

Specimen ID Boring, Depth(m)	Length (in.)	Diameter (in.)	De^2 (in^2)	Gauge Failure Load (psig)	P (lb)	Is	F	Is(50)	C	Compressive Strength (psi)	Loading with respect to Fracture	Failure Mode
TH4-1, 2, 5.5	2.667	2.411	5.813	1300	2691.0	462.9	1.1	503.6	25.0	12,580	Parallel	C
TH4-2, 1, 7.9	2.682	2.426	5.885	1400	2898.0	492.4	1.1	537.1	25.1	13,460	N/A	S
TH4-2, 2, 7.1	2.998	2.425	5.881	3000	6210.0	1056.0	1.1	1151.7	25.1	28,860	Perpendicular	S
TH4-3, 1, 7.7	2.997	2.433	5.919	400	828.0	139.9	1.1	152.8	25.1	3,830	Perpendicular	C
TH4-3, 2, 8.9	2.766	2.434	5.924	1500	3105.0	524.1	1.1	572.5	25.1	14,370	Perpendicular	C
TH2-2, 2, 7.35	2.960	2.008	4.032	1800	3726.0	924.1	1.0	925.8	23.1	21,390	Perpendicular	S
TH2-2, 3, 8.25	3.029	1.996	3.984	2400	4968.0	1247.0	1.0	1245.9	23.0	28,710	N/A	S
TH2-3, 2, 8.4	2.429	2.410	5.808	2400	4968.0	855.4	1.1	930.2	25.0	23,240	Perpendicular	S
TH2-3, 3, 9.6	2.861	2.405	5.784	2800	5796.0	1002.1	1.1	1088.8	25.0	27,180	Parallel	C
TH2-5, 2, 13.27	2.711	2.010	4.040	2800	5796.0	1434.6	1.0	1437.8	23.1	33,230	Prependicular	S
TH2-5, 3, 12.4	2.781	1.994	3.976	2400	4968.0	1249.5	1.0	1247.8	23.0	28,750	N/A	S
TH2-6, 2, 9.2	3.012	2.437	5.939	1000	2070.0	348.5	1.1	381.0	25.1	9,570	N/A	S

Notes:
L: Sample Length
D: Sample Diameter
De^2: Exquivalent Diameter = D^2
P: Gauge Failure Load * Piston area (2.07 in^2)
Is: Point Load Index Strength = P/De^2
F: Size Correction Factor to 2.0 in = (De/2.0)^0.45
Is(50): Size Corrected Index Strength = F* Is
C: Factor to Estimate Compressive Strength related to Core Diameter
Compressive Strength in psi = C * Is(50)

Failure Modes:
F: Fracture Controlled
S: Substance Controlled
C: Combination Substance & Fracture

Data Entered By: HN Date: 11/12/2000
Data Checked By: [signature] Date: 11/14/00
Filename: WCPTLOA3

ADVANCED TERRA TESTING, inc.

Table D-3
(continued)

DIAMETRAL POINT LOAD TEST
ASTM D 5731

CLIENT: URS Greiner Woowward Clyde JOB NO.: 2093-92

LOCATION: Bear tooth Highway DATE TESTED: 11/8/00 HN
PAGE: 4 of 4

Specimen ID Boring, Depth(m)	Length (in.)	Diameter (in.)	De^2 (in^2)	Gauge Failure Load (psig)	P (lb)	Is	F	Is(50)	C	Compressive Strength (psi)	Loading with respect to Fracture	Failure Mode
TH2-6, 3, 8.85	2.849	2.404	5.779	1800	3726.0	644.7	1.1	700.4	25.0	17,480	N/A	S
TH2-7, 2, 10.0	2.890	2.002	4.008	1000	2070.0	516.5	1.0	516.7	23.1	11,920	Perpendicular	C
TH2-7, 3, 10.7	2.822	2.018	4.072	2400	4968.0	1219.9	1.0	1224.9	23.1	28,360	N/A	S
TH2-8, 2, 14.1	2.947	2.394	5.731	800	1656.0	288.9	1.1	313.3	24.9	7,800	Perpendicular	C
TH2-8, 3, 15.45	2.890	2.407	5.794	2800	5796.0	1000.4	1.1	1087.4	25.0	27,150	N/A	S
TH2-11, 2, 15.3	3.172	2.419	5.852	3000	6210.0	1061.3	1.1	1156.1	25.0	28,940	N/A	S
TH2-11, 3, 14.5	3.220	2.427	5.890	3000	6210.0	1054.3	1.1	1150.2	25.1	28,830	N/A	S

Notes:
L: Sample Length
D: Sample Diameter
De^2: Exquivalent Diameter = D^2
P: Gauge Failure Load * Piston area (2.07 in^2)
Is: Point Load Index Strength = P/De^2
F: Size Correction Factor to 2.0 in = (De/2.0)^0.45
Is(50): Size Corrected Index Strength = F* Is
C: Factor to Estimate Compressive Strength related to Core Diameter
Compressive Strength in psi = C * Is(50)

Failure Modes:
F: Fracture Controlled
S: Substance Controlled
C: Combination Substance & Fracture

Data Entered By: HN Date: 11/12/2000
Data Checked By: CJ Date: 11/14/00
Filename: WCPTLOA4

ADVANCED TERRA TESTING, inc.

Appendix E

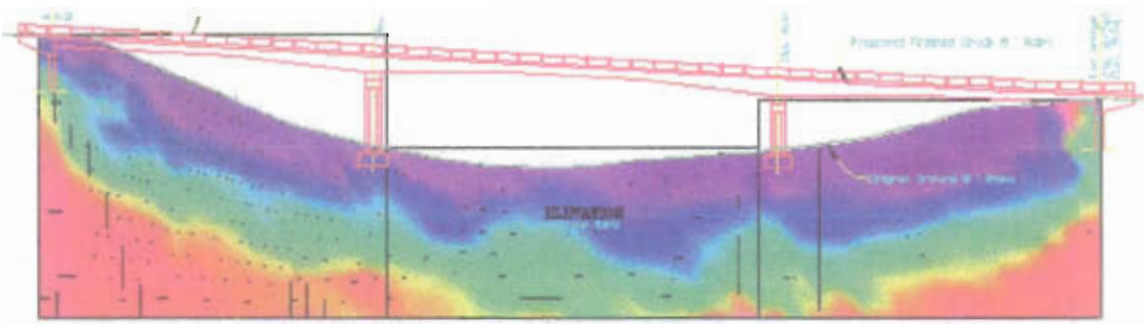
Appendix E
Seismic Tomography

Final Report

Tomographic Ground Imaging For the Beartooth Highway U.S. 212, Wyoming

Submitted to:

URS Corporation
Stanford Place III, Suite 1000
4582 S. Ulster St.
Denver, CO 80237-2637



Submitted by:

NSA Engineering, Inc.
15000 West Sixth Avenue, Suite 100
Golden, CO 80401
(303) 277-9920

December 8, 2000

Disclaimer

NSA's findings are based upon and have been developed in accordance with generally accepted scientific principles and professional judgment with resultant subjective interpretations. Professional judgments expressed herein are based on the facts currently available within the limits of existing data, scope of work, budget, and schedule.

This report completes the Project "Tomographic Ground Imaging for the Beartooth Highway, U.S. 212, Wyoming, NSA Proposal #PRV(00)61R3, for the URS Corporation.

Bart Hoekstra
Field Operations Manager - Seismics
NSA Engineering, Inc.

EXECUTIVE SUMMARY

As part of the geotechnical study for a new bridge alignment and widening of Highway 212 west of the summit of Beartooth Pass, Wyoming, the URS Corporation retained NSA Engineering, Inc. (NSA), of Golden, Colorado to utilize their RockVision3D™ seismic tomography to assess geotechnical site conditions, particularly with regard to the overburden/rock interface and rock quality. Specific objectives of this survey include...

- Mapping of fill/talus/rock interfaces,
- Identification of possible "weak" zones in the portion of the study area that will require blasting.

The seismic survey was performed in three separate areas defined as the bridge, knob, and side hill. The survey methods applied involved cross-borehole, surface-to-borehole, and surface-to-surface source-to-receiver configurations.

Conclusions:

The study effectively characterized the subsurface ground conditions. Based on the tomographic images obtained from the seismic survey, combined with available geotechnical and geologic information and engineering experience, the survey results, as shown in Figures A and B, are summarized as follows:

1. Bridge Area:
   - The bedrock surface, as shown in Figure 7, generally follows the ground surface elevation, with a slightly deeper trough to the west of THB-4.
   - The trough may indicate the presence of a fault zone running down the length of the ravine.
   - A steeply dipping bedrock surface at an approximate 45° angle and oriented NW-SE was identified. It is suggested that the dip angle be further investigated using the tomography method with additional boreholes drilled perpendicular to the bridge alignment, as originally proposed.

2. Knob Area:
   - The tomographic velocity contours of 3,000 m/s approximate the shape of the bedrock surface.
   - The rock mass in this area was intact, and no low-velocity zones were detected.

3. Side Hill Area:
   - A general trend of higher velocity zones that indicate the bedrock surface (high-strength material) was identified in the eastern section of the road.
   - The velocity profile sections TH2-3 and TH2-4 in Figure 21 show the general structure of the bedrock surface.

20. Projected elevation view of a vertical slice of the velocity tomogram along the center line of the proposed "Z" wall.
21. Velocity tomograms at three locations of vertical slices perpendicular to alignment.

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10. Velocity tomogram parallel to ravine axis at Station 41+444 looking east.
11. Source and receiver locations for the knob area.
12. Time-distance plot – knob area.
13. Ray path coverage – knob area.
14. Velocity tomogram along vertical plane through THB-5 looking toward the west in knob area at Station 41+520.
15. Horizontal slice of velocity tomogram at existing roadbed elevation (2692 m).
16. Source and receiver locations for side hill area.
17. Time-distance plot for the side hill area.
18. Ray path coverage for the side hill area.
19. Plan view of horizontal slice of tomographic velocities at the approximate elevation of the base of "Z" wall (2688 m)

Figures (Continued)

20. Projected elevation view of a vertical slice of the velocity tomogram along the center line of the proposed "Z" wall.
21. Velocity tomograms at three locations of vertical slices perpendicular to alignment.

SECTION 1.0 INTRODUCTION

NSA Engineering, Inc. (NSA) was contracted by the URS Corporation to conduct a cross-borehole and surface seismic tomography survey along an area of Highway 212 near the summit of Beartooth Pass in Wyoming. The tomographic study was conducted as part of an overall geotechnical study included in the design phase for a highway widening and realignment project. The objective of the tomographic survey was to...

- Map the fill/talus/rock interfaces underlying the existing highway,
- Map zones of anomalous high or low velocity within the rocks of the knob area that will require removal.

NSA applied its RockVision3D™ seismic tomographic imaging system to construct 3-D tomographic images to achieve these objectives.

This report includes a description of the test procedures, data analysis, and interpretation of results.

Seismic tomography techniques have been used for geological and geotechnical site characterization including mapping the extent of fractured zones, geologic anomalies, contact zones, and karst features. The same type of behavior may be noted in rocks of varying lithology, as harder, more competent materials propagate seismic waves at higher velocity and lower attenuation than softer, less competent or less consolidated rocks. Initiating a seismic event underground, such as triggering an air gun, results in the generation of several different seismic wave types. The fastest waves are compressional, or P-waves. P-waves propagate with particle motion parallel to the direction of propagation. Slower shear or S-waves propagate with particle motion orthogonal to the direction of propagation. These different wave modes have different sensitivities to fracture orientations and degrees of consolidation.

Tomography is a method of generating images of structural features within a body by propagating energy through the rock mass from multiple viewing angles and reconstructing images representative of the interior. The path followed by the seismic wave from the source to the receiver is represented as a ray. Velocity tomography is based on the time required for each ray to travel through the rock from the source to the receiver.

NSA engineers conducted a tomographic survey employing a variety of configurations of downhole sources and receivers and surface sources and receivers. The surface sources were a combination of hammer blows and Kinepak™ explosive charges. All explosive detonations were conducted under the supervision of John Nichol of the URS Corporation.

SECTION 2.0 SURVEY PROCEDURE

A three-dimensional seismic survey was conducted using the RockVision3D™ tomographic imaging system. This section provides a description of the equipment and procedures used to collect the data for the survey.

2.1 Equipment

A summary of the equipment used for this survey is shown in Table 1.

Table 1. Equipment used for survey.

Component	Equipment	Specifications
Seismic Recording System	Geometrics R-48 Strataview	48 channel, 24 bit A/D, 110 dB dynamic range, 4,000 samples/sec, 10 Hz Locut filter
Downhole receivers	Sercel DH-5 Hydrophones	Two strings of 12 phones, 2 m spacing, 24 dB preamp, Frequency Bandwidth from 7 Hz To 4.7 kHz
Surface Receivers	Mark Products 40 Hz. Geophones	24 single-element phones
Seismic Source	Detonator Caps	Downhole only
	Hammer Blows	Surface Only
	Kinepak™ - 1/4 to 1\2 lb	Surface and downhole

The data quality varied widely across the site. In most cases, the detonator caps were tried first in the borehole, and in many cases provided excellent data; however, in some areas there was insufficient energy to accurately obtain a first arrival time. In those cases, Kinepak™ was used. In all cases, however, the Kinepak™ destroyed the borehole below the shot, making reentry below the level of the shot impossible. The detonator caps were not as destructive and, in many cases, left the boreholes intact. However, in most boreholes in which the caps were used, the casing eventually cracked, and the borehole was unable to hold water. At this point, Kinepak™ was used. The charges were lowered into the hole using a metal chain to ensure that the proper depths were obtained.

Hammer blows were also attempted first at most shotpoints and, in most cases, provided adequate signal. However, on some of the long-distance shots, Kinepak™ shots were required. Examples of the data received from these three types of sources (cap, Kinepak™, and hammer) are shown in Figure 1.

Traffic control was handled by subcontractors to URS, and, generally, one lane was shut down at all times so that the boreholes were accessible, but traffic could traverse the survey area with minimal delay. Prior to the activation of explosive charges, all traffic was stopped for safety purposes. In general, the traffic was very light, and caused little disruption in the data acquisition efforts.

Section 2.2 Data Acquisition

A schedule of the data acquisition activities is shown in Table 2.

Table 2. Schedule of data acquisition activities.

Date	Activity
Sep 8	Mobilize to Cooke City, WY
Sep 9	Knob Area – Borehole THB-5
Sep 10	Knob Area – Borehole THB-5 Side Hill Area – Boreholes TH2-1 to TH2-3 and TH2-4
Sep. 11	Side Hill Area – Boreholes TH2-4 to TH2-9 and TH2-7 – Boreholes TH2-8 to TH2-7 and TH2-9
Sep 12	Side Hill Area – Geophones Only using Boreholes TH2-4 and TH2-2
Sep 13	Bridge Area – Boreholes THB-4 to THB-2 and THB-5
Sep 13-14	Demobilize to Denver, CO

The hydrophones were lowered into the boreholes until the bottom of the hole was reached. At this point, the length of the string was measured. In order to obtain better coverage in the knob area, the hydrophones were raised on alternate shots by 1 m and then lowered on the next shot to effectively obtain 1-m sampling along the length of the borehole.

The surface geophones were planted in the side of the road along the side hill area between boulders or in soil where possible. The quality of the geophone coupling to the ground was highly variable, and, in some cases, the coupling in the areas between the boulders was poor.

The process of data acquisition for each seismic record was as follows:

1. The location of the source operator was verified by the operator of the seismograph via radio communication with the source operator.
2. The location of the source and receiver was noted in the field notes.
3. Traffic was stopped if necessary (i.e., for explosives detonation).
4. The noise monitor on the seismograph was viewed to ensure receiver operation and low noise conditions.
5. The source was activated
6. The resulting record was reviewed for the ability to make first arrival picks. In the case of hammer blows, up to eight blows were averaged or "stacked" to improve signal to noise. If an explosive charge did not provide adequate signal, a larger charge was used.

7. The data were saved to hard disk
8. The traffic was allowed to proceed.

After the acquisition of each shot and prior to the removal of geophones, the locations were surveyed using a Trimble GPS receiver, with differential corrections obtained from the Omnistar Satellite. The GPS unit was able to obtain x, y coordinates within accuracies on the order ±1 ft, but the vertical accuracies were worse and required correction using the topographic data provided by URS. It is believed that the final elevation accuracies were within ±1.5 ft. The final positions used for this interpretation for the source and geophone locations are provided in Appendix A.

SECTION 3.0 DATA ANALYSIS

NSA Engineering's tomographic imaging software was used to construct an inversion model from seismic energy propagated through the medium. Within the software, measured waves are propagated through a finite difference mesh to determine energy transmission through the medium. Mesh properties are iteratively adjusted using a variation of SIRT (Simultaneous Iterative Reconstruction Technique) until the model reaches equilibrium. The variation of SIRT used in RockVision3D™ allows stable and rapid convergence to the most probable statistical solution.

As the data were collected, they were loaded into the RockVision3D™ software package for on-site signal processing and tomogram generation. Data analysis requires several steps. The first step is a general assessment of the data quality on a record-by-record basis. Acceptable data are identified by the presence of a signal that has sufficient amplitude to be easily distinguished from background noise. This allows the arrival time of the direct seismic wave to be picked with confidence. Data were then filtered to remove undesired noise, such as noise from wind as well as car and truck noise. This process was carried out using a digital filter before automated arrival time picking was implemented. The raw waveform data were filtered to remove high- and low-frequency noise. Marginal data were identified and discarded from the analysis process. Approximately 90% of the data provided adequate signal-to-noise so that a travel time pick could be made.

The presence of boulders in the subsurface impacted the data quality. Boulders can produce erratic travel times due to the fact that the energy will travel from boulder to boulder and not necessarily in a straight path. Boulders that are in a loose matrix also attenuate the signal, resulting in low-frequency, poor signal-to-noise data.

The picks were corrected for pre-trigger recording, zero times from the trigger hydrophone, and velocity filtered. The signal acquisition times were mapped to source and receiver locations, and rays were generated to match source and receiver locations with the arrival pick.

Photographs of the three separate areas surveyed for this project are shown in Figure 2. The results and the processing for each of these zones are discussed separately in the following sections. Figure 3 shows the locations of the imaging areas in relation to the road and boreholes.

Section 3.1 Bridge Area

Area Description

The first was the bridge area, extending from stations 41+400 to 41+540. This survey was conducted using four boreholes (THB-1, THB-2, THB-4, and THB-5), along with surface geophone and surface sources. The surface sources were a combination of hammer blows and Kinepak™ charges. The ground was rocky, and, in the case of the boulder slope, coupling of the explosive energy into the ground was not very good and resulted in low-frequency signal. Low frequency signal reduces the accuracy of the travel time pick, which, in turn, can cause errors in the final tomogram output. Figure 4 shows the locations of the sources and receivers. The sources are shown in red and the receivers in blue. The source and receiver locations were deployed in a pattern to follow the approximate alignment of the proposed bridge, but with some extent perpendicular to the alignment. This was to compensate for the fact that the boreholes were aligned approximately in a 2-D manner along the alignment, and information on the dip of the bedrock was desired.

Data Analysis

The data were picked using the B-Spline automatic picker and then manually reviewed and edited. The picks were then edited to remove picks with apparent velocities less than 500 m/s and more than 5,000 m/s. The geometries were input into the program, and the resulting time-distance plot is shown in Figure 5.

The picks and their associated source and receiver locations were input into the tomographic inversion portion of the program. In order to constrain the inversion, a forward model was constructed using the surface locations of the sources and receivers and the depth of bedrock obtained from the boreholes. A bedrock velocity of 4,000 m/s and an overburden velocity of 500 m/s were used. A minimum number of points were used to construct the model to minimize the amount of constraint on the final result. The final model was constructed using a 2-m pixel size (a 2x2x2 m cube).

The final output of the inversion process is a model of the velocity volume that best fits the travel time data. The travel paths of the seismic energy from the sources to receivers (ray paths) are shown in Figure 6. Figures 7, 8, and 9 are slices that are cut through the tomographic volume. The colors represent the velocity at points inside the volume, with "cool" colors (purple through green) as lower velocity. The "hotter" colors represent higher velocity. Figure 7 is a vertical section that follows along the centerline of the bridge alignment. Figures 8 and 9 show vertical sections 2 m offset north and south (left and right

of the centerline) from the section in Figure 7. Figure 10 is a section that parallels the axis of the ravine and passes through borehole THB-2 at approximately Station 41+444. .

These figures show a bedrock surface that generally follows the ground surface, with a slightly deeper trough to the west of THB-4. This may be indicative of a fault zone running down the length of the ravine. In Figure 10, a section (location of the section is shown in Figure 4) that runs parallel to the center of the ravine (approximately NW-SE) shows steeply dipping bedrock, with a dip of approximately 45°. This is a rough estimate of the dip; a more accurate estimate using the tomographic method would have required boreholes oriented perpendicular to the bridge alignment as originally planned.

Section 3.2 Knob Area

Site Description

The knob area extends from approximately station 41+540 to 41+640 around the curve of a prominent granitic rock outcrop. The outcrop was nearly vertical in some areas. The data were acquired in this area using a combination of hammer blows and Kinepak™ charges, with downhole receivers in Borehole THB-5. The Kinepak™ was used primarily for shots that were farther away. In order to reach the locations for the shots along the steep sections of the outcrop, an experienced climber was roped off from the top of the knob and climbed down to the shot locations. Approximately 90 shots were obtained to image the zone. The location of the sources and receivers is shown in Figure 11.

Data Processing and Analysis

The data were processed in a similar manner to the data from the bridge area. The time-distance plot and ray paths are shown in Figures 12 and 13. A forward model was constructed in a similar manner to the forward model of the bridge area by using surface locations of the source and the depth of rock obtained from the borehole. In the case of the knob area, however, a rock velocity of 4,500 m/s was used, and an overburden velocity of 800 m/s was used for the fill area below the road. Figure 14 shows a slice through the volume along an approximately SSW to NNE trend through borehole THB-5. The location of this section is shown in Figure 13. The dip of the bedrock continues along the trend of the rock above the road. There appears to be a depression underlying the road that may be due to construction activities performed in the construction of the road. A horizontal slice through the volume at the approximate elevation of the existing road (2692 m) is shown in Figure 15.

Section 3.3 Side Hill Area

Site Description

The side hill area, extending from station 41+600 to 41+905, is the easternmost area surveyed and extended along a generally straight section along a steep rock outcrop as

shown in Figure 2c. The hydrophones were placed in boreholes, and geophones were placed along the road. Shots were located along the road between the geophones as well as in the boreholes. After discussions with URS personnel, it was decided to place the source and geophones down the slope, as there was an interest in mapping the bedrock surface to the south of the boreholes (below the road surface). Therefore, along the western section of this area, geophones and sources were placed on the fill slope below the road, and sources were placed in borehole TH2-2. The location of the sources and receivers is shown in Figure 16.

Data Processing and Analysis

The data were processed in a similar manner to the data from the other two areas. The time-distance plots and ray paths are shown in Figures 17 and 18. A forward model was constructed in a similar manner to the forward model of the bridge area by using surface locations of the source and the depth of rock obtained from the boreholes as well as fixing the depth of rock that directly underlay the road on the northern side. A rock velocity of 4,000 m/s was used, and an overburden velocity of 800 m/s was used for the fill area below the road. Figure 19 shows a horizontal slice through the tomographic volume at the approximate elevation of the base of the "Z" wall (2688 ft). Figure 20 shows a projected vertical slice along the approximate alignment of the "Z" wall. The profiles perpendicular to the road are shown in Figure 21, relative to design drawings overlain on the tomograms. The section through TH2-1 was on the edge of the data coverage area and does not appear to define the bedrock surface. The two sections through TH2-3 and TH2-4 do show the general structure of the bedrock surface, although a high-velocity artifact appears on the surface of the TH2-4 transect that is related to the processing of the data and not of the geology.

SECTION 4.0 CONCLUSIONS

A tomographic survey was conducted as part of a geotechnical design study for the widening and realignment of Highway 212 near the summit of Beartooth Pass, near Cook City, WY. Surface-to-borehole, cross-borehole, and surface-to-surface source-to-receiver configurations were used. The work was conducted in conjunction with URS personnel, who provided the capability and all explosive source material.

The data were processed using RockVision3D™ software to produce velocity tomograms. The tomograms showed the general structure of the bedrock surface underlying the study area. However, the data collected were limited by the terrain and the geometry of the boreholes and were, therefore, limited in the amount of information perpendicular to the alignment outside the lines between the boreholes. The boulder fill also impacted the quality of the final result, because of the erratic travel paths of the seismic energy through the boulders, as well as causing high attenuation of the signal. The attenuation resulted in poor signal-to-noise ratios in many areas.

Several new geometries were tried for this project due to the difficult site conditions. It appears that the use of geophones on the northern side of the road (nearer to the rock) and sources at and below the road was the most effective in imaging the geology in the eastern part of the survey (side hill area). In addition, the use of geophones scattered along the bridge alignment appeared to effectively image the geology and the structure of the bedrock surface. However, the limited extent of the array perpendicular to the alignment decreased the accuracy of the dip estimate perpendicular to the alignment.

The use of multiple shots into a single borehole appeared to image the bedrock surface in the knob area. The use of multiple receiver locations (i.e., additional boreholes) would have improved the ability of the method to image the geology.

APPENDIX A
Coordinates of Surface Sources and Geophones

UTM Coordinate Summary

Description	Easting, m	Northing, m	Elevation, m
Sources - Group 1: Knob			
A1	136403.74	4986375.37	2683.70
A2	136412.28	4986380.45	2686.00
A3	136417.05	4986384.96	2694.90
A4	136424.16	4986388.58	2694.90
A4.5	136432.69	4986390.17	2703.00
A5	136435.31	4986393.20	2708.00
A6	136450.01	4986400.91	2723.00
B1	136407.77	4986362.50	2681.10
B2	136417.29	4986366.82	2689.40
B3	136424.61	4986371.70	2694.00
B4	136432.15	4986377.25	2693.80
B4.5	136440.00	4986380.83	2704.50
B5	136450.46	4986381.90	2716.40
B6	136455.07	4986389.54	2725.00
C1	136423.04	4986344.57	2681.90
C2	136427.99	4986350.91	2688.00
C3	136435.23	4986360.34	2693.30
C4	136439.92	4986366.84	2693.30
C5	136449.79	4986374.79	2712.00
C6	136460.81	4986386.96	2727.50
D1	136442.90	4986335.91	2681.20
D2	136446.83	4986344.82	2688.60
D3	136449.33	4986353.99	2693.50
D4	136453.18	4986361.10	2692.90
D5	136455.81	4986369.74	2710.60
D6	136464.27	4986386.07	2728.10
E1	136460.64	4986336.66	2683.40
E2	136462.10	4986342.96	2688.80
E3	136464.72	4986350.98	2693.20
E4	136466.59	4986357.38	2692.70
E4.5	136462.91	4986369.56	2713.90
E5	136467.90	4986372.61	2715.20
E6	136469.64	4986385.87	2730.30
F1	136480.79	4986336.85	2683.80
F2	136479.64	4986342.41	2687.80
F2A	136478.95	4986342.73	2687.80
F3	136480.28	4986351.16	2692.90

F4	136480.36	4986356.79	2691.60
F4.5	136475.41	4986361.69	2697.20
F5	136476.52	4986369.30	2707.00
F6	136478.99	4986384.30	2725.00
Receivers - Group 1: Knob			
A-2	136429.49	4986365.64	2693.60
Sources - Group 2: TH2-1 to TH2-4			
Shot	136503.05	4986366.28	2693.50
Shot	136520.44	4986364.95	2692.60
Shot	136538.80	4986372.40	2692.50
Shot	136549.80	4986378.10	2693.50
Shot	136563.26	4986386.00	2694.70
Shot	136577.46	4986393.84	2695.50
Hammer	136594.24	4986398.10	2694.40
TH2-3	136551.65	4986382.08	2695.30
Receivers - Group 2: TH2-1 to TH2-4			
TH2-1	136506.40	4986361.01	2693.50
TH2-4	136588.29	4986400.83	2697.00
Geophone	136586.20	4986406.58	2697.40
Geophone	136582.73	4986404.75	2697.20
Geophone	136580.50	4986403.30	2697.00
Geophone	136577.56	4986401.98	2696.80
Geophone	136572.84	4986400.18	2696.60
Geophone	136569.28	4986396.92	2696.40
Geophone	136565.43	4986395.36	2696.25
Geophone	136561.83	4986393.37	2696.00
Geophone	136558.26	4986391.50	2695.80
Geophone	136555.86	4986390.67	2695.60
Geophone	136551.00	4986387.97	2695.40
Geophone	136547.71	4986386.12	2695.20
Geophone	136543.61	4986385.01	2695.00
Geophone	136539.90	4986383.45	2694.80
Geophone	136536.64	4986381.05	2694.60
Geophone	136532.63	4986379.91	2694.40
Geophone	136529.05	4986378.44	2694.20
Geophone	136525.47	4986376.49	2694.00
Geophone	136521.85	4986374.71	2693.80
Geophone	136517.96	4986373.39	2693.60
Geophone	136516.19	4986371.78	2693.40
Geophone	136511.45	4986368.75	2693.20
Geophone	136507.68	4986367.34	2693.00
Geophone	136505.70	4986366.14	2693.00

Sources - Group 3: TH2-4 to TH2-7

TH2-5	136588.63	4986397.60	2695.80
TH2-6	136646.53	4986427.74	2700.00
Hammer	136589.80	4986406.90	2697.40
Hammer	136600.52	4986413.20	2697.90
Hammer	136611.21	4986418.38	2698.50
Shot	136625.63	4986416.90	2697.60
Shot	136635.70	4986422.00	2698.30
Shot	136647.18	4986425.66	2699.40
Shot	136657.53	4986428.25	2699.10
Shot	136668.04	4986433.08	2700.00
Shot	136679.63	4986438.38	2700.50

Receivers - Group 3: TH2-4 to TH2-7

TH2-7	136688.40	4986440.91	2700.20
Geophone	136591.23	4986408.65	2697.50
Geophone	136594.85	4986410.72	2697.60
Geophone	136598.38	4986412.68	2697.80
Geophone	136601.85	4986414.71	2698.00
Geophone	136605.09	4986415.29	2698.20
Geophone	136608.80	4986416.99	2698.40
Geophone	136612.50	4986418.57	2698.60
Geophone	136616.03	4986420.18	2698.80
Geophone	136619.40	4986422.24	2698.90
Geophone	136623.18	4986424.22	2699.00
Geophone	136626.85	4986425.91	2699.20
Geophone	136629.91	4986427.61	2699.40
Geophone	136633.87	4986429.15	2699.60
Geophone	136637.69	4986430.62	2699.80
Geophone	136641.33	4986432.89	2700.00
Geophone	136644.30	4986434.56	2700.20
Geophone	136648.20	4986436.63	2700.40
Geophone	136651.81	4986438.29	2700.60
Geophone	136655.24	4986440.17	2700.80
Geophone	136658.81	4986441.35	2701.00
Geophone	136662.63	4986442.58	2701.10
Geophone	136665.86	4986444.48	2701.20
Geophone	136668.99	4986445.57	2701.30
Geophone	136672.33	4986447.36	2701.40

Sources - Group 4: TH2-7 to TH2-9

TH2-8	136740.50	4986464.32	2704.80
Hammer	136767.82	4986470.52	2706.10
Hammer	136754.80	4986470.40	2705.50
Hammer	136743.38	4986470.26	2705.00

Hammer	136732.47	4986470.06	2704.50
Hammer	136715.67	4986466.98	2703.70
Hammer	136703.91	4986461.96	2703.10
Hammer	136693.27	4986458.09	2702.70
Hammer	136682.20	4986453.20	2702.10
Shot	136684.61	4986438.69	2699.40
Shot	136684.61	4986438.69	2699.40
Shot	136697.44	4986443.56	2698.50
Shot	136712.01	4986453.68	2701.60
Shot	136727.81	4986454.63	2699.60
Shot	136740.31	4986457.26	2700.30
Shot	136762.01	4986457.22	2700.50
Shot	136770.66	4986457.12	2700.70
Shot	136785.86	4986456.64	2701.10
Shot	136804.93	4986455.78	2703.30
Shot	136825.30	4986449.13	2700.50

Receivers - Group 4: TH2-7 to TH2-9

TH2-7	136688.40	4986440.91	2700.20
TH2-9	136778.53	4986465.55	2706.10
Geophone	136680.61	4986452.57	2702.00
Geophone	136684.15	4986454.26	2702.20
Geophone	136687.70	4986456.14	2702.40
Geophone	136691.71	4986457.31	2702.60
Geophone	136695.52	4986458.62	2702.80
Geophone	136699.28	4986460.32	2703.00
Geophone	136702.79	4986461.35	2703.10
Geophone	136706.14	4986463.29	2703.20
Geophone	136709.50	4986465.41	2703.40
Geophone	136713.20	4986466.91	2703.60
Geophone	136717.94	4986467.16	2703.80
Geophone	136721.45	4986468.77	2704.00
Geophone	136725.80	4986469.50	2704.20
Geophone	136729.50	4986470.00	2704.40
Geophone	136734.60	4986470.80	2704.60
Geophone	136737.88	4986469.93	2704.80
Geophone	136740.60	4986470.50	2705.00
Geophone	136745.30	4986470.20	2705.10
Geophone	136749.40	4986470.40	2705.20
Geophone	136753.70	4986470.30	2705.40
Geophone	136757.10	4986470.40	2705.60
Geophone	136761.54	4986470.37	2705.80
Geophone	136765.67	4986470.54	2706.00
Geophone	136769.91	4986470.81	2706.20

Sources - Group 5: Slope between TH2-1 and TH2-4

TH2-1	136506.40	4986361.01	2693.50
TH2-2	136508.57	4986358.16	2692.10
TH2-3	136551.65	4986382.08	2695.20
TH2-4	136588.29	4986400.83	2697.00
Hammer	136530.47	4986370.91	2694.10
Hammer	136542.79	4986377.01	2694.90
Hammer	136556.10	4986383.00	2695.30
Hammer	136570.10	4986390.30	2696.00
Hammer	136588.90	4986400.00	2697.00
Hammer	136533.35	4986365.88	2690.90
Hammer	136545.58	4986372.04	2691.40
Hammer	136558.55	4986377.80	2691.90
Hammer	136572.88	4986384.49	2692.00
Hammer	136589.41	4986392.41	2692.60
Hammer	136590.33	4986386.85	2688.20
Hammer	136571.45	4986380.09	2689.40
Hammer	136558.21	4986374.86	2689.60
Hammer	136547.17	4986365.29	2686.00
Hammer	136534.69	4986359.35	2685.30
Hammer	136538.87	4986354.35	2680.20
Hammer	136560.96	4986366.03	2682.80
Hammer	136594.38	4986381.87	2683.80

Receivers - Group 5: Slope between TH2-1 and TH2-4

Geophone	136591.67	4986398.84	2695.90
Geophone	136588.45	4986397.66	2695.80
Geophone	136587.81	4986393.11	2693.50
Geophone	136587.75	4986389.81	2691.00
Geophone	136586.20	4986385.56	2688.60
Geophone	136584.48	4986382.78	2687.30
Geophone	136583.55	4986380.45	2686.00
Geophone	136580.20	4986383.10	2688.90
Geophone	136576.19	4986384.77	2691.20
Geophone	136573.20	4986386.19	2693.00
Geophone	136570.84	4986384.76	2692.00
Geophone	136569.97	4986380.84	2690.20
Geophone	136568.14	4986376.09	2687.50
Geophone	136563.92	4986373.54	2686.90
Geophone	136561.93	4986375.80	2689.10
Geophone	136557.47	4986377.91	2692.10
Geophone	136552.53	4986378.62	2693.70
Geophone	136551.51	4986373.43	2690.60
Geophone	136550.77	4986368.67	2687.20
Geophone	136550.45	4986364.19	2684.40

Geophone	136546.81	4986362.00	2683.80
Geophone	136541.43	4986364.35	2687.00
Geophone	136537.07	4986365.54	2689.70
Geophone	136532.01	4986366.75	2691.70

Sources - Group 6: A-2 to A-3 (east bridge)

A-4	136393.41	4986394.13	2686.00
Hammer	136423.71	4986373.91	2694.10
Hammer	136410.12	4986391.04	2693.80
Hammer	136385.09	4986403.93	2686.20
Hammer	136350.39	4986423.18	2689.20
Hammer	136349.64	4986417.25	2686.70
Hammer	136383.35	4986397.67	2684.20
Hammer	136406.65	4986388.10	2690.50
Hammer	136419.85	4986369.82	2692.20
Hammer	136351.20	4986412.20	2684.00
Hammer	136350.60	4986414.60	2685.20
Hammer	136351.80	4986462.20	2690.20
Hammer	136351.32	4986428.17	2691.00
Blast	136407.21	4986383.70	2689.00
Blast	136353.04	4986425.40	2689.90

Receivers - Group 6: A-2 to A-3 (east bridge)

A-2	136429.49	4986365.64	2693.60
A-3	136351.54	4986420.20	2687.90
Geophone	136422.10	4986366.92	2692.40
Geophone	136419.36	4986368.96	2692.00
Geophone	136419.32	4986371.74	2692.60
Geophone	136414.56	4986373.22	2689.70
Geophone	136414.30	4986377.21	2691.20
Geophone	136410.54	4986378.98	2689.20
Geophone	136408.78	4986384.42	2690.20
Geophone	136408.53	4986387.31	2691.40
Geophone	136403.93	4986389.67	2689.60
Geophone	136400.17	4986392.05	2688.50
Geophone	136393.25	4986392.27	2685.20
Geophone	136388.88	4986394.01	2684.40
Geophone	136391.77	4986399.29	2687.00
Geophone	136387.34	4986398.94	2685.60
Geophone	136381.53	4986402.40	2685.00
Geophone	136379.97	4986406.32	2685.40
Geophone	136377.31	4986410.08	2685.50
Geophone	136370.56	4986411.02	2684.20
Geophone	136365.62	4986413.00	2684.30
Geophone	136359.38	4986412.04	2683.50

Geophone	136356.87	4986417.22	2686.00
Geophone	136355.69	4986420.34	2687.50
Geophone	136351.21	4986422.65	2690.00
Geophone	136344.86	4986420.43	2688.90

Sources - Group 7: A-1 to A-4 (west bridge)

A-1	136305.09	4986424.06	2702.70
A-4	136393.41	4986394.13	2686.00
Hammer	136307.11	4986423.83	2702.60
Hammer	136337.10	4986421.10	2691.20
Hammer	136364.30	4986413.00	2684.20
Hammer	136364.88	4986410.21	2683.30
Hammer	136337.33	4986416.38	2688.80
Hammer	136309.58	4986419.86	2700.80

Receivers - Group 7: A-1 to A-4 (west bridge)

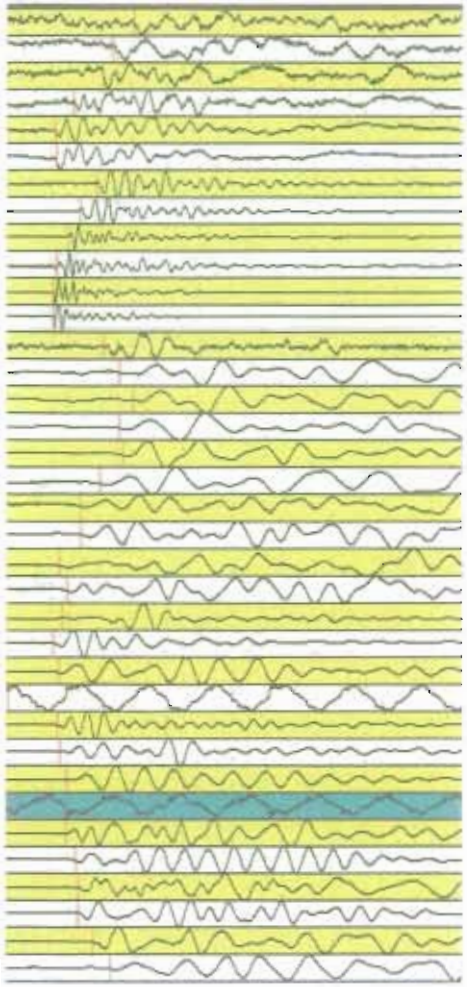
A-3	136351.54	4986420.20	2687.90
Geophone	136393.25	4986392.27	2685.20
Geophone	136388.88	4986394.01	2684.40
Geophone	136391.77	4986399.29	2687.00
Geophone	136387.34	4986398.94	2685.60
Geophone	136381.53	4986402.40	2685.00
Geophone	136379.97	4986406.32	2685.40
Geophone	136377.31	4986410.08	2685.50
Geophone	136370.56	4986411.02	2684.20
Geophone	136365.62	4986413.00	2684.30
Geophone	136359.38	4986412.04	2683.50
Geophone	136356.87	4986417.22	2686.00
Geophone	136355.69	4986420.34	2687.50
Geophone	136351.21	4986422.65	2690.00
Geophone	136344.86	4986420.43	2688.90
Geophone	136307.99	4986421.69	2702.70
Geophone	136313.19	4986424.28	2702.40
Geophone	136316.72	4986425.70	2701.90
Geophone	136323.11	4986422.79	2697.60
Geophone	136327.82	4986424.37	2696.70
Geophone	136330.55	4986424.81	2695.90
Geophone	136331.27	4986419.65	2692.80
Geophone	136335.99	4986418.44	2690.20
Geophone	136339.87	4986421.64	2690.50
Geophone	136344.48	4986421.01	2689.20

FIGURES

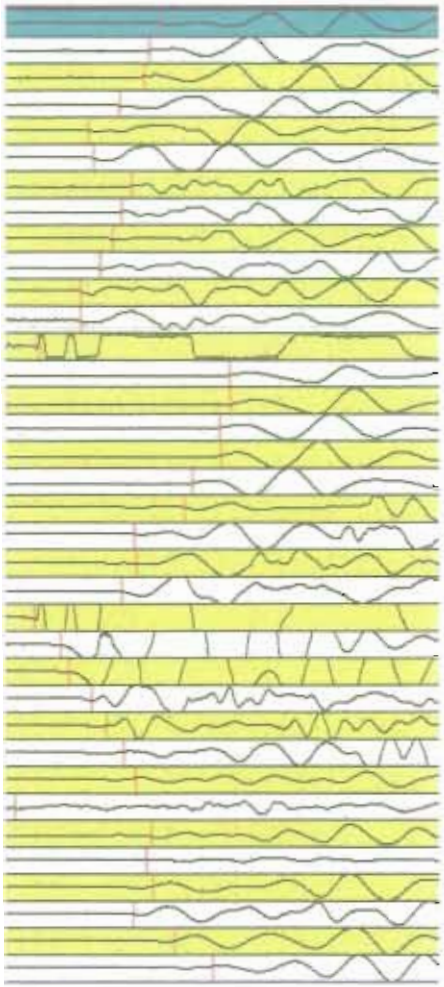
Note regarding the interpretation of figures:

Several of the following figures show output from NSA's RockVision3D™ tomographic software package. There are several things to note regarding these images.

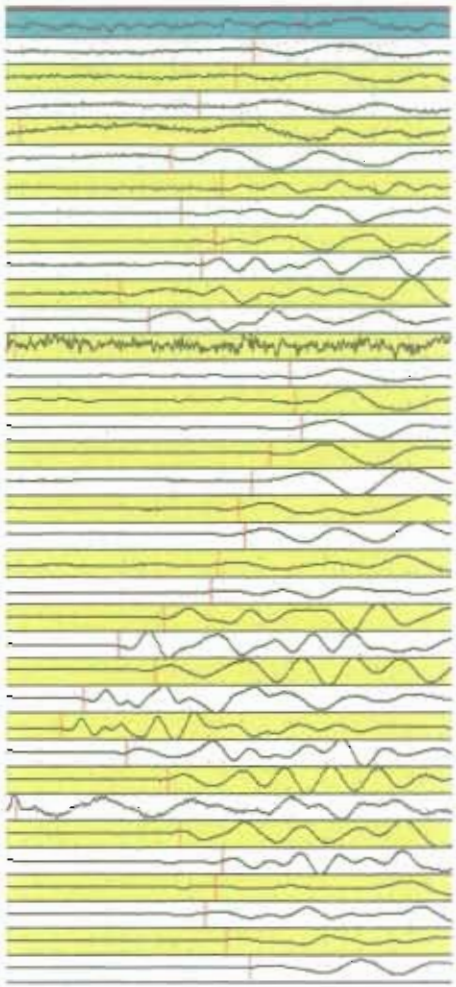
- The velocity scale bars are linear from the minimum to maximum velocity.
- Black lines appear on the color images. These are artifacts of the rendering process and do not represent actual distances.
- Several of the figures are projections of curved surfaces onto a flat plane, and, therefore, distances on these figures are approximate.



1a. Detonating cap in borehole.



1b. Kinepak™ in borehole.



1c. Hammer blow on surface.

Figure 1. Example seismic records.

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	Project engineer: B. Hoekstra
	Date: December 8, 2000

Figure 2a. View looking to the east across the ravine - bridge area.

Figure 2b. View of knob looking to the east.

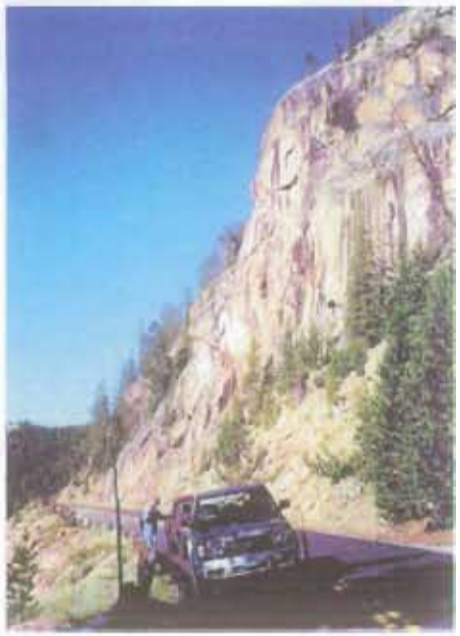

Figure 2c. View looking to the west along side hill area.

Figure 2. Pictures of survey area.

NSA Engineering, Inc. Ground Imaging Technology	Project: Hwy 212
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	Date: December 8, 2000

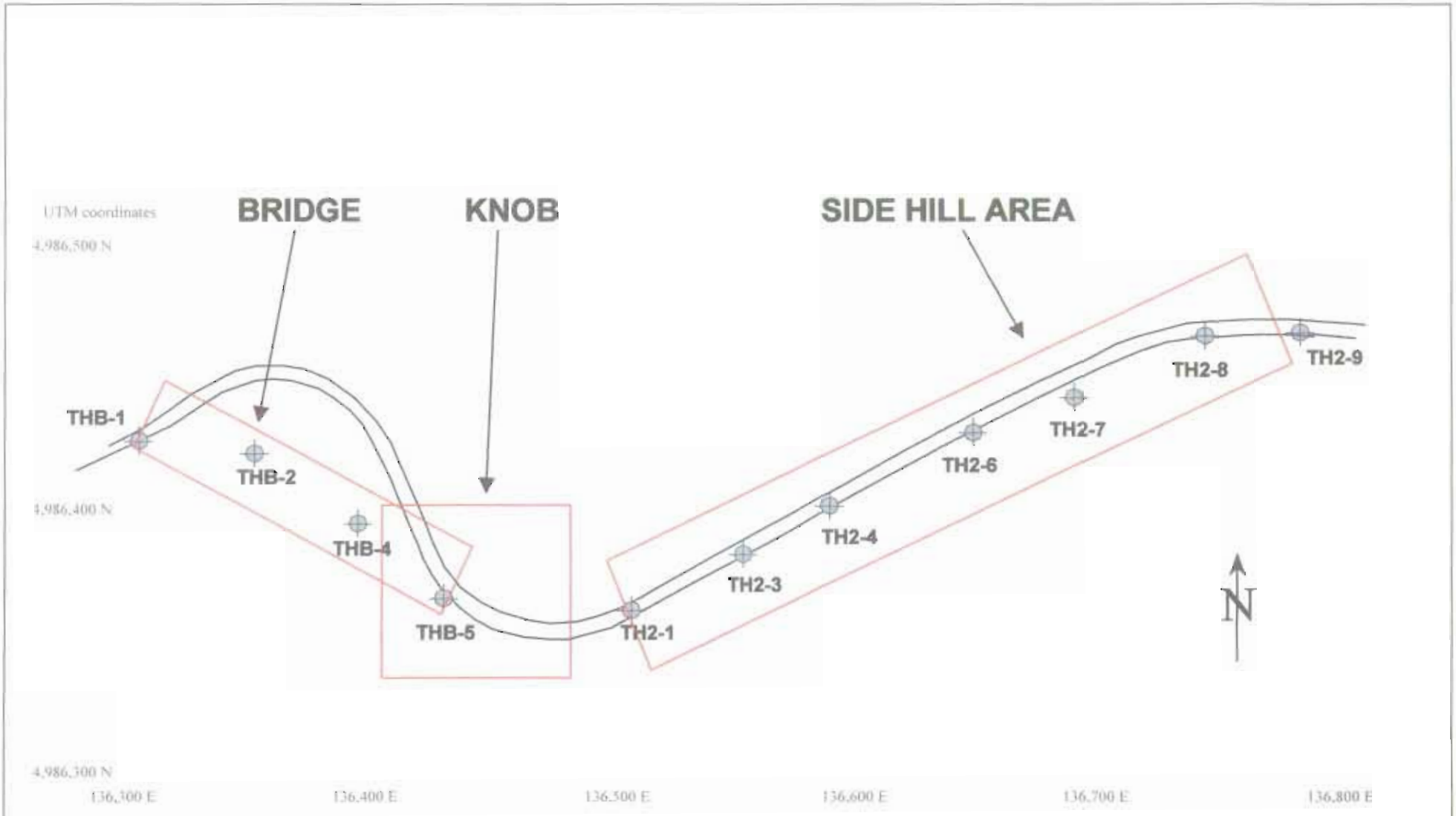

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Figure 3 - Location of imaging areas in relation to road and boreholes.

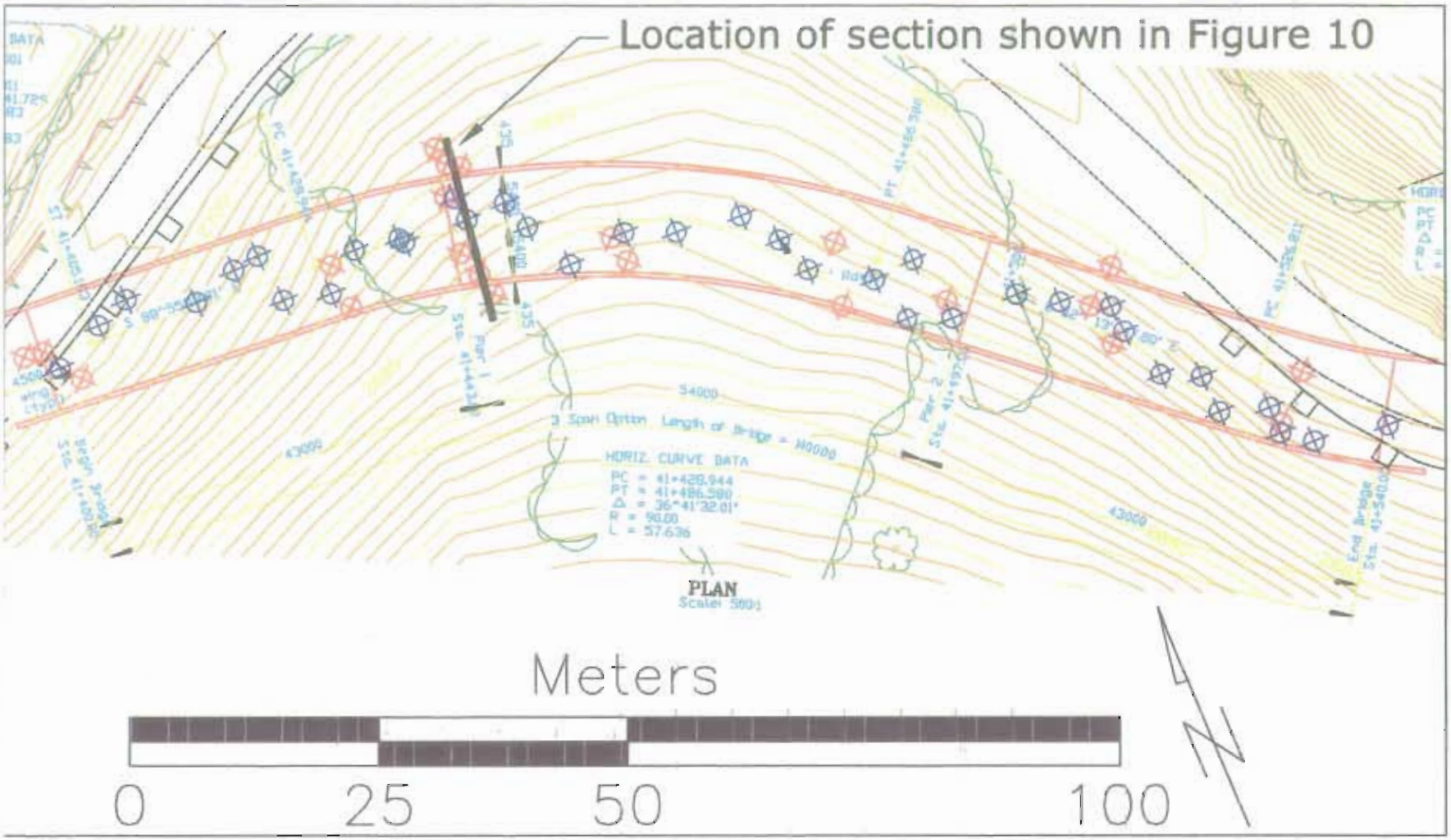


Figure 4. Source and receiver locations for the bridge area.

NSA Engineering, Inc. Ground Imaging Technology	Project: Highway 212
	Project Engineer: B. Hoekstra
	Client URS Corporation
	Date: Dec. 8, 2000

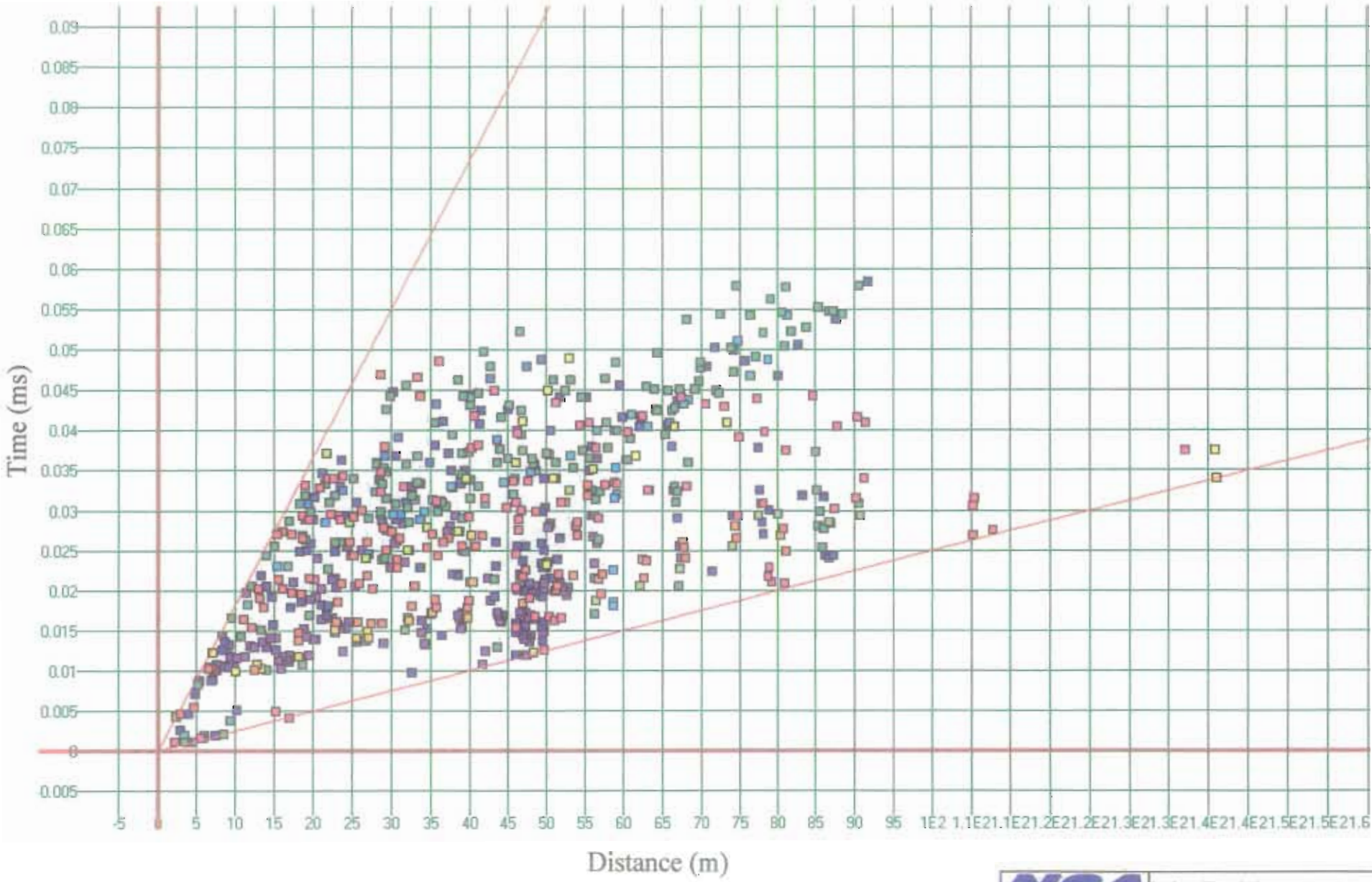


Figure 5. Bridge area time vs. distance plot.

NSA Engineering, Inc. Ground Imaging Technology	Project: Hwy 212
	For: URS Corporation
	Project engineer: B. Hoekstra
	Date: December 8, 2000

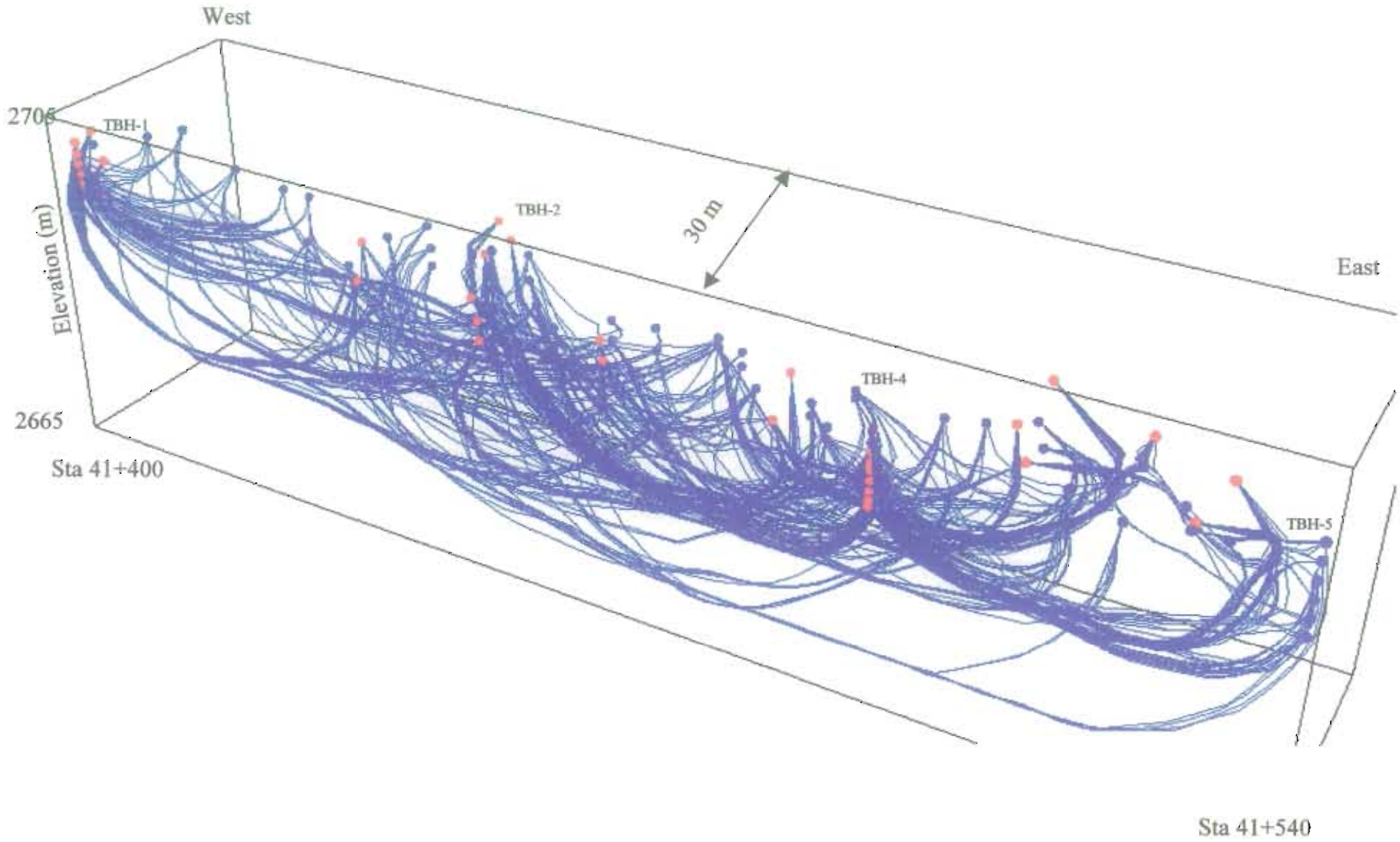
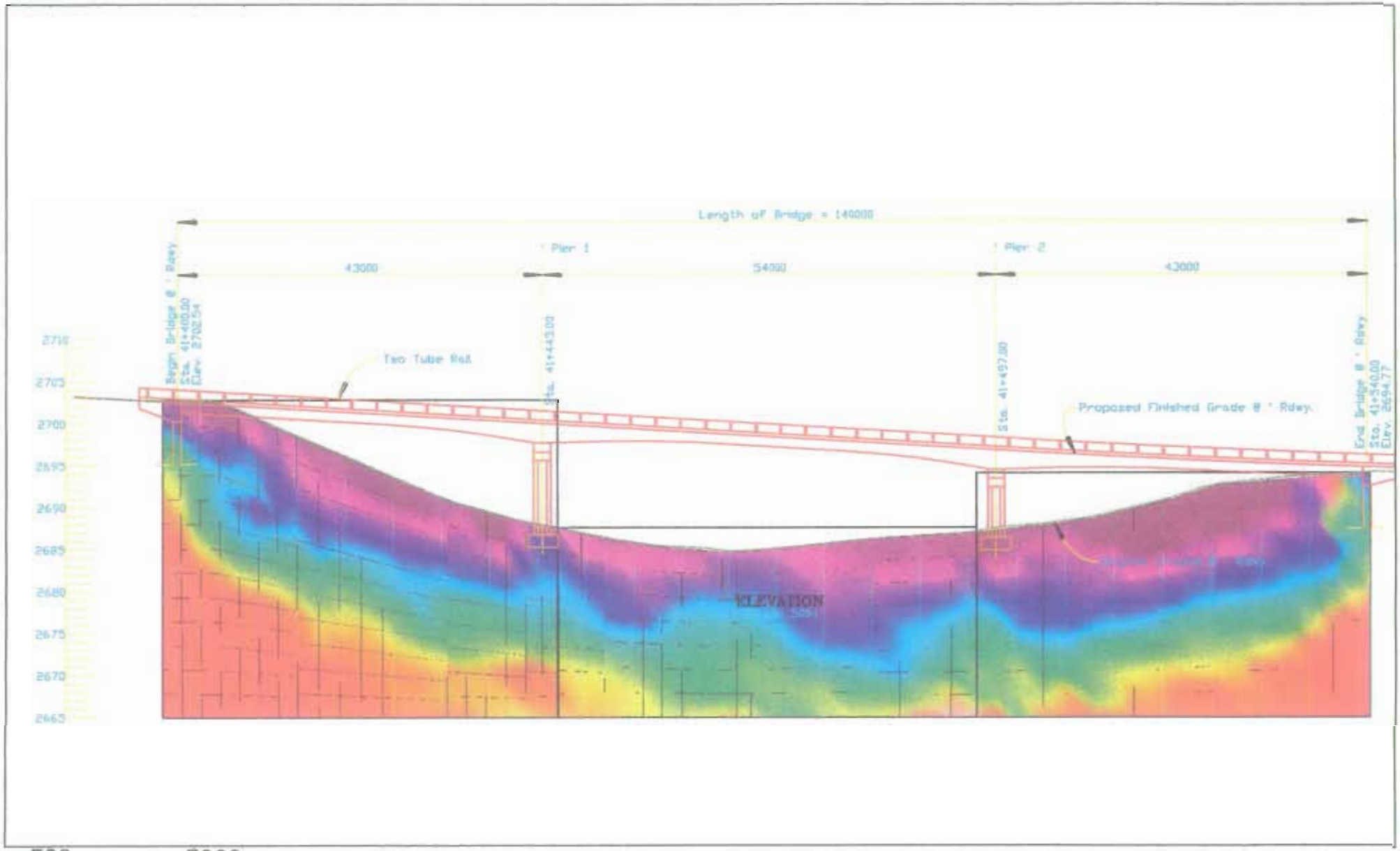


Figure 6. Bridge area ray paths.

NCA Engineering, Inc. Ground Imaging Technology	Project: Hwy 212
	For: URS Corporation
	Project engineer: B. Hoekstra
	Date: December 8, 2000

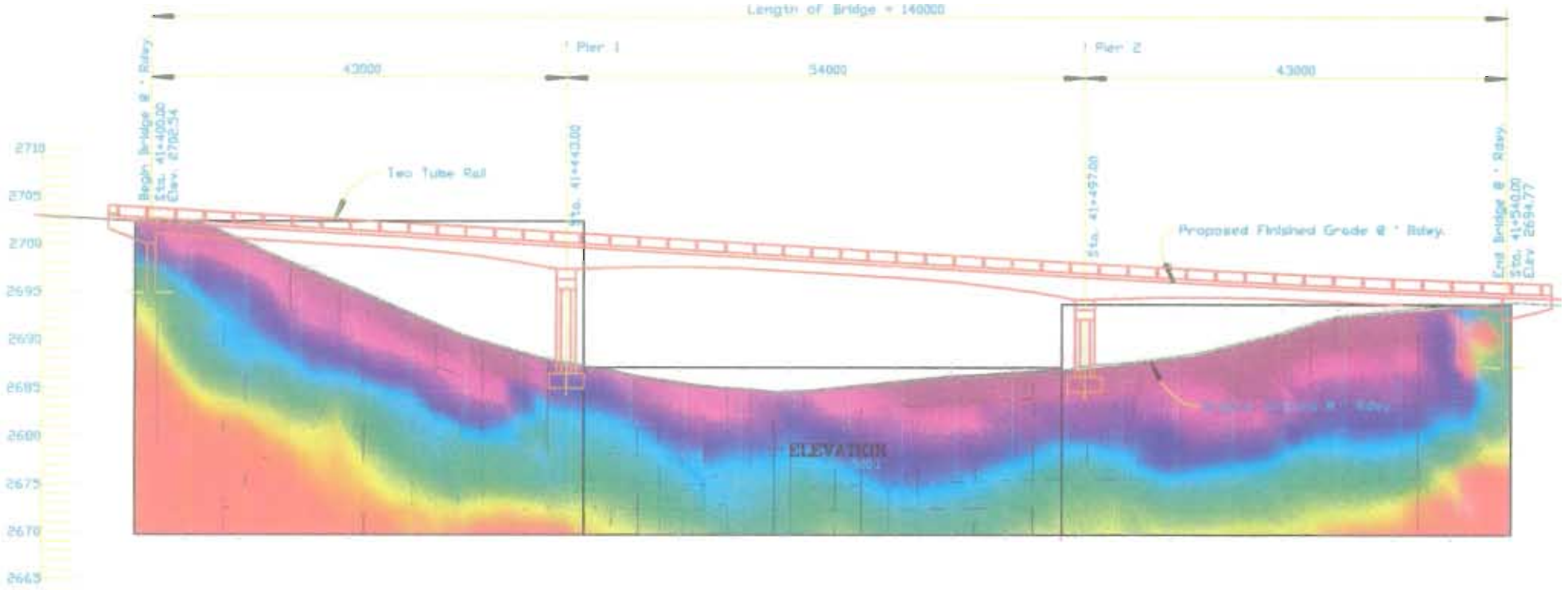


500 5000

Velocity (m/s)

NSA Engineering, Inc.	Project: Highway 212
	Project Engineer: B. Hoekstra
	Client URS Corporation

Figure 7. Projected vertical slice along centerline of bridge alignment.



500 5000

Velocity (m/s)

Figure 8. Projected vertical slice 2 m north of bridge centerline.

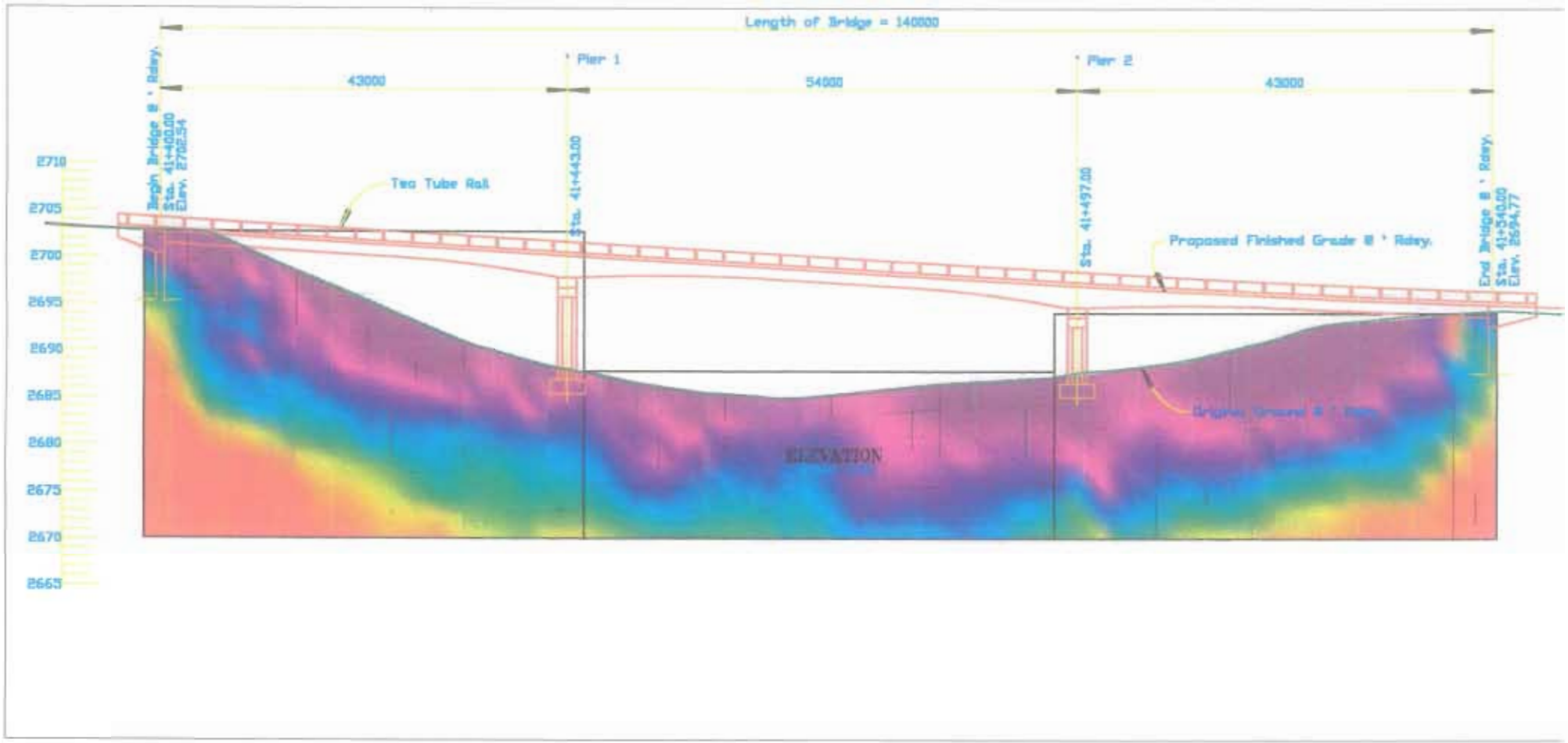
NSA Engineering, Inc. Ground Imaging Technology

Project: Highway 212

Project Engineer: B. Hoekstra

Client URS Corporation

Date: Dec. 8, 2000



ELEVATION

500 5000

Velocity (m/s)

Figure 9. Projected vertical slice 2 m south of bridge centerline.

NSA Engineering, Inc. Ground Imaging Technology

Project: Highway 212

Project Engineer: B. Hoeks

Client URS Corporation

Date: Dec. 8, 2000

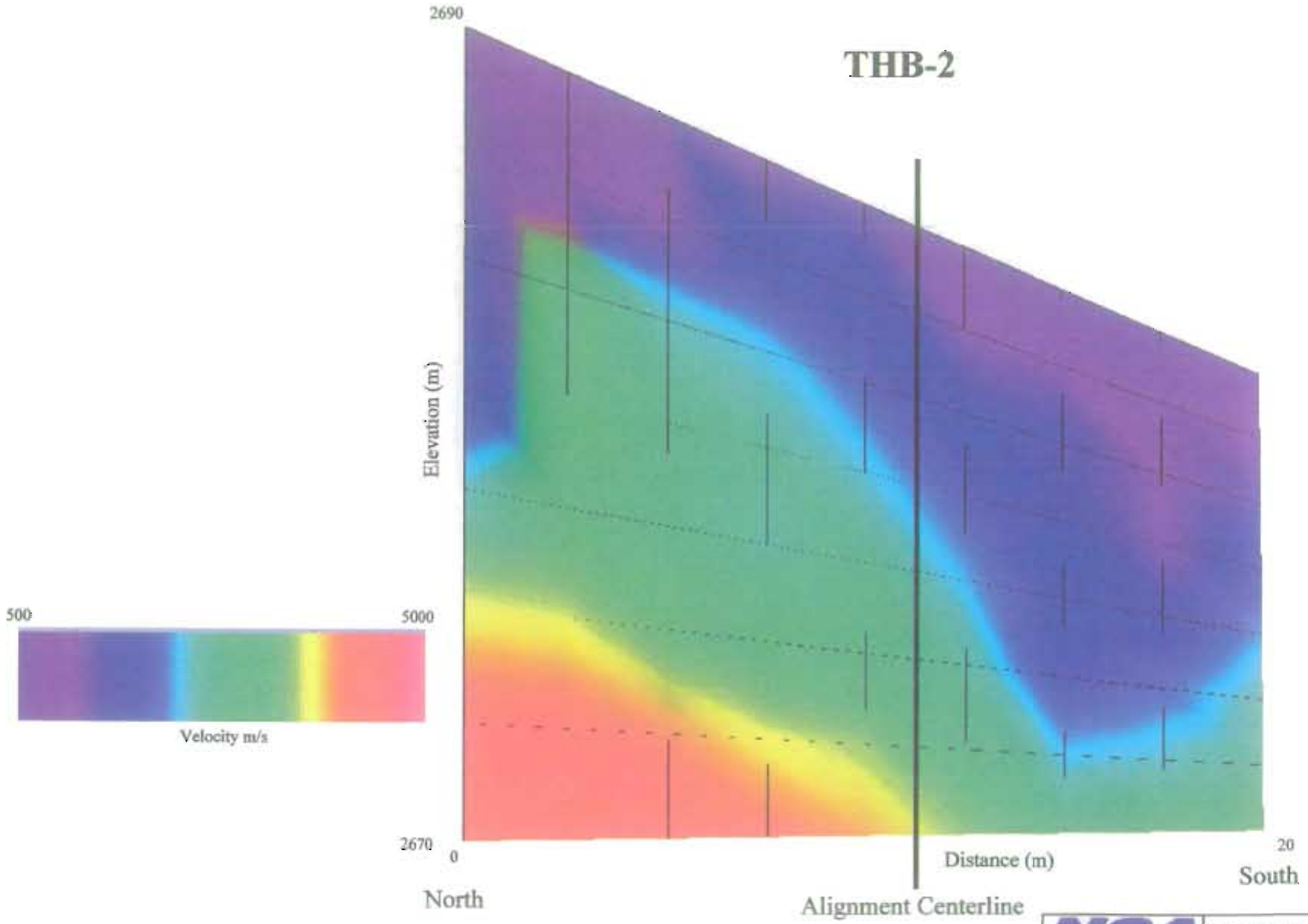


Figure 10. Velocity tomogram parallel to ravine axis at Sta. 41+444 looking east.

NSA Engineering, Inc. Ground Imaging Technology	Project: Hwy 212
	For: URS Corporation
	Project engineer: B. Hoekstra
	Date: December 8, 2000

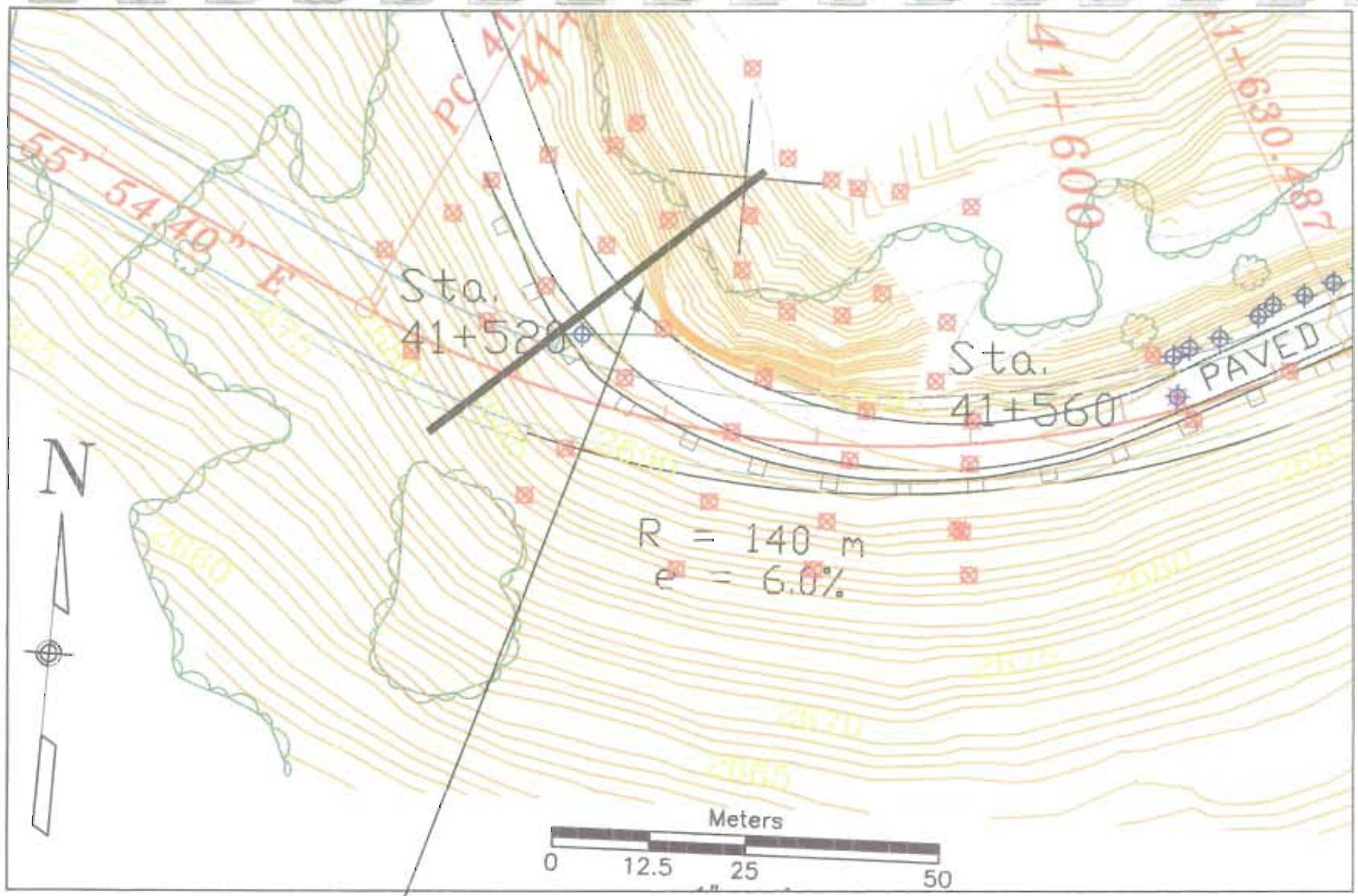


Figure 11. Source and receiver locations knob area.

NSA Engineering, Inc. Ground Imaging Technology	Project: Highway 212
	Project Engineer: B. Hoekstra
	Client URS Corporation
	Date: Dec. 8, 2000

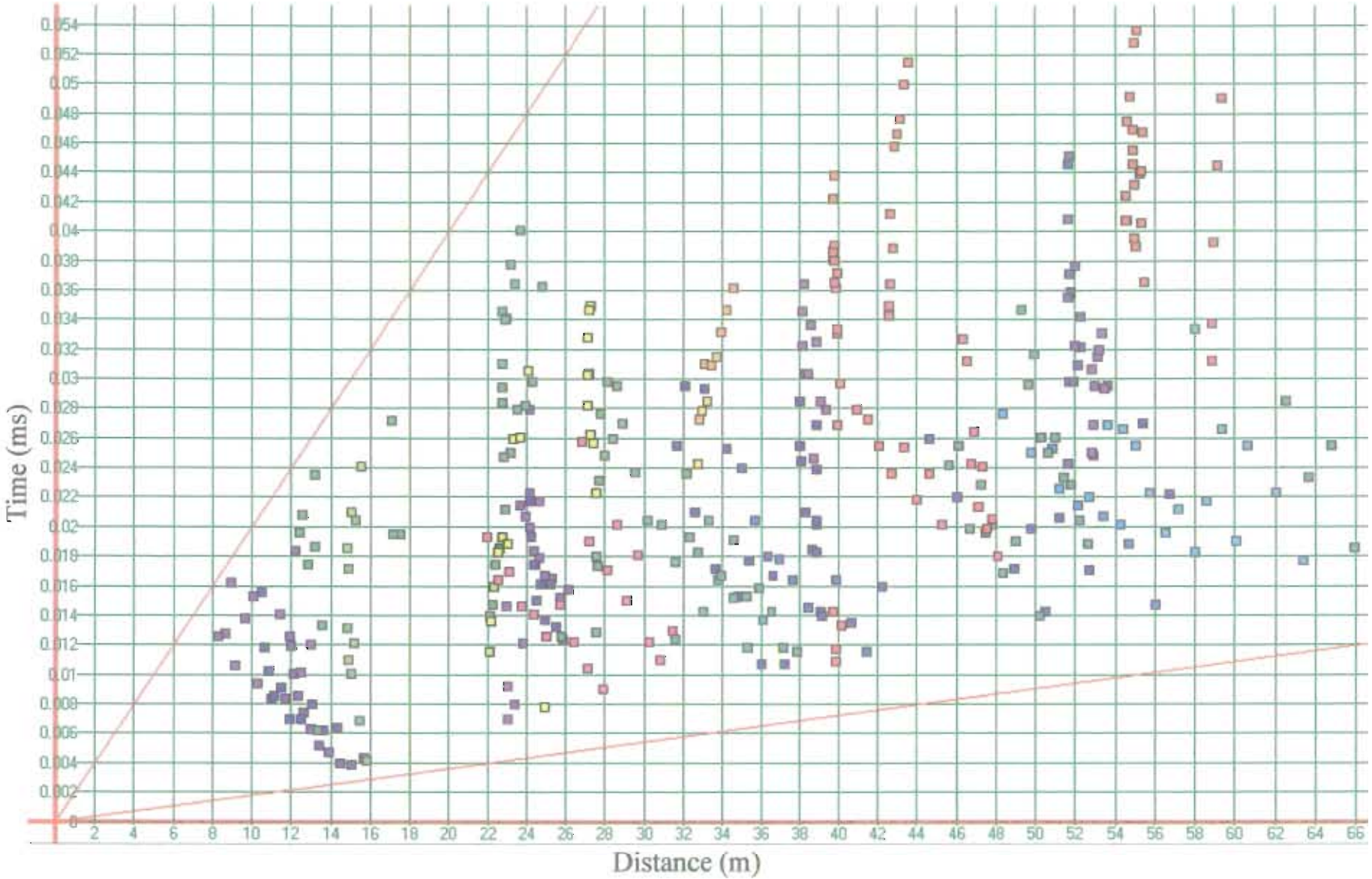


Figure 12. Time-distance plot - knob area.

NSA Engineering, Inc. Ground Imaging Technology	Project: Hwy 212
	For: URS Corporation
	Project engineer: B. Hoekstra
	Date: December 8, 2000

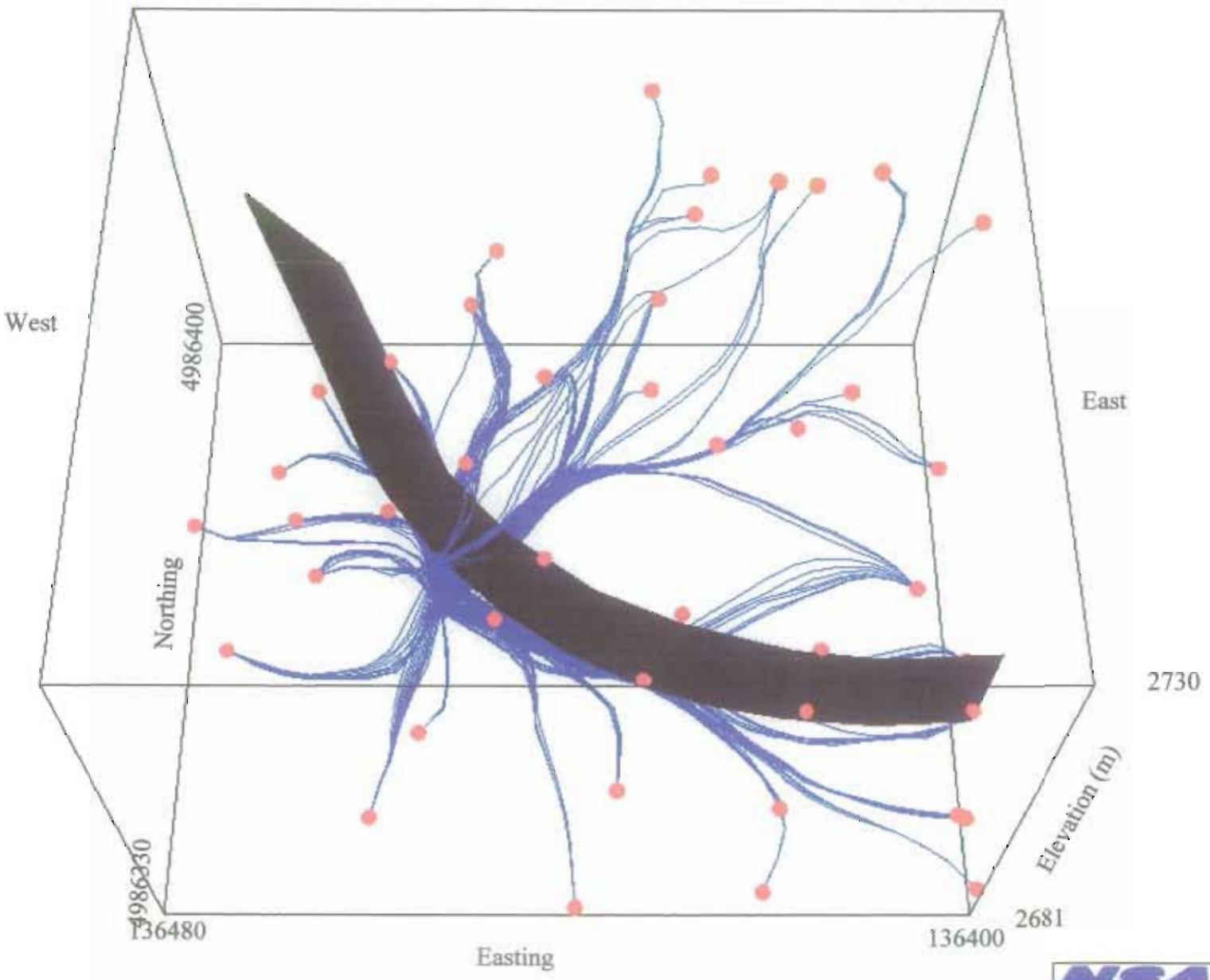


Figure 13. Ray path coverage - knob area.

NSA Engineering, Inc. Ground Imaging Technology	Project: Hwy 212
	For: URS Corporation
	Project engineer: B. Hoekstra
	Date: December 8, 2000

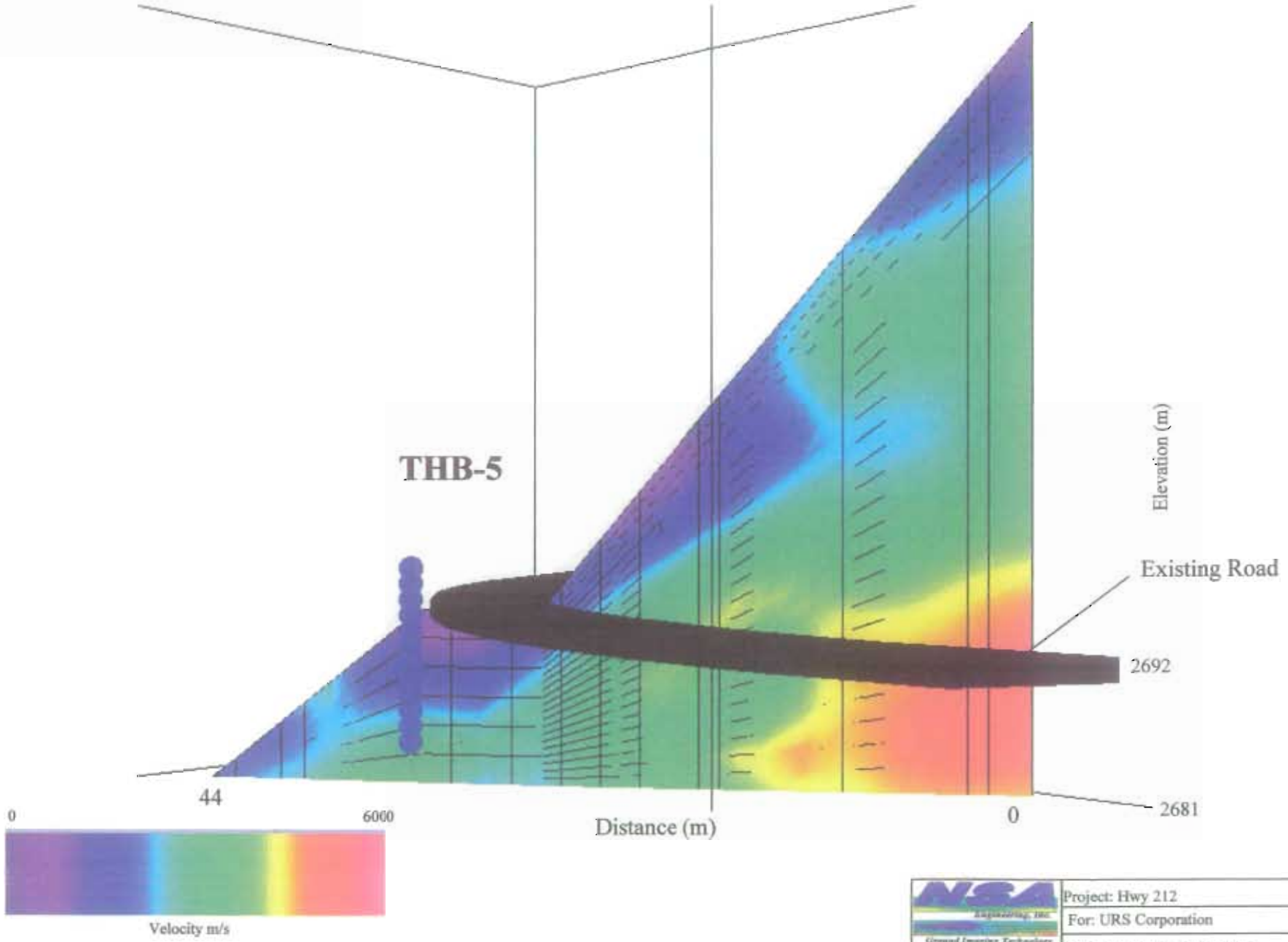
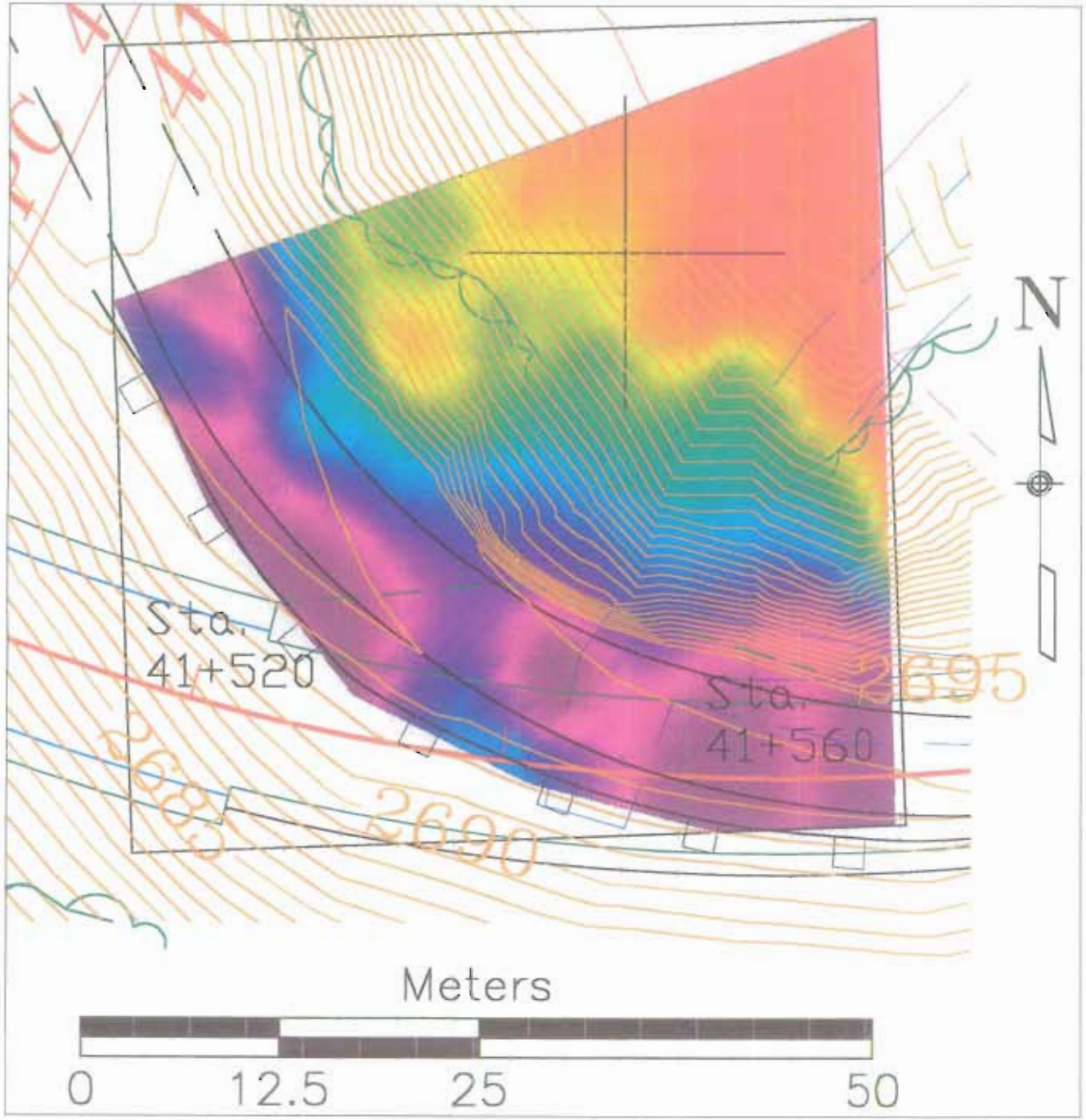


Figure 14. Velocity tomogram along vertical plane through THB-5 looking toward the west in knob area at Station 41+520.

NSA Engineering, Inc. Ground Imaging Technology	Project: Hwy 212
	For: URS Corporation
	Project engineer: B. Hoekstra
	Date: December 8, 2000



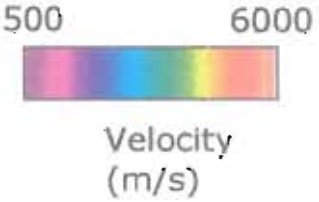


Figure 15. Horizontal slice of velocity tomogram at existing roadbed elevation (2692 m).

NSA Engineering, Inc. Ground Imaging Technology	Project: Highway 212
	Project Engineer: B. Hoekstra
	Client URS Corporation
	Date: Dec. 8, 2000

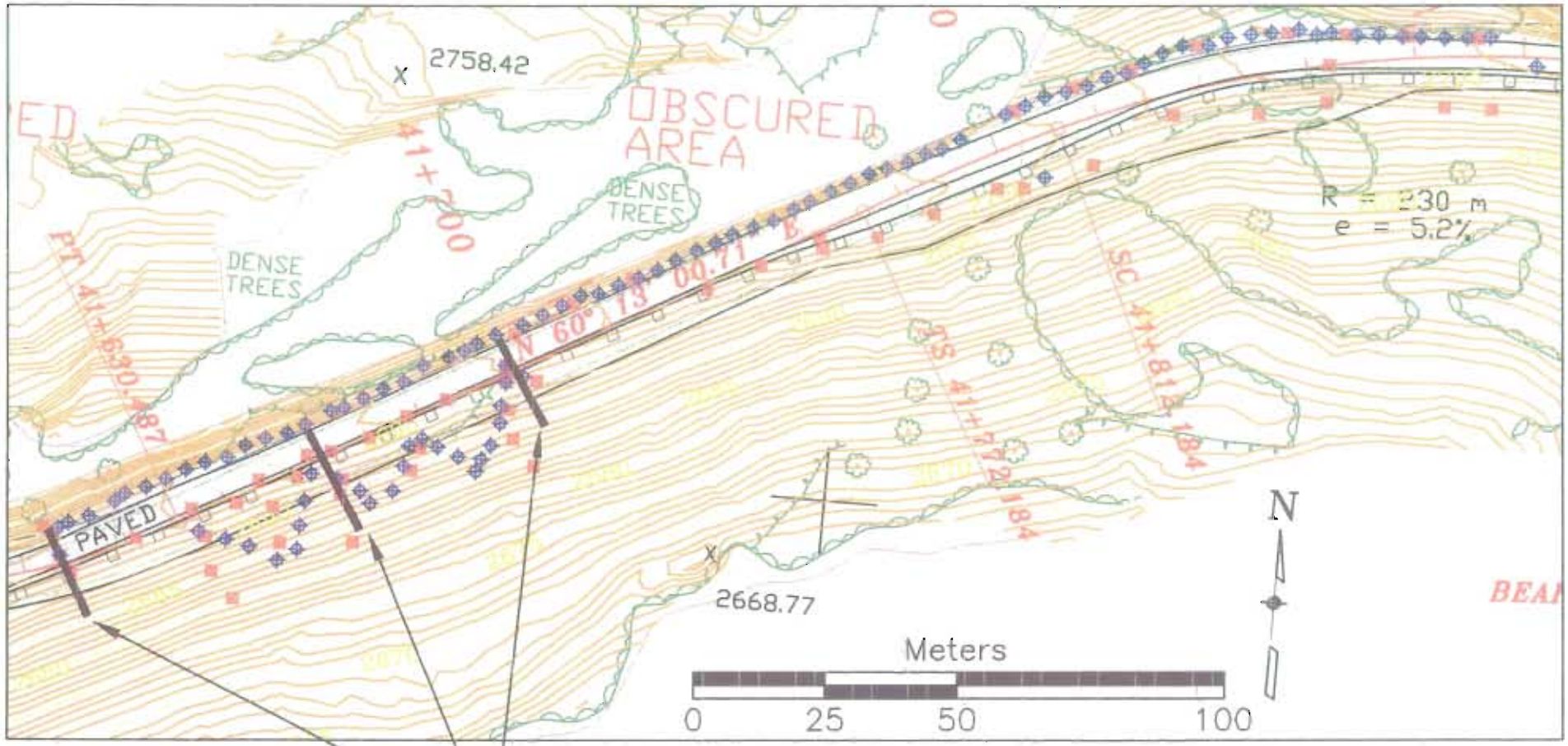


Figure 16. Source and receiver locations for side hill area.

NSA Engineering, Inc.	Project: Highway 212
	Project Engineer: B. Hoekstra
	Client URS Corporation
Ground Imaging Technologies	Date: Dec. 8, 2000

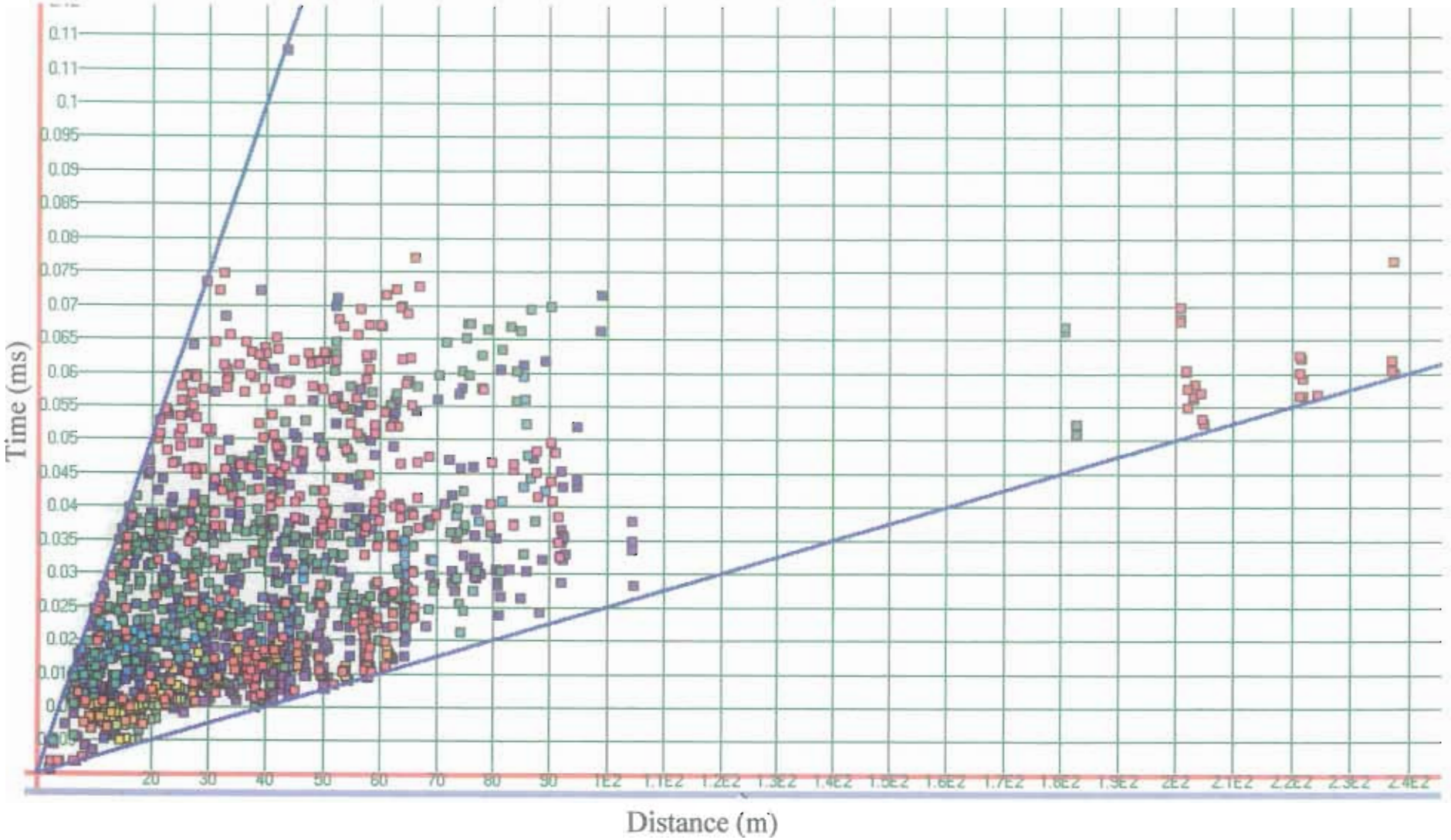


Figure 17. Time-distance plot for side hill area.

NSA Engineering, Inc. Ground Imaging Technology	Project: Hwy 212
	For: URS Corporation
	Project engineer: B. Hoekstra
	Date: December 8, 2000

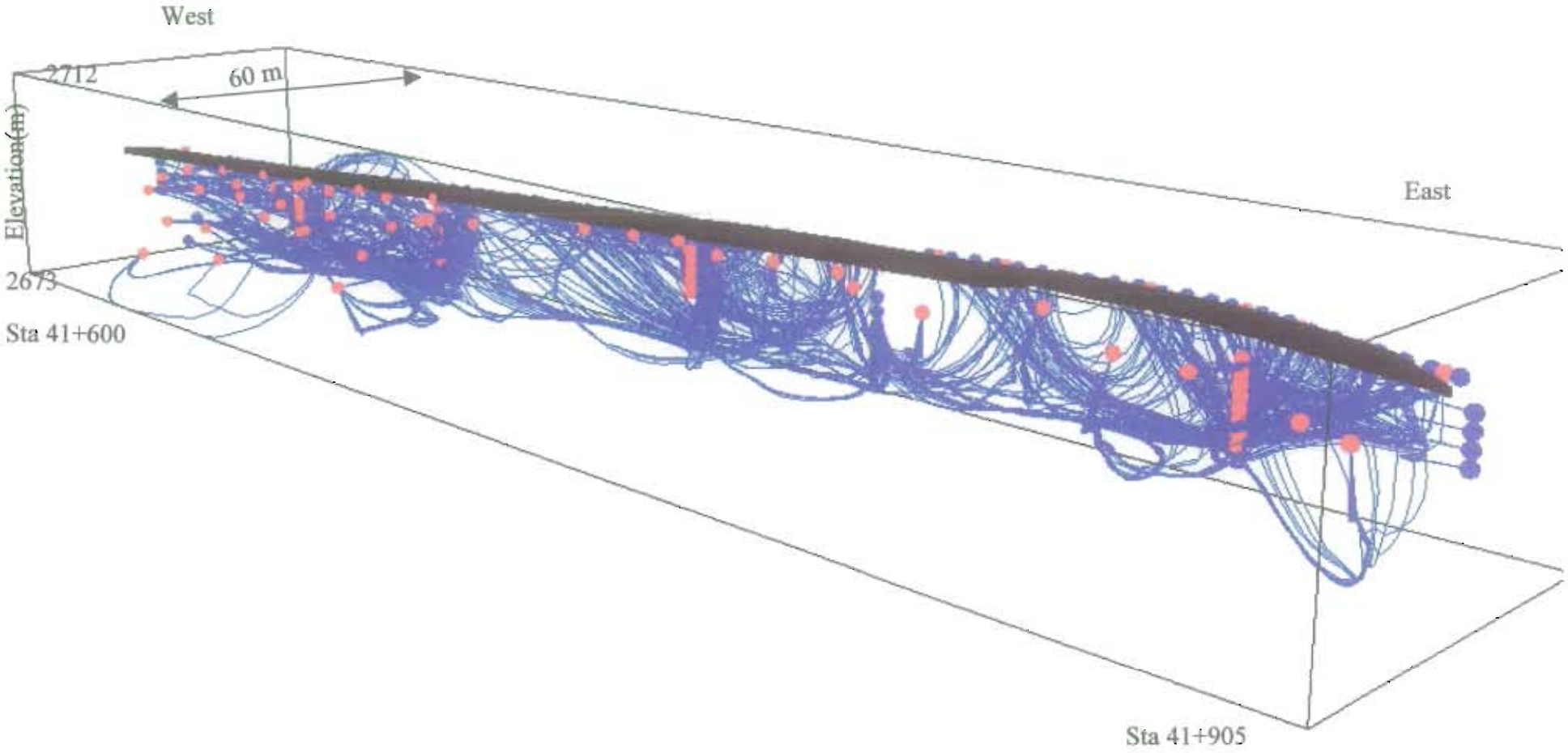
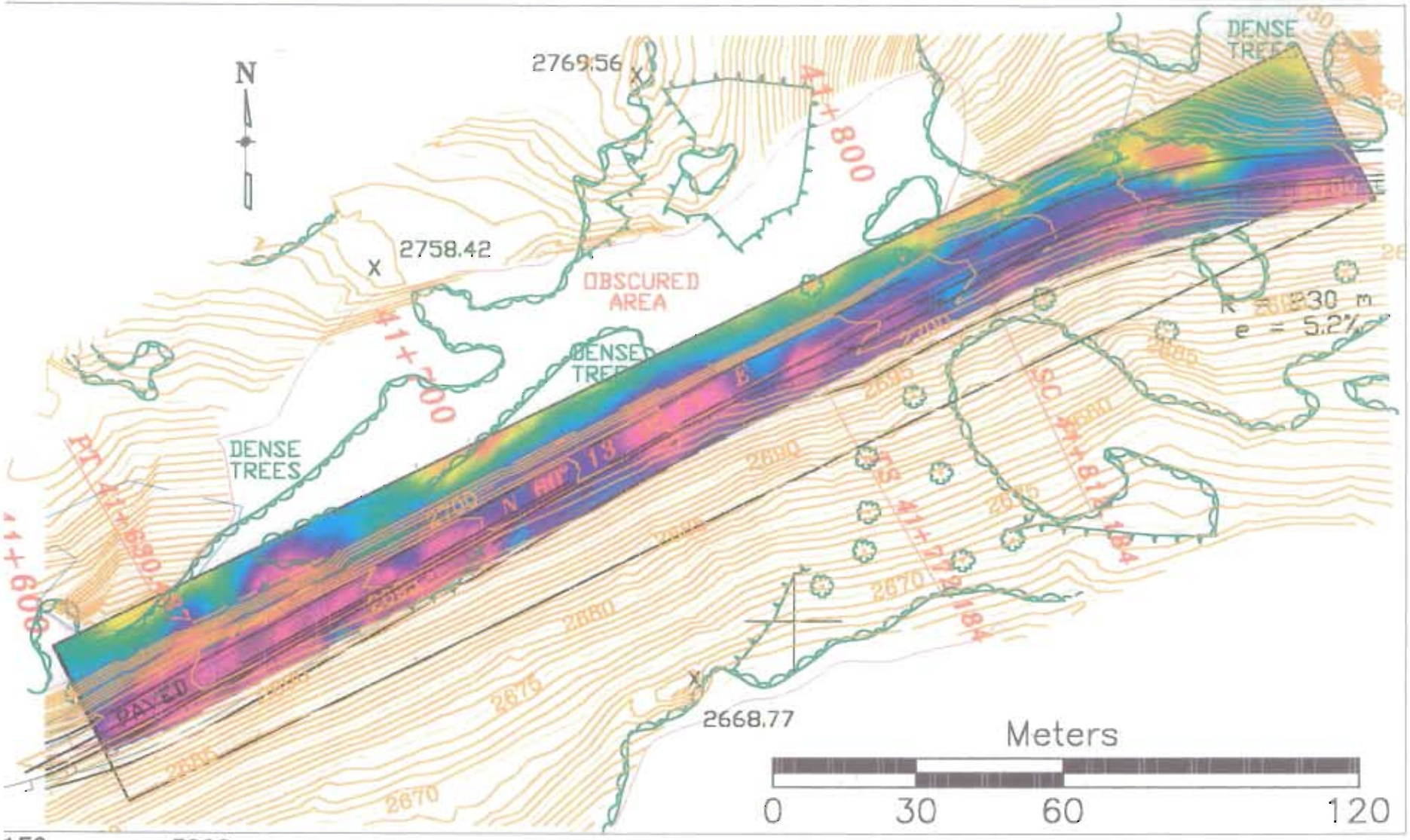


Figure 18. Ray path coverage for side hill area.

NSA Engineering, INC. Ground Imaging Technology	Project: Hwy 212
	For: URS Corporation
	Project engineer: B. Hoekstra
	Date: December 8, 2000



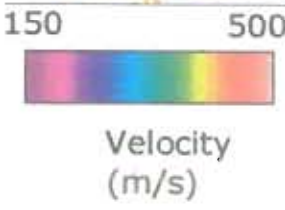


Figure 19. Plan view of horizontal slice of tomographic velocities at the approximate elevation of the base of "Z" wall (2688 m)

NSA Engineering, Inc. Ground Imaging Technology	Project: Highway 212
	Project Engineer: B. Hoekstra
	Client URS Corporation
	Date: Dec. 8, 2000

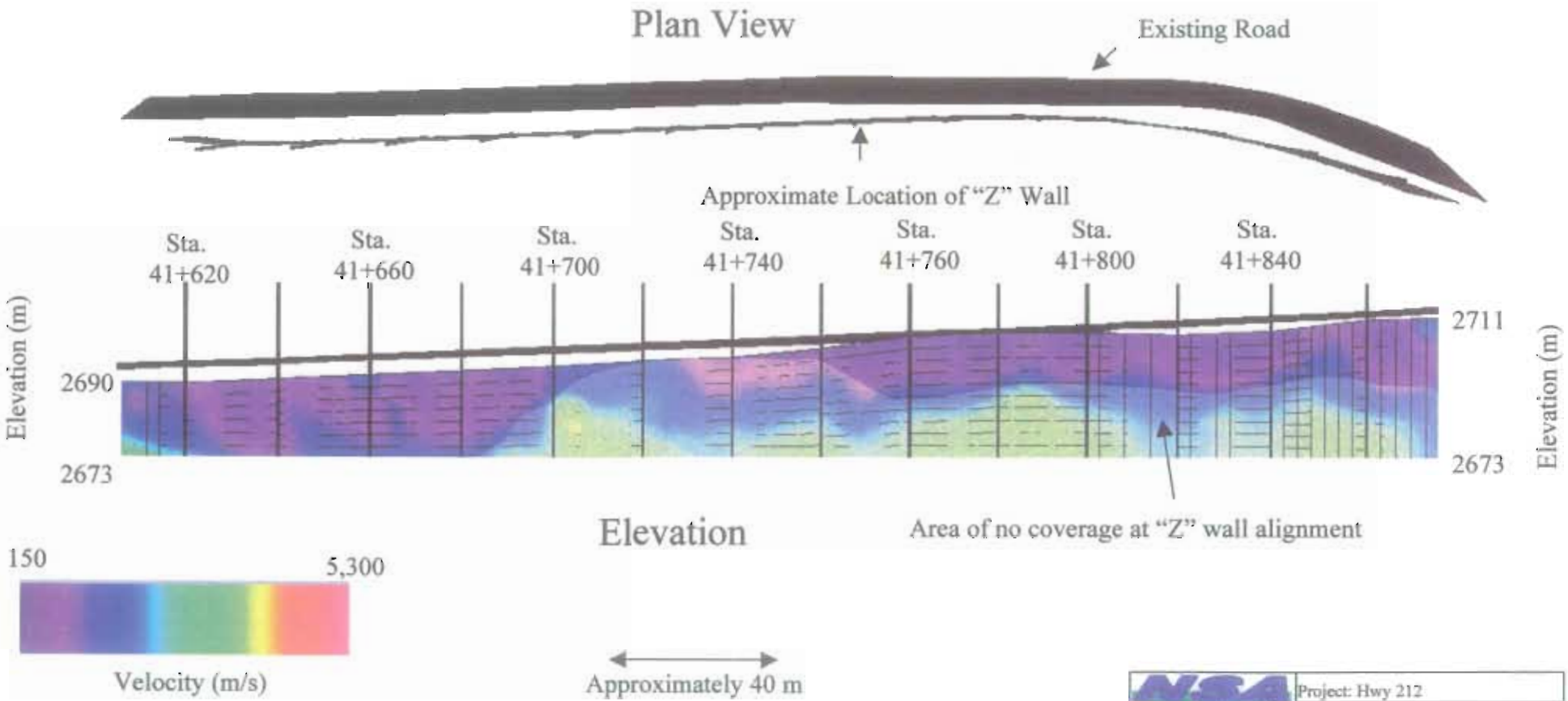
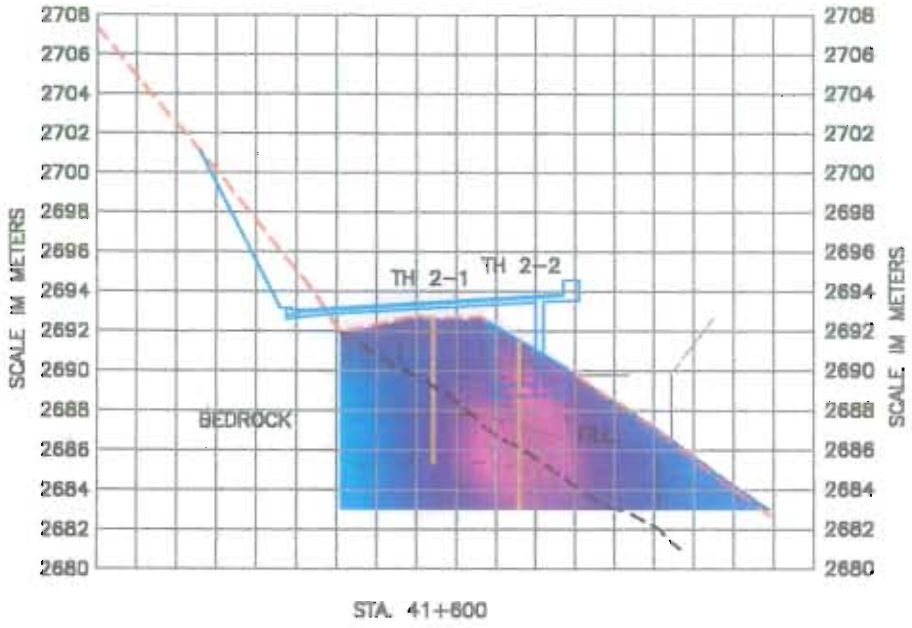
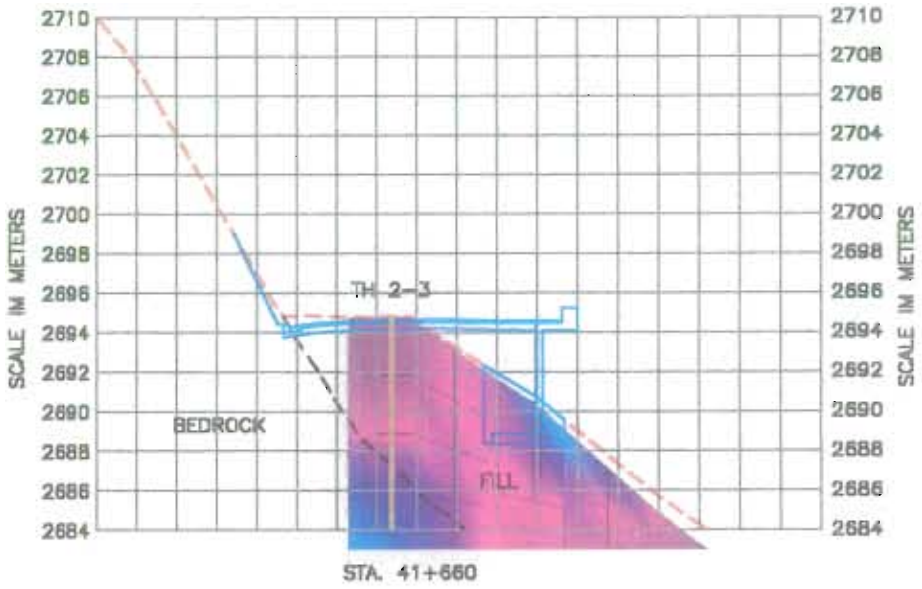
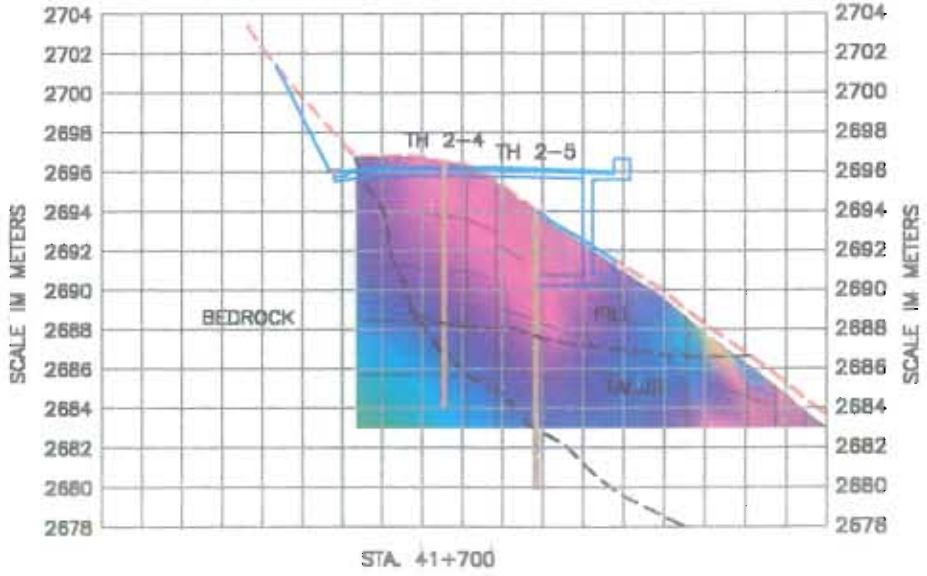


Figure 20. Projected elevation view of a vertical slice of the velocity tomogram along the center line of the proposed "Z" wall.

NSA Engineering, Inc. Ground Imaging Technology	Project: Hwy 212
	For: URS Corporation
	Project engineer: B. Hoekstra
	Date: December 8, 2000







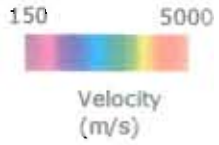


Figure 21. Velocity tomograms at three locations of vertical slices perpendicular to the alignment

NSA Engineering, Inc.	Project: Highway 212
	Project Engineer: B. Hoelstra
	Client URS Corporation
Ground Imaging Technology	Date: Dec. 8, 2000

Appendix F

Appendix F
Seismic Refraction

F.1 INTRODUCTION

Geophysical techniques consisting of seismic refraction surveys were utilized as part of the geotechnical evaluation for proposed improvements along a section of the Beartooth Highway. The objectives of the geophysical survey were to:

- obtain seismic velocity information to facilitate estimation of the subsurface excavatability,
- estimate overburden thickness, and
- aid in characterization of the bedrock surface.

This information was obtained in selected areas along the length of the Site 4 retaining wall.

F.2 SEISMIC REFRACTION METHOD

F.2.1 Description of Method

Seismic refraction for engineering applications is most often used to infer geological boundaries as indicated by interfaces with seismic velocity contrasts. Seismic refraction data may be used to characterize:

- The thickness of alluvial or colluvial deposits, or weathered rock,
- The depth to the water table and/or competent subsurface layers,
- The configuration of the alluvial-bedrock contact, and
- Relative excavatability (inferred from seismic velocities)

The seismic refraction method consists of transmitting seismic energy into the ground and recording the arrival of direct or refracted sound waves at various distances along the earth's surface. The seismic energy travels within geologic units with a characteristic compressional seismic velocity that is dependent on the density, compressibility, porosity, and fluid content of the geologic layer. The seismic refraction method analyzes the first arrival times of the seismic compression wave (p-wave) versus distance from the source. This analysis is based on Snell's Law which predicts the behavior of a seismic raypath at a geologic interface. By measuring seismic velocities, as inferred from the recorded first-arrival travel times, and by determining seismic velocity contrasts, an interpretation can be made of the configuration and depths of subsurface seismic layers, which often infers or correlates to geologic units. A schematic of the method is shown in Figure F-1.

To record seismic refraction data, a seismic source, cables, geophones, and a seismograph are required. The seismic source may be a sledge hammer, buried explosives or an elastic wave generator, depending on the depth of investigation and attenuation properties of the near-surface material. Geophones implanted in the ground translate vibrations into an electrical signal displayed on the seismograph. The seismic record (seismogram) is a plot of the seismic data

with response amplitude versus time recorded. Data can be output on hard copy records and/or saved on personal computer (PC)-compatible disks.

A minimum of three seismic sources are usually used for each seismic refraction setup or spread, one source on each end of the spread, and one in the middle. If the target layer is likely overlain by a relatively thick overburden, offset shots must be used to ensure that sufficient coverage of the target layer is made. However, the use of offset sources can be problematic if topographic or other physical constraints exist in the investigation area.

Data collected can be processed and analyzed with one of several interactive seismic refraction interpretation packages. These packages, which operate on a PC, make all necessary topographic corrections, construct time-distance plots, allow calculation of apparent layer velocities, and calculate depths at each geophone location. From the seismogram, the arrival time of the compression wave for each geophone is selected and plotted versus the distance of each geophone from the seismic source, commonly denoted a time-distance plot. Analysis of the time-distance plot allows calculation of seismic compression wave velocity of the subsurface material(s), or several velocities for multiple layers if they are present. Application of Snell's Law governing the angle of refraction at an interface between layers with different seismic velocities permits calculation of upper layer thicknesses. From the analyses and results, a cross-section of the seismic layers is produced. Cross-sections from individual seismic spreads can be tied to available geologic or geophysical borehole information, as well as to other seismic spreads, to make a final geologic interpretation of the surveyed area.

F.2.2 Method Limitations

There are several potential limitations of the seismic refraction method that must be kept in mind when using it and interpreting the results. One limitation involves the primary assumption made in refraction interpretation that the seismic velocity of the subsurface increases with depth. If the velocity of a layer is less than that of the layer immediately overlying it, the observed travel time due to the deeper layer will be slower and not easily measured, yielding a first-arrival travel time of the faster layer. A decrease in velocity with depth can cause layers to be hidden or shadowed by shallower, faster velocity layers. This can lead to depth estimates that may be in error.

Another limitation of the method is that a target layer must be sufficiently thick to be detected. The thickness required depends on the layer depth, the velocity contrast with overlying and underlying layers, and the field parameters utilized during data acquisition and recording. Additionally, the degree of subsurface weathering can make the interpretation of discrete seismic interfaces more difficult.

In some instances, the seismic refraction method is limited solely by physics. That is, the target geometry can be such that refracted raypaths to a target structure are either limited, or non-existent. For example, a steep-walled narrow valley (on the order of 30m wide) and a deep incised channel resulting in a fluctuating, but very deep (on the order of 40 to 60m), bedrock surface will have limited ray paths. The completion of a refraction line in the geometry of the

canyon would limit the availability of seismic refraction raypaths to penetrate to the bedrock, resulting in failure to resolve or characterize such a target. Refractions could still occur but would be from shallower layers, and not from the desired bedrock interface.

We must also keep in mind that the depth to the refractor identified and mapped in seismic refraction results is the shortest distance between the refractor and the geophones represented by the smallest travel time. For most geometrics, this distance is a true vertical depth. However, for a geometry in which the seismic refraction is conducted along strike of a steeply-dipping interface, the shortest distance to the refractor may not be truly vertical. In such a scenario, the refractor 'depth" will be a minimum depth, and likely not the true depth vertically below the detecting geophones.

F.3 FIELD PROGRAM

Three seismic refraction spreads (denoted SP1 through SP3) totaling approximately 300 linear meters were completed during the field program. The final locations of the seismic refraction spreads were chosen based on the project objectives and to maximize coverage along a specific stretch of the highway. Final location of the seismic refraction spreads are shown in Figure 10-1.

Seismic refraction data were collected using a signal-enhancing, Geometrics Strataview Model R48 seismograph, the same instrument used in the seismic tomography work. The seismic refraction survey utilized 24 of the available 48 channels on the seismograph. A geophone spacing of 4 or 5 meters was used on the lines resulting in seismic spread lengths of 92 or 115 meters. Seismic sources were used at both ends of each spread for forward and reverse travel times and at locations within each spread to increase near-surface velocity control. On each of the seismic lines, offset seismic source locations were used to enhance coverage of the bedrock surface. For most of the seismic sources, seismic energy was produced by a sledgehammer impacting an aluminum plate, or by small explosive charges. Explosive charges consisted of approximately 200g of Kinepak initiated by electric blasting cap. Each seismic spread had five to eight seismic source locations.

Since the seismic lines were conducted adjacent to the highway, data were collected at times in which traffic was not present, or at times in which traffic was stopped, to minimize the seismic noise. However, because the overburden material was attenuative for the propagation of seismic waves, multiple sledgehammer blows were required and stacked together for each record or explosives were necessary. Overall, the seismic refraction data were of moderate to good quality.

F.4 DATA PROCESSING AND INTERPRETATION

Processing of the seismic refraction data involved the construction of a time-distance plot for each seismic spread. To construct the time-distance plots, the first compressional wave arrival at each geophone location was plotted versus the source-to-geophone distance. Velocities of the seismic layers were calculated based on the slope of the best-fit lines through the plotted

compressional-wave time-distance data. The intercept of each velocity slope at time zero was used to calculate depths to particular seismic interfaces.

Interpretation of refraction survey data involved the computation of average velocities over surveyed volumes of subsurface material. For the upper seismic layer in which the direct seismic wave arrived at the geophone first, the velocity observed was the true average velocity between the energy source and the geophone. For deeper interfaces in which the refracted seismic wave arrived at the geophone first, the velocity observed is usually an apparent velocity. If the refractor surface is flat-lying, the apparent velocity will be equal to the true average velocity. If, however, the refractor surface is dipping or has a variable surface, the true average velocity can be estimated utilizing the apparent velocities obtained from the data collected form both the forward and reverse seismic energy sources.

Time-distance plots were constructed and interpreted using the software program GREMIX, an interactive seismic-refraction processing routine developed by Interpex Limited. GREMIX allowed interactive plotting of each travel time plot, selection of velocity slopes, and identifications of forward and reverse shot pairs. With elevation information from each geophone, the program calculated seismic layer thicknesses for each geophone travel time. Final interpretations are displayed for each seismic line in cross-section form in Figures F-2 through F-4. A composite plot of all three spreads is shown in Figure F-5.

Each of the seismic refraction plots display three panels. The top panel displays a plot of the arrival time data and is the p-wave arrival at each geophone versus distance along the seismic spread. The middle panel, which is the depth section, is the interpreted subsurface model based on the input arrival time data. The lower panel, which is the velocity section, displays the seismic velocities associated with the interpreted depth section. The depth and velocity sections are plotted in terms of seismic layers, which represent the interfaces at which a distinct velocity contrast is interpreted to exist. These seismic layer interfaces often relate to geologic boundaries as well, depending on the seismic velocity contrast observed in the geologic section. Note that distances portrayed in the plotted panels are slope distances. Table F-1 summarizes the seismic refraction results.

F.5 GEOPHYSICAL RESULTS

The travel time data, resulting cross sections and seismic velocities are plotted for the seismic profiles in Figures F-2 through F-5.

All of the seismic refraction spreads indicate a two-layer model of the subsurface. The near-surface seismic layer has a seismic velocity ranging from 370 meters/second (m/s) to 740 m/s. The thickness of the near-surface seismic layer varies from about 4 meters to a maximum of about 12 meters. The refractor layer has a seismic velocity ranging from 2,820 m/s to 4,570 m/s.

Based on the seismic velocities and considering drilling information, seismic layer 1 probably consists of loose to medium dense gravel, cobbles and boulders and perhaps weathered bedrock material. Seismic layer 2 consists of granite bedrock materials. The seismic interface mapped

suggests higher competency in the materials below the interface, and generally appears to correlate with bedrock materials in which there is an RQD that is non-zero.

Because the seismic spreads were conducted outboard (northeast) of the borings (by a meter or so), and because the bedrock appears to drop off significantly in the outboard (northeast) direction, we expect the boring results to differ some from the seismic results. Correlating the seismic refraction results with drilling information does indicate that the seismic refraction results are consistently about a meter or so deeper than bedrock results indicated in the drilling.

Table F-1

Seismic Refraction Results

Seismic Line	Stationing	Seismic Layer	Velocity Range (meters/second)	Thickness of Seismic Layer (meters)
SP1	56+920 to 57+012	1	370 – 720	3.9 – 7.4
		2	3200 – 4180	
SP2	57+012 to 57+104	1	450 – 710	3.6 – 8.8
		2	3370 – 4410	
SP3	57+104 to 57+219	1	410 – 540	12.0
		2	2560 – 3020	

F.6 PREVIOUS SEISMIC REFRACTION STUDIES

In the fall of 1999, seismic refraction work was conducted over a portion of the highway between stationing 41+600 and 42+000. Results of that work were previously reported. The seismic cross-sections for that work are reproduced here to clarify several issues. At the time of the 1999 work, several stationing coordinate systems were in use corresponding to the various design alternatives, and these multiple systems were reflected in the seismic results. To prevent confusion, we have recreated the cross sections with the current stationing.

A comparison was made of the previous seismic refraction results will drilling information reported on in this document. It appears that the refractor depths determined in the previous work may be minimum refractor depths, and perhaps not true vertical depths.

This scenario, which is further discussed in Section A.2.2, may suggest that the true position of the refractor interface mapped in the previous work may be up dip of the seismic line location, and not a vertical depth directly beneath the geophone receivers. Table F-2 summaries the previous seismic refraction results.

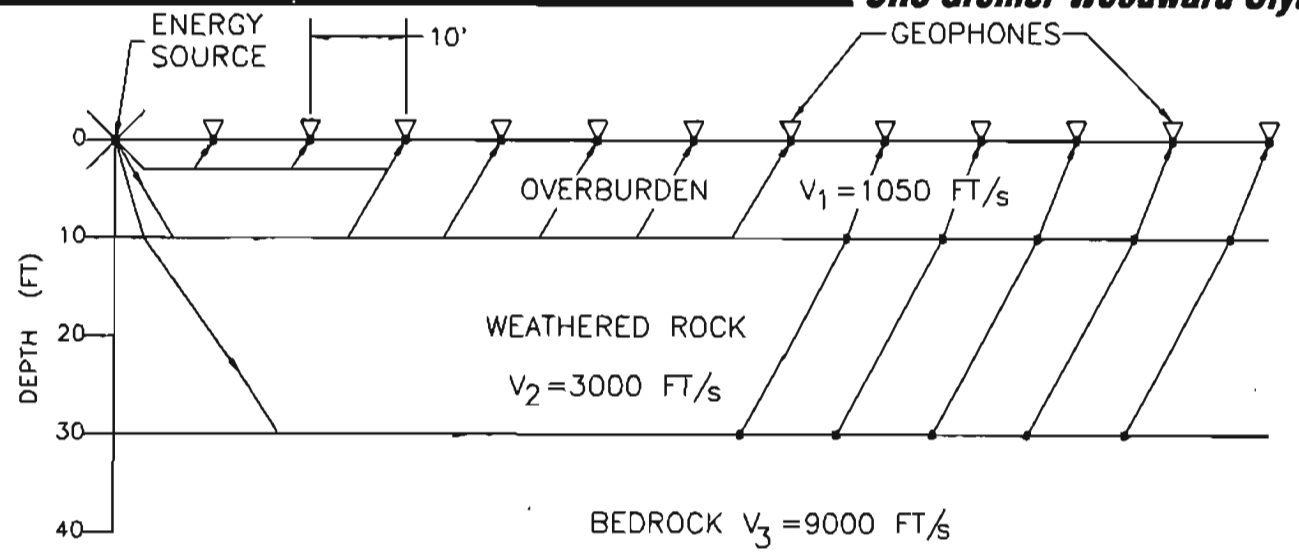
Table F-2

Seismic Refraction Results

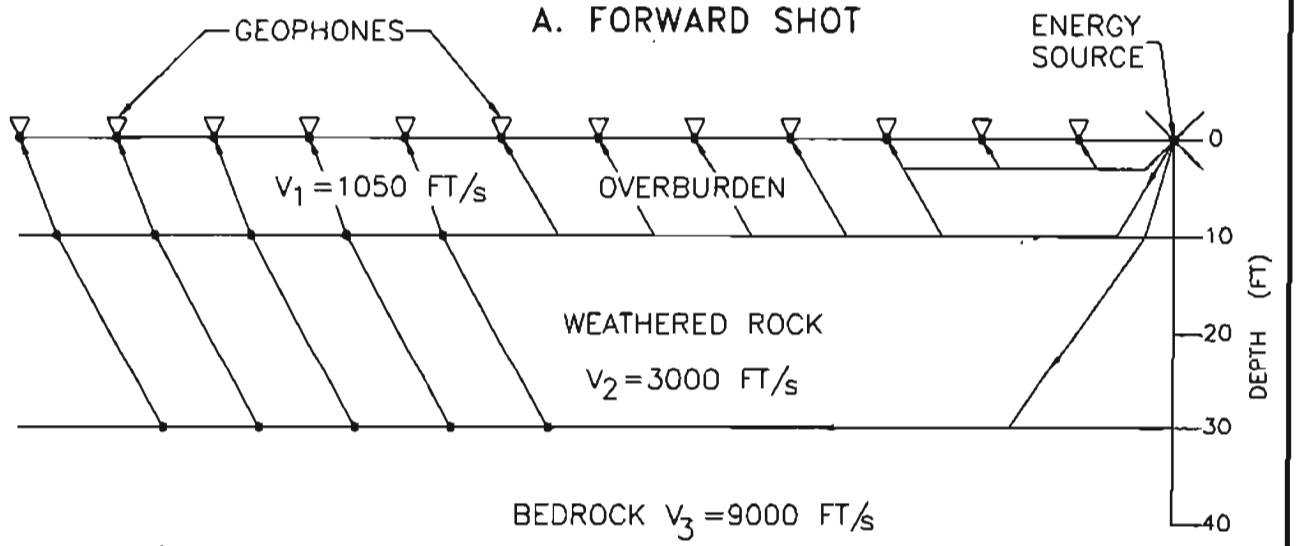
Seismic Line	Stationing	Seismic Layer	Velocity Range (meters/second)	Thickness of Seismic Layer (meters)
SL1	41+615 to 41+684	1	340 – 360	3 – 5.5
		2	4930 – 5360	
SL2	41+684 to 41+753	1	350 – 400	5 – 8
		2	3770 – 4290	
SL3	41+753 to 41+822	1	360 – 450	6 – 8
		2	3040 – 4420	
SL4	41+822 to 41+891	1	350 – 630	5 – 9
		2	4910 – 5670	
SL5	41+891 to 41+960	1	340 – 380	6.5
		2	5200 – 6220	

F.7 LIMITATIONS

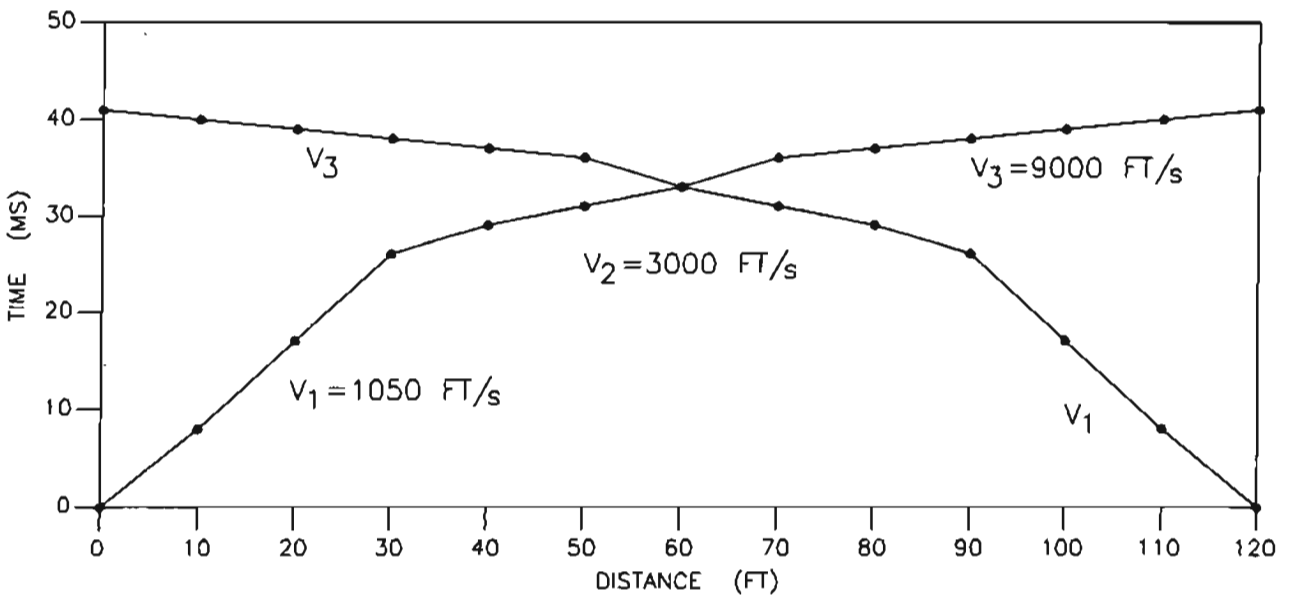
This work was conducted in accordance with reasonable and accepted engineering geophysics practice, and the interpretations and conclusions are rendered in a manner consistent with other consultants in our profession. However, all geophysical techniques have some level of uncertainty and limitations. No other representations to the client is expressed of implied, and no warrant or guarantee is included or intended.



A. FORWARD SHOT



B. REVERSE SHOT

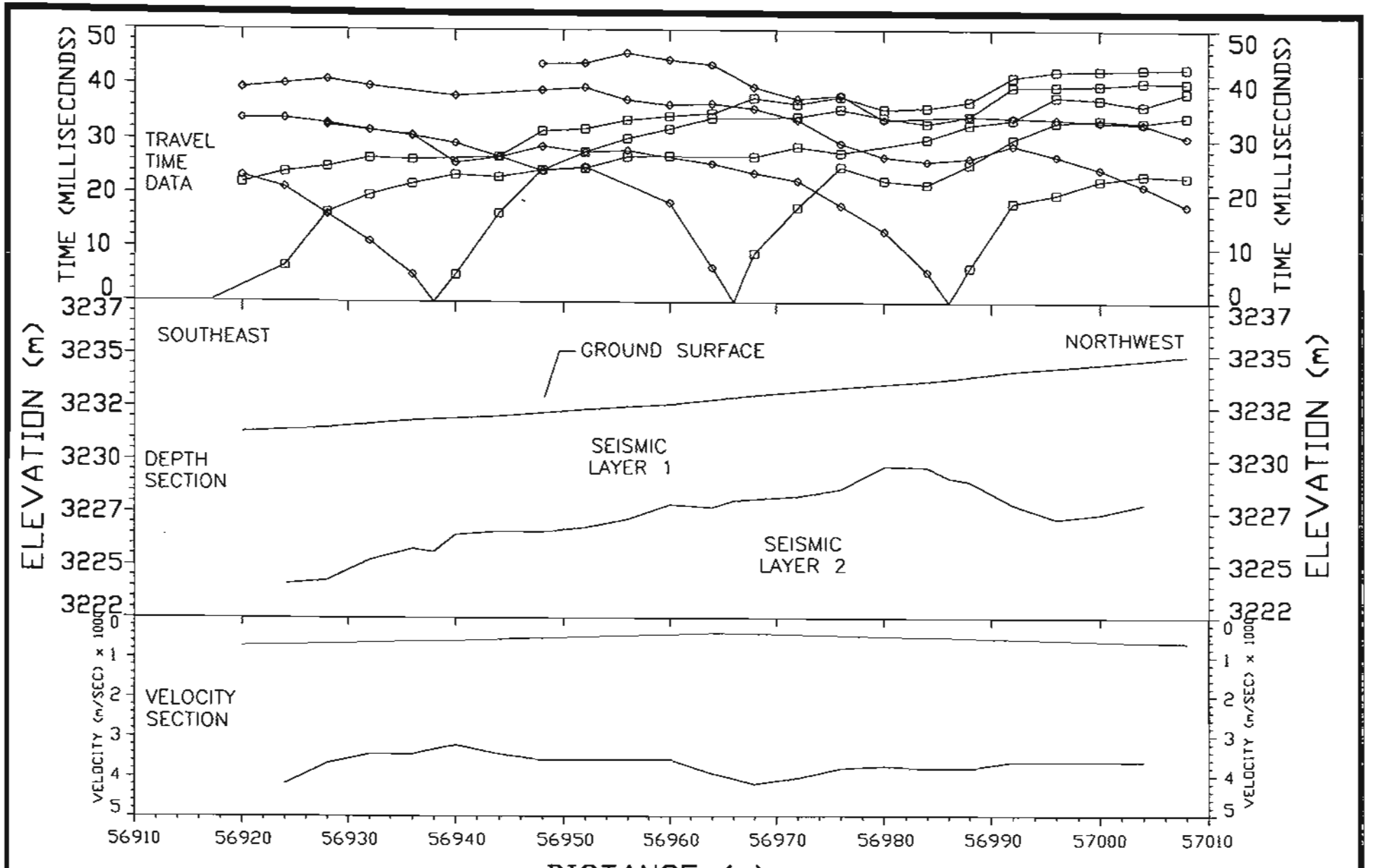


C. TIME-DISTANCE PLOT OF FORWARD AND REVERSE SHOTS

Job No. : 68FHAT0027.00	AN EXAMPLE OF THE SEISMIC REFRACTION METHOD
Prepared by : J.J.N.	
Date : 12/12/00	

BTS-001

FIG. F-1

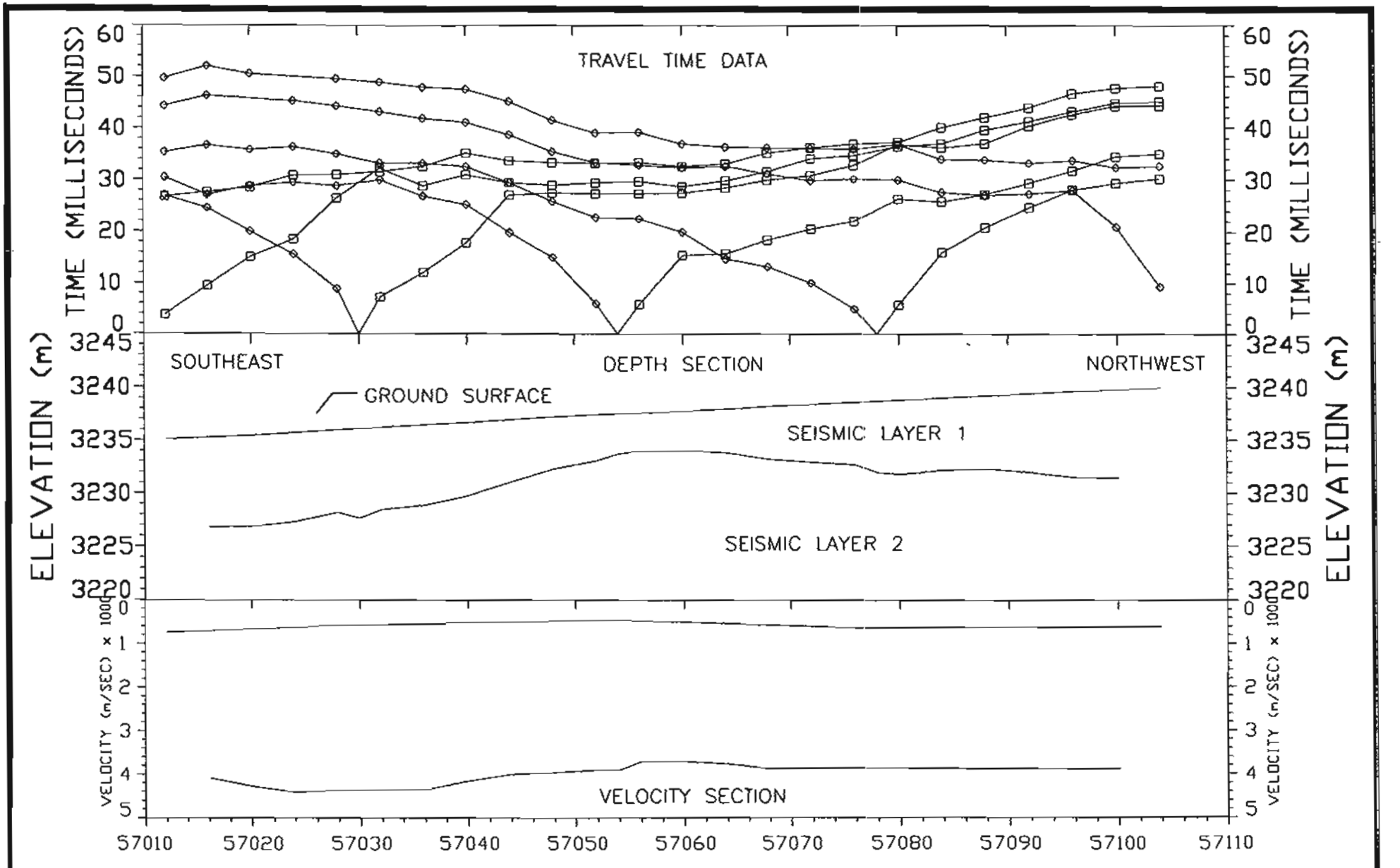


NOTES:

1. ELEVATION ASSUMED AS 3231m AT STA. 56+920
2. STATIONING USED CORRESPONDS TO THAT MARKED BY FHWA

for: Project No. 68FHAT002700	Federal Highway Administration
by: URS	Beartooth Highway (U.S. 212)
	STATIONING 56+920 TO 57+012
Data SetSP1 / Date: Sept. '00	
Equipment: Strataview / Spread: sp1	FIG. F-2

BTS-02

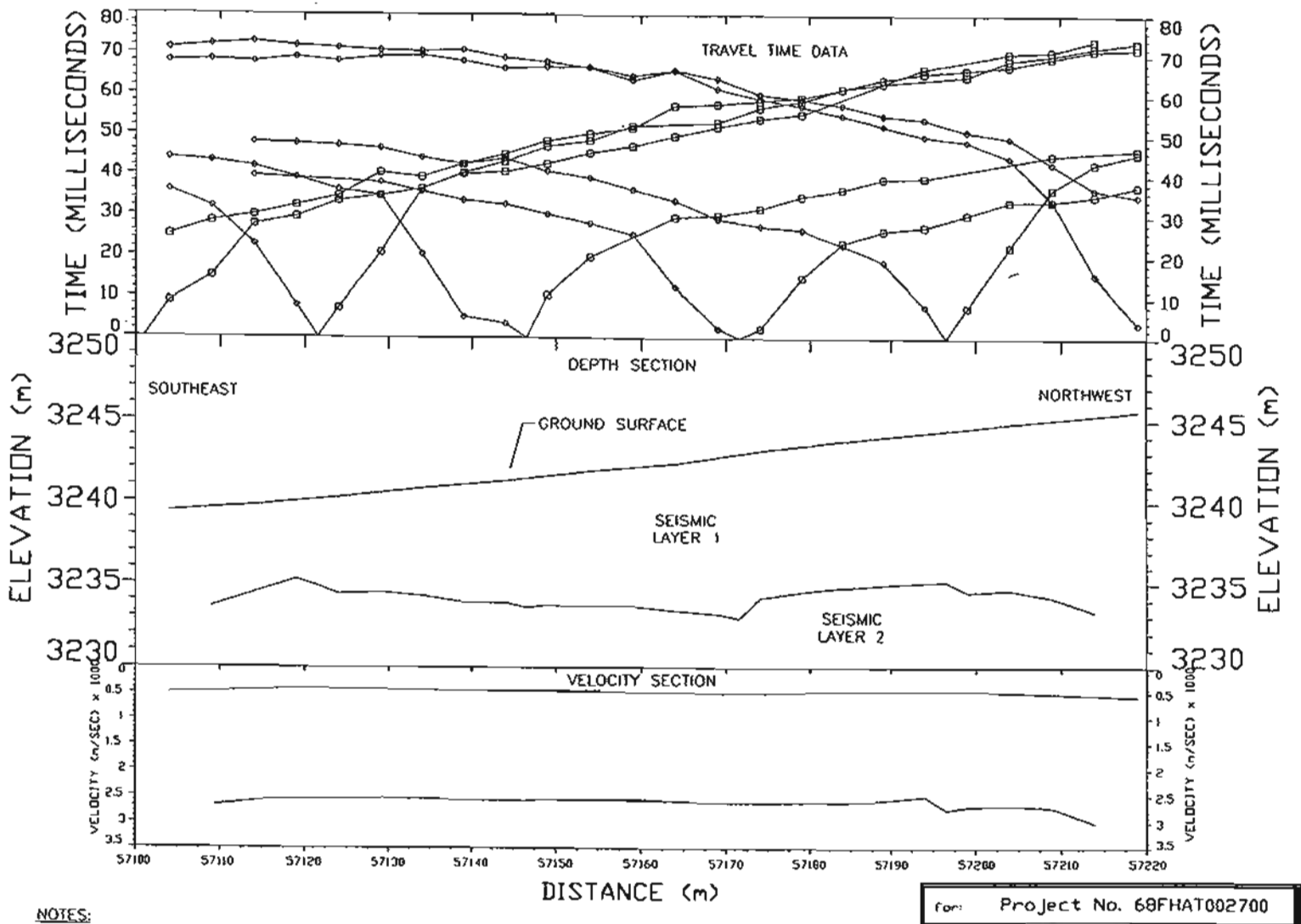


NOTES:

1. ELEVATION ASSUMED AS 3235. m AT STA. 57+012
2. STATIONING USED CORRESPONDS TO THAT MARKED BY FHWA

For: Project No. 68FHAT002700	Federal Highway Administration
by: URS	Beartooth Highway (U.S.212) STATIONING 57+012 TO 57+104
Data SetSP2 / Date: Sept. '00	
Equipment: Stratavisor / Spread: sp2	Fig. F-3

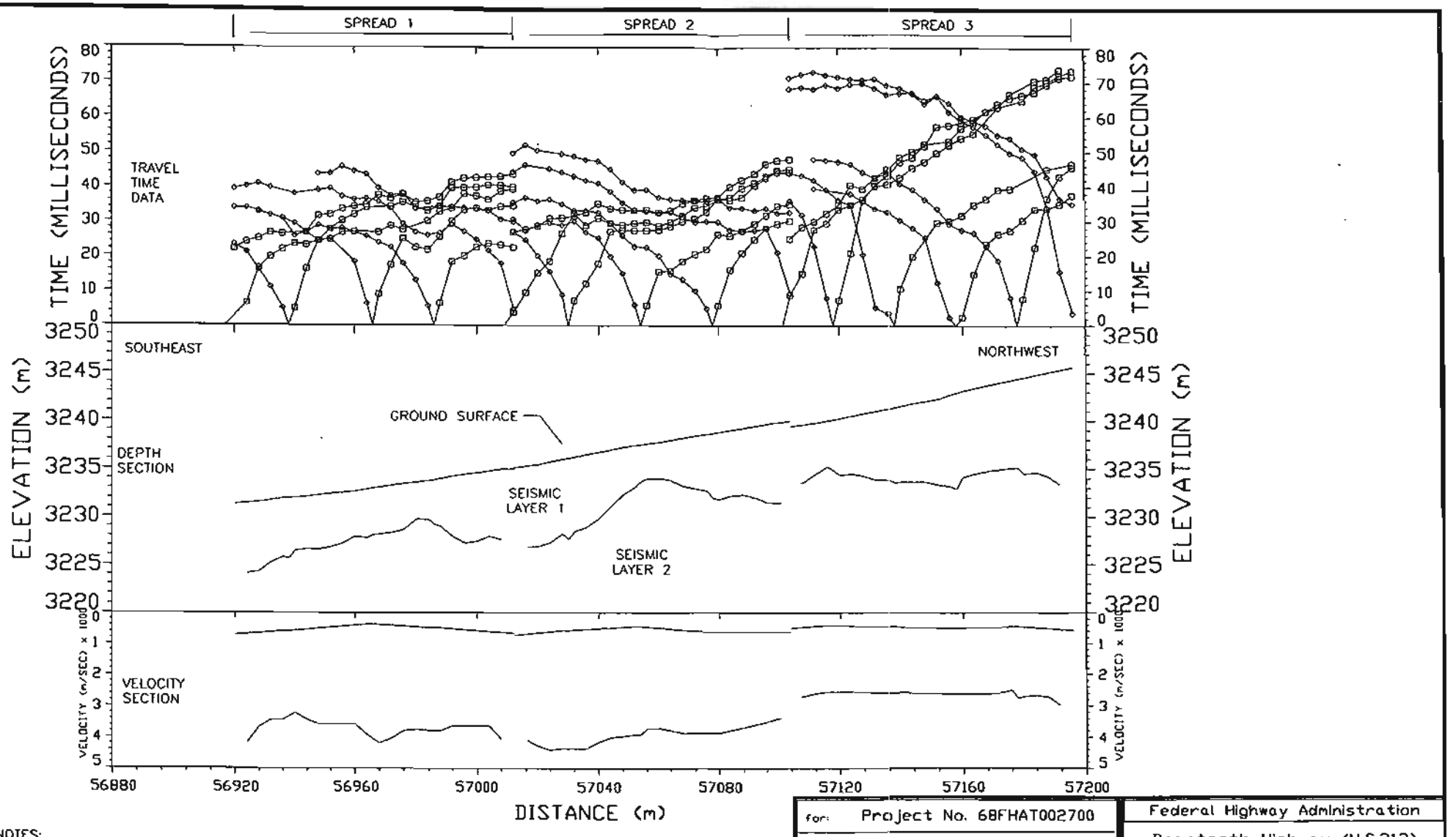
BTS-04



NOTES:

1. ELEVATION ASSUMED AS 3239.4m AT STA. 57+104
2. STATIONING USED CORRESPONDS TO THAT MARKED BY FHWA

For: Project No. 68FHAT002700		Federal Highway Administration
by: URS		Beartooth Highway (U.S.212) STATIONING 57+104 TO 57+219
Data SetSP3	Date: Sept. '00	
Equipment: Stratavlew	Spread: sp3	FIG. F-4

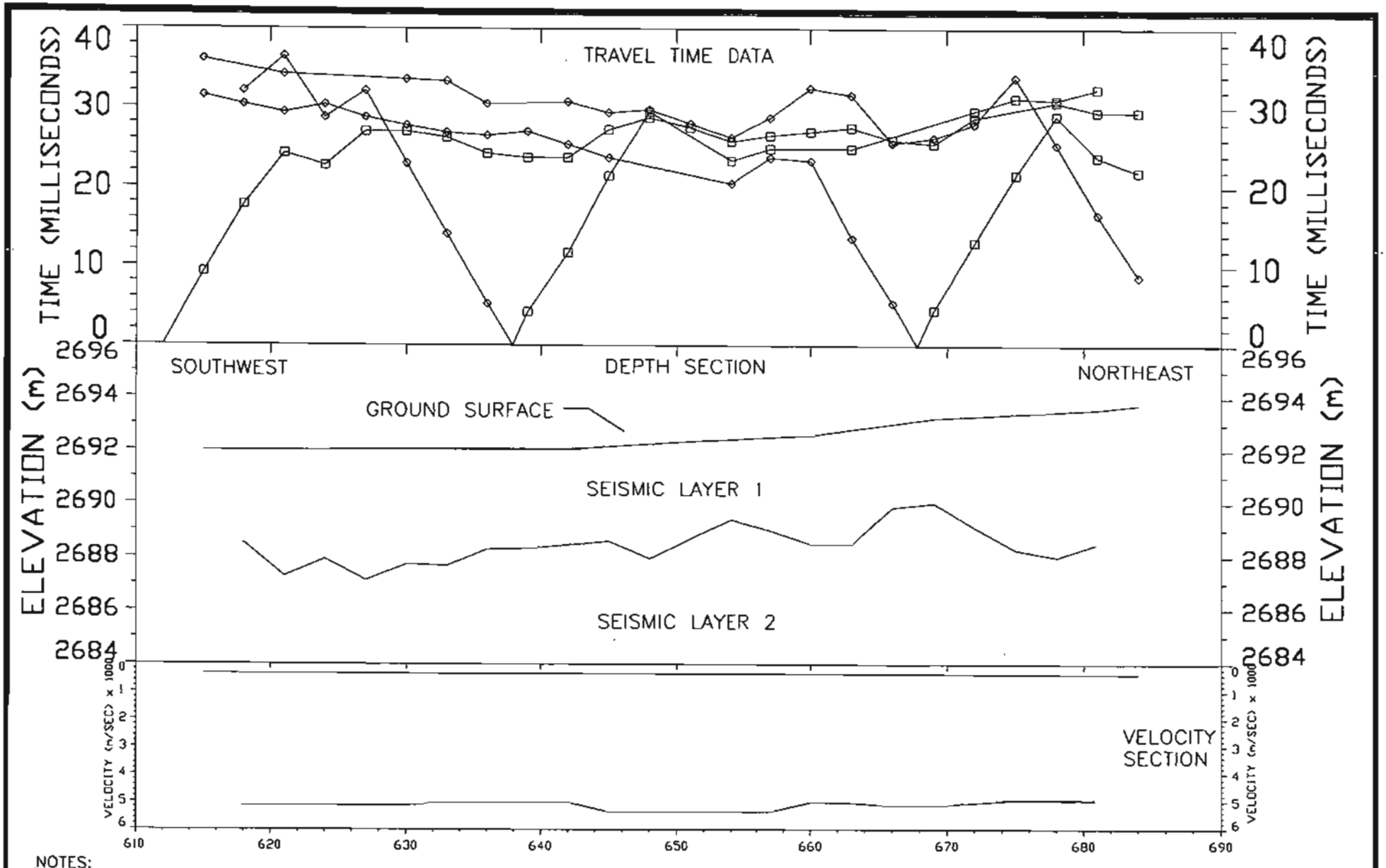


NOTES:

1. ELEVATION ASSUMED AS 3231 AT STA. 56+920
2. STATIONING USED CORRESPONDS TO THAT MARKED BY FHWA

for: Project No. 68FHAT002700	Federal Highway Administration
by: URS	Beartooth Highway (U.S.212) STATIONING 56+920 TO 57+219
Data SetSP3 / Date: Sept. '00	
Equipment: Strataview / Spreads: 1,2,3	Fig. F-5

BTS-03



NOTES:

1. GROUND ELEVATION ASSUMED AS 2692M AT STA. 41+615.
2. STATIONING USED CORRESPONDS TO THAT STAKED IN FIELD FOR MAINTENANCE; 41000 ELIMINATED FROM LABELING.
3. REFRACTOR INTERFACE REPRESENTS SHORTEST DISTANCE TO REFRACTOR. DISTANCE MAY NOT BE VERTICAL DEPTH.

for: Project No. 68FHAT002200	
by: URS Greiner Woodward Clyde	
Data SetSL1	Date: Sept. '99
SCALE 1:400	Spread: SL1

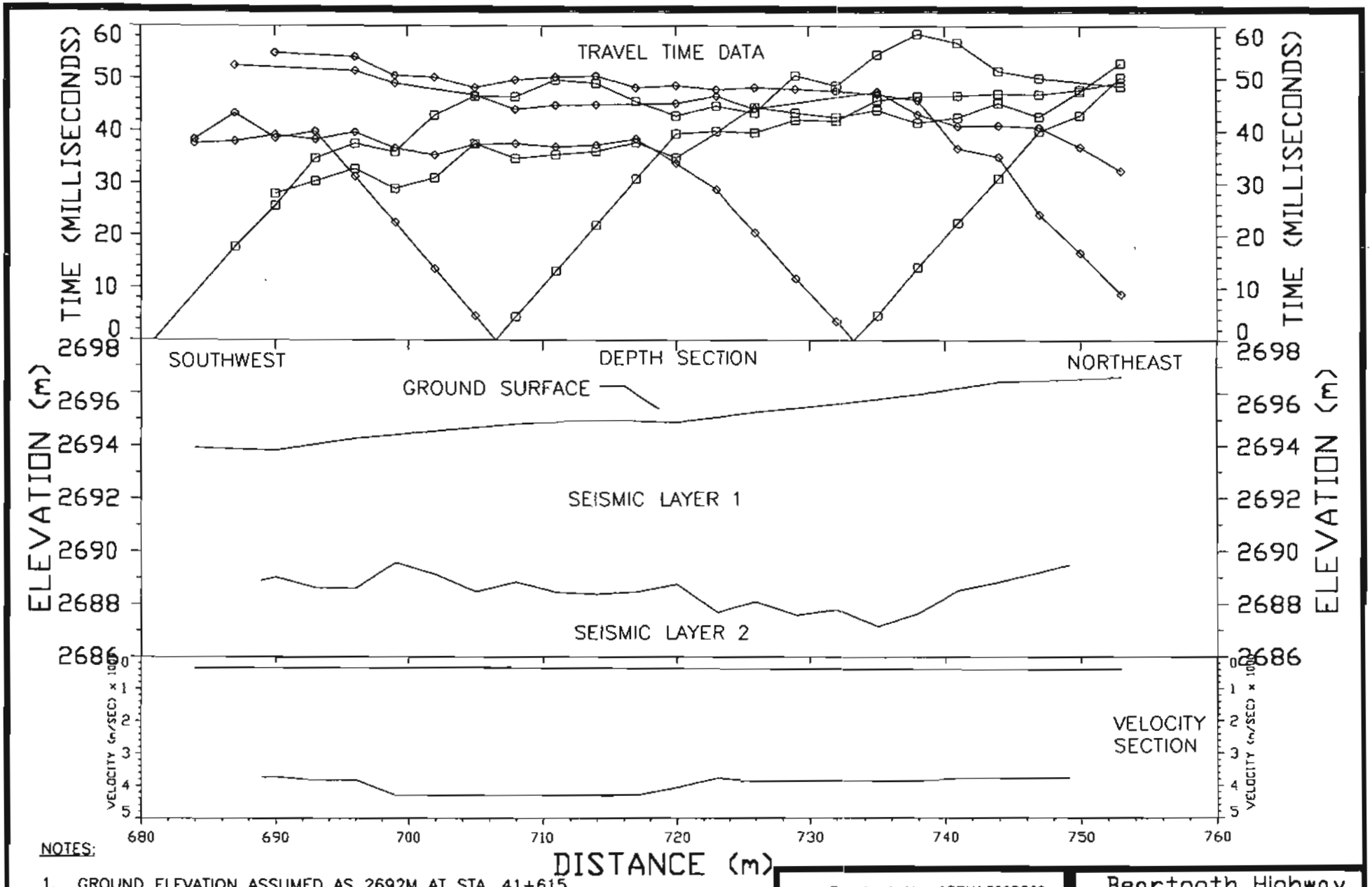
Beartooth Highway

Retaining Wall

Stationing 41+615 to 41+684

Figure F-6

BT-01



NOTES:

1. GROUND ELEVATION ASSUMED AS 2692M AT STA. 41+615.
2. STATIONING USED CORRESPONDS TO THAT STAKED IN FIELD FOR MAINTENANCE; 41000 ELIMINATED FROM LABELING.
3. REFRACTOR INTERFACE REPRESENTS SHORTEST DISTANCE TO REFRACTOR. DISTANCE MAY NOT BE VERTICAL DEPTH.

For: Project No. 68FHAT002200	
by: URS Greiner Woodward Clyde	
Data SetSL2	Date: Sept. '99
SCALE 1:400	Spread: SL2

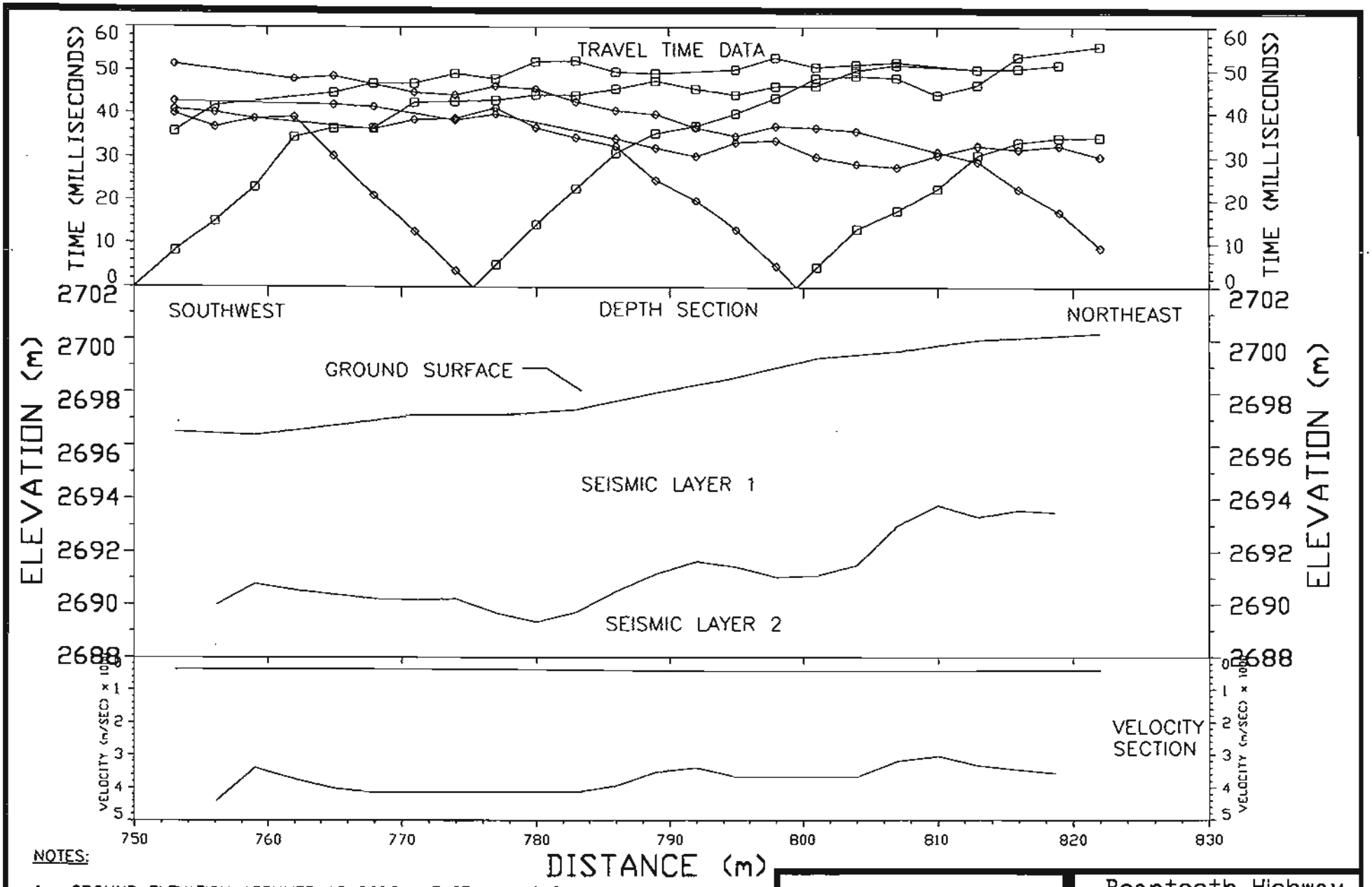
Beartooth Highway

Retaining Wall

Stationing 41+684 to 41+753

Figure F-7

BT-02



NOTES:

1. GROUND ELEVATION ASSUMED AS 2692M AT STA. 41+615.
2. STATIONING USED CORRESPONDS TO THAT STAKED IN FIELD FOR MAINTENANCE; 41000 ELIMINATED FROM LABELING.
3. REFRACTOR INTERFACE REPRESENTS SHORTEST DISTANCE TO REFRACTOR. DISTANCE MAY NOT BE VERTICAL DEPTH.

for: Project No. 68FHAT002200	
by: URS Greiner Woodward Clyde	
Data SetSL3	Date: Sept. '99
SCALE 1:400	Spread: SL3

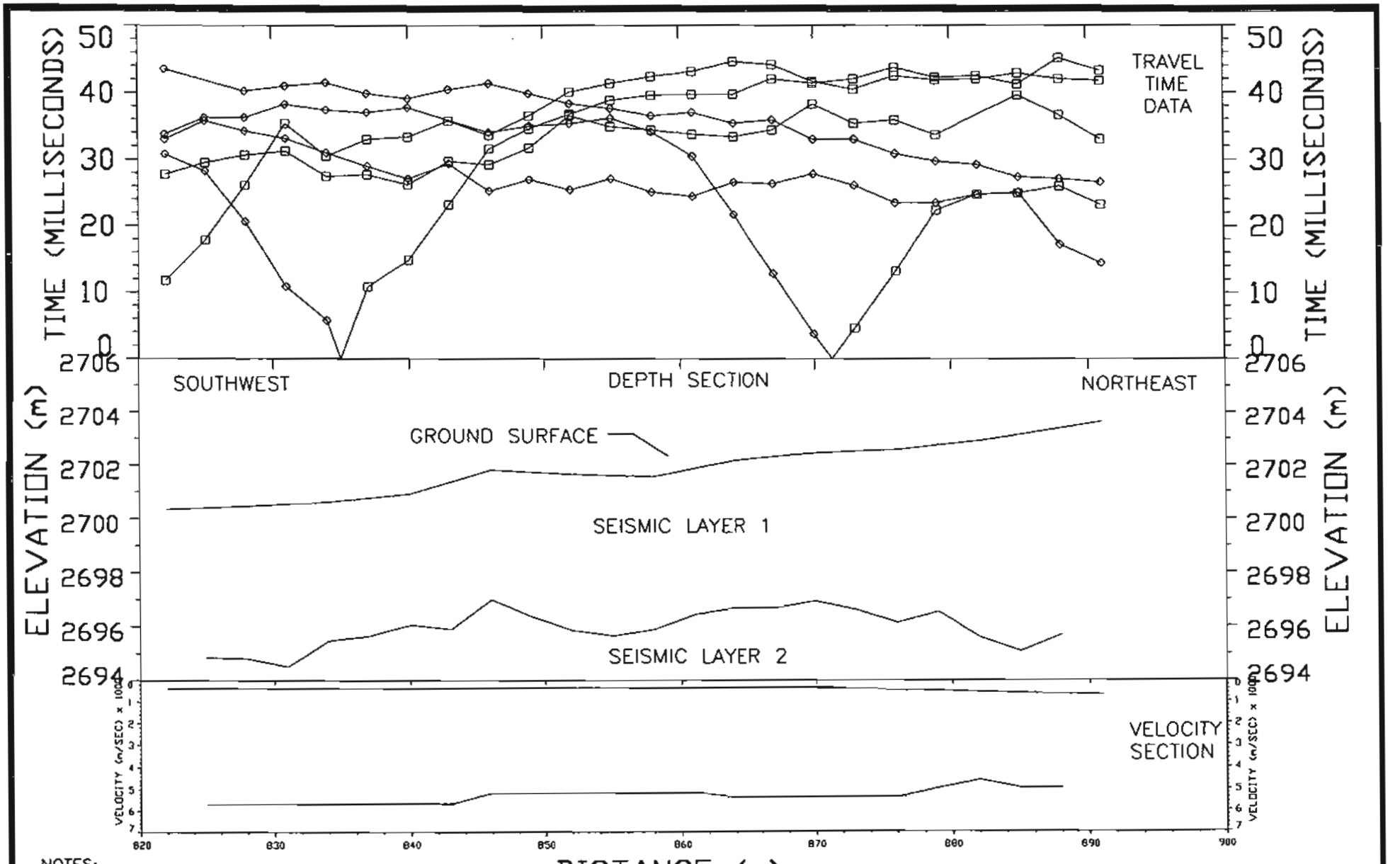
Beartooth Highway

Retaining Wall

Stationing 41+753 to 41+822

Figure F-8

BT-03



NOTES:

1. GROUND ELEVATION ASSUMED AS 2692M AT STA. 41+615.
2. STATIONING USED CORRESPONDS TO THAT STAKED IN FIELD FOR MAINTENANCE; 41000 ELIMINATED FROM LABELING.
3. REFRACTOR INTERFACE REPRESENTS SHORTEST DISTANCE TO REFRACTOR. DISTANCE MAY NOT BE VERTICAL DEPTH.

For: Project No. 68FHAT002200	
by: URS Greiner Woodward Clyde	
Data SetSL4	Date: Sept. '99
SCALE 1:400	Spread: SL4

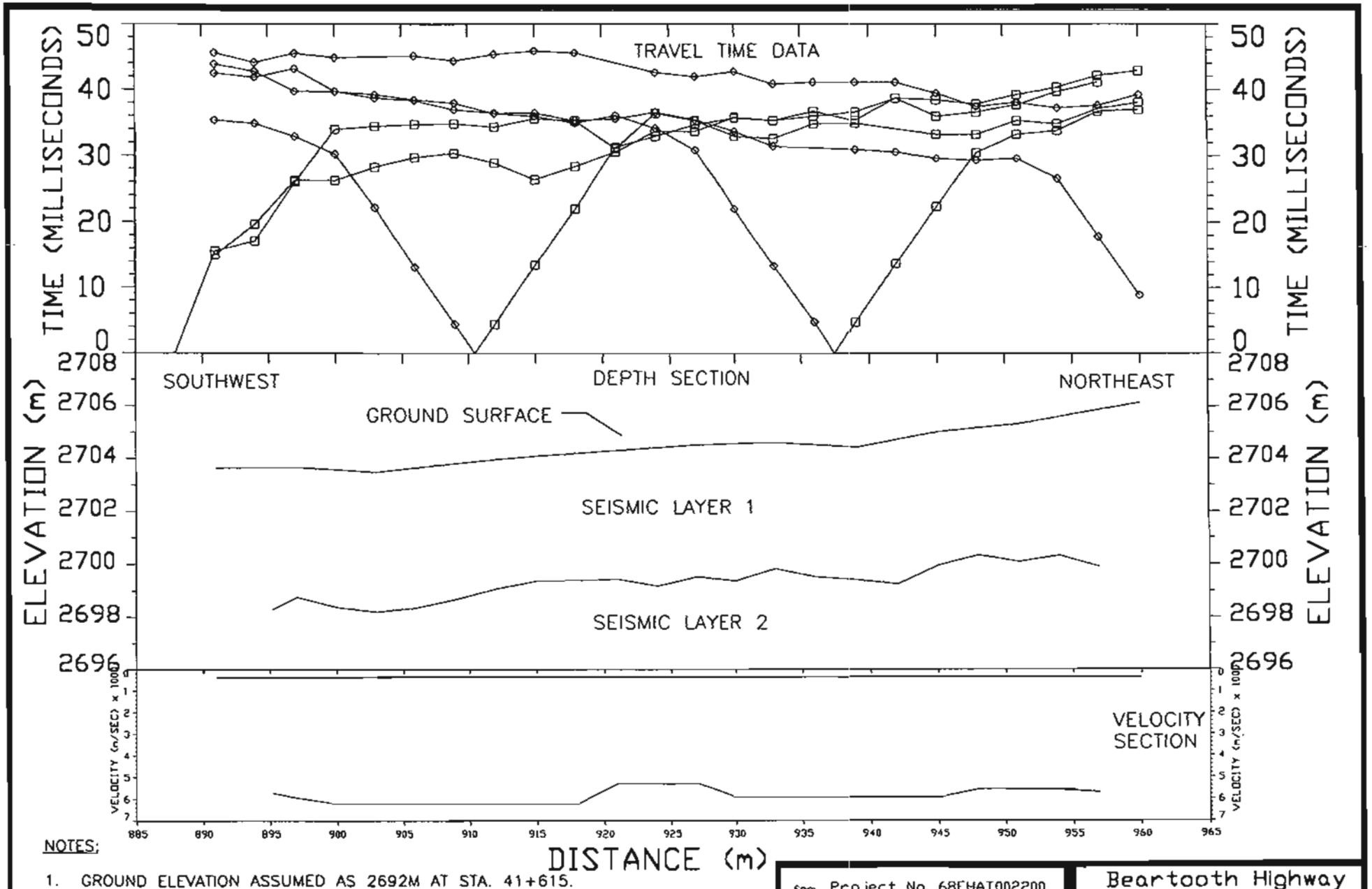
Beartooth Highway

Retaining Wall

Stationing 41+822 to 41+891

Figure F-9

BT-04



NOTES:

1. GROUND ELEVATION ASSUMED AS 2692M AT STA. 41+615.
2. STATIONING USED CORRESPONDS TO THAT STAKED IN FIELD FOR MAINTENANCE; 41000 ELIMINATED FROM LABELING.
3. REFRACTOR INTERFACE REPRESENTS SHORTEST DISTANCE TO REFRACTOR. DISTANCE MAY NOT BE VERTICAL DEPTH.

for: Project No. 68FHAT002200	
by: URS Greiner Woodward Clyde	
Data Set SL5	Date: Sept. '99
SCALE 1:400	Spread: SL5

Beartooth Highway

Retaining Wall

Stationing 41+891 TO 41+960

Figure F-10

BT-05

Appendix G

Appendix G
Cut Mapping

Table G-1
ROCK MASS CHARACTERIZATION

Rockmass Area (Shown on Figure 4-4)	Rock Type	Rock Weathering	Estimated Intact Rock Compressive Strength (MPa)	Estimated RQD	Joint Set Orientation			Spacing (m)	Continuity (m)	Aperture	Infilling		Roughness	Surface Shape
					No.	Strike	Dip				Type	Amount		
A	Granite	SW	25-100	25-75	1	N70E	50NW	0.1-1	1-3	T-N	Fe	Su	SR	PL
					2	65SE	65SE	0.1-1	1-4	T-N	Fe	Su	SR-R	PL
					3	N30W	90	0.3-3	1-4	T-MW	Fe	Su	SR-R	PL
B	Granite	SW	25-100	25-75	1	N50E	50SE	0.3-1.5	5-10	N-W	Fe	Su	SR-R	PL
					2	N55E	60NW	0.1-1	5-7	T-N	Fe	Su	SR	PL
					3	N35W	80SW	0.2-2	2-7	N-W	Fe	Su	SR	PL
					4	N10E	70SE	0.5-1	2-4	T-MW	Fe	Su	SR	PL
C	Granite Gneiss	SW-F	100-200	75-100	1	N70E	90	0.5-8	3-20	T-N	H	Fi	R	PL-Wa
					2	N30W	80SE	1-2	2-7	T-N	H	Fi	R	PL-Wa
					3	N70E	10SE	2-5	3-20	T-N	H	Fi	R	PL-Wa
					4	Random		--	3-20	T-N	H	Fi	R	PL-Wa
D	Granite Gneiss	SW-F	100-200	75-100	1	N60E	50SE	2-10	4-30	T-W	H-Mn	Fi-Su	R	PL-Wa
					2	N35W	90	0.3-4	2-10	T-W	H-Fe	Fi-Sp	SR-R	PL-Wa
					3	N70E	80NW	0.3-4	4-10	T-N	H-Fe	Fi-Sp	R	PL-Wa
					4	Random		--	4-20	T-W	H-Mn	Fi-Sp	R	PL-Wa
E	Granite Gneiss	SW-F	100-200	75-100	1	E-W	65SW	0.5-4	2-25	T-W	Fe-Mn	Su-Sp	R	PL-Wa
					2	N30E	90	0.5-7	4-40	T-W	Fe-Mn	Fi-Su	R	PL
					3	N30W	75NE	4-10	2-8	T-W	Fe	Su	R	PL
					4	N40E	50NW	0.3-1	2-4	T-N	Fe	Su	SR-R	PL
					5	N70E	90	4-10	4-8	T-N	H-Mn	Fi-Sp	R	PL-Wa
					6	Random		--	4-20	T-N	H-Mn	Fi-Su	R	PL-Wa
F	Granite Gneiss	SW	25-100	50-75	1	N30E	20NW	0.2-1	2-15	N-MW	Fe	Su	SR-R	PL
					2	N60E	70SE	0.5-4	4-10	T-MW	Fe	Su	SR-R	PL
					3	N70E	45SE	1-4	2-7	T-MW	Fe	Su	SR-R	PL
					4	N30W	90	1-7	2-8	T-MW	Fe	Su	SR-R	PL
					5	Random		--	2-10	T-MW	Fe	Su	SR-R	PL-Wa
G	Granite Gneiss	SW-MW	10-100	25-50	1	N25E	60NW	0.1-1	2-15	N-W	Fe	Su	SR-R	PL
					2	N30W	90	1-7	2-10	N-W	Fe	Su	SR	PL
					3	N70E	40SE	1-7	1-8	N-W	Fe	Su	SR	PL
					4	Random		--	2-10	N-W	Fe	Su	SR-R	PL-Wa
H	Granite Gneiss	SW-MW	10-100	25-50	1	N20W	30SW	0.1-1	2-15	N-W	Fe	Su-Sp	SR-R	PL-Wa
					2	N60E	90	2-4	4-20	N-W	Fe	Su-Sp	SR	PL
					3	N70E	45NW	0.1-0.8	2-4	N-W	Fe	Su-Sp	SR-R	PL
					4	Random		--	2-15	N-W	Fe	Su-Sp	SR-R	PL-Wa

Notes:
1. Adapted from Table 1, URS (1999)

Table G-1
ROCK MASS CHARACTERIZATION

LEGEND

ROCK WEATHERING

F	Fresh or unweathered, rock shows no discoloration, loss of strength, or other effect of weathering or alteration.
SW	Slightly weathered, rock is discolored, typically iron staining on joint surfaces, not noticeably lower in strength than fresh rock.
MW	Moderately weathered, rock is discolored into rock fabric and noticeably weakened, 2-inch-diameter sample cannot be broken by hand.

JOINT APERTURE (mm)

T	Tight (0)
N	Narrow (1-2)
MW	Moderately Wide (2-10)
W	Wide (>10)

JOINT INFILLING TYPE

Fe	Iron Oxide
H	Healed
Mn	Manganese Oxide

JOINT INFILLING AMOUNT

Su	Surface Stain
Sp	Spotty
Fi	Filled

JOINT SURFACE SHAPE

Pl	Planar
Wa	Wavy
St	Stepped

JOINT SURFACE ROUGHNESS

SR	Slightly Rough - small asperities on surface feel like silt to fine sand.
R	Rough - asperities are visible and feel like sand.

JOINT SPACING (m)

W	Wide (0.6-1.8)
M	Moderate (0.2-0.6)
C	Close (0.06-0.2)
IRR	Irregular

ROCK STRENGTH

Strong Rock	Requires one hammer blow to fracture
Very Strong Rock	Requires many hammer blows to fracture
Extremely Strong Rock	Can only be chipped with hammer blows

Table G-2
RECORD OF LINE SURVEY

Table G-2a. Line 1

Line 1	Station 53+460 - 53+474	Discontinuity Reference Number (1) (No. of occurrences shown)									
Distance (m)	Mechanical (Blast) Fractures	11	12	13	14	15	16	17	18		
1	1	2									
2	5		1								
3	2			4							
4	2			2							
5	2			2	2						
6	8										
7	4										
8	3					2					
9	2						2				
10	1							1	1		
11	1										
12	1										
13	3										
14	covered										

Table G-2b. Line 2

Line 2	Station 53+860 - 53+880	Discontinuity Reference Number (1) (No. of occurrences shown)									
Distance (m)	Mechanical (Blast) Fractures	19	20	21	22	23	24	25	26		
1	2	1									
2	2		2								
3	1		2								
4	2										
5	covered										
6	covered										
7	1										
8	2			2							
9	1				1						
10	2										
11	3					1					
12	3										
13	1						2				
14	4										
15	2										
16	2										
17	6										
18	1							1			
19	2										
20	2								1		

Table G-2
RECORD OF LINE SURVEY

Table G-2c. Line 3

Line 3	Station 54+380 - 54+400	Discontinuity Reference Number [1] (No. of occurrences shown)									
Distance (m)	Mechanical (Blast) Fractures	1	2	3	4	5	6	7	8	9	10
1	1	2									
2	0		1								
3	2		2								
4	1			1							
5	2			3							
6	covered										
7	covered										
8	3				2						
9	2			2							
10	4					5					
11	covered										
12	covered										
13	covered										
14	5										
15	1						1				
16	0							2			
17	1								2		
18	1									2	
19	2										3
20	2										

Notes:

1. For example, for line 1 at a distance of 1m, there were 2 occurrences of discontinuity No. 11 (see table G-2D for descriptions of discontinuities)
2. Line surveys were conducted approximately 1.5 meters above the road elevation at the stations indicated.
3. Discontinuity descriptions are consistent with face mapping and test hole logs.
4. Mechanical (Blast) fractures are identified by their limited persistence and fresh appearance.
5. Strike and dip values recorded represent an average of multiple readings where possible.
6. Areas indicated as covered were covered with soil or plants.

Table G-2
RECORD OF LINE SURVEY

Table G-2d. Description of Discontinuities in Line Surveys

No.	Discontinuity Type	Strike	Dip	Aperture	Filling	Infilling Amount	Surface Shape	Roughness	Joint Spacing	RQD	Strength
1	J	278	51 S	MW	Fe	Su	Wa	SR	W	90-100	Strong to Extremely Strong
2	J	233	75 S	N	Fe	Su	Pl	SR	W	90-100	Strong to Extremely Strong
3	J	145	54 N	N	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong
4	J	274	56 S	N	Fe	Su	St	R	M	90-100	Strong to Extremely Strong
5	J	52	90 S	N	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong
6	J	214	52 S	N	Fe	Su	Pl	SR	M - W	90-100	Strong to Extremely Strong
7	J	220	78 S	N	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong
8	J	222	79 N	N	Fe	Su	Pl	SR	M - C	90-100	Strong to Extremely Strong
9	J	239	51 N	N	Fe	Su	Wa	R	W	90-100	Strong to Extremely Strong
10	J	229	90 S	N	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong
11	J	40	68 N	N	Fe	Su	Pl	SR	W	90-100	Strong to Extremely Strong
12	J	38	70 N	N	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong
13	J	354	85 N	N	Fe	Su	Pl	SR	W - IRR	90-100	Strong to Extremely Strong
14	J	3	72 S	N	Fe	Su	Pl	SR	W - IRR	90-100	Strong to Extremely Strong
15	J	42	80 W	N	Fe	Su	Pl	SR	W - IRR	90-100	Strong to Extremely Strong
16	J	75	46 W	N	Fe	Su	Pl	SR	W - IRR	90-100	Strong to Extremely Strong
17	J	78	52 W	N	Fe	Su	Pl	R	W - IRR	90-100	Strong to Extremely Strong
18	J	20	86 N	N	Fe	Su	Pl	SR	W - IRR	90-100	Strong to Extremely Strong
19	J	83	85 W	N	Fe	Su	Pl	SR	W - IRR	90-100	Strong to Extremely Strong
20	J	65	68 W	N	Fe	Su	Pl	SR	W - IRR	90-100	Strong to Extremely Strong
21	J	70	88 W	N	Fe	Su	Pl	SR	W - IRR	90-100	Strong to Extremely Strong
22	J	42	42 W	N	Fe	Su	Pl	R	W - IRR	90-100	Strong to Extremely Strong
23	J	32	86 S	W	Fe	Su	Pl	R	W - IRR	90-100	Strong to Extremely Strong
24	J	38	50 W	N	Fe	Su	Pl	SR	W - IRR	90-100	Strong to Extremely Strong
25	J	47	34 W	N	Fe	Su	Pl	SR	W - IRR	90-100	Strong to Extremely Strong
26	J	31	82 S	W	Fe	Su	Pl	R	W - IRR	90-100	Strong to Extremely Strong

Note: See Key to Terms in Table G-1.

Table G-3
Summary of Cut Slope Analysis and Recommendations

Approximate Location	Kinematically Possible Failure Mode	Discontinuities (Reference)	Dip of Sliding	Direction of Sliding	Approximate Dip Direction of Cut	Alternative Slope Ratios (V:H)	Approximate Maximum Cut Slope Height (m)	Recommended Cut Slope Ratio (V:H)	Basis of Recommendations
Station 41+510 to 51+540	Wedge (Almost Planar)	346°, 42°W (#1, Fig. G-1a) 99°, 84°S (#2, Fig. G-1a)	42°	256°	245°	1:0.25 1:0.5 1:1	14 20 Unknown	1:1.5 (soil) 2:1 (rock)	• 0 to 6 meter horizontal cut • Controlled blasting employed • Scaling employed • 3meter (min.) rockfall collection ditch • Spot bolting for identified and unidentified wedges
Station 41+540 to 41+580	No unstable wedges identified	(Fig. G-1b)			150° to 245°	1:0.25 1:0.5 1:1	16 20 35	2:1 (10:1 slope for 3 m max. height)	• 2 to 6 meter horizontal cut • Controlled blasting employed • Scaling employed • Maximum practical rockfall collection ditch, 2 meter (min.) • Spot bolting for unidentified wedges on 1:0.5 slope, pattern bolting on 1:0.1 slope
Station 41+580 to 41+720	Wedge (Almost Planar)	N70E, 45SE (Area F, Table G-1) N30W, 90° (Area F, Table G-1)	44°	149°	150°	1:0.25 1:0.5 1:1	6 10 45	2:1 (10:1 slope for 3 m max. height)	• 0 to 2 meter horizontal cut • Controlled blasting • Scaling • Maximum practical rockfall collection ditch, 2 meters (min.) • Spot bolting for identified and unidentified wedges on 1:0.5 slope, pattern bolting on 1:0.1 slope
Station 41+720 to 41+800	Wedge (Almost Planar)	250°, 61S (#7, Fig. G-2) N30E, 90 (Area E, Table G-1)	61°	160°	150°	1:0.25 1:0.5 1:1	15 to 20 55 70	Cut slope not recommended	• Potential for cut slope to destabilize existing slope on large failure wedge
Station 41+800 to 41+980	Not Analyzed							Use existing slope as much as practical. 1:1.5 (soil) (or reinforce) 2:1 (rock)	• Alternating colluvium and bedrock • New slopes will be difficult to construct and maintain • Minimal benefit to alignment

Table G-3
Summary of Cut Slope Analysis and Recommendations

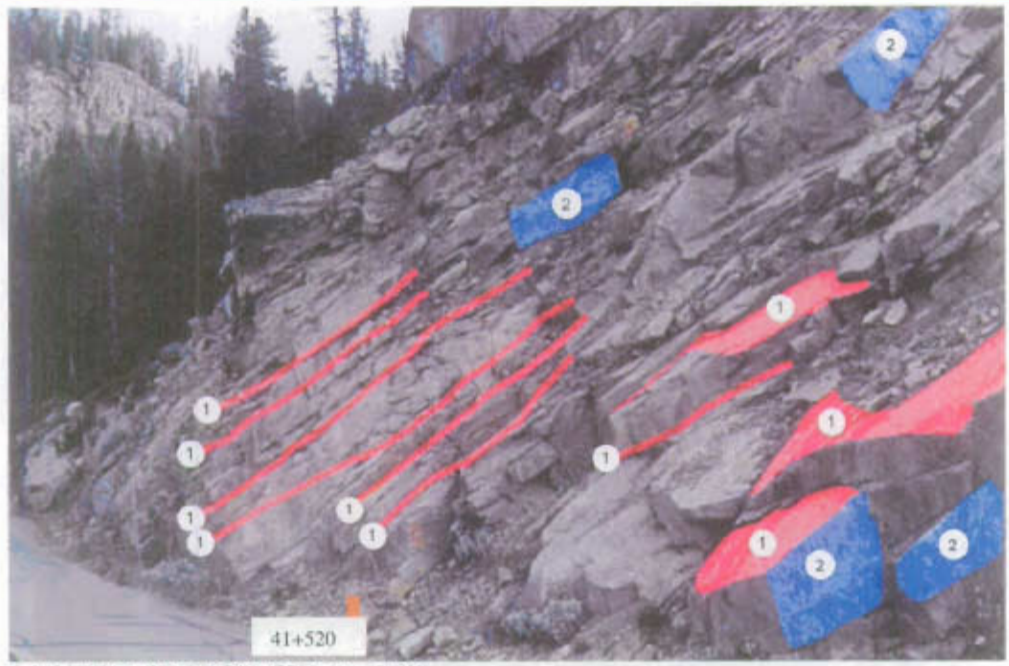
Approximate Location	Kinematically Possible Failure Mode	Discontinuities (Reference)	Dip of Sliding	Direction of Sliding	Approximate Dip Direction of Cut	Alternative Slope Ratios (V:H)	Approximate Maximum Cut Slope Height (m)	Recommended Cut Slope Ratio (V:H)	Basis of Recommendations
Station 41+980 to 42+060	Wedge	52°, 59°S (#1, Fig. G-3) 153°, 84°S (#2, Fig. G-3)	57°	162°	185°	1:0.25 1:0.5 1:0.75	5 7 7	1:0.5 (pattern bolting) 1:0.75 (spot bolting) 1:1 (no bolting)	• Controlled blasting employed • Scaling employed • 2 meter (min.) rockfall collection ditch • Occasional to no bolting for 1:1 slope, spot bolting for 1:0.75 slope, pattern bolting for 1:0.5 slope
Station 53+485	No unstable wedges identified				300°	0.25:1 0.5:1 1:1	10 12 12	4:1	Strong, moderately to widely fractured rock, with wedges that are stable, some slide into the slope, many areas with steeply dipping into the slope joints do not form unstable wedges
53+535	No unstable wedges identified				215°	1:0.25 1:0.5 1:1	6 8 9	4:1	Strong, moderately to widely fractured rock, with wedges that are stable, some slide into the slope, many areas with steeply dipping into the slope joints do not form unstable wedges
53+660	No unstable wedges identified				280°	1:0.25 1:0.5 1:1	7 7 6	4:1	Strong, moderately to widely fractured rock, with wedges that are stable, some slide into the slope, many areas with steeply dipping into the slope joints do not form unstable wedges
53+800	No unstable wedges identified				245°	1:0.25 1:0.5 1:1	4 4 4	4:1	Strong, moderately to widely fractured rock, with wedges that are stable, some slide into the slope, many areas with steeply dipping into the slope joints do not form unstable wedges
53+840	No unstable wedges identified				245°	1:0.25 1:0.5 1:1	6 8 8	4:1	Strong, moderately to widely fractured rock, with wedges that are stable, some slide into the slope, many areas with steeply dipping into the slope joints do not form unstable wedges

Table G-3
Summary of Cut Slope Analysis and Recommendations

Approximate Location	Kinematically Possible Failure Mode	Discontinuities (Reference)	Dip of Sliding	Direction of Sliding	Approximate Dip Direction of Cut	Alternative Slope Ratios (V:H)	Approximate Maximum Cut Slope Height (m)	Recommended Cut Slope Ratio (V:H)	Basis of Recommendations
53+940	No unstable wedges identified				250°	1:0.25 1:0.5 1:1	5 7 8	4:1	Strong, moderately to widely fractured rock, with wedges that are stable, some slide into the slope, many areas with steeply dipping into the slope joints do not form unstable wedges
53+980	No unstable wedges identified				270°	1:0.25 1:0.5 1:1	4 4 4	4:1	Strong, moderately to widely fractured rock, with wedges that are stable, some slide into the slope, many areas with steeply dipping into the slope joints do not form unstable wedges
54+320	No unstable wedges identified				195°	1:0.25 1:0.5 1:1	4 6 7	4:1	Strong, moderately to widely fractured rock, with wedges that are stable, some slide into the slope, many areas with steeply dipping into the slope joints do not form unstable wedges
54+380	None - stable wedges	211°, 42°N (#20, Fig. G-10) 73°, 49°S (#23, Fig. G-10)	20°	235°	200°	1:0.25 1:0.5 1:1	10 11 11	4:1	Strong, moderately to widely fractured rock, with wedges that are stable, some slide into the slope, many areas with steeply dipping into the slope joints do not form unstable wedges
54+420	No unstable wedges identified				245°	1:0.25 1:0.5 1:1	10 12 14	4:1	Strong, moderately to widely fractured rock, with wedges that are stable, some slide into the slope, many areas with steeply dipping into the slope joints do not form unstable wedges
54+460	None - stable wedges	60°, 39°W (#30, Fig. G-12) 35°, 40°S (#31, Fig. G-12)	10°	47°	245°	1:0.25 1:0.5 1:1	6 6 6	4:1	Strong, moderately to widely fractured rock, with wedges that are stable, some slide into the slope, many areas with steeply dipping into the slope joints do not form unstable wedges
54+560	No unstable wedges identified				270°	1:0.25 1:0.5 1:1	5 7 8	4:1	Strong, moderately to widely fractured rock, with wedges that are stable, some slide into the slope, many areas with steeply dipping into the slope joints do not form unstable

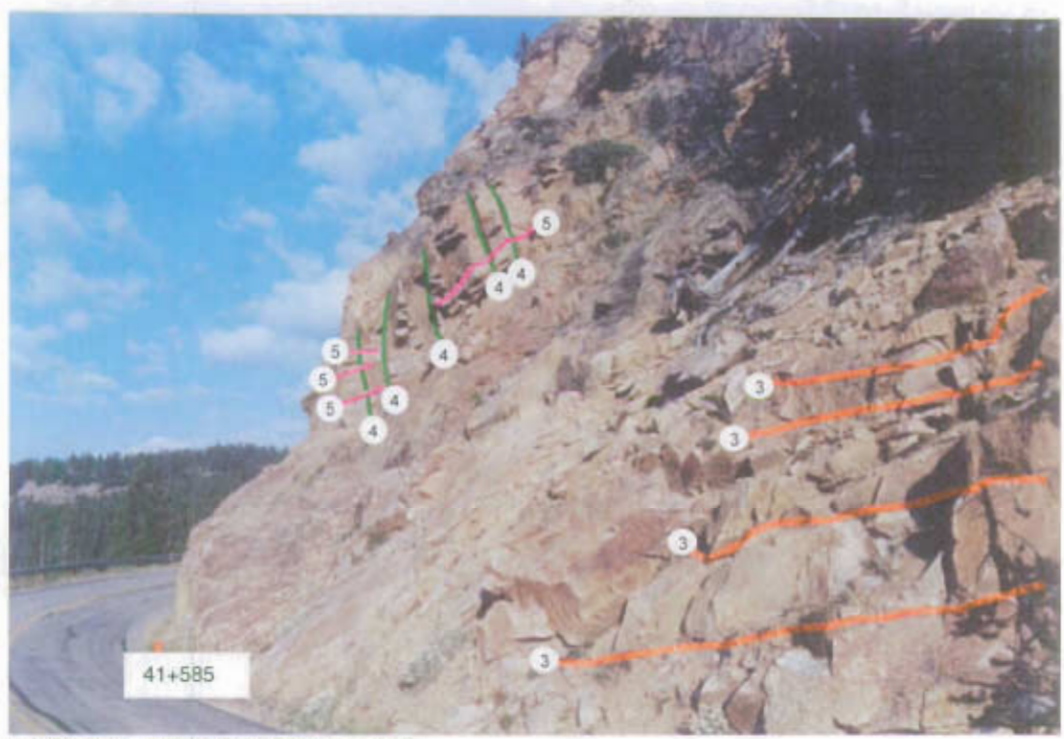
Table G-3
Summary of Cut Slope Analysis and Recommendations

Approximate Location	Kinematically Possible Failure Mode	Discontinuities (Reference)	Dip of Sliding	Direction of Sliding	Approximate Dip Direction of Cut	Alternative Slope Ratios (V:H)	Approximate Maximum Cut Slope Height (m)	Recommended Cut Slope Ratio (V:H)	Basis of Recommendations
									wedges
54+664	None - stable wedges	100°, 15°W (#35, Fig. G-13) 154°, 86°N (#36, Fig. G-13)	12°	153°	275°	1:0.25 1:0.5 1:1	8 9 10	4:1	Strong, moderately to widely fractured rock, with wedges that are stable, some slide into the slope, many areas with steeply dipping into the slope joints do not form unstable wedges
54+720	None -stable wedges	105°, 32°W (#37, Fig. G-14) 39°, 58°N (#38, Fig. G-14)	25°	146°	25	1:0.25 1:0.5 1:1	4 5 6	4:1	Strong, moderately to widely fractured rock, with wedges that are stable, some slide into the slope, many areas with steeply dipping into the slope joints do not form unstable wedges



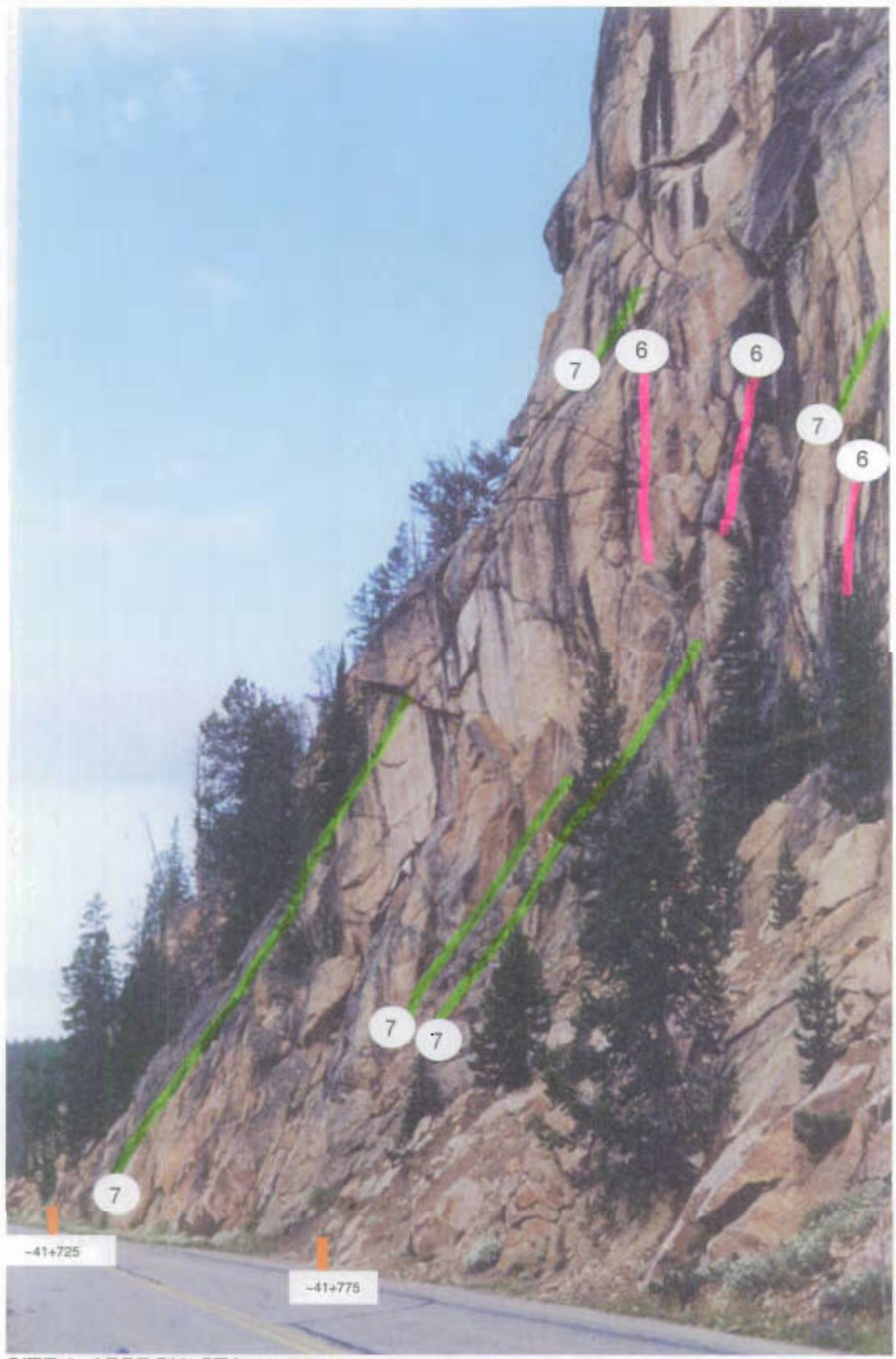
(a) SITE 2, APPROX. STA 41+520

Number	Discontinuity Type	Strike	Dip	Width	Filling	Amount	Surface Shape	Roughness	Spacing	Est. RQD	Strength (see p. B-54)
1	J	346	42 W	N - NW	Fe	Su	Pl	SR	C	50	Moderately Strong to Very Strong
2	J	99	84 S	N	Fe	Su	Pl	SR	W	50	Moderately Strong to Very Strong



(b) SITE 2, APPROX. STA 41+585

Number	Discontinuity Type	Strike	Dip	Width	Filling	Amount	Surface Shape	Roughness	Spacing	Est. RQD	Strength (see p. B-54)
3	J	134	1 W	N	Fe	Su	Pl	SR	C	75	Moderately Strong to Very Strong
4	J	109	8 E	W	Fe	Su	Pl	SR	M	75	Moderately Strong to Very Strong
5	J	92	1 W	N	Fe	Su	Pl	SR	M	75	Moderately Strong to Very Strong

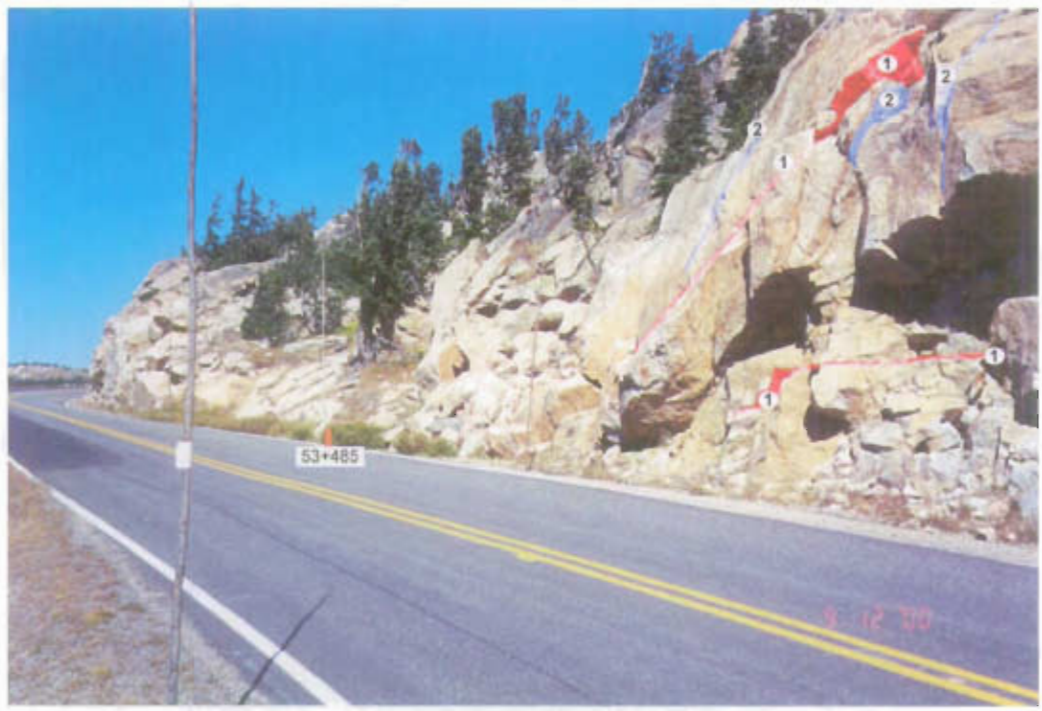


SITE 2, APPROX. STA 41+775

Number	Discontinuity Type	Strike	Dip	Width	Filling	Amount	Surface Shape	Roughness	Spacing	Est. RQD	Strength (see p. B-54)
6	J	312	-90	N	Fe	Su	Pl	SR	M	90-100	Moderately Strong to Very Strong
7	J	250	61 S	N	Fe	Su	Pl	SR	M - C	90-100	Moderately Strong to Very Strong

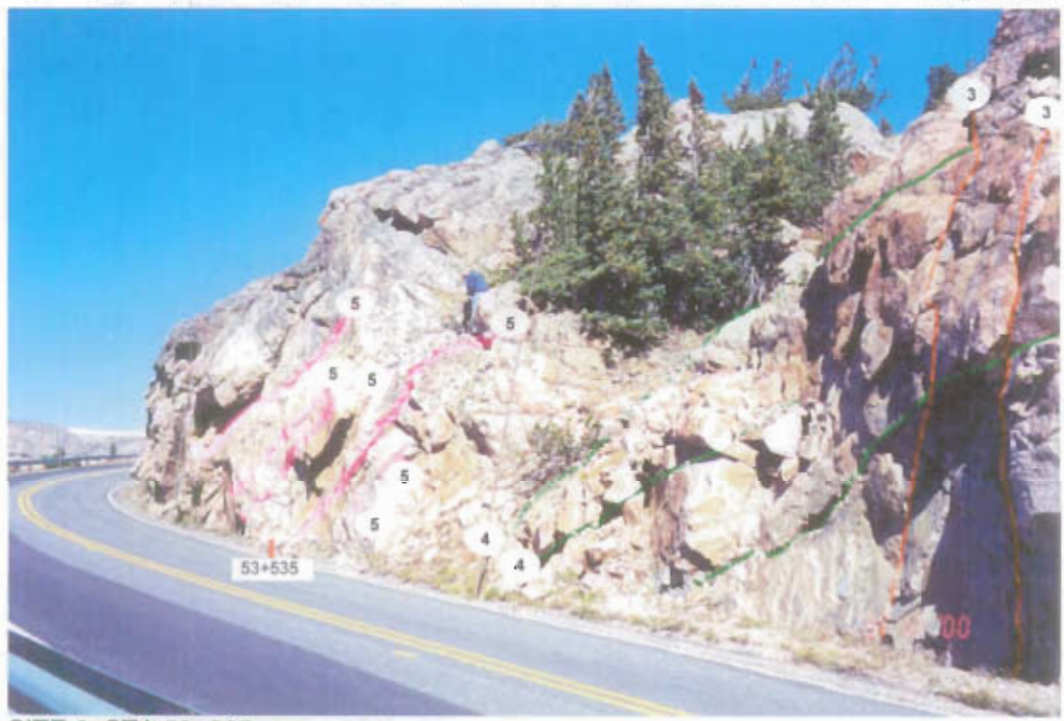
SITE 2 through cut, APPROX. STA 42+020

Number	Discontinuity Type	Strike	Dip		Width	Filling	Amount	Surface Shape	Roughness	Spacing	Est. RQD	Strength (see p. B-54)
1	J	52	59	S	MW - N	Fe	Su	Pl	SR	C	50	Moderately Strong to Very Strong
2	J	153	84	S	MW - N	Fe	Su	Wa	SR	M - C	50	Moderately Strong to Very Strong
3	J	61	46	N	MW - N	Fe	Su	Wa	SR	M - C	50	Moderately Strong to Very Strong



SITE 3, STA 53+485

Number	Discont-inuity Type	Strike	Dip		Width	Filling	Amount	Surface Shape	Rough-ness	Spacing	Est. RQD	Strength (see p. B-54)
1	J	110	63	N	MW	Fe	Su	Pl	SR	W	90-100	Strong to Extremely Strong
2	J	242	82	W	MW	Fe	Su	Pl	SR	W	90-100	Strong to Extremely Strong

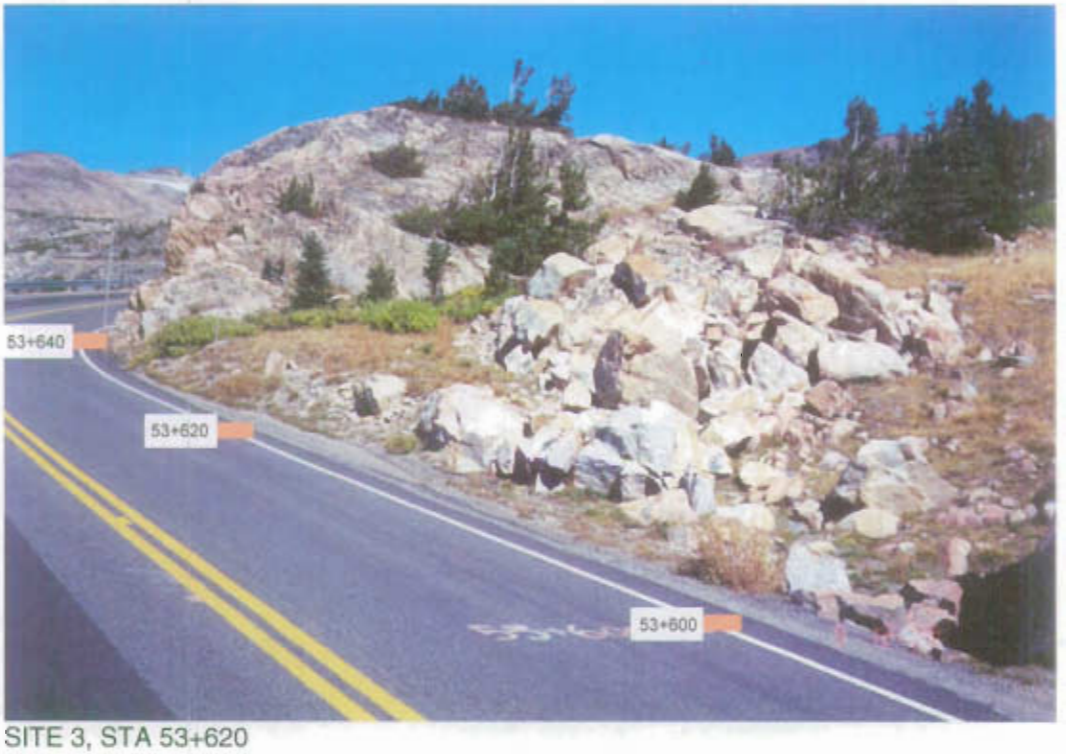


SITE 3, STA 53+535

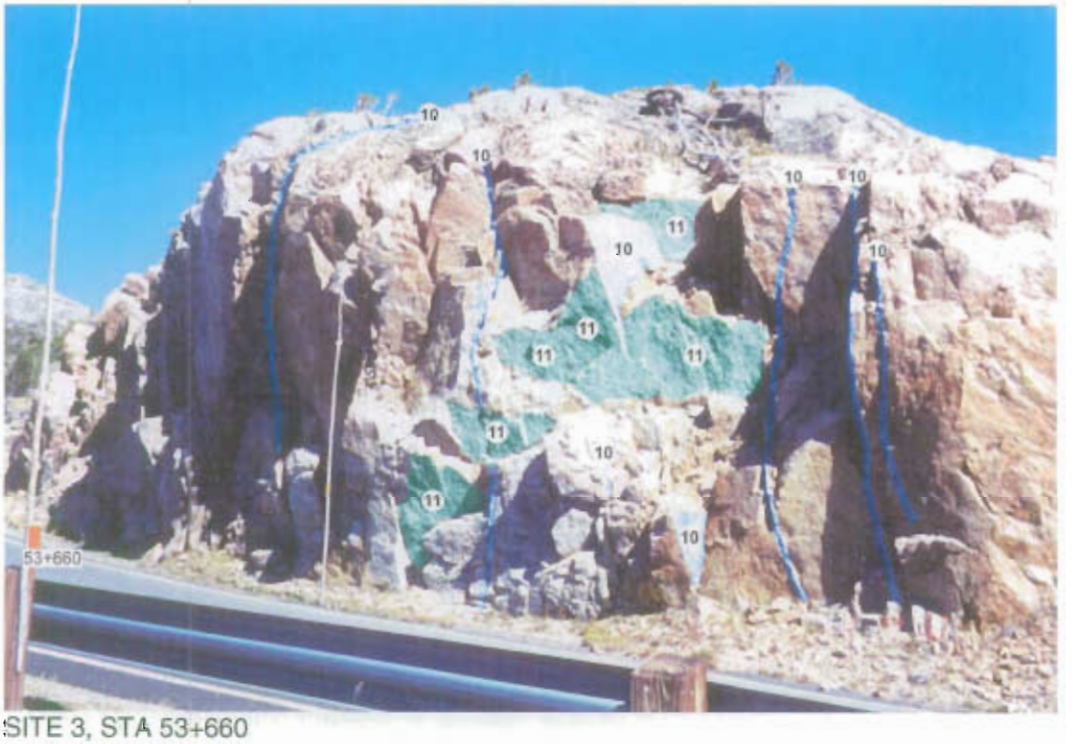
Number	Discont-inuity Type	Strike	Dip		Width	Filling	Amount	Surface Shape	Rough-ness	Spacing	Est. RQD	Strength (see p. B-54)
3	J	63	84	W	N	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong
4	J	29	42	N	MW	Fe	Su	St	R	M	90-100	Strong to Extremely Strong
5	J	12	54	W	N	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong

SITE 3, STA 53+535

Number	Discont -inuity Type	Strike	Dip	Width	Filling	Amount	Surface Shape	Rough- ness	Spacing	Est. RQD	Strength (see p. B-54)
6	J	78	64 N	MW	Fe	Su	Pl	SR	M - W	90-100	Strong to Extremely Strong
7	J	211	28 S	N	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong
8	J	236	81 N	N	Fe	Su	Pl	SR	M - C	90-100	Strong to Extremely Strong
9	J	169	64 N	N	Fe	Su	Wa	SR	W	90-100	Strong to Extremely Strong

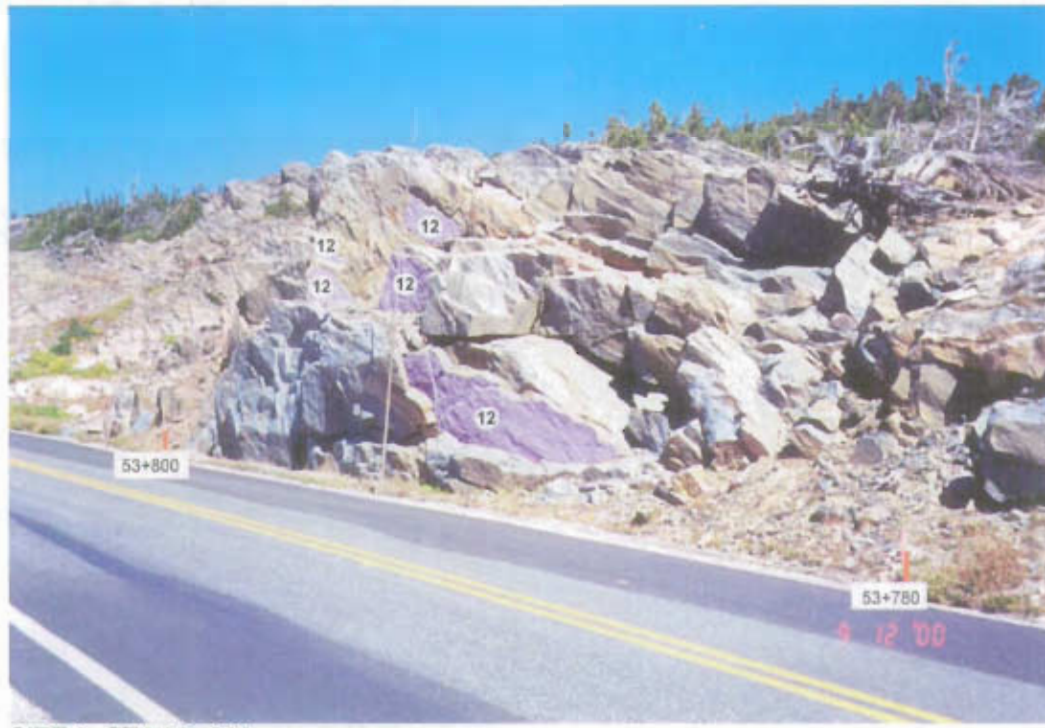


SITE 3, STA 53+620



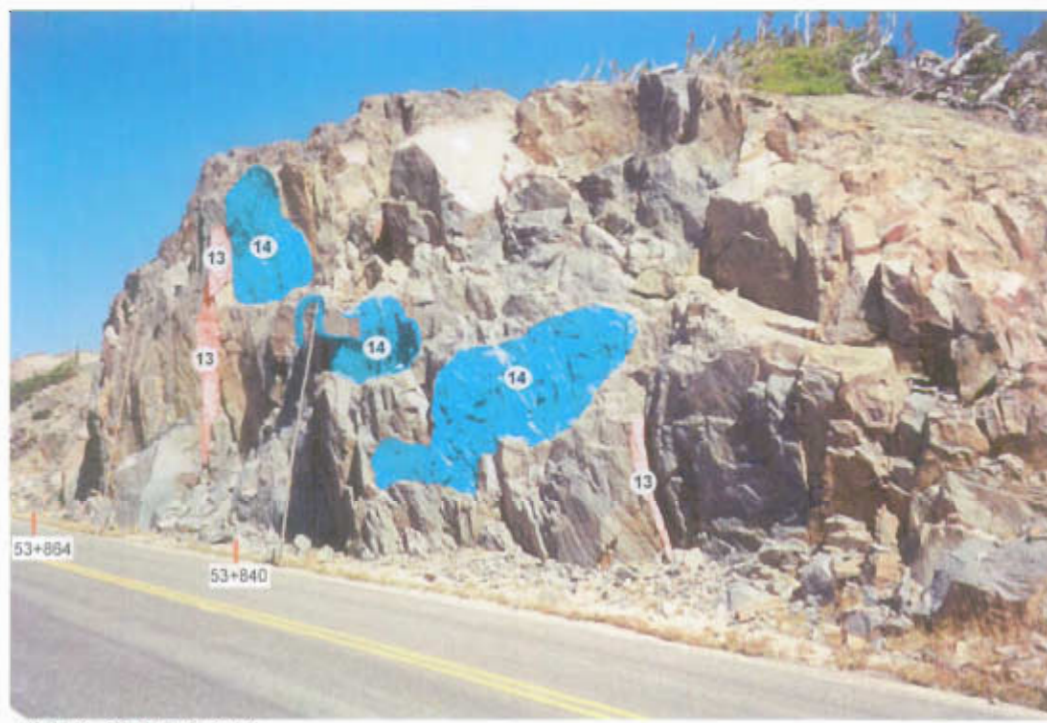
SITE 3, STA 53+660

Number	Discontinuity Type	Strike	Dip		Width	Filling	Amount	Surface Shape	Roughness	Spacing	Est. RQD	Strength (see p. B-54)
10	J	279	79	S	N	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong
11	J	173	81	W	N	Fe	Su	Pl	SR	W	90-100	Strong to Extremely Strong



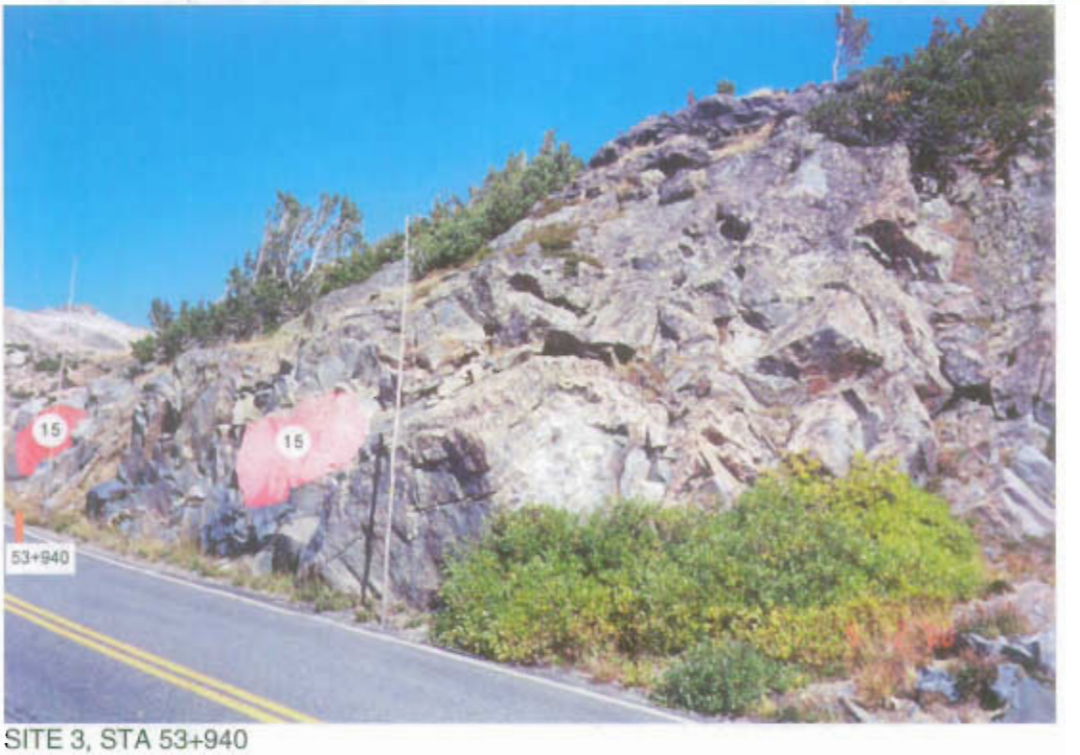
SITE 3, STA 53+800

Number	Discont-inuity Type	Strike	Dip		Width	Filling	Amount	Surface Shape	Rough-ness	Spacing	Est. RQD	Strength (see p. B-54)
12	J	148	71	N	N	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong



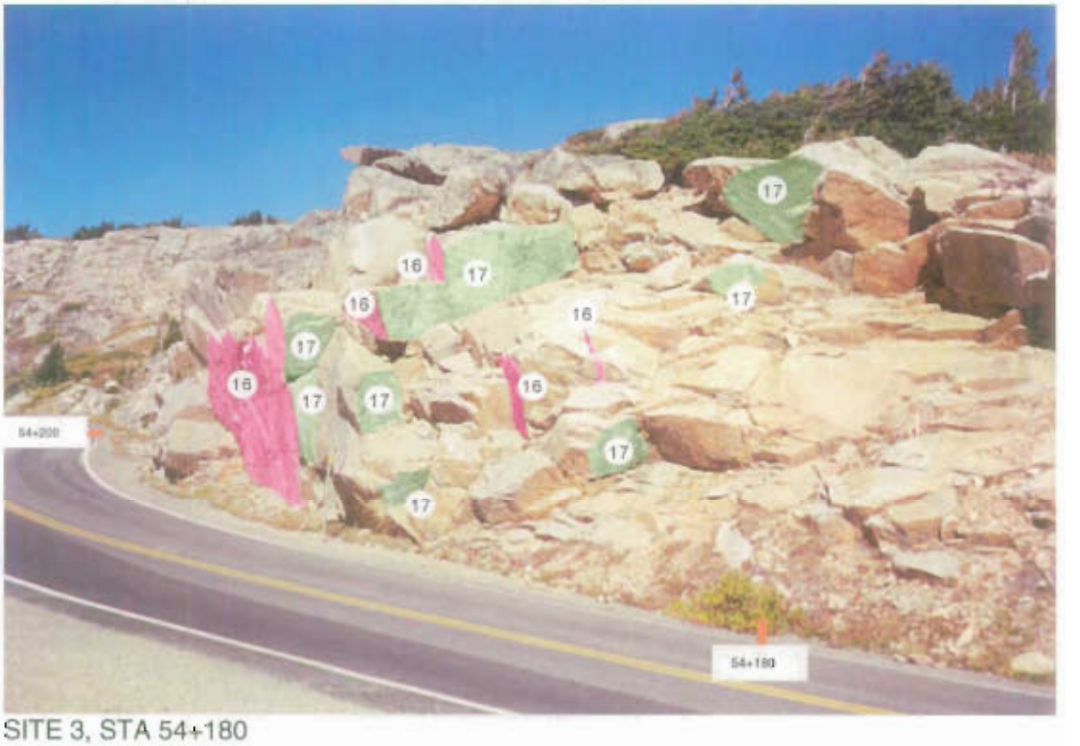
SITE 3, STA 53+840

Number	Discont-inuity Type	Strike	Dip		Width	Filling	Amount	Surface Shape	Rough-ness	Spacing	Est. RQD	Strength (see p. B-54)
13	J	67	81	S	N	Fe	Su	Pl	SR	W - IRR	90-100	Strong to Extremely Strong
14	J	321	88	N	N	Fe	Su	Pl	SR	W - IRR	90-100	Strong to Extremely Strong



SITE 3, STA 53+940

Number	Discont -inuity Type	Strike	Dip	Width	Filling	Amount	Surface Shape	Rough-ness	Spacing	Est. RQD	Strength (see p. B-54)
15	J	313	81 S	N	Fe	Su	Pl	SR	W - IRR	90-100	Strong to Extremely Strong



SITE 3, STA 54+180

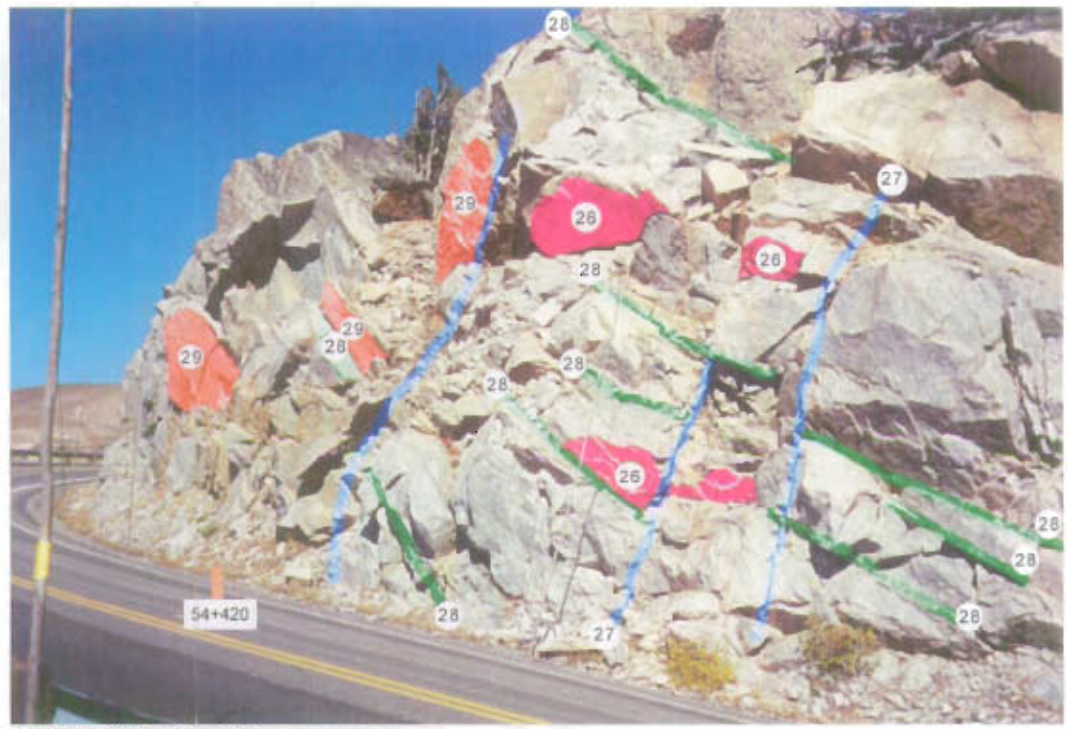
Number	Discont -inuity Type	Strike	Dip	Width	Filling	Amount	Surface Shape	Rough-ness	Spacing	Est. RQD	Strength (see p. B-54)
16	J	327	76 N	N	Fe	Su	Pl	SR	W - M	90-100	Strong to Extremely Strong
17	J	66	87 S	N	Fe	Su	Pl	SR	W - M	90-100	Strong to Extremely Strong

SITE 3, STA 54+320

Number	Discont -inuity Type	Strike	Dip		Width	Filling	Amount	Surface Shape	Rough- ness	Spacing	Est. RQD	Strength (see p. B-54)
18	J	326	45	N	N	---	---	PI	SR	W	90-100	Strong to Extremely Strong
19	J	52	86	N	N	Fe	Su	PI	SR	M	90-100	Strong to Extremely Strong

SITE 3, STA 54+380

Number	Discont -inuity Type	Strike	Dip		Width	Filling	Amount	Surface Shape	Rough- ness	Spacing	Est. RQD	Strength (see p. B-54)
20	J	211	42	N	MW	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong
21	J	70	88	N	N	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong
22	J	352	74	N	N	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong
23	J	73	49	S	MW	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong
24	J	62	70	S	N	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong
25	J	312	74	N	N	Fe	Su	Pl	SR	W - IRR	90-100	Strong to Extremely Strong

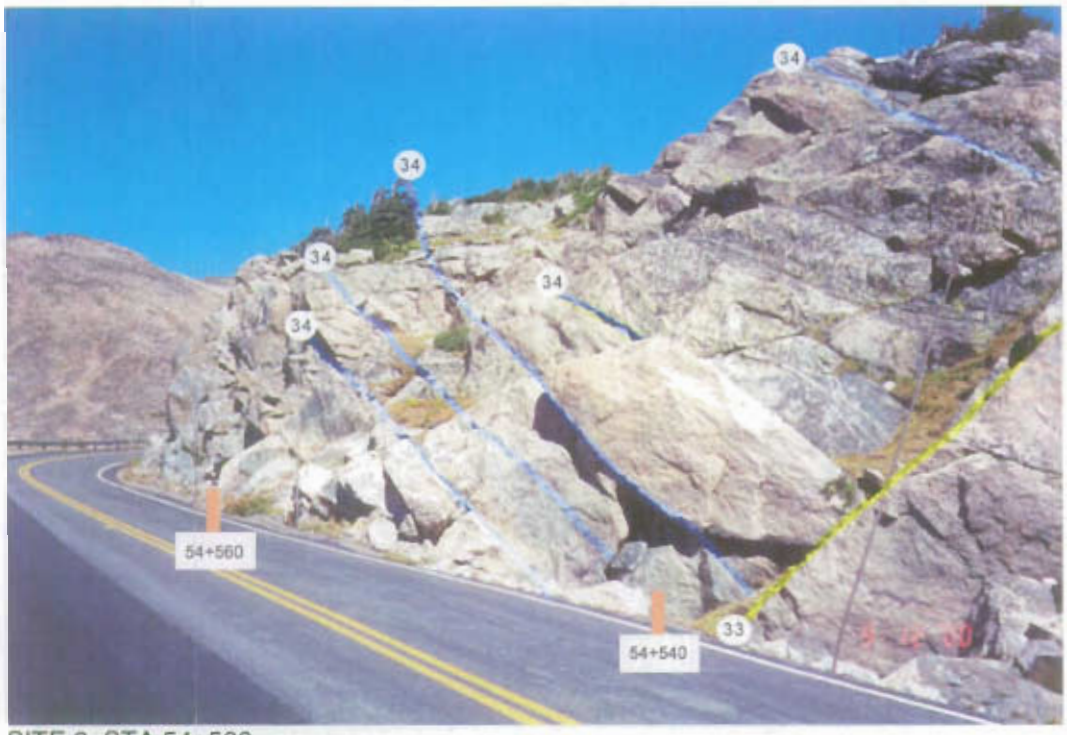


SITE 3, STA 54+420

Number	Discont -inuity Type	Strike	Dip	Width	Filling	Amount	Surface Shape	Rough- ness	Spacing	Est. RQD	Strength (see p. B-54)
26	J	128	63 N	N	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong
27	J	17	75 W	N	Fe	Su	Pl	SR	W	90-100	Strong to Extremely Strong
28	J	15	47 S	MW	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong
29	J	136	78 N	N	Fe	Su	Pl	SR	W	90-100	Strong to Extremely Strong

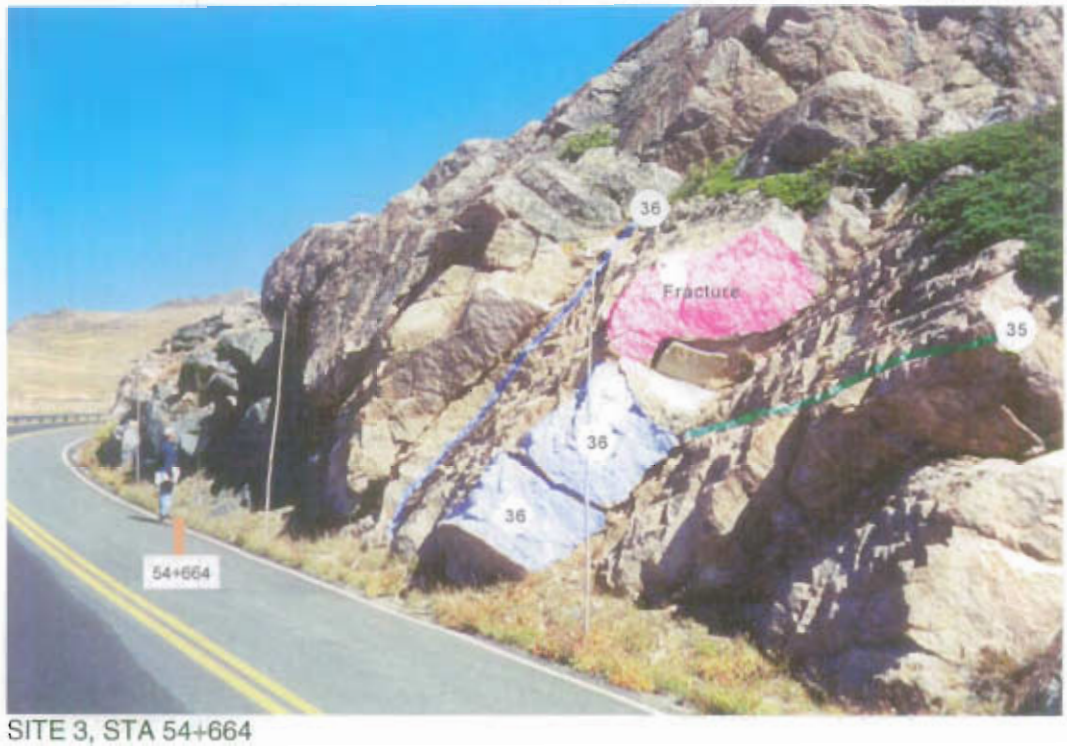
SITE 3, STA 54+460

Number	Discont -inuity Type	Strike	Dip		Width	Filling	Amount	Surface Shape	Rough- ness	Spacing	Est. RQD	Strength (see p. B-54)
30	J	60	39	W	N	Fe	Su	Pl	SR	M - W	90-100	Strong to Extremely Strong
31	J	35	40	N	N	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong
32	J	132	86	N	N	Fe	Su	Pl	SR	M - W	90-100	Strong to Extremely Strong



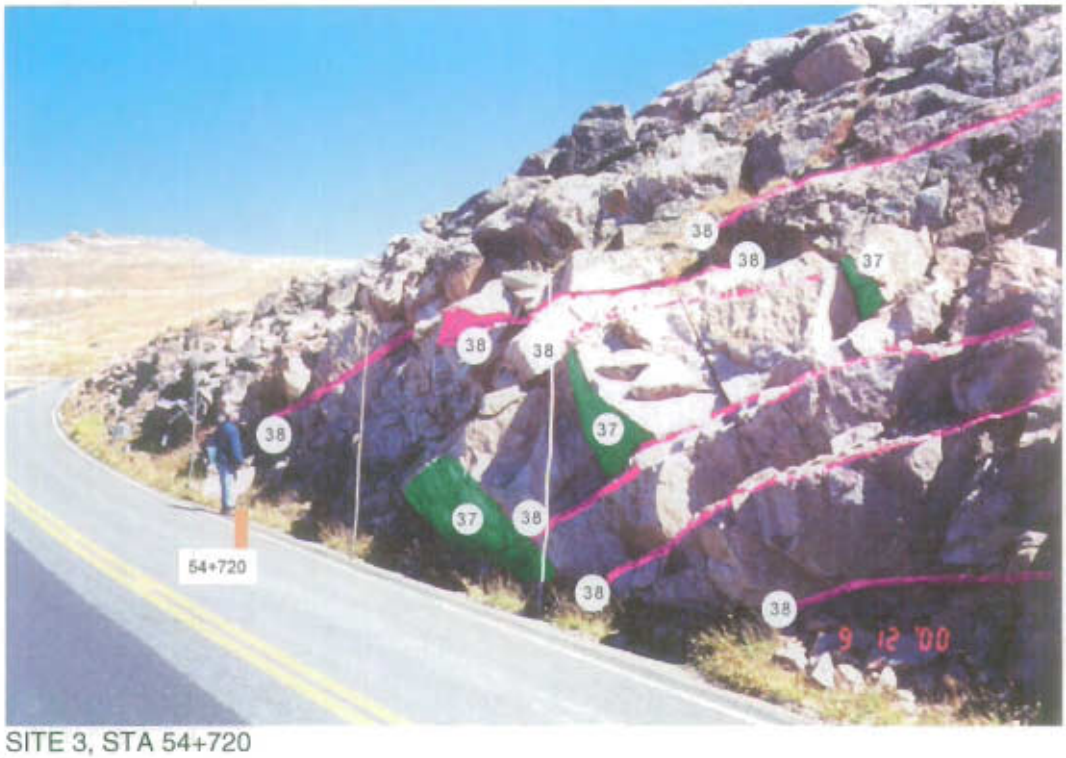
SITE 3, STA 54+560

Number	Discont-inuity Type	Strike	Dip		Width	Filling	Amount	Surface Shape	Rough-ness	Spacing	Est. RQD	Strength (see p. B-54)
33	J	20	43	N	N	---	---	Pl	SR	M	90-100	Strong to Extremely Strong
34	J	70	46	S	N	---	---	Pl	SR	W - IRR	90-100	Strong to Extremely Strong



SITE 3, STA 54+664

Number	Discont-inuity Type	Strike	Dip		Width	Filling	Amount	Surface Shape	Rough-ness	Spacing	Est. RQD	Strength (see p. B-54)
35	J	100	15	W	N	Fe	Su	St/Wa	SR	C	90-100	Strong to Extremely Strong
36	J	154	86	N	N	Fe	Su	Wa	SR	W	90-100	Strong to Extremely Strong



SITE 3, STA 54+720

Number	Discont -inuity Type	Strike	Dip		Width	Filling	Amount	Surface Shape	Rough- ness	Spacing	Est. RQD	Strength (see p. B-54)
37	J	105	32	W	N	---	---	Pl	SR	M	90-100	Strong to Extremely Strong
38	J	39	58	N	N	Fe	Su	Pl	SR	M	90-100	Strong to Extremely Strong

Appendix H

Appendix H
Slope Stability Analysis

Slope stability analyses were performed for Station 39+800 using the computer program UTEXAS3. Analyses were completed for both the existing conditions and a case where the slope below the existing roadway was regraded to a 4:1 (Horizontal:Vertical) slope and a 2 m deep trench drain was installed at the upslope edge of the road. Searches were performed for circular surfaces and factors of safety were evaluated using Spencer's method.

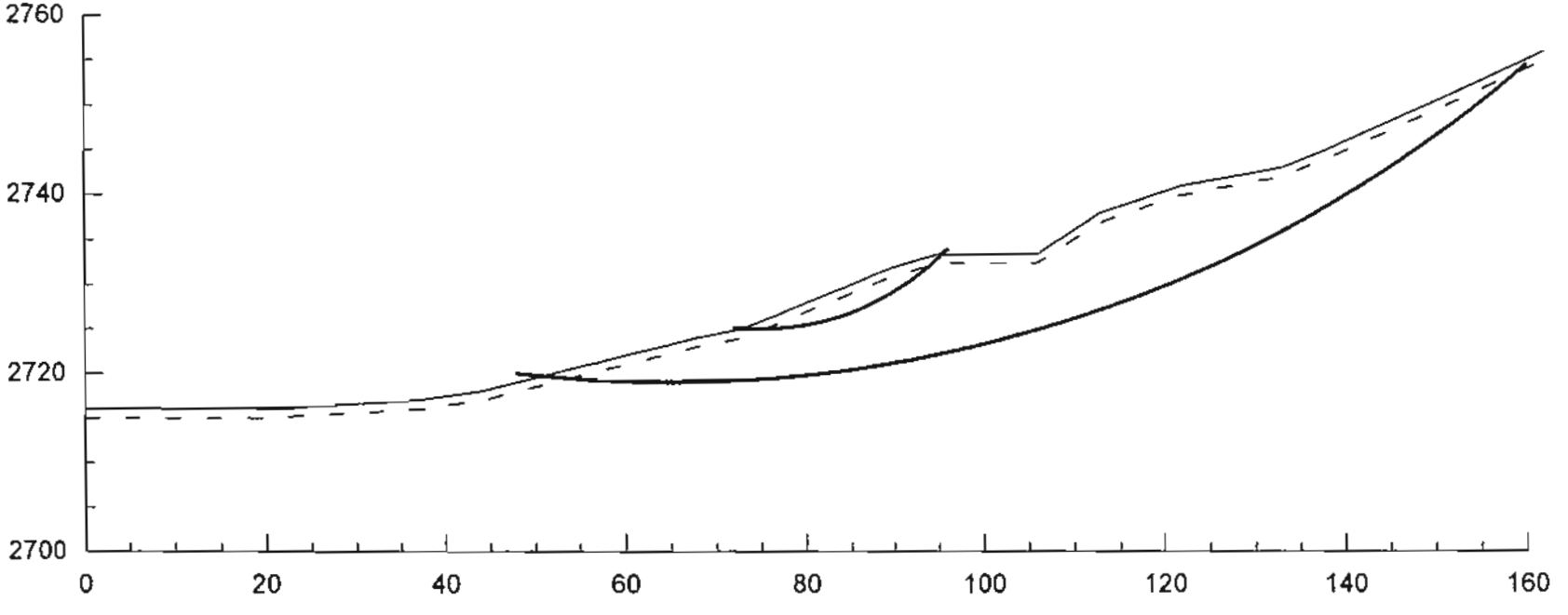
For existing conditions, two phreatic levels were used in the evaluations. The first phreatic level, located 1 m below the existing ground surface, resulted in an estimated lowest factor of safety of 1.05. This factor of safety corresponds to both the smaller, local failure surface below the existing roadway and the deeper, global failure surface shown on Figure H-1. Drained material properties were used in the analysis, as shown in the table on Figure H-1. For a slightly lower phreatic level (located 2 m below the existing ground surface), the estimated lowest factor of safety for both surfaces shown is 1.25.

For the regraded slope with the trench drain, the estimated factors of safety for a surface extending below the existing roadway is 1.4 (1.36). The estimated lowest factor of safety for a deeper, global surface that extends above the roadway is 1.2 (1.22).

Material	c' (kPa)	ϕ' (°)	Watertable[1]	F.S.[2]
Lean Sandy Clay	0	28	1.0 m (shown)	1.05
	0	28	2.0 m	1.25

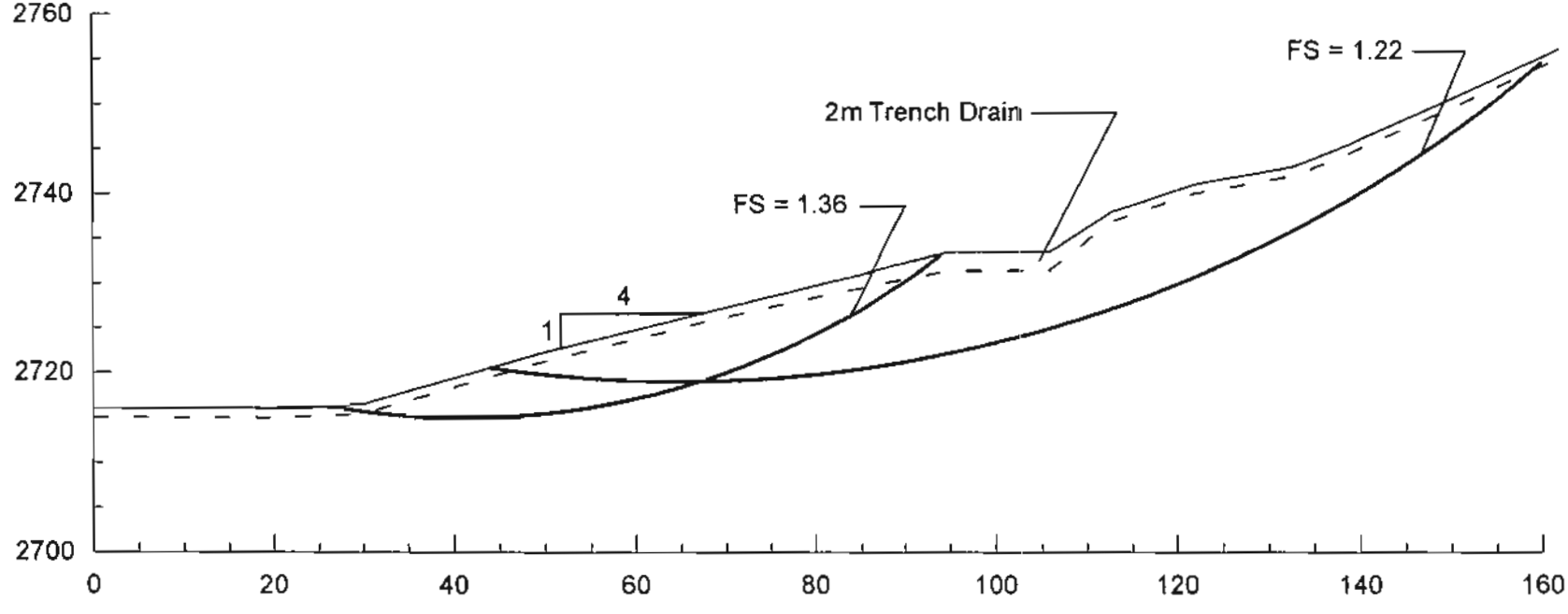
[1] values are meters below ground surface

[2] Safety factors are for both surfaces



Project No. 68FHAT0027	FH-4-1(0) Beartooth Highway	Stability Evaluation Existing Conditions Station 39+800	Figure H-1
	URS		

BTH-fig.GRPwith beartooth_geom.xls / 01/15/01 / jhr



Project No. 68FHAT0027	FH-4-1(0) Beartooth Highway	Stability Evaluation Regrade to 4H:1V with 2m trench drain	Figure H-2
URS			

BTH-fig2.GRFwith bth_regrade.xls / 01/15/01 / jlw