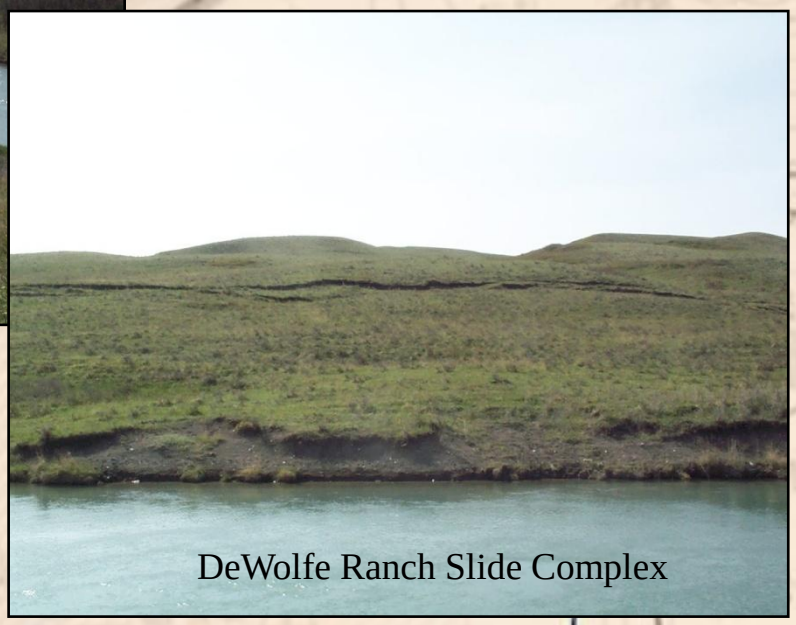
PRELIMINARY GEOTECHNICAL REPORT
FOR CANAL BANK INSTABILITIES
ST. MARY RIVER DIVERSION AND CONVEYANCE FACILITIES

AUGUST 2008 Final

"Lifeline of the Hi-line"

Drop No. 2 Slide

Big Cut Slides



DeWolfe Ranch Slide Complex

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- Instructions On Preparation of USBR Landslide Register (7 Pages)
- Logs of Exploration Test Pits (8 Pages)
- Laboratory Test Data (12 Pages)
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- Classification of Soils for Engineering Purposes

PRELIMINARY GEOTECHNICAL REPORT
CANAL BANK INSTABILITIES
ST. MARY RIVER DIVERSION AND CONVEYANCE FACILITIES

1.0 EXECUTIVE SUMMARY

Eventual rehabilitation of the St. Mary River Diversion and Conveyance Facilities will most likely include realignment or reconstruction of nearly 26 miles of earthen canal. Slope movements and instabilities have been a regular occurrence since original construction in the early 1910's. In 2003, the United States Bureau of Reclamation (USBR) estimated that overall rehabilitation costs could vary from $72 to $98 million depending on the capacity of the rehabilitated canal and related structures. Of that, the USBR estimated $21 million would be necessary to stabilize portions of the one-bank, contour canal during these rehabilitation efforts. The bank stabilization portion represents 21 to 29% of the overall costs and was based on several design and landslide assumptions. This study was initiated by the Montana Department of Natural Resources and Conservation (DNRC) and was intended to address several of the unknowns and provide a basis for future studies.

The USBR maintains a Landslide Register for all landslide and embankment instabilities identified as either currently impacting or those that have the potential to impact their projects. As of November 2007, there were 16 landslides listed for the St. Mary River Diversion Project. Only one slide was listed in the Landslide Register prior to 1995 although at least 14 unstable areas were reportedly monitored by USBR staff prior to that time. Periods of elevated precipitation in 1995 and 2004 triggered many new and existing slides in marginally-stable areas.

Primary factors contributing to the slope instabilities include low soil shear strengths, elevated soil moisture contents, hydrostatic/artesian forces, surcharge loads, slope geometry and toe erosion. The original typical section reportedly consisted of 1½:1 (H:V) backslopes and 2:1 fill slopes (USBR, 2003) and was most likely dictated to minimize the magnitude of excavation that was performed by the horse-drawn fresnos during original construction. In our opinion, these slope angles are too aggressive for the marginal soil strengths and the destabilizing nature of the canal water and groundwater seepage. Also, the excavation of the canal prism and periodic seepage flows related to

elevated precipitation is the primary triggering mechanism for the majority of the landslides and stabilities comprising the canal backslopes.

Based on our past landslide experience and our observations for this project, most of the backslope instabilities can be mitigated during future canal rehabilitation. Flatter fill and backslopes consistent with the native soils and current Standards will increase overall stability. Slope rounding over steep backslopes comprised predominantly of stable bedrock will facilitate gentler, overall slopes and reduce the magnitude of excavation in marginal soils. Canal lining in seepage-sensitive fill sections should be considered to enhance the long-term stability of these downslope embankments. A gravel, gravity toe buttress can be employed to enhance toe stability and increase resisting forces where existing subsurface conditions warrant its use. Many of these potential enhancements can be further improved if the alignment and grade can be raised and shifted over onto the subject hillsides and backslopes above the existing canal.

A Preliminary Grade and Alignment (PGA) study needs to be performed in order to determine the magnitude of lateral shift and elevation gain that could be obtained between Spider Lake and Big Cut. This would alleviate the majority of the instabilities currently identified along the existing St. Mary River Diversion and Conveyance Facilities. Once a new alignment has been proposed, a detailed geotechnical investigation program is warranted to address these subsurface unknowns.

2.0 INTRODUCTION

2.1 Background

Future rehabilitation of the St. Mary River Diversion and Conveyance Facilities will likely include realignment or reconstruction of nearly 26 miles of earthen canal. Original canal construction reportedly spanned from 1912 to 1915 and was completed predominantly using horse-drawn fresnos. Larger excavations were accomplished using steam shovels. Several canal sections have since been reconstructed or realigned due to operational issues or geotechnical failures. Slope movements are presently on-going in both the natural soils within the backslopes and those comprising the embankment fill sections.

In 2001, the United States Bureau of Reclamation (USBR) prepared an appraisal-level study to rehabilitate the St. Mary River Facilities. Additional studies began that same year to further refine alternatives and to establish costs for a rehabilitated facility having various capacities ranging from 500 to 1,000 cfs. The USBR's estimate projected costs which ranged from $72 to $98 million (USBR, 2003) depending on final capacity. The USBR considers the cost estimating work to be feasibly level. A significant portion of the rehabilitation costs, $21 million, was estimated to address canal bank stabilization. This value represented 21 to 29% of the overall rehabilitation costs and is independent of the rehabilitated capacity. This cost estimate was based on a generalized understanding of the geology, assumed quantities, and the assumption that the rehabilitated canal would basically follow the existing alignment and grade.

2.2 Purpose and Scope

The cost of slope remediation represents a significant portion of the overall rehabilitation costs and comprises many unknowns and assumptions. The Montana Department of Natural Resources and Conservation (DNRC) initiated this preliminary study to summarize the scope of the problem and to provide possible recommendations. The scope of work for this study included the following:

- coordinate with USBR and Blackfeet officials to obtain the necessary environmental clearances to conduct a preliminary subsurface soils investigation;
- conduct a site visit with DNRC and USBR staff in order to discuss and to delineate the previously identified instabilities and areas for potential instability;
- perform follow-up topographical surveying of each slide that was field delineated;
- conduct a preliminary subsurface soils investigations within several slides; and
- prepare a short summary report of field observations, findings and recommendations.

3.0 CANAL BANK INSTABILITIES

3.1 Introduction

Numerous landslides, slope movements and failures have occurred and are on-going in both the natural soils comprising the backslopes above the canal prism and within the fill embankment defining of the downslope bank. The original construction details reportedly consists of 1½:1 (H:V) cut slopes and 2:1 fill slopes. These slopes along with inherent seepage and imposed seasonal phreatic surfaces are often too excessive for the native, fine-grained soils that predominantly comprise the slopes and fill sections. Also, original early 19th century construction techniques, their inherent limitations, and inadequate surface preparation are most likely the main contributing factors of the fill embankment instabilities.

Regionally, the natural slopes and subsurface soil and groundwater conditions exhibit an elevated propensity for slope failure. Evidence of naturally-occurring slope movements is common and abundant. In fact, the original canal alignment and construction was reportedly initiated along the east side of the St. Mary River but had to be abandoned and relocated to its present alignment due to slope failures triggered by the canal excavations. As a result, the current western alignment necessitated an inverted siphon structure in order to cross the St. Mary River near the mouth the of Spider Lake drainage.

3.2 Regional Geologic Setting and Physiography

As stated in previous geologic reports prepared by the USBR (2003), the general geology of the area comprising diversion and conveyance facilities varies considerably. At the beginning of the project, from the diversion dam to approximately the Powell Creek underdrain culvert (Figure 2), the geologic setting is best described as predominantly stream alluvium and alluvial fan deposits comprised of coarse gravels and cobbles. These granular soils are relatively thick and overlie Cretaceous-aged sedimentary bedrock. The USBR reports the bedrock unit to be a non-marine mudstone (Two Medicine Formation) near the diversion dam.

Along the canal and downstream from the Powell Creek culvert, the earthen canal traverses increasing occurrences of fine-grained, glacial till and drift in varying thicknesses overlying earlier alluvial deposits and/or sedimentary bedrock.

Near the St. Mary River siphon, the local geology varies from the glacial till and/or alluvium underlain by the Cretaceous-aged, Virgelle Sandstone Formation. The glacial deposits mostly blanket the valley sidewalls and consist generally of high-plasticity, lean clay with sand and with lesser occurrences of silty sand with gravel. The alluvium closer to the river can be described as cleaner material, typically composed of coarse gravel with sand, cobbles and boulders. The Virgelle Sandstone is yellowish-brown to gray, fine to medium-grained, and slightly to moderately fractured. It is described as soft to moderately hard depending on the degree of weathering. The USBR report further states that in this area, the sandstone stikes north-northwest, dips about 25 degrees toward the northeast, and has been thrust over the adjacent and younger Cretaceous-aged Two Medicine Formation.

Downstream from the St. Mary River siphon, along the canal, the physiographic terrain changes to predominately glacial till of varying thickness underlain by Cretaceous and Tertiary-aged sedimentary bedrock. Spider Lake drainage represents a hanging valley as it joins the St. Mary River drainage. Alluvial and fluvial deposits are less common and most likely limited to drainage lowlands. Occasional occurrences of terrace and outwash gravel deposits are known. Bedrock outcrops are observed along the hillsides and occasionally in the bottom of the canal when dewatered. In general, the stratagraphic sequence is older to younger from west to east. High angle normal faults and reverse faults are likely and are associated with the folding, faulting and thrusting of the Rocky Mountains.

A generalized geologic map of the area is presented in Figure 1 below.

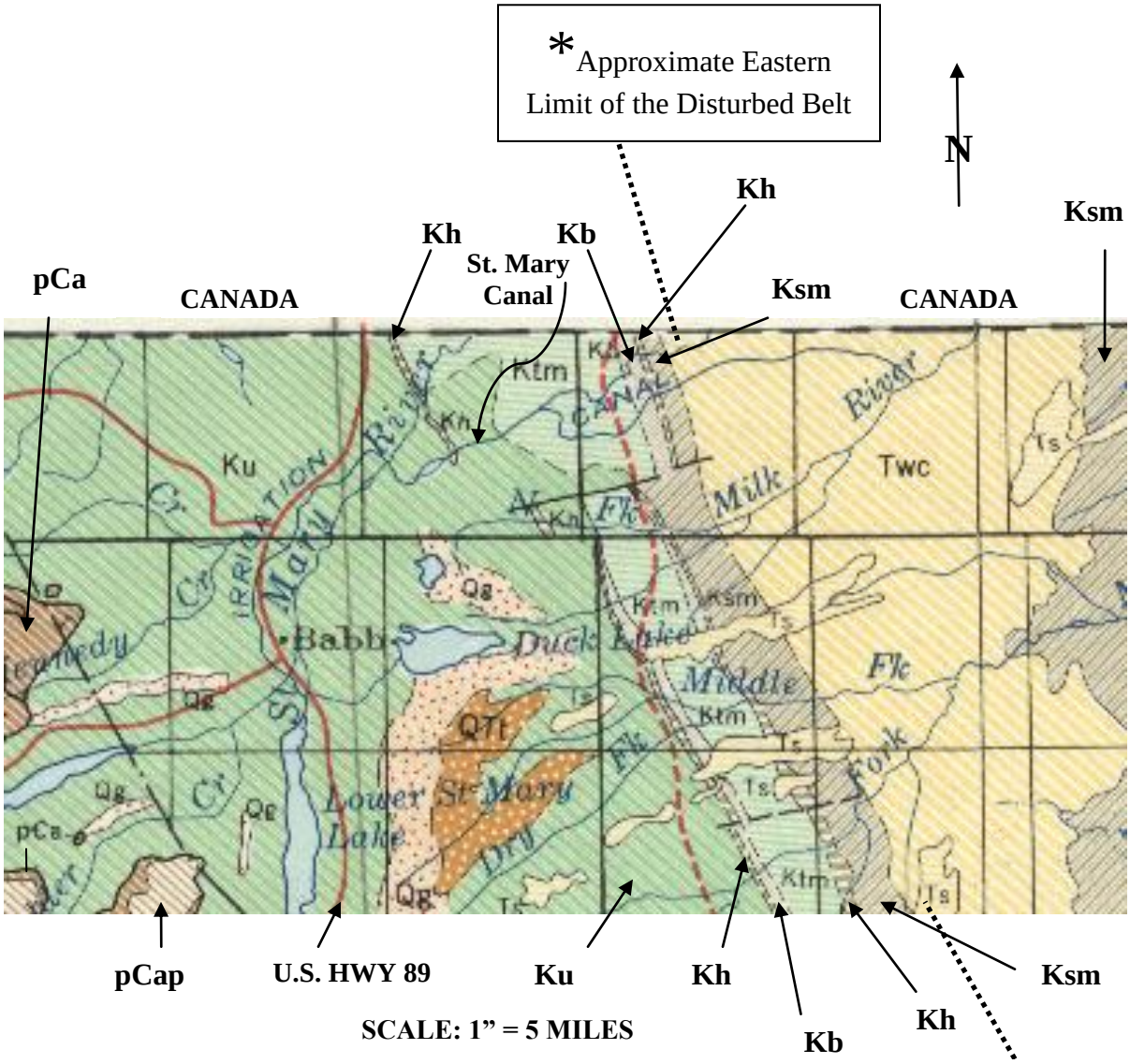


Figure 1. Geologic map of the area comprising the St. Mary River Diversion and Conveyance Facilities (from USGS Geol. Map of MT MR2235, 1955). *

Q_g - Pleistocene Glacial Drift
Morainal and outwash plain deposits of mountain glaciers; mainly ill-sorted and poorly rounded boulders, cobbles, pebbles and sand; may include alluvium in places.

Q_{Tf} – Terrace Deposits
Gravel, sand, and silt of terrace remnants

T_s - Tertiary Sedimentary Rocks Undifferentiated
Poorly consolidated gravel, sand, silt, clay and other - tuff, lignite, bentonite, deposits, in valleys.

T_{wc} - Tertiary (Paleocene) Willow Creek Formation
Variegated clay and soft sandstone, chiefly maroon to chocolate brown; local lenses of purple-gray nodular limestone.

K_{sm} - Upper Cretaceous St. Mary River Formation
Greenish-gray clay with local nodular limestone and cross-bedded sandstone.

K_h - Upper Cretaceous Horsethief Sandstone
Shaley sandstone grading upward into massive brownish cliff-forming sandstone with local concentrations of magnetite in beds near top.

K_b - Upper Cretaceous Bearpaw Shale
Dark gray and brownish clay shale; thick units of non-fissile bentonitic shale; contains some thick bentonite beds.

K_{tm} - Upper Cretaceous Two Medicine Formation
Greenish-gray clay with local nodular limestone and cross-bedded sandstone; locally some coal in lower part.

K_u - Cretaceous Undifferentiated
(disturbed belt – subdivision difficult – contains
K_{vi} Virgelle formation Gray to buff massive cliff forming sandstone with iron-stained concretions in the upper part)

pC_{ap} – Appekunny Argillite
Gray quartzitic argillite and quartzite

pC_a – Altyn Limestone
Dolomite and magnesian limestone

Fault observed _____ inferred --------

3.3 USBR Landslide Register

The USBR maintains a Landslide Register for all landslide and embankment instabilities identified as either currently impacting or those that could impact USBR projects. For the St. Mary River Diversion and Conveyance Facilities, only one slide was listed in the Register prior to 1995. This slide, known as the St. Mary Canal Slide, was a long area extending from approximately Sta. 650+00 to 800+00. No individual or specific slides were delineated. However, USBR staff reports that up to 14 unstable areas were monitored prior to 1995. This general reach was historically known to have a high propensity for instabilities and slope movements. In 1995, this area was replaced with discrete, individual slides identified by either local landmarks or approximate canal stationing. Since then, several other stabilities have been identified and added to the Register. However, to date, no slide for the St. Mary River Diversion Project has ever been removed from the Register.

In 1995, heavy precipitation triggered many of the former slides and several new ones. The single slide referenced in the Register was replaced with nine individual slides. In 1996, two new slides were added including the St. Mary River siphon crossing instabilities. An additional slide was included in the Register in 1997. In 2002, three more slides were added. The latest slide was added in 2005.

Presently, the USBR has identified and annually monitors at least 16 areas of instability that either currently impact or have historically had the potential to impact the facilities. USBR geologists conduct annual inspections to observe the known slides. In the last three to four years, little significant landslide activity has been observed although periodic maintenance has been required to maintain canal capacity in many of these areas. The impact of the existing instabilities on a rehabilitated canal section is the greatest monetary unknown influencing realignment issues, treatment options and construction uncertainties.

Significant movements of the natural backslopes would tend to close off the canal prism thereby reducing capacity and leading to over-topping and a possible progressive canal breach. Instability and subsequent movement of embankment fill sections leads to increased seepage, additional settlement

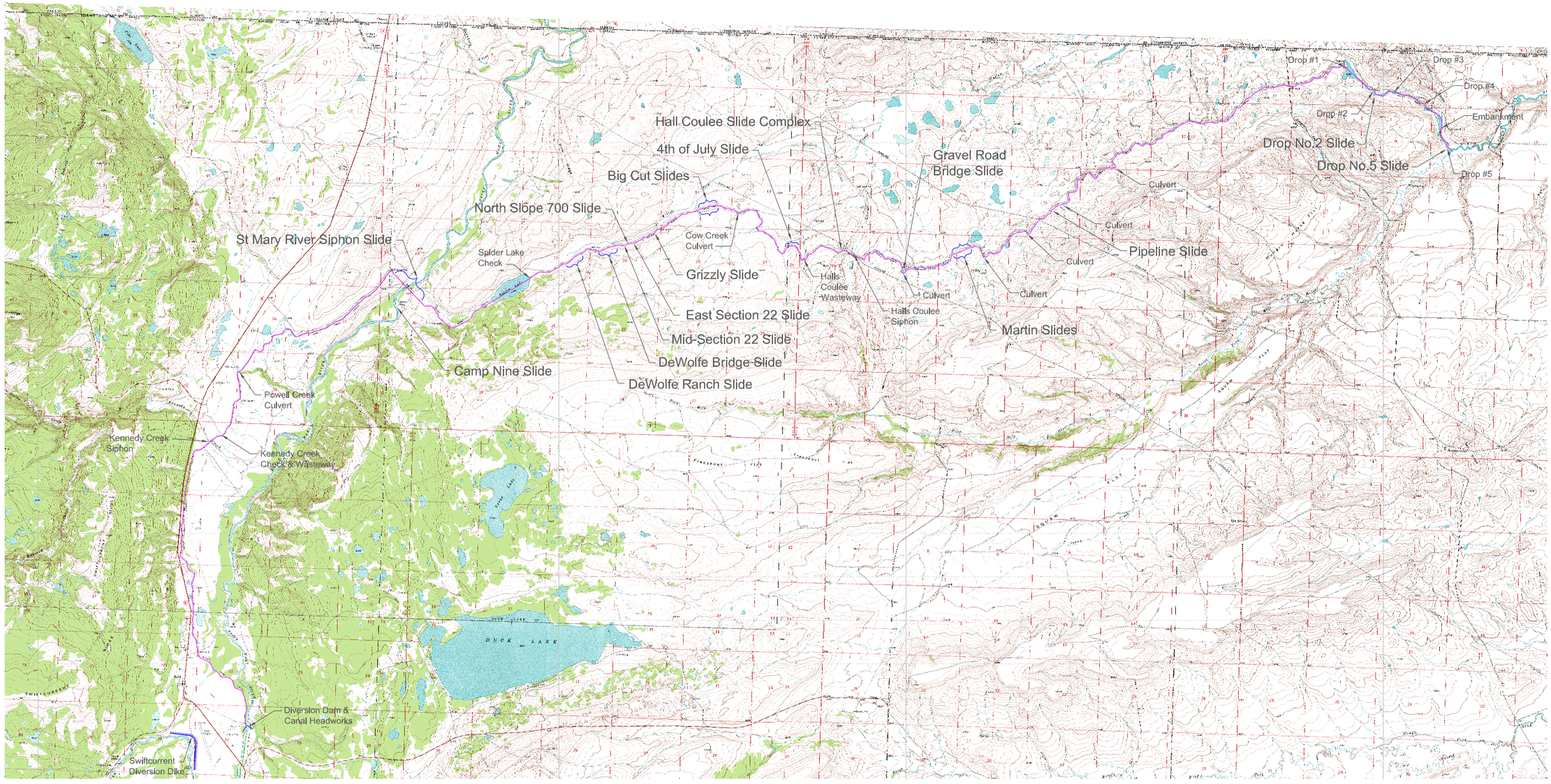
and movement, and the potential for catastrophic bank failure. As of November 2007, the known instabilities listed in the USBR Landslide Register and their locations are listed in Table 1 below. Their locations are also shown on Figure 2.

Table 1 - Summary of Slope Instabilities Identified by USBR

Slide Name (USBR)	Approximate Station[(1)]	Location
Camp Nine Slide	N/A	Natural Slope
St. Mary River Siphon	486+00 to 518+00	Natural Slope
DeWolfe Ranch Slide	646+50 to 659+50	Backslope
DeWolfe Bridge Slide	672+00 to 688+20	Backslope
Mid-Section 22 Slide	693+50	Backslope
North Slope 700 Slide	698+00	Embankment Fill
East Section 22 Slide	703+00	Backslope
Grizzly Slide	730+00 to 733+50	Backslope
Big (Deep) Cut Slides	759+00 to 773+00	Cut Slopes, Both Sides
4th of July Slide	852+00 to 865+00	Embankment Fill
Hall Coulee Slide Complex	911+00 to 914+00	Backslope
Gravel Road Bridge Slide	975+00 to 981+00	Embankment Fill
Martin Slides	1022+00 to 1034+00	Cut Slopes, Both Sides
Pipeline Slide	1121+50 to 1125+00	Backslope
Drop No. 2 Slide	1453+50	Reservoir Slope
Drop No. 5 Slide	1523+50	Backslope

Note: [(1)] Based on TD&H Engineering topo survey completed 2007.

The downslope embankment failures are most likely related to inadequate ground surface preparation prior to fill placement, insufficient compaction of embankment material and excessive fill slope angles. These inherent deficiencies are compounded by seasonal seepage from the canal. Backslope failures are most likely related to the natural soil comprising the cut section and soil shear strength softening that occurs due to saturation. During wet years, increased soil moistures and groundwater flows increase instabilities and movement. Removing the sloughed material, whether by planned maintenance or internal canal erosion, increases instability as does rapid drawdown conditions experienced within the canal prism during unplanned or emergency dewatering.



N

N.T.S.

FIGURE 2

REVISIONS

BY	DATE	DESCR
BY	DATE	DESCR
BY	DATE	DESCR

TD&H

THOMAS, DEAN & HOSKINS, INC.
ENGINEERING CONSULTANTS
GREAT FALLS–BOZEMAN–KALISPELL
SPOKANE
LEWISTON
MONTANA
WASHINGTON
IDAHO

DRAWN BY:	WAB
DESIGNED BY:	EAJ
QUALITY CHECK:	
DATE:	01.10.08
JOB NO.	04-167
FIELDBOOK	

DNRC - CARDD
ST. MARY DIVERSION FACILITIES

LANDSLIDE LOCATION MAP

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The USBR Landslide Register contains additional information about each listed slide. The information is qualitative and semi-quantative and is updated annually as needed by the visiting USBR geologist. Table 2 below contains some of the pertinent data from the Landslide Register for each of the listed slides. Explanations and descriptions of the categories and data codes are provided in the Appendix. Each listed slide is discussed in further detail below.

Table 2 - Summary Of Pertinent Slide Data From The 2007 USBR Landslide Register

Slide Area ID	Active or Inactive	First Movement	Last Movement	Failure Speed	Volume (1000's yd3)	Process	Rock Type	Collapse State	Measured	Primary USBR Structure Risk	Secondary USBR Structure Risk	Other Risk Property	Risk to Like	Overall Risk	Treatment Performed	Additional Treatment Required?
Camp Nine	A	61095	CREEPS	S	25	4	SQ	7	N	--	HIH	--	NIL	HIH	Y	*
St. Mary River Siphon	A	******	CREEPS	S	--	4	SQ	0	N	HIH	HIH	LOW	LOW	HIH	Y	Y
DeWolfe Ranch	A	******	CREEPS	S	300	4	SQ	1	N	HIH	--	--	NIL	HIH	Y	*
DeWolfe Bridge	A	******	CREEPS	S	110	4	SQ	1	N	MOD	--	--	NIL	MOD	Y	*
Mid Sec 22	A	06**95	CREEPS	S	30	4	SQ	1	N	MOD	--	--	NIL	MOD	Y	Y
North Slope 700	A	***05	CREEPS	S	0.3	13	F	2	N	HIH	HIH	LOW	LOW	HIH	N	Y
East Sec 22	A	***95	CREEPS	S	1	4	SQ	1	N	MOD	--	--	NIL	MOD	Y	*
Grizzly	A	06**02	CREEPS	M	0.5	16	F	3	N	MOD	--	--	LOW	MOD	N	*
Big Cut	A	******	CREEPS	S	250	4	SQ	0	N	NIH	--	--	NIL	HIH	Y	Y
4th of July	A	******	CREEPS	S	80	4	SQ	4	N	HIH	MOD	--	NIL	HIH	Y	*
Hall Coulee	A	******	CREEPS	S	200	4	SQ	2	N	HIH	--	MOD	NIL	HIH	N	Y
Gravel Road Bridge	A	******	CREEPS	S	0.1	16	F	8	N	MOD	MOD	--	NIL	MOD	Y	Y
Martin	A	******	CREEPS	S	250	4	SQ	1	N	HIH	--	--	NIL	HIH	Y	*
Pipeline	A	06**95	CREEPS	S	10	4	SQ	2	N	MOD	--	--	NIL	MOD	Y	*
Drop No. 2	A	****95	06**02	S	1	4	SQ	2	N	MOD	--	--	NIL	MOD	N	*
Drop No. 5	A	06**02	CREEPS	M	1	16	SQ	0	N	MOD	--	--	NIL	MOD	N	*

See Appendix for explanations and definitions of USBR categories and data codes.

3.3.1 Camp Nine Slide. This slide is located along the west side of the USBR's Camp Nine maintenance facility (Figure 2). The slide first occurred June 10, 1995 as a result of high runoff flows in the St. Mary River. Erosion and under-cutting of alluvial soils initiated the slope failure. The erosion is caused by a sharp bend in the river created by an abrupt outcrop of bedrock slightly downstream. Slope failure resulted in the loss of a portion of the perimeter security fencing and storage area. The slump was reshaped and toe slope protection consisting of riprap jetties was added in 1998. In general, the stabilization efforts implemented by the USBR appear to be diverting and attenuating the erosive nature of the river. Immediately upstream of the Camp Nine Slide, on-going slump and slide movements are occurring due to similar fluvial erosive forces and their effects. A summary of the annual observations made by the USBR geologists since 1999 is provided below.

- 1999, August - No additional movement was noted. Some repair riprap has been moved by the St. Mary River.
- 2000, July - Only minor erosion is presently occurring at the site.
- 2001, July - No additional movement was noted this year. Some of the riprap repair was previously moved by high river flows, but no additional riprap appears to have moved this year.
- 2002, June - There is no perceptible change in the appearance of the upstream portion of the slide. Portions of the riprap barriers remain, but most have been displaced by the river flows.
- 2003, June - Areas of concern display no perceptible change.
- 2004, July - At this time, both areas of concern display no perceptible change.
- 2005, July - Evidence of continued soil creep upstream of the subject slide is present as periodic, fresh scarp propagation, though this area does not appear to have moved significantly since last year's inspection.
- 2006, July - Scarps are visible throughout the slump area upstream of the subject slide. The unstable area does not appear to have moved significantly from previous inspections.

The Camp Nine Slide does not have a direct impact on the ability to divert and deliver St. Mary River water to the Milk River Basin. The slide is essentially the result of naturally-occurring erosion caused by the St. Mary River. The continual loss of ground will only have an impact on the maintenance camp. In our opinion, USBR's approach to attenuate or control the river erosion by use

of slope armoring and in-stream diversion is the correct approach. Instability of the slope may be exacerbated by face seepage during wet seasons. By regrading the slope in 1995, the USBR has reduced the destabilizing effects of this occurrence.

3.3.2 St. Mary River Siphon Slides. The St. Mary River Siphon Slides (Figure 2) were added to the register in 1996 following significant siphon and ground movements. However, both the north and south hillsides traversed by the double pipe siphon structure have a long history of movements dating back to the late 1910's. Significant precipitation events in 1995 most likely led to the problems realized in early 1996. On the southside, reported movements of between 3 and 6 inches resulted in pipe buckling near the bottom and separation of the pipe on the hillside. The instabilities of the St. Mary River Siphon Slide are being currently investigated and studied by TD&H Engineering (TD&H, 2008) under contract with DNRC. The annual observations by the USBR geology staff during their inspections are summarized below starting with the 1999 report.

- 1999, August - No unusual movement was noted this year, but continued leakage occurs around the slip joints.
- 2000, July - Leakage from the north hillside does not appear to be as bad as during recent past inspections. On the south hillside, leakage seems to be the same as the past inspections. Three piezometers installed on the south hillside in 1999 showed a sharp drop in water levels very soon after seasonal canal dewatering indicating a strong correlation and contribution to the observed instabilities.
- 2001, July - Leaking continues to occur from the slip joints; particularly on the south hillside. In some areas, leakage is causing erosion of siphon supports.
- 2002, June - Many of the joints continue to leak. Some remediation work has been performed on the siphon.
- 2003, June - Joints continue to leak.
- 2004, July - Pipes continue to seep. Some maintenance work and minor construction to repair portions of the siphon have been completed.
- 2005, July - Not mentioned in 2005 Inspection Report.
- 2006, July - Recent repair work targeted weak or leaking joints.

The subsurface soil and groundwater conditions comprising the north and south hillsides of the St. Mary River siphon crossing are varied and complex. Slope movements on both hillsides have been monitored using slope inclinometers by TD&H Engineering since the Fall of 2005, under contract with the Montana DNRC. The instrumentation monitoring indicates that slope movements are variable and that the velocities fluctuate seasonally. These seasonal variations coincide with rises in soil moisture and groundwater levels and are most likely the combined result of natural factors and canal operations.

Significant ground movements could lead to catastrophic separation or rupture of the siphon pipe. Due to the topography, pipe failure could cause rapid and severe erosion. Failure of one pipe could cause slope erosion, thereby undermining and initiating failure of the second, adjacent pipe. The TD&H/DNRC studies are on-going and an updated progress report will be prepared in December 2008. Preliminary data indicates that movements are seasonal and related to rises in soil moisture and groundwater. Slope movements accelerate and peak in middle summer to early fall and slow and stop during the winter months. Preliminary recommendations and designs include installing a series of passive, horizontal drains on the southslope to alleviate and reduce the seasonal pore pressures acting along the base of the failure plane.

3.3.3 DeWolfe Ranch Slide Complex. The DeWolfe Ranch Slide Complex begins approximately 3,000 feet downstream from the former Spider Lake check structure (Figure 2). This area of instability extends approximately 1,450 feet from Sta. 645+00 to 659+50 as shown on Figure 3. The combined area encompasses nearly 5 acres. This slide was placed on the Landslide Register in 1995 following significant and rapid movements as described by USBR Staff. The slide area reportedly exhibited slow and periodic creep prior to 1995. The annual inspections performed by USBR are summarized as follows:

- 1999, August - There appeared to be little, if any, movement since the last inspection.
- 2000, July - No observed movements since 1999 inspection.
- 2001, July - There appears to be no recent movement and the vegetation appears to be stabilizing the toe and slide scarps.
- 2002, June - There appears to be no recent movement and the vegetation appears to be

stabilizing the landslide surface.

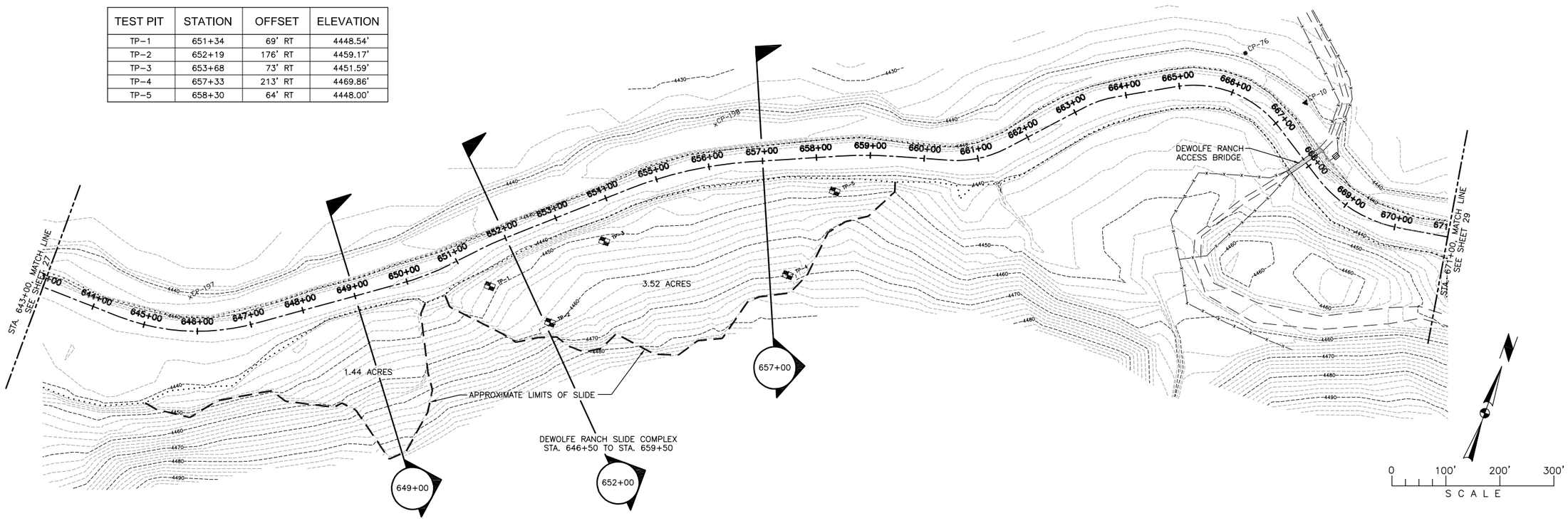
- 2003, June - A portion of the encroaching slide was excavated from the canal by USBR maintenance staff at the end of the 2002 season. No recent movement has been noted.
- 2004, July - No recent movement has been noted since last year.
- 2005, July - It appears this area has moved about 12 inches since the last inspection, marked by a fresh scarp estimated at 12 to 16 inches high.
- 2006, July - Little or no movement was noted from the 2005 inspection.

The middle portion of the eastern section of the DeWolfe Ranch Slide is shown below in Photo 1.

Photo 1. Photo showing the middle portion of the eastern section of the DeWolfe Ranch Slide Complex. Looking south from canal embankment. (06/04/07)

In October 2007, five test pits were excavated within the slide complex to provide a preliminary understanding of the subsurface soil and groundwater conditions. The locations of the test pits are shown on Figure 3. The test pits were excavated by USBR personnel using a Kobelco 914 tracked excavator and logged by TD&H geotechnical staff. Test pits varied in depth from approximately 8 to 19.5 feet below the ground surface.

TEST PIT	STATION	OFFSET	ELEVATION
TP-1	651+34	69' RT	4448.54'
TP-2	652+19	176' RT	4459.17'
TP-3	653+68	73' RT	4451.59'
TP-4	657+33	213' RT	4469.86'
TP-5	658+30	64' RT	4448.00'



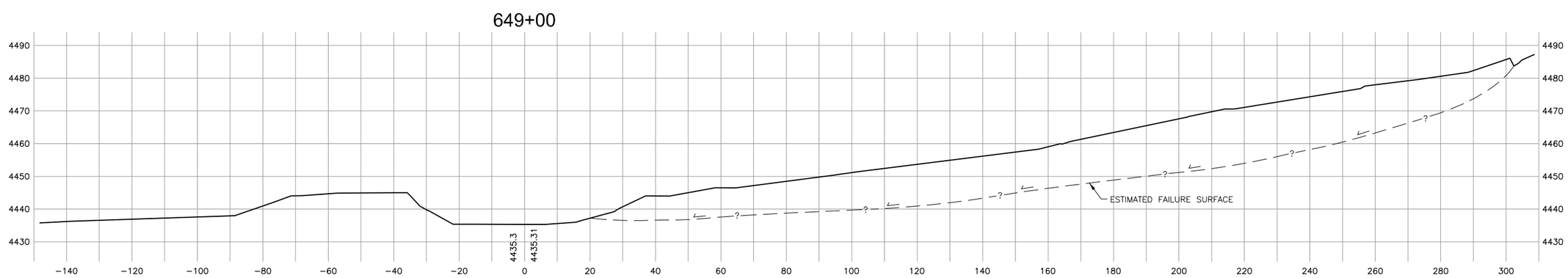


FIGURE 3

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DEWOLFE RANCH SLIDE COMPLEX

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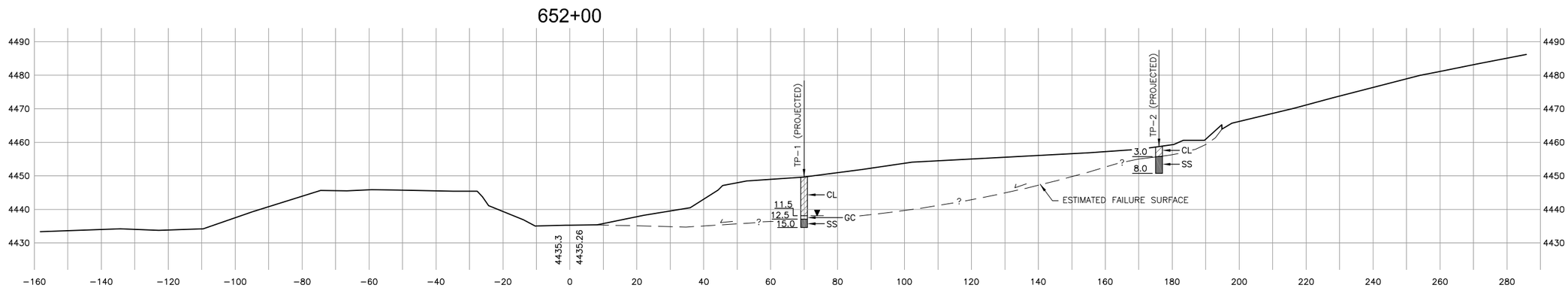

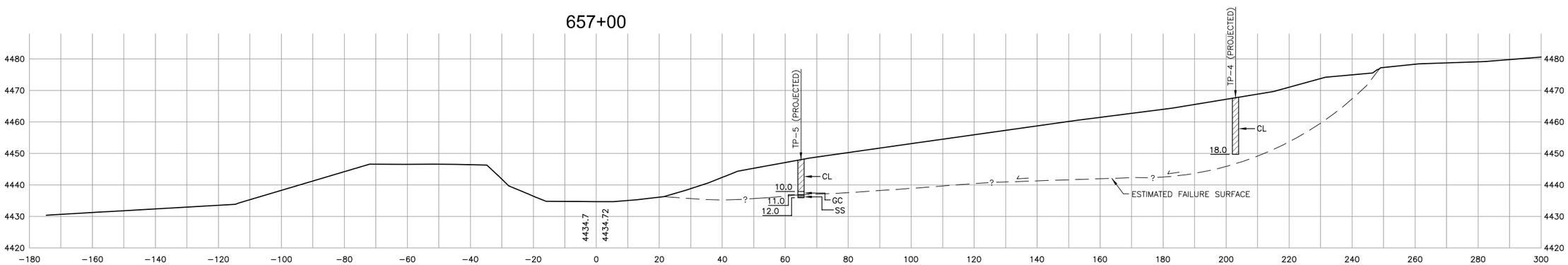

Summary of Subsurface Soil Conditions Within DeWolfe Ranch Slide Complex

Symbol	Soil Layer	TP-1	TP-2	TP-3	TP-4	TP-5
	Topsoil and Organic Layer	0'–0.8'	0'–0.3'	0'–0.8'	0'–0.8'	0'–0.8'
CL	Glacial Till/Plastic Clay	0.8'–11.5'	0.3'–3.0'	0.8'–19.5'	0.8'–18.0'	0.8'–10.0'
GC	Clayey Gravel w/Cobbles	11.5'–12.5'	NE	NE	NE	10.0'–11.0'
SS	Sandstone Bedrock	12.5'–15.0'	3.0'–8.0'	NE	NE	11.0'–12.0'
▼	Groundwater	11.5'	NE	Very Moist @ 19.5'	NE	NE

FIGURE 3

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DEWOLFE RANCH SLIDE COMPLEX

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In general, the subsurface conditions consist of glacial till underlain by sandstone bedrock. The glacial till varies from high-plasticity, lean clay to fat clay. The concentration of sand in the glacial till varied both with depth and laterally. Four samples of the till obtained from the test pits contained between 1 and 12 percent gravel, between 7 and 35 percent sand, and between 53 and 92 percent silt and clay. The glacial till clay exhibited liquid limits of 24 to 62 percent and plastic indexes of 10 to 42 percent. The natural moisture content varied from 15 to 32 percent.

One sample obtained from TP-3 at a depth of 4 to 4.5 feet, classified as clayey sand. This material contained 2 percent gravel, 70 percent sand and 28 percent silt and clay. The fines were plastic as exhibited by a liquid limit of 38 percent and a plastic index of 23 percent. The natural moisture content of the clayey sand was 18 percent.

Sandstone bedrock was encountered at depth in test pits TP-1, TP-2, and TP-5. The sandstone is olive to light greenish-brown and medium-grained. In TP-1, the sandstone was moderately weathered at the upper contact becoming harder and less weathered with depth. In TP-2, the sandstone was encountered very near the ground surface, was harder and less weathered, and exhibited a high degree of fracturing. In TP-1 and TP-5, the contact between the glacial till and underlying bedrock was marked by a coarse, cobbley layer of clayey gravel. This layer is approximately 1-foot thick and contained rounded to subrounded cobbles up to 10 inches in diameter.

Photo 2 below shows the headscarp and failure surface observed while excavating Test Pit TP-2. Failure is occurring atop the intact sandstone bedrock encountered below the slide plane. The headscarp marks the location where the failure plane intersects the ground surface.

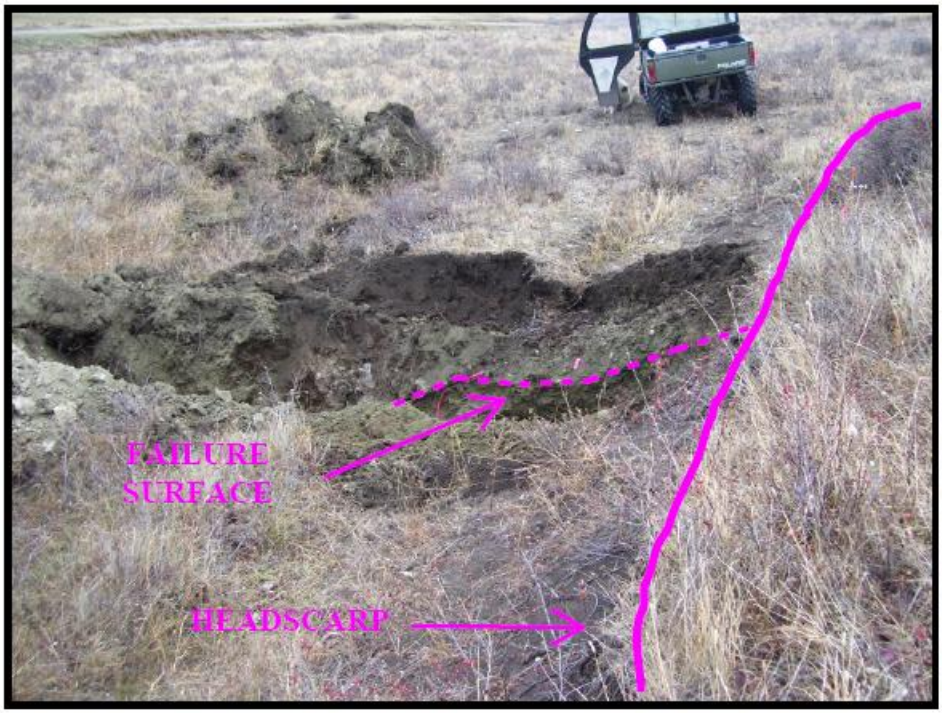


Photo 2. Shows the failure surface and headscarp observed in Test Pit TP-2. (10/23/07)

The subsurface soil conditions encountered within the DeWolfe Ranch Slide Complex are summarized below. More detailed logs of the Test Pits and the supporting lab data are included in the Appendix.

Table 3 - Summary of Subsurface Soil Conditions Within DeWolfe Ranch Slide Complex

Soil Layer	TP-1	TP-2	TP-3	TP-4	TP-5
Topsoil and Organic Layer	0' - 0.8'	0' - 0.3'	0' - 0.8'	0' - 0.8'	0' - 0.8'
Glacial Till/Plastic Clay	0.8' - 11.5'	0.3' - 3'	0.8' - 19.5'	0.8' - 18'	0.8' - 10'
Clayey Gravel w/Cobbles	11.5' - 12.5'	NE	NE	NE	10' - 11'
Sandstone Bedrock	12.5' - 15'	3' - 8'	NE	NE	11' - 12'
Groundwater	11.5'	NE	V. Moist @ 19.5'	NE	NE

NE- Not Encountered

The subsurface soil conditions are projected onto the cross-sections shown on Figure 3. Based on the subsurface soil conditions, it appears that the failure plane coincides with the soil-bedrock contact. The natural undisturbed slopes range from approximately 15 to 30 percent. Parallel to

slope movements, disturbed ground slopes range from 10 to 16 percent. Photo 3 shows encroachment of slide toe into canal prism.

Photo 3. Photo shows slide toe encroachment of the DeWolfe Ranch Slide into canal prism at approximate Sta. 654+00. (01/04/08)

3.3.4 DeWolfe Bridge Slide Complex. The DeWolfe Bridge Slide Complex begins immediately downstream of the DeWolfe Bridge at approximately Sta. 672+50 as shown on Figure 4. This slide was reportedly added to the Register in 1996. The slide complex encompasses two distinct areas, has a combined area of 1.91 acres, and extends to Sta. 688+00. USBR geology staff made the following comments as part of their annual inspection visits.

- 1999, August - Does not appear to have moved much since last year's inspection.
- 2000, July - Does not appear to have moved since the last inspection.
- 2001, July - Does not appear to have moved much since last year's inspection. Toe of the slide adjacent to the canal displays some calving due to erosion. Scarps near the top of the slide are still well-defined, but no discernable movement has occurred.
- 2002, June - The scarps near the top of the slide are still well-defined, but no discernable

movement has occurred.

- 2003, June - Scarps near the top of the slide are still well-defined, but no discernable movement has occurred. Near the upstream end of the slide, the area has been excavated and no movement was detected this year.
- 2004, July - Does not appear to have moved much since last year's inspection.
- 2005, July - It does not appear that movement has occurred since the last inspection.
- 2006, July - The head scarp is prominent, indicative of soil creep though additional movement since the 2005 inspection is not discernible. Grasses and sage continue to grow across the unstable area.

In October 2007, three test pits were excavated within the DeWolfe Bridge Slide Complex to provide a preliminary understanding of the subsurface soil and groundwater conditions. The locations of the test pits are shown on Figure 4. The test pits were excavated by USBR personnel using a Kobelco 914 excavator and logged by TD&H geotechnical staff. Test pits varied in depth from approximately 4 to 9.5 feet below the ground surface.

In general, subsurface conditions consist of glacial till underlain by silty, clayey sand with cobbles and then sandstone. The glacial till varies from high-plasticity, lean clay to fat clay. The concentration of sand in the glacial till varied both with depth and laterally. The glacial till clay soil is comparable to that encountered in the DeWolfe Ranch Slide Complex.

A sample obtained from TP-6 at a depth of 7 to 8 feet, classified as silty, clayey sand with gravel. This material contained 30 percent gravel, 32 percent sand and 38 percent silt and clay. The fines were slightly plastic as exhibited by a liquid limit of 24 percent and a plastic index of 7 percent.

Sandstone bedrock was encountered at depth in each of the three test pits. The sandstone is olive to light greenish-brown and medium-grained. In TP6 and TP-7, the sandstone was moderately weathered at the upper contact becoming harder and less weathered with depth. In TP-8, the sandstone was encountered very near the ground surface, was harder and less weathered, and exhibited a high degree of fracturing.

The subsurface soil conditions encountered within the DeWolfe Bridge Slide Complex are summarized below. More detailed logs of the Test Pits and the supporting lab data are included in the Appendix.

Table 4 - Summary of Subsurface Soil Conditions Within DeWolfe Bridge Slide Complex

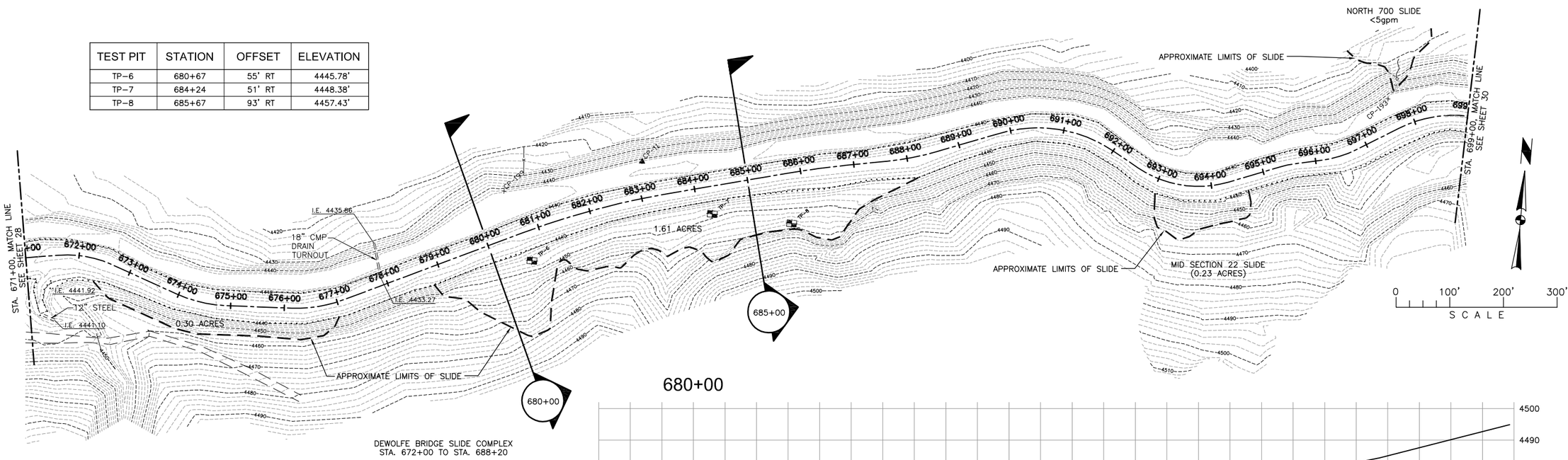
Soil Layer	TP-6	TP-7	TP-8
Topsoil and Organic Layer	0' - 0.8'	0' - 0.5'	0' - 0.3'
Glacial Till/Plastic Clay	0.8' - 5'	0.5' - 3'	NE
Silty, Clayey Sand w/Cobbles	5' - 8'	3' - 9'	NE
Sandstone Bedrock	8' - 9'	9' - 9.5'	0.3' - 4'
Groundwater	NE	NE	NE

NE- Not Encountered

The subsurface soil conditions are projected onto the cross-sections shown on Figure 4. Based on these subsurface conditions, it is apparent that the failure surface coincides with the soil-bedrock contact. Photo 4 below shows the eastern portion of the eastern section of the DeWolfe Bridge Slide near Sta. 685+00.

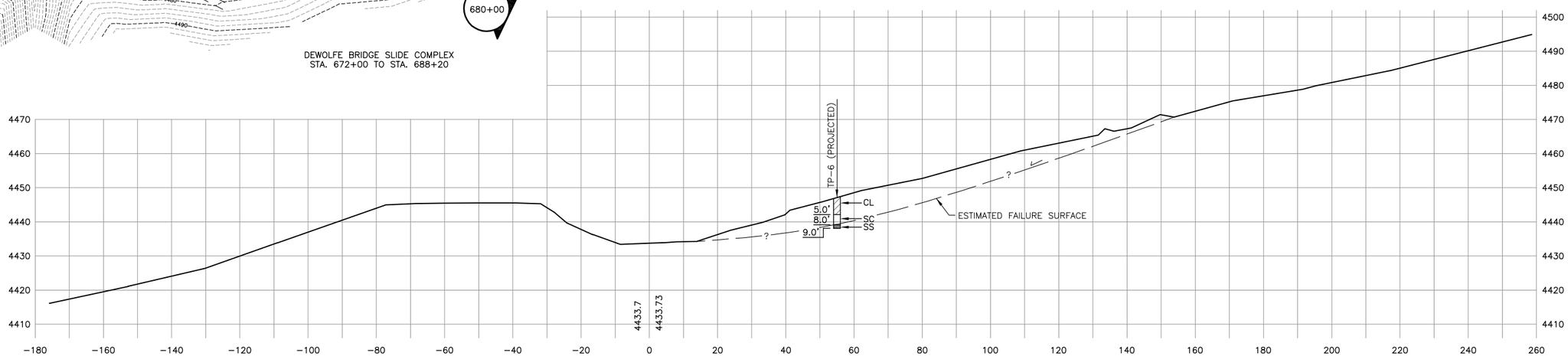
Photo 4. Looking downstream and across canal at DeWolfe Bridge Slide near Sta. 685+00. Note survey lath denoting TP-7. (01/04/08)

TEST PIT	STATION	OFFSET	ELEVATION
TP-6	680+67	55' RT	4445.78'
TP-7	684+24	51' RT	4448.38'
TP-8	685+67	93' RT	4457.43'

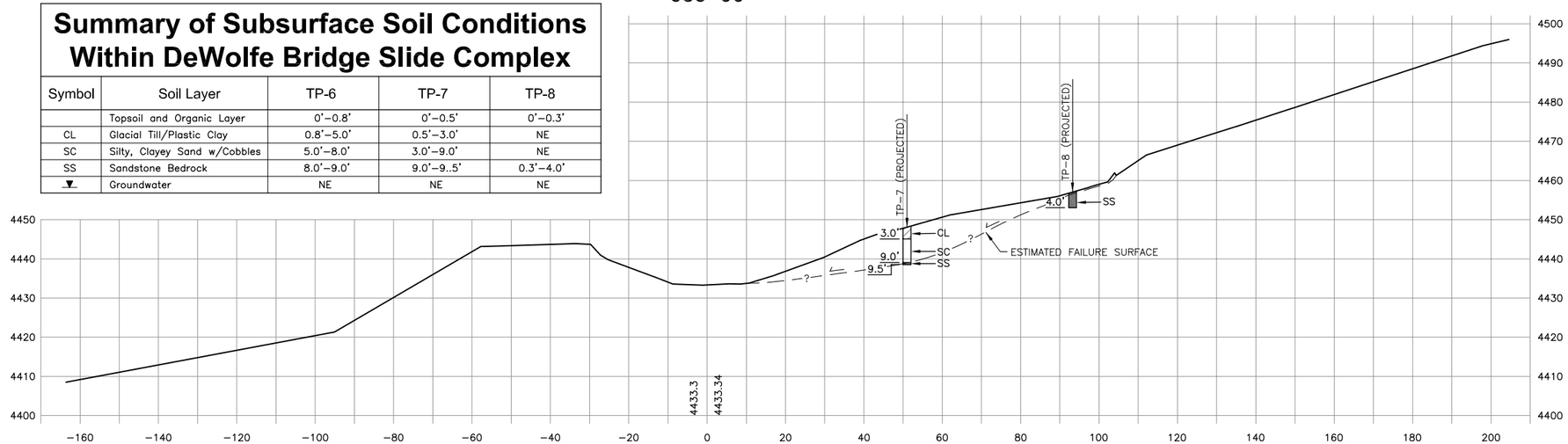


DEWOLFE BRIDGE SLIDE COMPLEX
STA. 672+00 TO STA. 688+20

680+00



685+00



Summary of Subsurface Soil Conditions Within DeWolfe Bridge Slide Complex

Symbol	Soil Layer	TP-6	TP-7	TP-8
	Topsoil and Organic Layer	0'–0.8'	0'–0.5'	0'–0.3'
CL	Glacial Till/Plastic Clay	0.8'–5.0'	0.5'–3.0'	NE
SC	Silty, Clayey Sand w/Cobbles	5.0'–8.0'	3.0'–9.0'	NE
SS	Sandstone Bedrock	8.0'–9.0'	9.0'–9..5'	0.3'–4.0'
▼	Groundwater	NE	NE	NE

FIGURE 4

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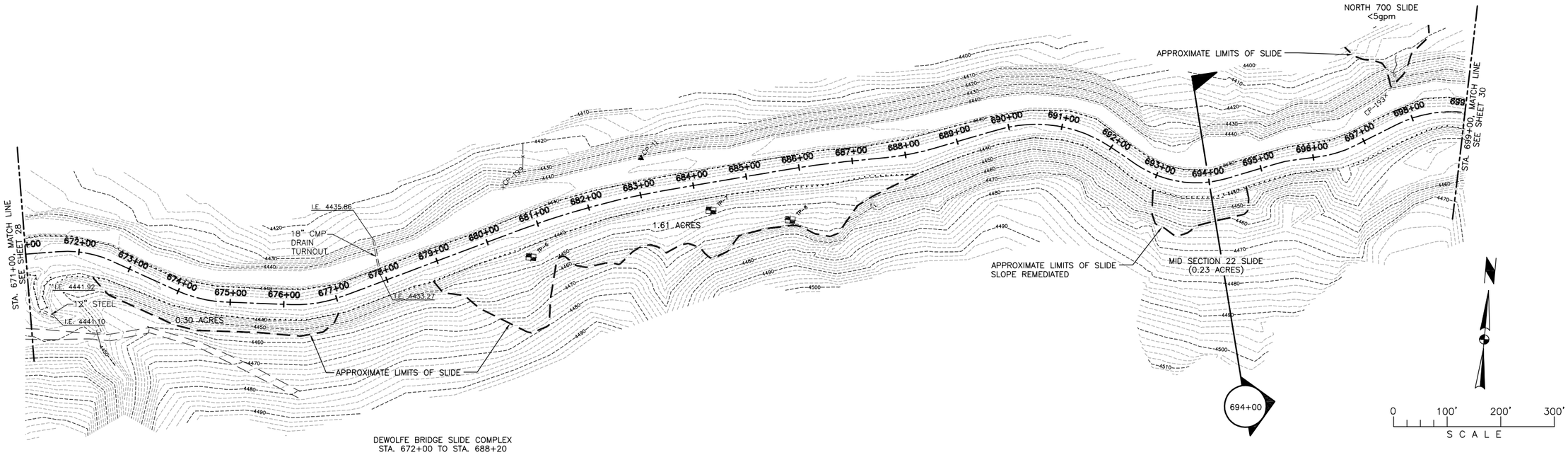
3.3.5 Mid-Section 22 Slide. This slide was added to the USBR Landslide Register in 1995. It is a relatively small, concentrated instability (0.23 acres) located between Sta. 693+00 and 694+50 as shown on Figure 5. Failure occurred in the natural soils comprising the backslope. Excerpts from the annual inspection reports are listed as follows starting in 1999.

- 1999, August - The east end of the slide was removed before the 1996 inspection and there appears to have been little movement since.
- 2000, July - No movement noted.
- 2001, July - No movement noted.
- 2002, June - No movement evident since last report.
- 2003, June - No change since the last inspection.
- 2004, July - Appears to be little change since last inspection.
- 2005, July - A portion of the slide reportedly regraded due to previous movement (2003). It does not appear to have moved since the last inspection.
- 2006, July - This area was previously graded due to soil creep and slumping in the past. Vegetation continues to proliferate across the graded area. Fresh scarps or other signs of recent movement were not evident.

The slide has reportedly exhibited very little movement since its significant displacement in 1995. The USBR's 2005 report references slope regrading and material removal performed in 2003. However the 2004 report makes no reference of this slope maintenance. Figure 5 shows the slide and provides a cross-section that reflects the slope remediation and regrading performed earlier. Photo 5 shows the Mid-Section 22 Slide.

Photo 5. Mid-Section Slide looking from across the canal near Sta. 693+50 Note area of past slope reduction and regrading. (01/04/08)



DEWOLFE BRIDGE SLIDE COMPLEX
STA. 672+00 TO STA. 688+20

694+00

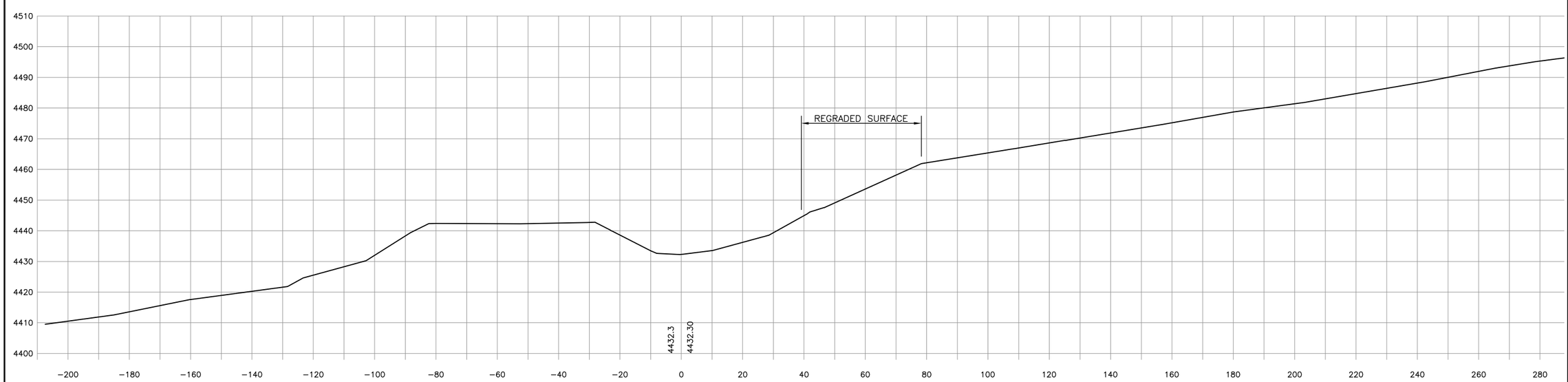


FIGURE 5

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MID SECTION 22 SLIDE

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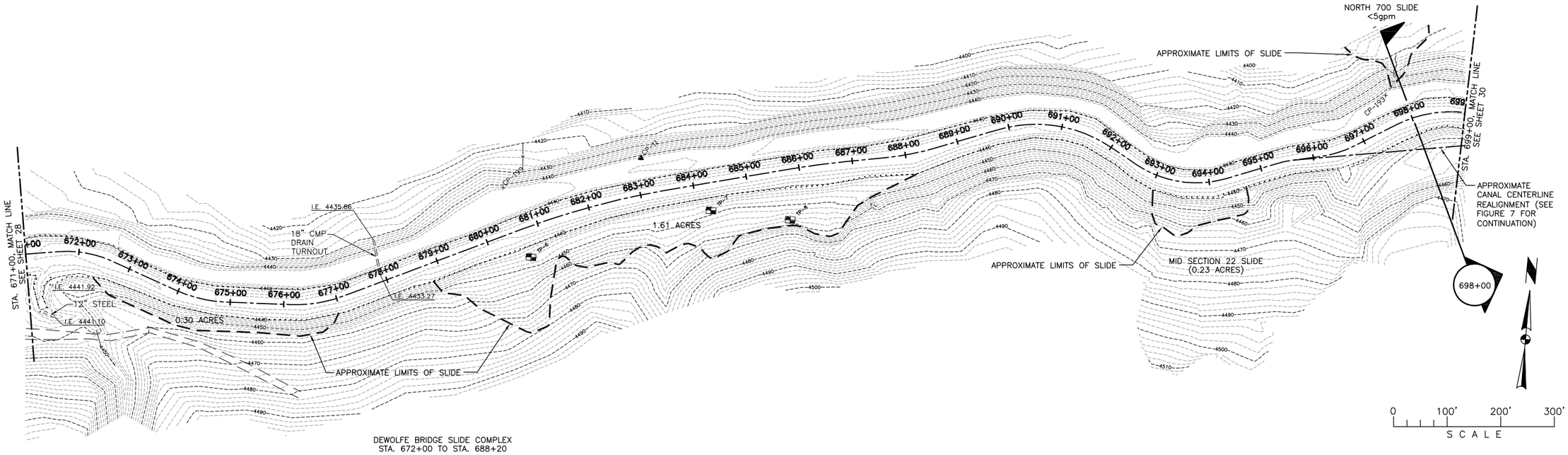
3.3.6 North Slope 700 Slide. This slide area was added to the USBR Landslide Register in 2005 and is situated on the downslope side of the canal embankment. It is located at approximately Sta. 698+00 between Mid-Section 22 and East Section 22 slides and is shown on Figure 6. USBR reports that seepage had been previously observed at this location and that tension cracks were observed at the crest of the embankment. Soil mass movements and erosion due to the heavy seepage have compounded the instability. Livestock and wildlife utilizing the seepage have detrimentally impacted the slump area and seepage egress. Heavy vegetation obscures the surface features of the instability. In 2006, a USBR geologist reports the presence of tension cracks up to 1½ inches wide at approximately 2 feet from the embankment crest and access road shoulder.

During the 2007-2008 off-season, USBR maintenance crews realigned approximately 600 feet of the canal in an attempt to improve stability of the North 700 Slide and reduce the risk of sudden and catastrophic failure. This alignment shift reduces the embankment fill slope angle and lengthens the seepage path. The photos below show the realignment project in January 2008. Several site visits by TD&H Engineering during the 2008 season suggest that the observed seepage at North 700 Slide is only slightly less than previous years.

Photo 6. Looking upstream at the canal realignment project intended to improve instability conditions at the North 700 Slide. (01/04/08)



DEWOLFE BRIDGE SLIDE COMPLEX
STA. 672+00 TO STA. 688+20

698+00

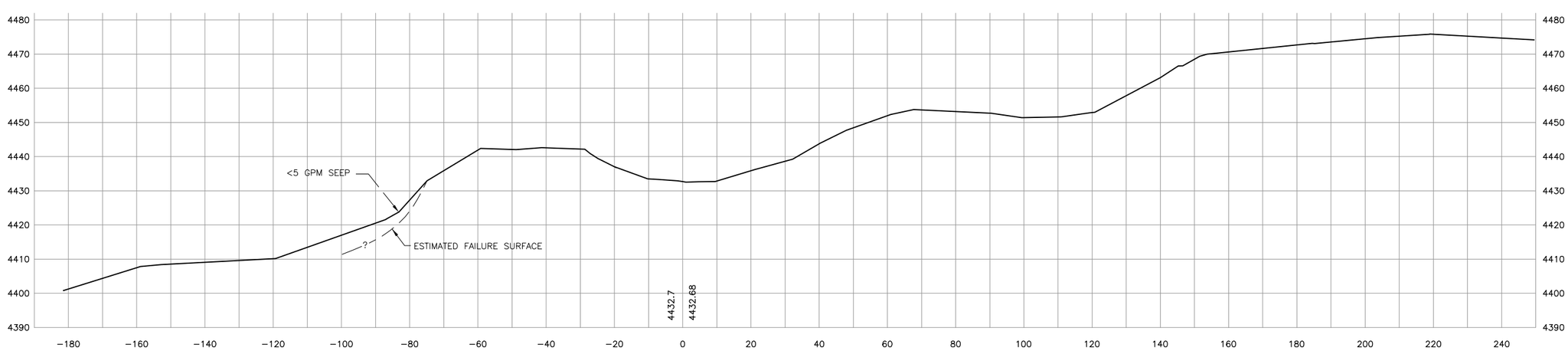


FIGURE 6

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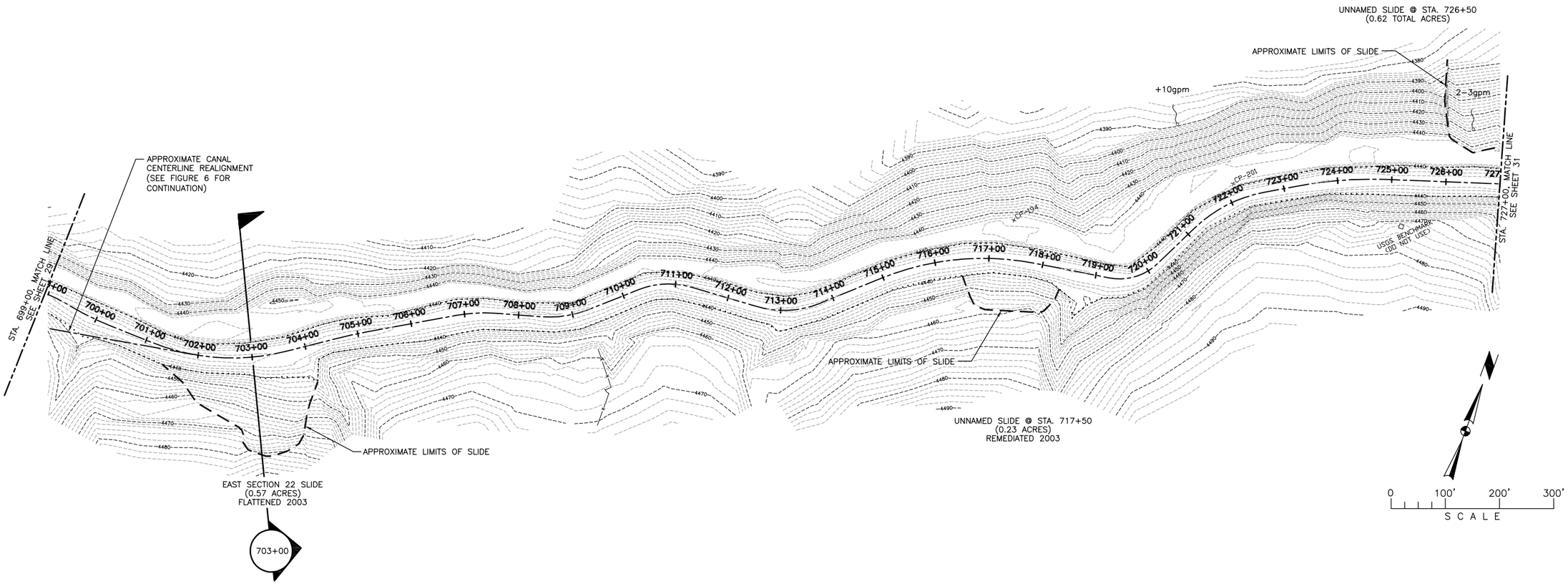
Photo 7. Looking downstream at the canal realignment project. Note East Section 22 Slide along right side. (01/04/08)

3.3.7 East Section 22 Slide. This slide is located in the natural soils comprising the backslope between approximately Sta. 701+50 and 704+00 (Figure 7). The slide was added to the Register in 1995 and presently covers 0.57 acres. It is reported that in 1996 material was excavated and removed from the toe and canal prism and that the slide again promptly failed. Excerpts from the individual USBR inspection reports starting in 1999 are provided as follows:

- 1999, August - The extreme eastern end of the slide reactivated last year, but does not seem to have moved this year. There appears to be no change in the top scarp since the 1998 inspection.
- 2000, July - There has only been very minor raveling at the back scarp since the last inspection.
- 2001, July - There appears to be no change in the top scarp since the 1998 inspection.
- 2002, June - Material moved from the center of the 1998 scarp and flowed toward the east end of the slide. This newly activated portion of the landslide extended 8 to 10 feet into the hillside from the original scarp. The toe has bulged out to match the original landslide movement in respect to the encroachment on the canal prism.



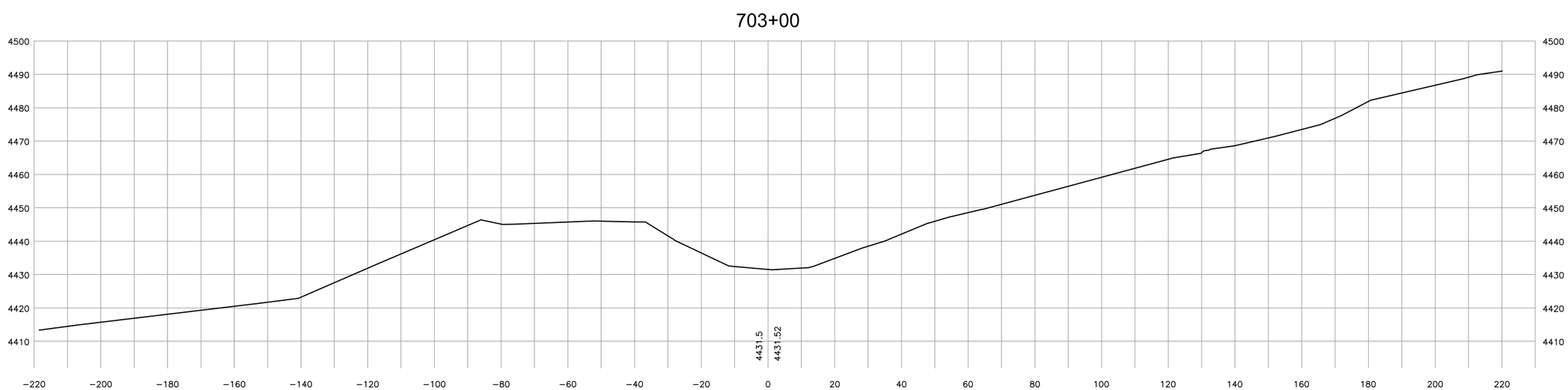


FIGURE 7

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EAST SECTION 22 SLIDE

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- 2003, June- Slide material that encroached into the canal prism in 2002 was excavated and removed on the eastern end of the slide by USBR crews and equipment at the end of last summer.
- 2004, July - Little to no movement was detected this year.
- 2005, July - The slump has a well-defined headscarp approximately 100 feet long. This slide does exhibit recent movement; the headscarp is extimated at 18 inches high.
- 2006, July - This area has shown movement in the last few years, though new movement does not appear to have taken place since the 2005 inspection. The headscarp has deteriorated slightly since the 2005 inspection.

This slide appears to have received considerable slope regrading and excavation of the slide mass. This is shown in Photo 8 below.

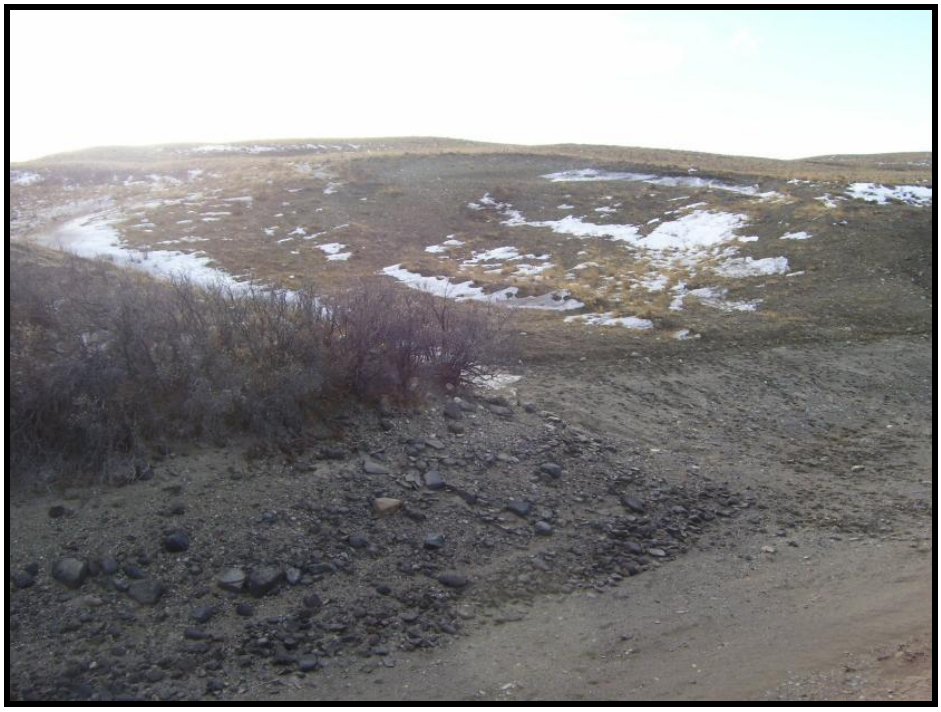


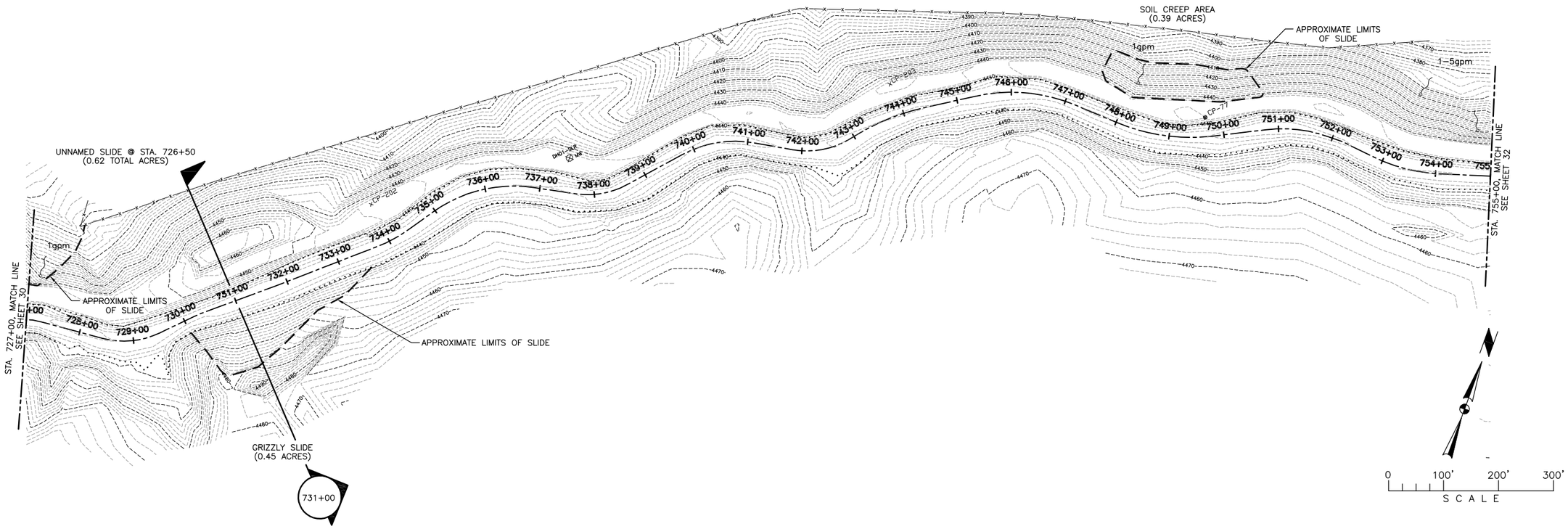
Photo 8. Photo shows East Section 22 Slide. Note the magnitude of previous slope regrading and excavation. (01/04/08)

3.3.8 Grizzly Slide. This slide was added to the Register in June 2002. Of initial interest was slumping of the left bank on the canal side which constitutes the downslope embankment and maintenance road. This slide encroached 3 feet into the roadway/left embankment. The volume was estimated to be about 75 cubic yards. The location provided included in the 2002 inspection report was approximately at Sta. 735+00 as determined from a landslide site map prepared by USBR staff. This area corresponds approximately to Sta. 717+50 as determined from the TD&H Engineering topo survey which is based on original USBR design stationing and is shown on Figure 7.

During the USBR's inspection in 2002, a second failure was noted. This second failure is situated along the backslope (right side) downstream of the embankment slump. Based on a site visit conducted in the Summer of 2007, the limits were delineated by USBR geology staff using the updated topographical survey prepared by TD&H Engineering. The slide extends from Sta. 730+00 to 735+00, covers approximately 0.45 acres and is shown on Figure 8. Initially, this small slide was also approximated at 75 cubic yards but has since expanded in size.

The embankment/maintenance road portion of the Grizzly Slide was rebuilt by USBR maintenance staff. Over time the backslope portion of the Grizzly Slide became the primary focus of attention in the annual write-ups. Pertinent excerpts from USBR's annual inspection reports are provided below starting with 2003.

- 2003, June - The road and shoulder were rebuilt in 2002 off-season with local equipment and material. No additional movement was detected this year. [Reference is made to repairs of the other Grizzly Slide.]
- 2004, July - No additional movement was detected. [The photos reference the embankment slide.]
- 2005, July - The roadway was repaired between 2003 and 2004, and has not posed a problem since. Slumping has occurred on steep slopes adjacent to outcrop along the right side of the canal. The slump does not appear to have moved all at once but rather over a couple of years yet was not discussed in previous inspection reports. The area has not moved since the 2004 inspection.



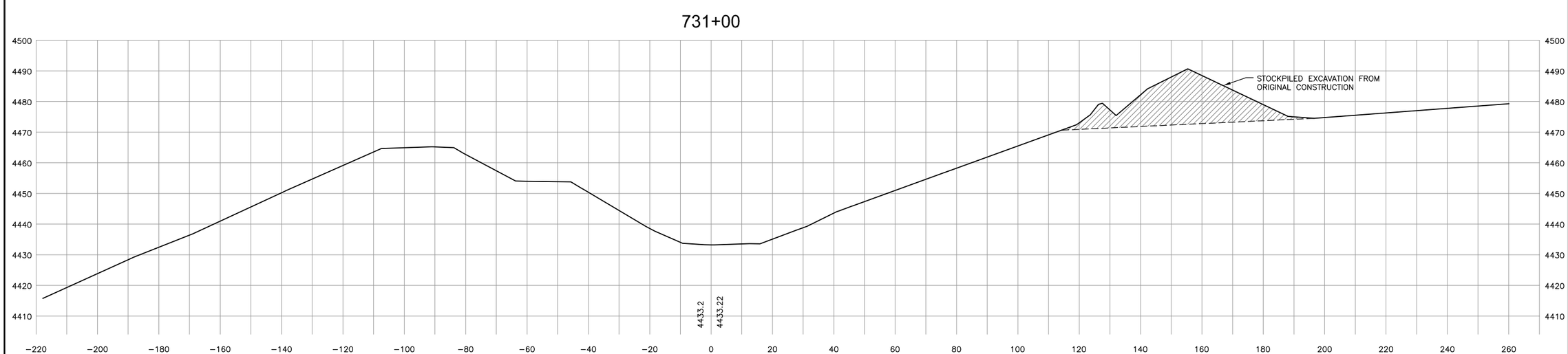


FIGURE 8

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GRIZZLY SLIDE

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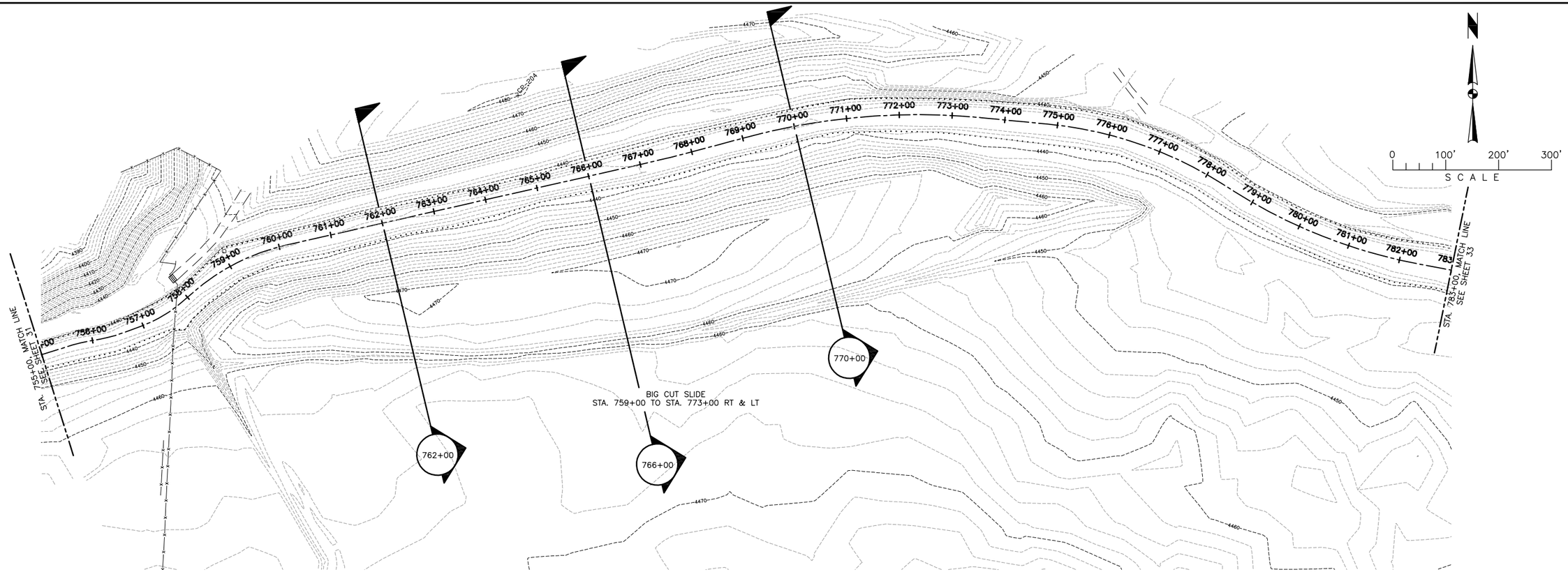
- 2006, July - Located on the right side of the canal, a head scarp up to four feet high defines the crest of this unstable area. Little change was noticeable since the 2005 inspection.

Photo 9. Looking upstream at the Grizzly Slide along the canal backslope. Note the excavation from original construction stock-piled at the top of the slide. (01/04/08)

3.3.9 Big Cut Slides. This is a very large slide area with a long history of movements necessitating regular maintenance. The slide area extends from Sta. 759+00 to 737+00 and is shown on Figure 9. The instability includes both sides of a very long, double backslope, excavation. Instability has been compounded by the placement of the original excavation material atop the backslopes.

Most of the slide area failed in 1995 and a large remediation program began in 1996 to remove material adjacent to the canal prism and regrade the backslopes. The slide area has been extensively excavated, yet continues to exhibit periodic movement. A compression pressure ridge forms in the center of the canal bottom between the two unstable slopes. Springs and seeps within the prism are reportedly observed by maintenance staff when the canal is dewatered.



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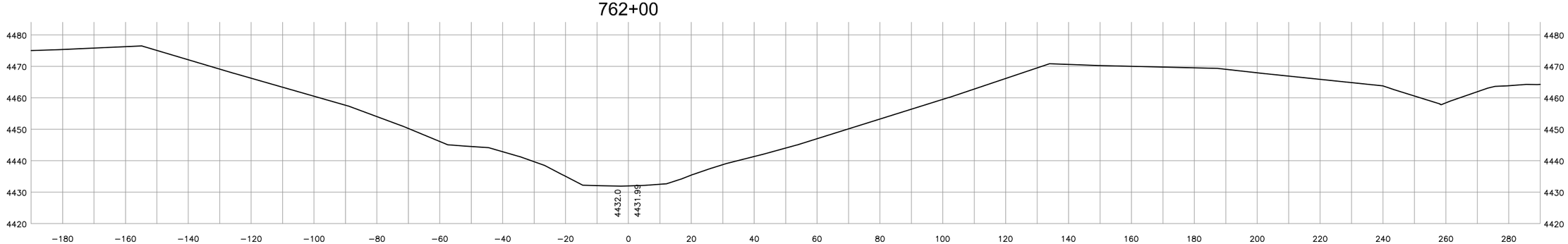

FIGURE 9

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BIG CUT SLIDE COMPLEX

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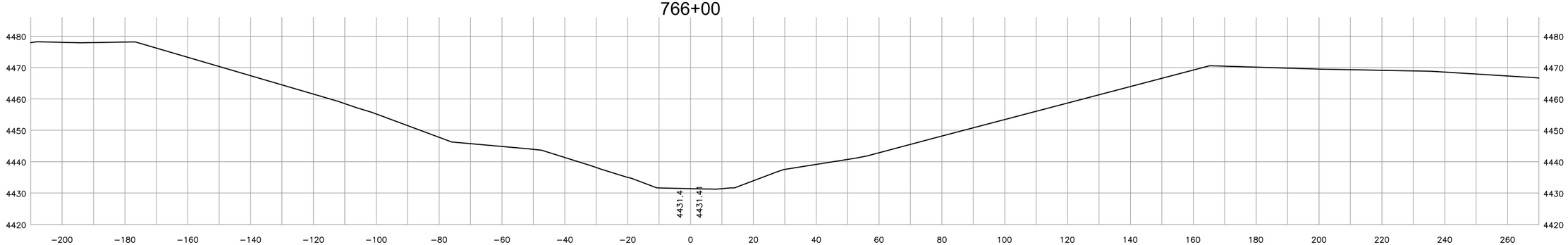

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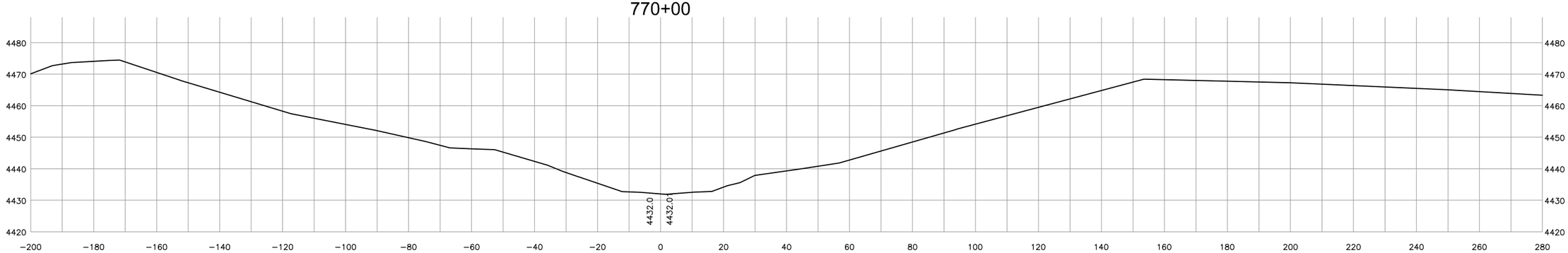

FIGURE 9

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The USBR's annual inspection reports are summarized as follows starting with 1999.

- 1999, August - Continued movement is evidenced by a few, narrow, weathered cracks parallel to the canal prism.
- 2000, July - The sides of the cut have revegetated over the last few years and little additional movement has occurred.
- 2001, July - Photo shows scarp which has formed as material continues to creep.
- 2002, June - Continued movement has been evidenced by large, offset cracks parallel to the canal prism. Much of the movement this year is by slow creep; some is likely to have been exacerbated by this year's fairly rapid drawdown of the canal.
- 2003, June - The slide area has been annually reworked. The site is currently being excavated on the west side of the canal. At one time there was severe cracking in the area.
- 2004, July - No movement was detected since remediation work completed last year.
- 2005, July - Repeated soil creep and slumping continues to require maintenance and mitigation. In 2004, material had to be removed from the canal. The area continues to creep into the canal from both sides, outlined by numerous scarps and tension cracks. Up to 14 inches of vertical movement with tension cracks to 2 inches accounts for an estimated 12 inches of lateral creep since last year's inspection.
- 2006, July - Maintenance is on-going along this section of the canal as both banks continue to creep inward. This is not new nor does it pose an immediate hazard to canal operation, but will continue as a maintenance issue.

Photo 10. Looking southeast at Big Cut Slides. (05/16/07)

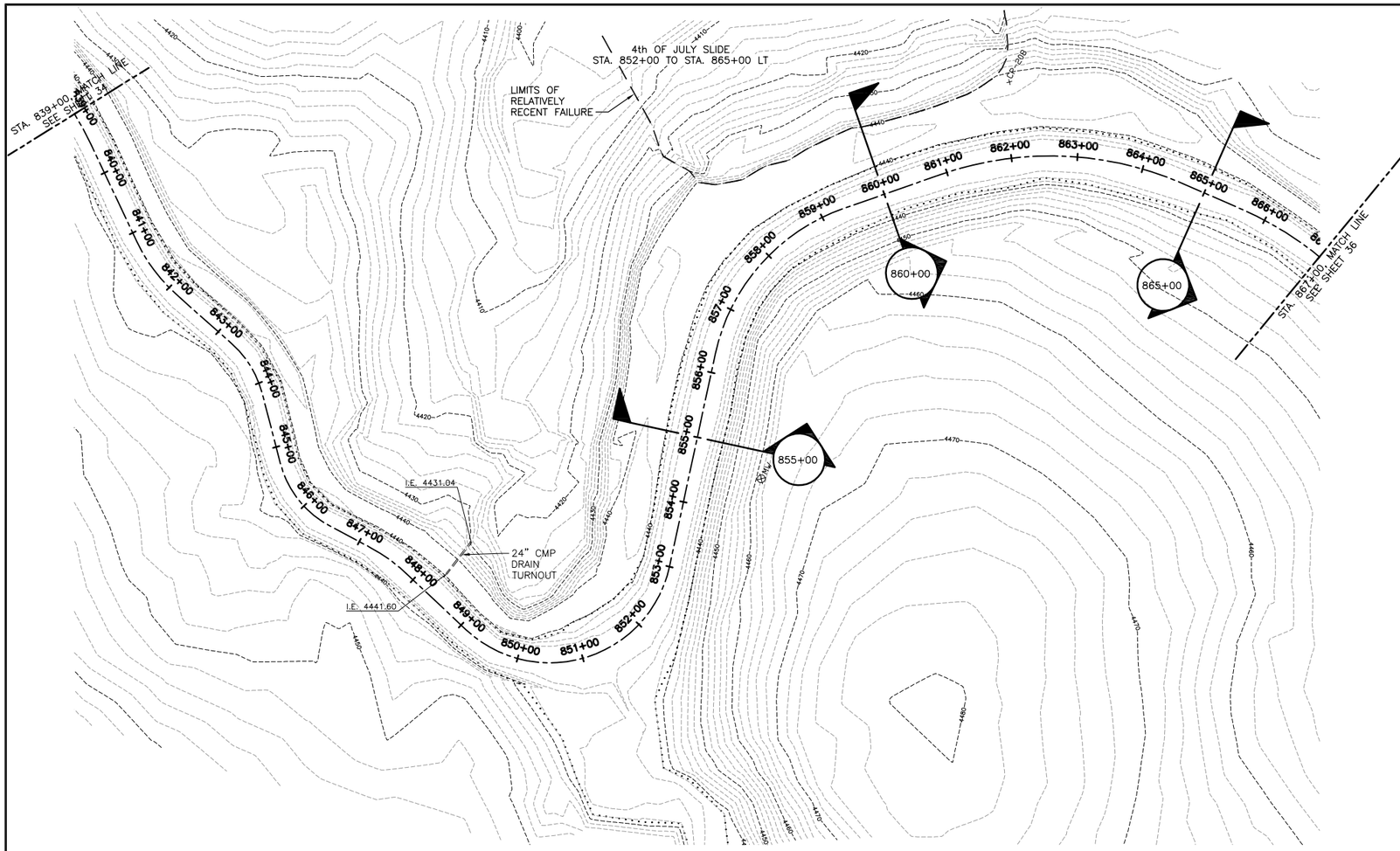
Photo 11. Looking downstream at canal prism of Big Cut Slides. Note compression pressure ridge formed along canal bottom. (01/04/08)

Photo 12. Looking northwest and upstream at Big Cut Slide. Note excavation stock-piled on top of south slope. (01/04/08)

3.3.10 4th of July Slide Complex. This area of instability is located from approximately Sta. 852+00 to 865+00 and is shown on Figure 10. The slide comprises the downslope fill embankment on a sharp bend of the contour canal. Lateral displacement and settlement of the embankment and maintenance roadway in this area has been a historical issue. The slide was given its name due to the time of year when movements historically began to occur each season.

In 1995, the USBR conducted an investigation to determine a method of stabilization. It is reported that, in June of 1995, movement nearly caused failure and loss of the embankment section. The investigation revealed that the slide consists of glacial till (plastic clay) over a sandy, shale bedrock. Instability is reportedly compounded by hydrostatic head pressure in a sandy, outwash layer at the top of the underlying bedrock.

The instability of 1995 was remediated during the off-season by shifting the canal alignment into the backslope and using the resulting excavation to buttress the toe. Since that time, the outer edge of the fill has continued to break off and slide into the adjacent valley (See Figure 10).



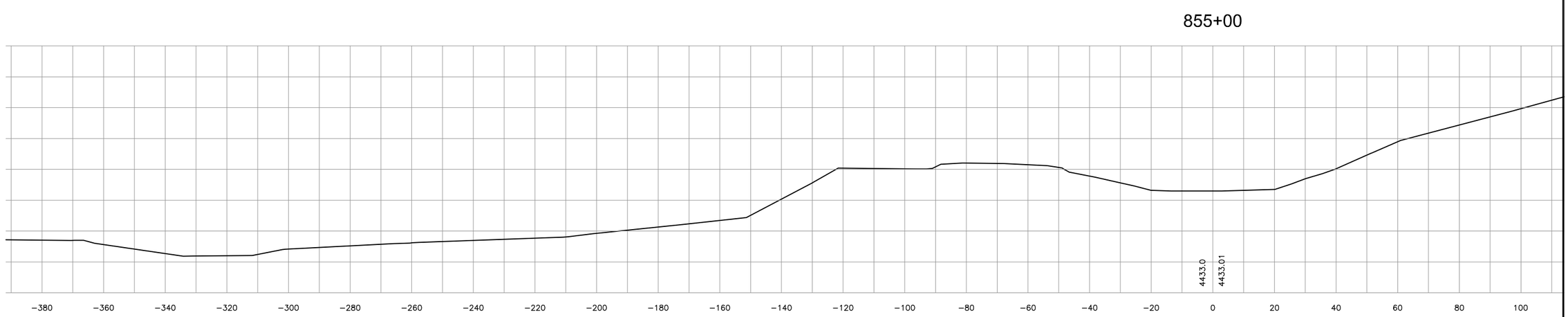


FIGURE 10

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4TH OF JULY SLIDE

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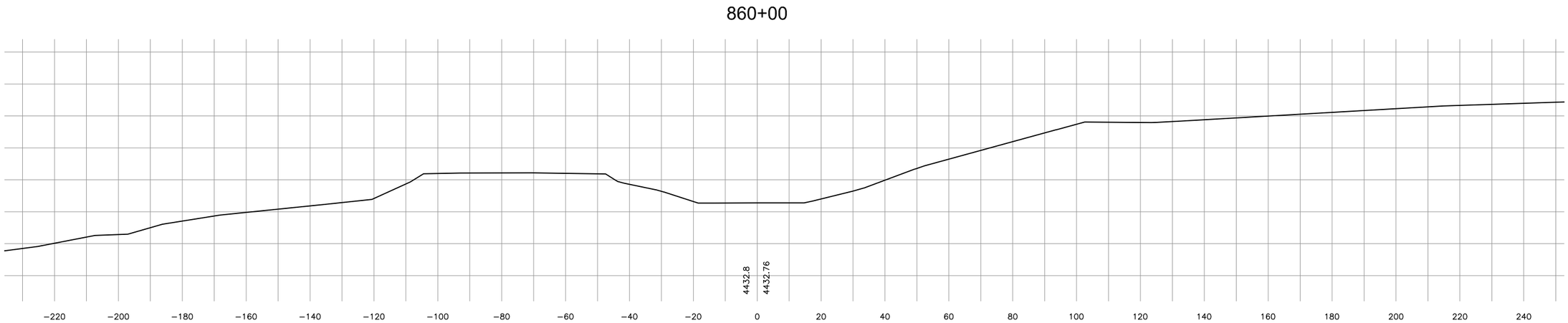

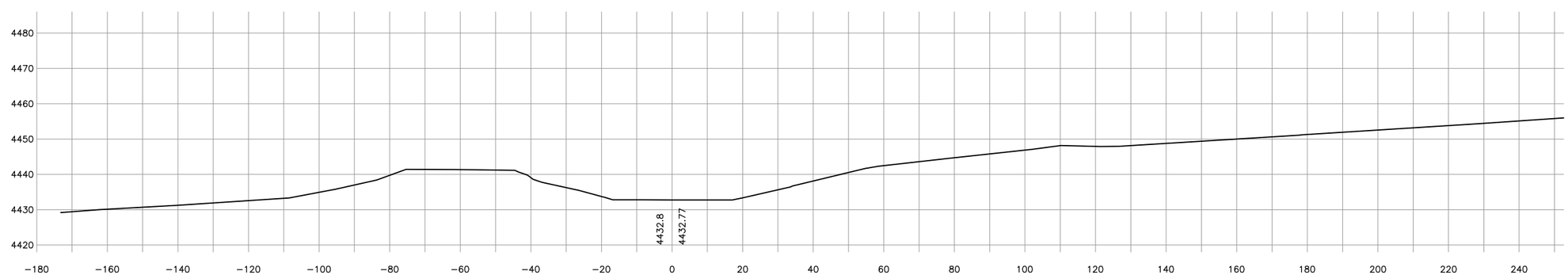

FIGURE 10

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4TH OF JULY SLIDE

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Photo 13. Looking eastward to downslope embankment fill of the 4th of July Slide. Note the relatively recent slump failure. (01/04/08)

A summary of comments from the annual inspection reports since 1999 by the USBR geologic staff is provided as follows:

- 1999, August - No discernable difference was noted from last year's inspection.
- 2000, July - There has been little or no movement since the last inspection.
- 2001, July - No discernable surface expression of recent landslide movement was noted at this time. This area appears to be unchanged since last inspected.
- 2002, June - No discernable surface expression of recent landslide movement was noted at this time.
- 2003, June - No discernable movement or recent landslide activity was noted this year.
- 2004, July - Little movement or recent landslide activity was noted this year.
- 2005, July - The 1995 realignment effectively moved the canal slightly south, reducing failure concern.
- 2006, July - Does not exhibit recent slope failure.

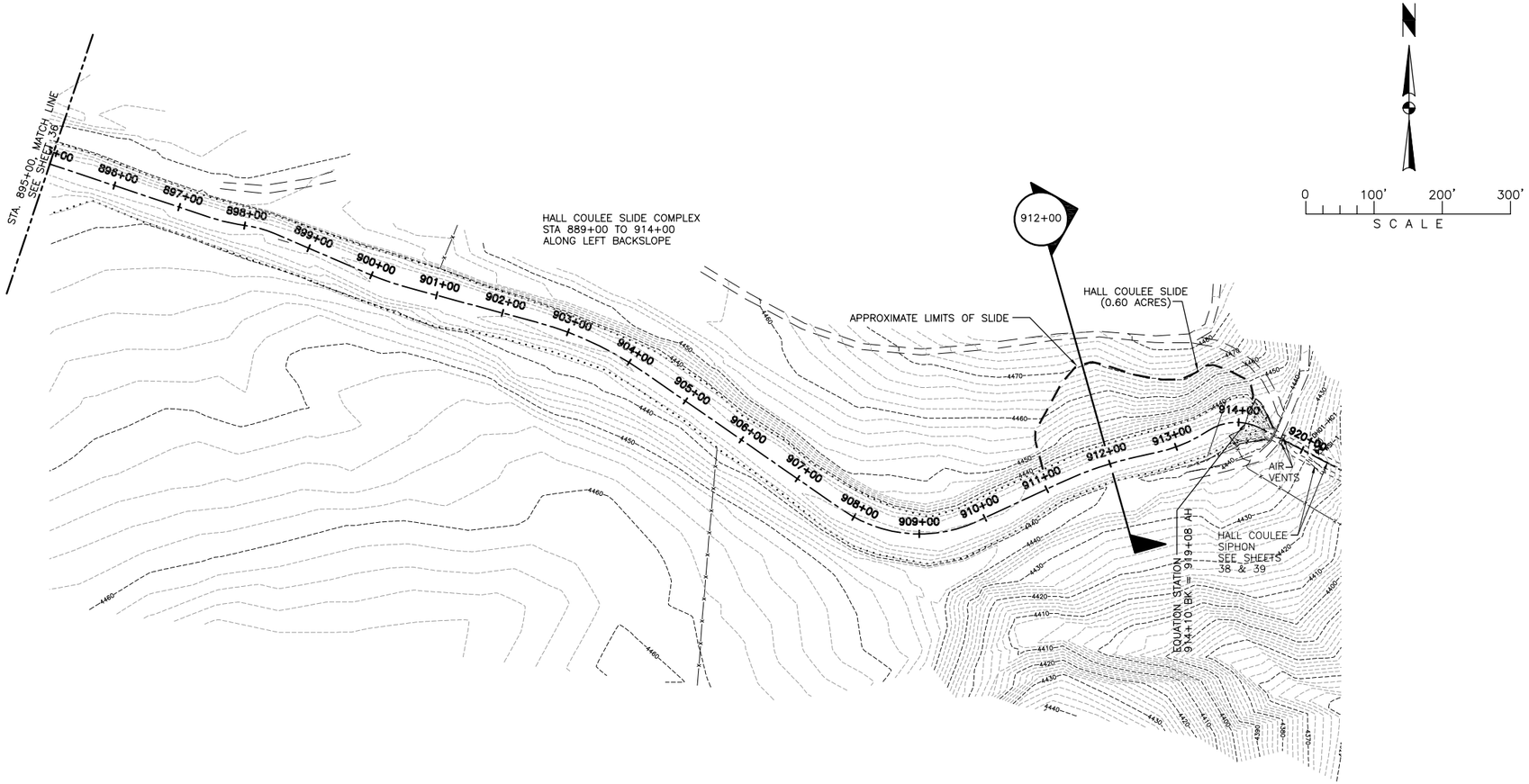
3.3.11 Hall Coulee Slide Complex. This slide area was added to the USBR Landslide Register in 2002. The slide complex extends a reported length of 2,500 feet from Sta. 889+00 to 914+00 along the left backslope. The area is shown on Figure 11. The USBR reports that the slide complex consists of an excavated hillside in the glacial till which mantles the Cretaceous-aged Horsethief Sandstone formation. The sandstone bedrock is exposed in the backslope adjacent to the siphon inlet. Heading upstream, the bedrock becomes shale and its outcrop exposure becomes progressively lower in the canal prism. Failure is either in the thin mantle overlying the bedrock or in the steep backslopes comprised of glacial till.

Photo 14 below shows that portion of the Hall Coulee Slide Complex adjacent to the siphon inlet structure. Failure at this location occurred in the thin mantle of glacial till overlying the stable bedrock and was reportedly remediated in 2003.

Photo 14. Looking northward at left backslope comprising the Hall Coulee Siphon Complex. Note the slope has been regarded and excavated to enhance stability. (01/04/08)

Excerpts from the USBR's annual inspection reports are provided below starting with 2002.

- 2002, June - The most recent movement is a series of small, arcuate slumps along the over-steepened excavation slope.



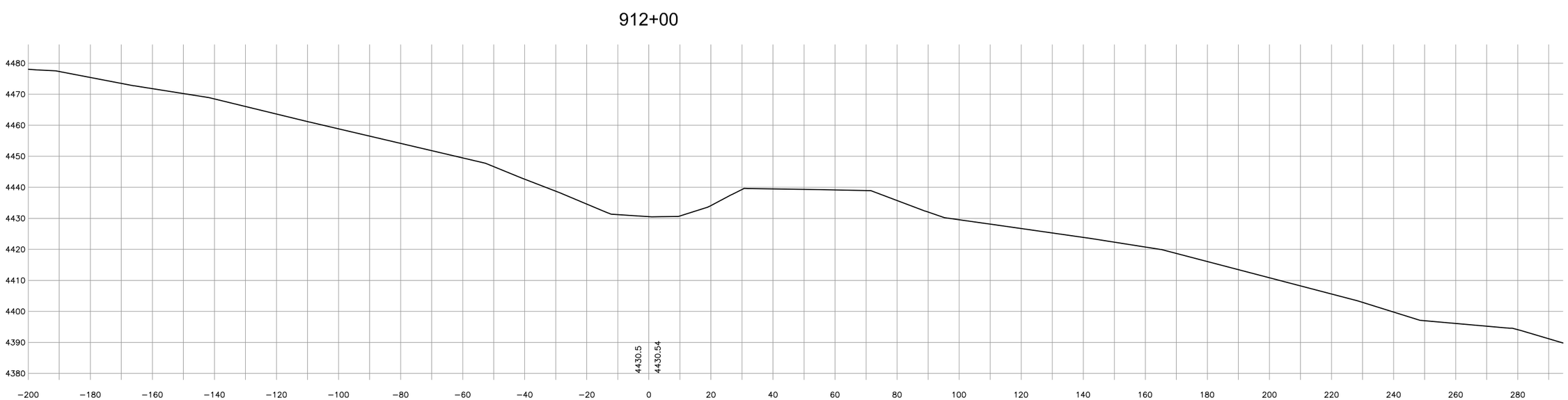


FIGURE 11

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HALL COULEE SLIDE COMPLEX

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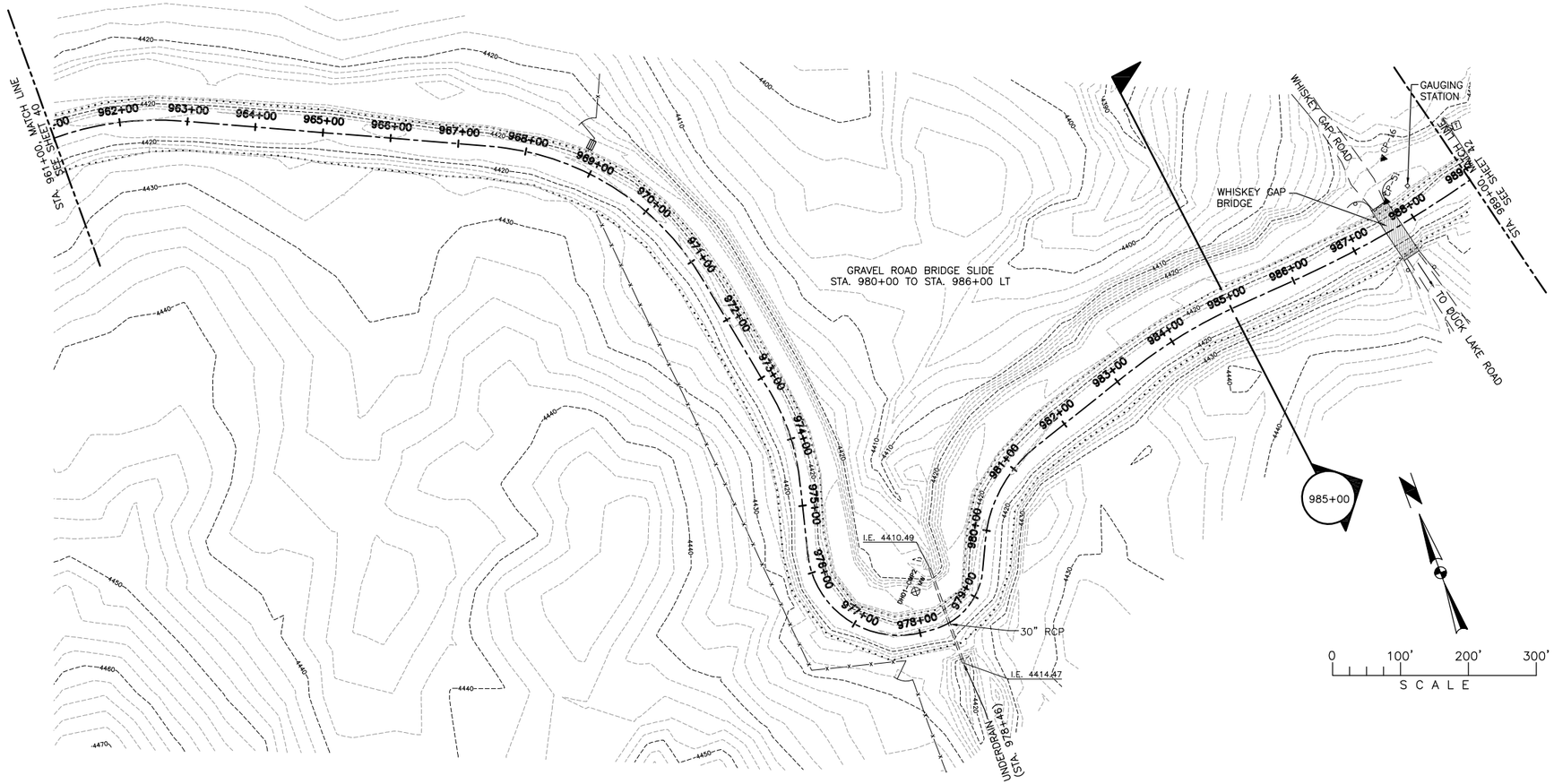
- 2003, June - No discernable movement was noted this year.
- 2004, July - Some movement was noted.
- 2005, July - Grading along the canal was completed in 2003 to mitigate an unstable section of the slope, and has not appeared to have moved since.
- 2006, July - Grading along both sides of the canal has reduced the quantity of material at risk of detrimentally affecting the canal through solifluction or slumping.

3.3.12 Gravel Road Bridge Slide. This slide was added to the USBR Landslide Register in 1997. The Bureau indicated that the slide is an old slide that reactivated in the embankment fill section of the canal. The original slide volume was estimated to be on the order of 75 cubic yards. The slide area was reportedly remediated following the 1997 inspection. The Gravel Road Bridge Slide area is located from Sta. 980+00 to 986+00 and is shown on Figure 12. The USBR's annual inspection reports are summarized as follows:

- 1999, August - The slide has displayed no movement this year.
- 2000, July - The slide is still exhibiting minor slumping.
- 2001, July - The slide has displayed no movements during the last three or four years. The repairs have been complete (1997) and the area appears to have stabilized.
- 2002, June - The slide has displayed no movement during the last five years. The area appears to have stabilized.
- 2003, June - In 2002, a small failure from a saturated excavation slope on the south side of the canal was noted. This slide is relatively small and located southeast of the original Gravel Road Bridge Slide. No movement has been noted this year.
- 2004, July - No movement has been noted this year.
- 2005, July - The original Gravel Road Bridge Slide, on the left embankment, has been mitigated and has not posed a stability concern. The subsequent Gravel Road Bridge Slide, on the right backslope, is approximately 70 feet wide along the toe and displays vertical movement up to 14 inches, estimated from scarp height.
- 2006, July - The head scarp appears slightly degraded in comparison with 2005 inspection finding.



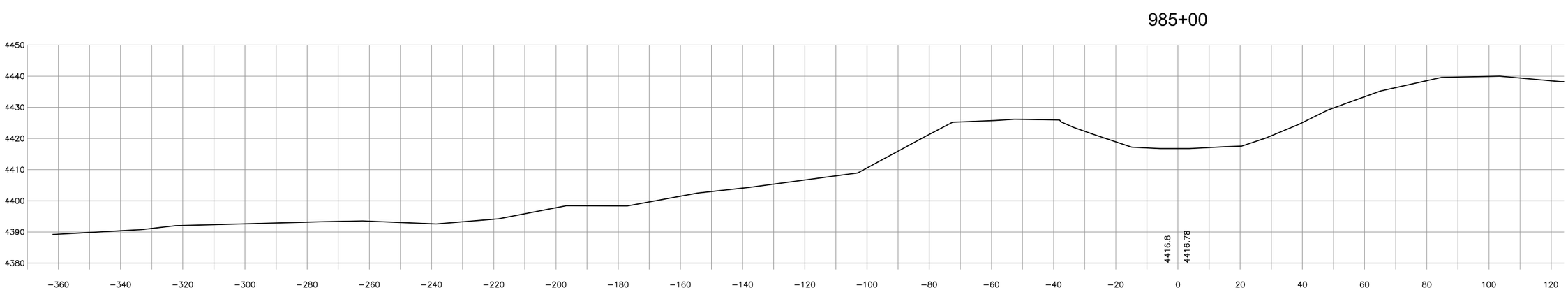


FIGURE 12

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GRAVEL ROAD BRIDGE SLIDE

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3.3.13 Martin Slides. The Martin Slides are similar to the Big Cut Slides in that the canal passes through a large, double backslope cut section initially with relatively steep backslopes on both sides. The potential for instability has been increased, in part, by the placement of the original excavation at the crest of the excavated backslopes. The Martin Slides extend approximately 1,200 feet from Sta. 1022+00 to Sta. 1034+00 on both the right and left sides as shown on Figure 13.

This area has a long history of displacements and those that occurred in the Summer of 1995 were reportedly very extensive. At that time, failed backslopes were reshaped and stabilized (see Photo 15). Movements continued and are exacerbated by above normal precipitation events.

Photo 15. Looking southeast at south backslope of Martin Slides. Note the slope surface has been regraded. (01/04/08)

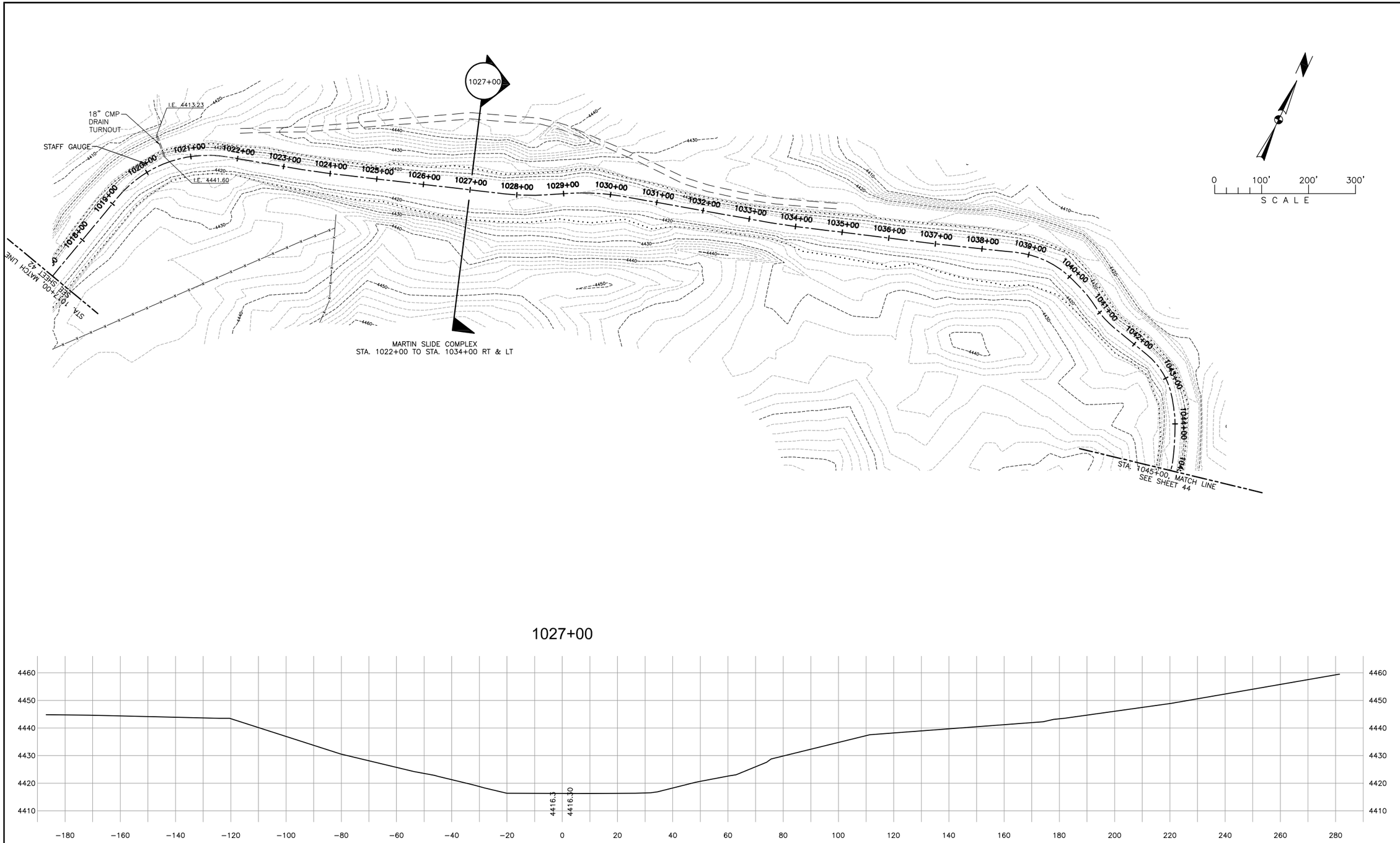

FIGURE 13

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The USBR's comments were summarized from the annual inspections for this slide area and are listed below.

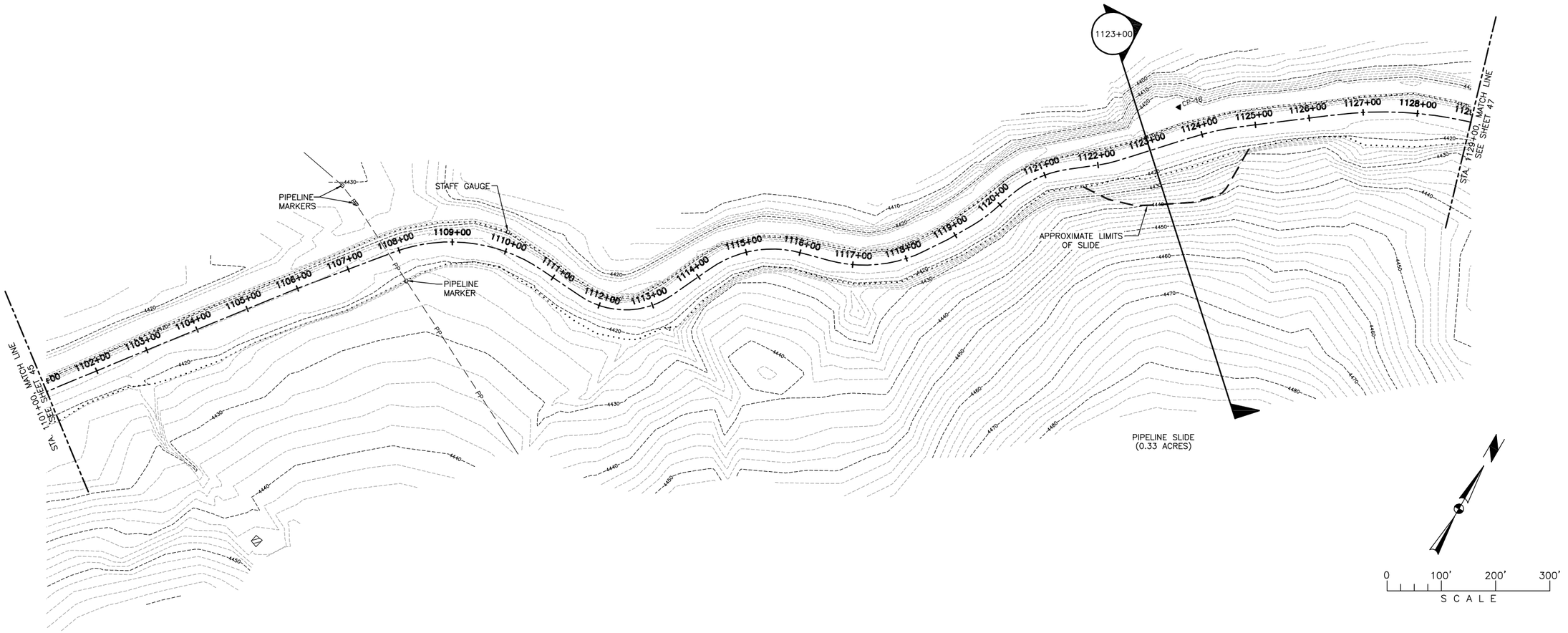
- 1999, August - The area was reshaped after the 1995 movement, and has remained relatively stable since.
- 2000, July - No new movement was noted since the last inspection.
- 2001, July - For the last three years, a large increase in vegetation on the slopes is the most noticeable change.
- 2002, June - High precipitation triggered new and massive failures. The failures have partially occluded the canal prism and both slopes failed into the canal. Extensive repairs were warranted.
- 2003, June - Repairs addressing the 2002 failures were reportedly completed in the Summer 2003.
- 2004, July - There appears to be some landslide activity since last year's inspection.
- 2005, July - Farthest downstream reaches of the Martin Slide area have been revegetated naturally and do not demonstrate recent movement. The upstream-most part of this slide has moved since the 2004 inspection. A fresh scarp up to 3 feet high, 70 to 100 feet along the crest, and up to 120 feet across the toe has appeared since the last inspection. This new failure has slumped into the canal a few feet.
- 2006, July - The relatively-fresh scarp noted in the 2005 inspection remains prominent.

3.3.14 Pipeline Slide. The Pipeline Slide is a small circular slide located on the right backslope in native glacial till soils. The slide is approximately 0.33 acres and is centered about Sta. 1123+00 as shown on Figure 14. The slide occurred in 1995 and was reshaped and remediated prior to the 1996 inspection. The USBR annual inspection reports are summarized as follows beginning with the 1999 report.

- 1999, August - The slide has remained relatively stable since the 1995 repairs.
- 2000, July - It has shown no further movement.
- 2001, July - The slide has remained relatively stable since the 1995 repairs.
- 2002, June - The original slide (approximately 200 feet long) has reactivated to

approximately 500 feet long. Most of the hillside adjacent to the excavation is demonstrating slow creep.

- 2003, June - The slide became active and enlarged in 2002. No discernable change since last year.
- 2004, July - Some movement has occurred since last year.
- 2005, July - The slide was previously graded to address on-going failure, but continues to creep. It has a headscarp up to 24 inches high along its length. Little movement is recognizable since last year's inspection.
- 2006, July - Grading previously addressed slope failure through the middle of the unstable area. Scarps are prominent on either side of the graded area.



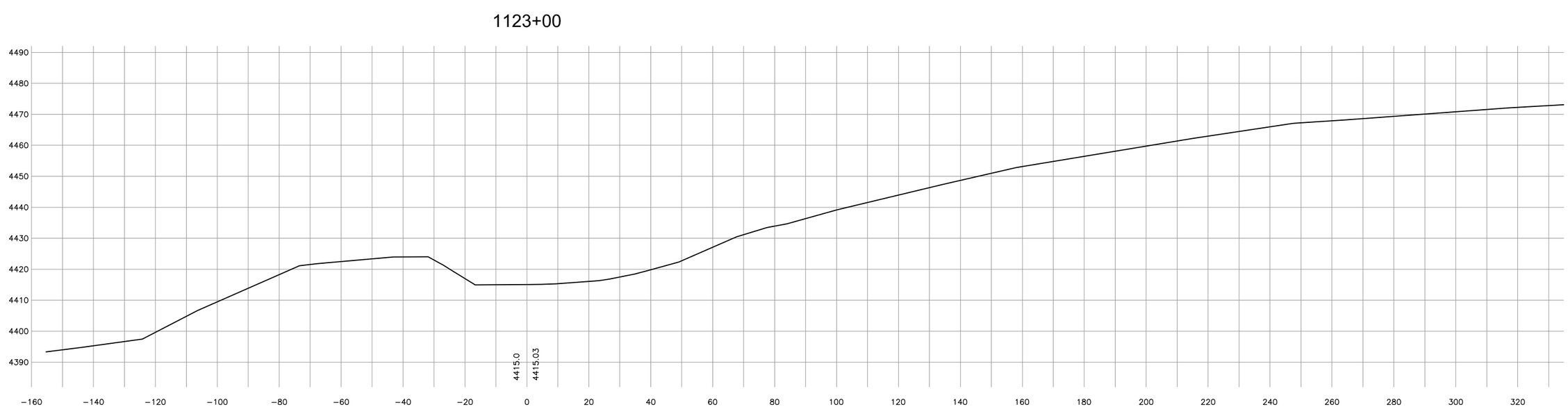


FIGURE 14

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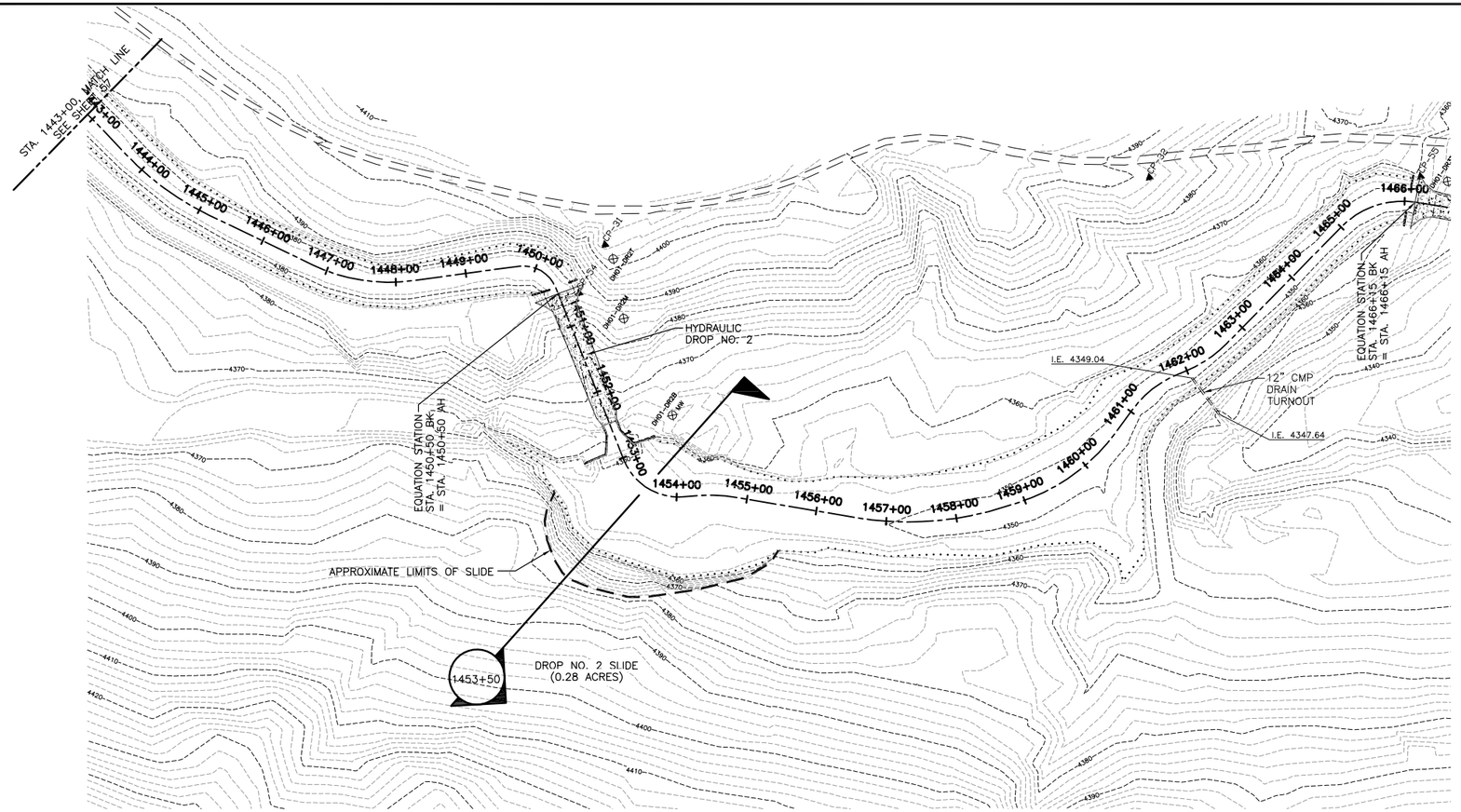
3.3.15 Drop No. 2 Slide. This is a small slide, approximately 0.28 acres, located in the stilling basin below the second hydraulic drop. The slide is shown on Figure 15 and in Photo 16. The slide is the result of erosion caused by general wave action and back eddies eroding the toe of the slope. The slide was added to the USBR Register in 1995. The USBR indicates the size of the slide to be 150 feet long by 20 feet high.

Photo 16. Looking south from top of hydraulic drop toward Drop No. 2 Slide. Note wave action created in the stilling basin. (05/16/06)

A history of the annual inspections starting with 1999 is summarized below.

- 1999, August - It does not appear to have changed significantly since the last inspection.
- 2000, July - It does not appear to have changed since the last inspection.



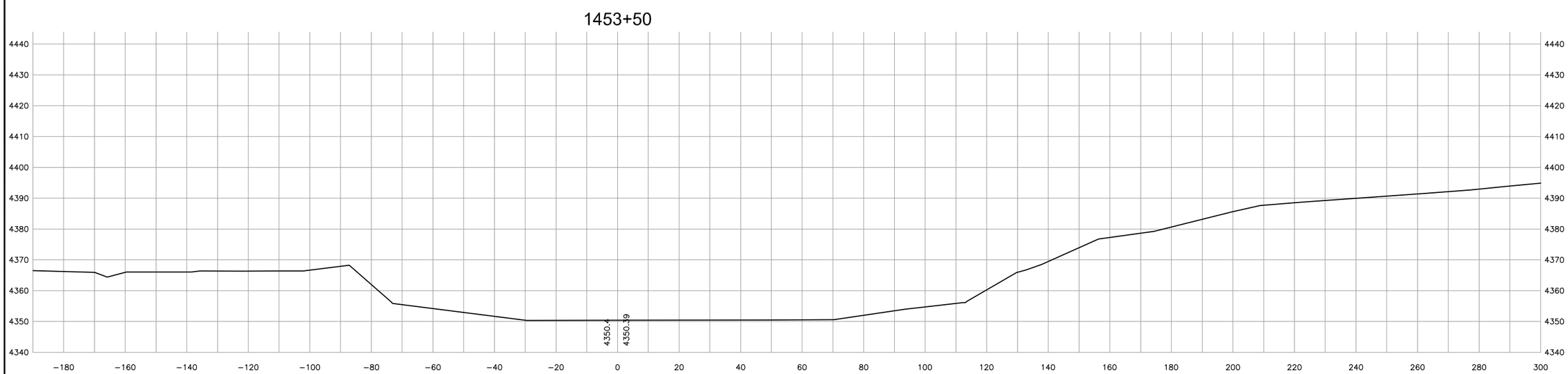


FIGURE 15

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DROP NO. 2 SLIDE

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- 2001, July - The area displays some recent erosion at the toe. The slide area displays no recent movement.
- 2002, June - Displays some recent movement due to erosion at the toe.
- 2003, June - The slide shows no change since last year. The slide is continuously undercut at the toe by erosion.
- 2004, July - The slide is continuously undercut at the toe by erosion.
- 2005, July - It does not appear to have moved or increased in size since the last inspection.
- 2006, July - Soil creep and bank failure continue resulting from over-steepened slopes and erosion by currents.

3.3.16 Drop No. 5 Slide. This slide was added to the Landslide Register in 2002. The USBR initially estimated the volume to be less 100 cubic yards. The slide is located upstream of the final hydraulic drop structure at approximately Sta. 1523+50. The area is shown on Figure 16 and has a surface area of approximately 0.31 acres. From the topo survey (Figure 16), it is apparent that the slide toe has encroached into the canal prism. This is supported by discussions with maintenance staff. They indicate that periodic excavations are warranted from the canal prism due to slope movements. This material is placed immediately downslope adjacent to the fill embankment. The slide is shown in Photo 17.

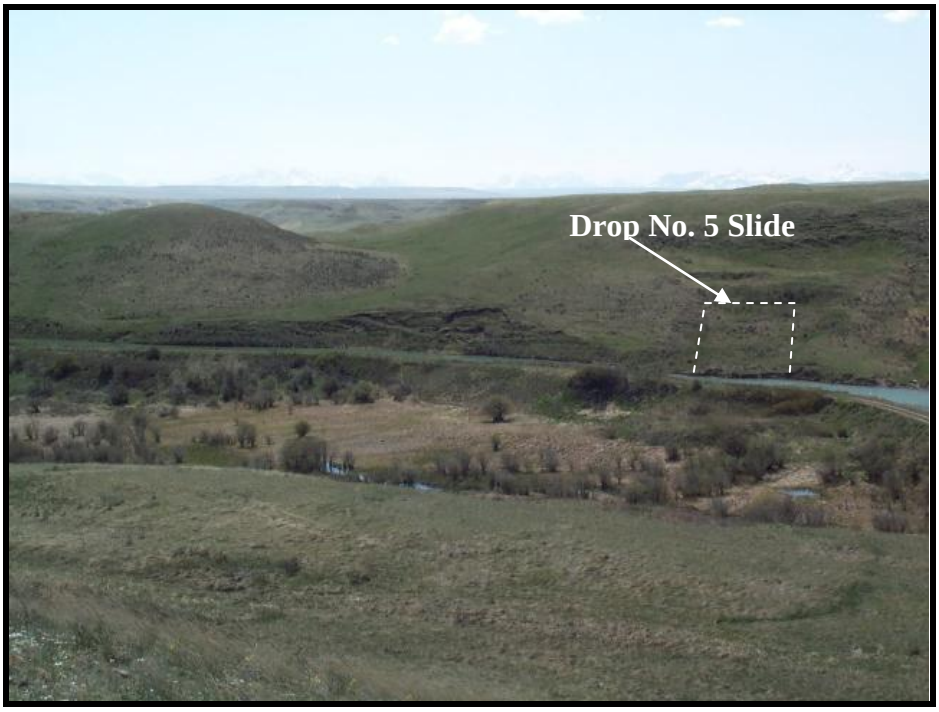
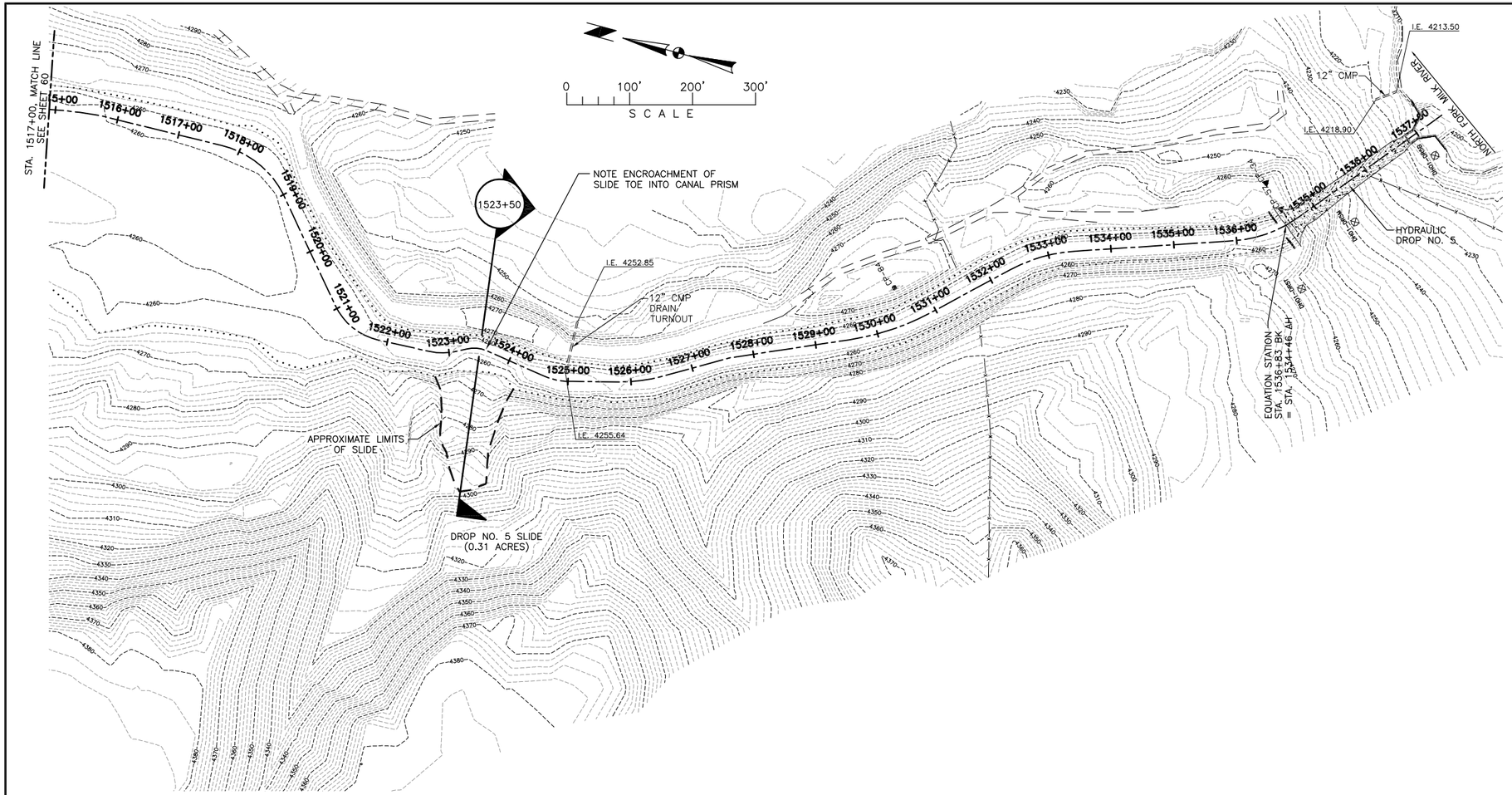


Photo 17. Looking west at Drop No. 5 Slide. Note nature of progressive failure and location relative to topographic swale. (05/16/06)

- 2003, June - Minor creep was noted. No discernable changes since the last inspection.
- 2004, July - Minor creep was noted. No discernable changes since the last inspection.
- 2005, July - No discernible movement has occurred.
- 2006, July - No reference to previously identified slide. New discussions regarding erosion and slumping in Drop No. 5 stilling basin (North Fork of Milk River).



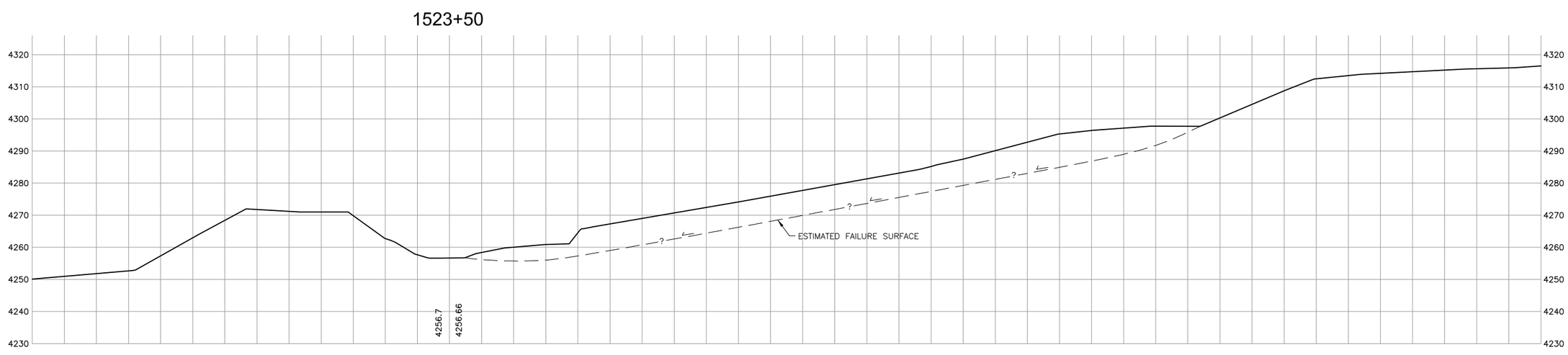


FIGURE 16

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DROP NO. 5 SLIDE

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4.0 GEOTEHCNICAL DISCUSSIONS

4.1 Slope Stability Factors.

Many of the 16 landslides currently listed on the Landslide Register have many similar characteristics that contribute to their instability. In general, environmental factors contributing to slope stability are classified as either driving forces or resisting forces. In theory, driving forces increase the potential for instability and resisting forces increase the Factor of Safety against slope movements. For slope equilibrium, the shear strength of the soil mass, especially along a developed the failure plane, must exceed the shear stresses applied to the soils. Typical examples of each force are listed below.

Table 5 - Slope Stability Factors

DRIVING FORCES	RESISTING FORCES
Surcharge Loads	Toe Buttresses
Seismic Loads	Shear Strength
Hydrostatic Forces	Supplemental Tensile Reinforcement
Gravity	Gravity

Activities that increase or reduce one of the forces will impact the stability of a given slope. These common activities are listed and discussed below.

Toe Erosion - Erosion at the base of a slope or at the toe of an active landslide reduces the resisting force. This is a primary factor currently impacting the Camp Nine and Drop No. 2 Slides. To some degree, the simple act of constructing the one-bank, contour canal has resulted in the excavation and removal of the slope toe and/or steepening of the pre-existing slope conditions with the new canal backslope. In many cases, this is sufficient to initiate failure in marginal soils. Also, toe erosion may occur internally to the canal due to currents and sharp bends. As would follow, slope stabilization can be achieved or improved by buttressing the toe of an instability or steep slope. This has been performed by the USBR on many downslope toes of fill embankment instabilities.

Shear Strength Degradation - Internal to the slope mass along the likely plane of failure and sliding, soil shear strength degradation can occur with increases in soil moisture and pore pressure. This can occur due to plastic swelling (void ratio increases), reduction in effective stresses, and decomposition and weathering of soil and rock constituents. Once failure occurs and movements begin, other factors can affect internal shear strength of the soil mass. These include loss of tensile strength continuity due to extension cracking in the clay soils, strain softening in over-consolidated clay soils, and slickenside propagation in plastic soils. Strain softening and the development of slickensides can be visualized in the figure below. Many over-consolidated clays (OCR>1) exhibit a peak strength when first strained and a much lower residual strength when sufficiently strained. This illustrates the propensity for some soils and slopes to exhibit progressive sliding and failure with very little change in applied stresses.

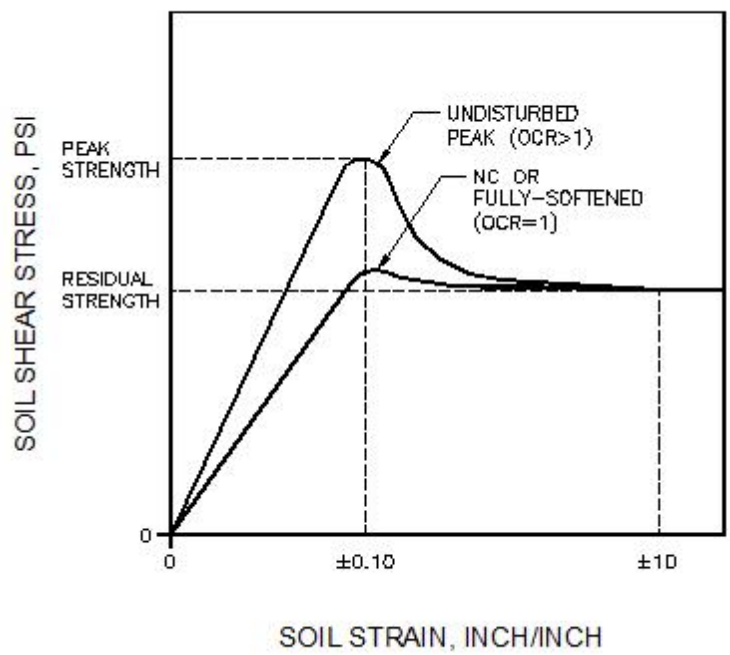


Figure 17. Stress Versus Strain Relationships for Plastic Clays

Surcharge Loads - The stock-piling of excavation or other loads placed at the top of a slope increases the driving forces by increasing the internal shear stress. This has been a contributing factor in Grizzly, Big Cut and Martin Slides. Conversely, unloading an unstable slope surface reduces the driving forces and increases the stability. This slope remediation technique has been

implemented by the USBR at many of the slides including Camp Nine, Mid-Section 22, East Section 22, Grizzly, Hall Coulee and Martin Slides.

Hydrostatic Forces - The destabilizing nature of water on a slope stability failure and its impacts are numerous. They include shear strength reductions (as discussed earlier) as well as increasing the soil unit weight, thereby adding weight to the slope mass (surcharging). Also, water can fill tension cracks and create a hydrostatic force thereby increasing the driving forces. Buoyant/artesian forces and elevated pore presures acting at the base of a slide mass can occur when saturated zones of permeable soil and rock are capped by relatively impervious soil, such as the glacial till. This is illustrated in Figure 18 below. The net effect is a reduction of shear strength by decreasing the effective stresses. In our opinion, this is a contributing factor to the St. Mary River Siphon (southside), DeWolfe Ranch and DeWolfe Bridge Slides.

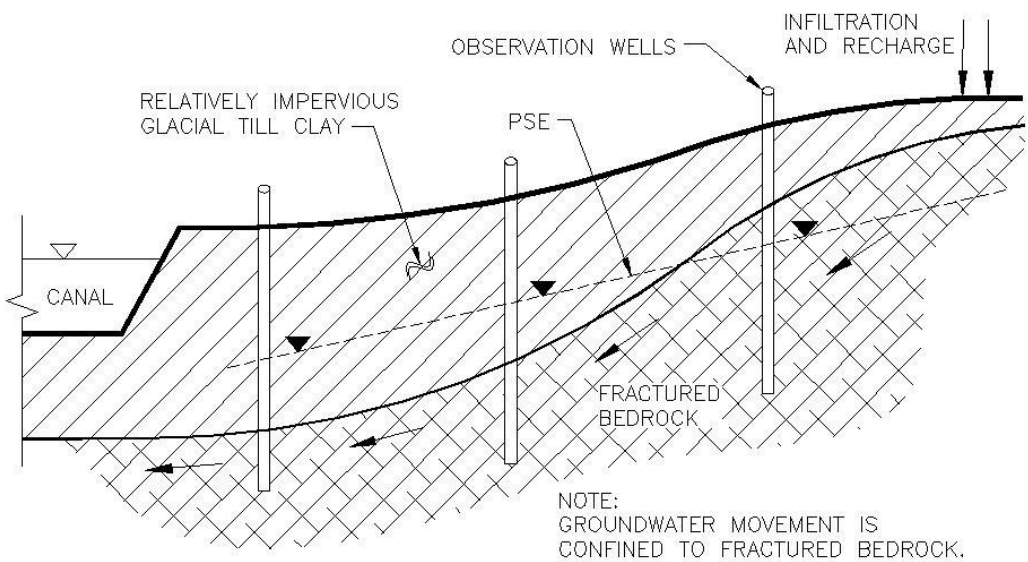


Figure 18. Example illustrates the potential development of artesian-related pore pressures in a slope capped with relatively impervious soil. The wells show the hypothetical rise of groundwater consistent with the PSE.

Rapid Drawdown - Destabilization and failure of slide toes and saturated canal banks can occur if a rapid drawdown of the canal is realized. This occurs as trapped pore pressures become relatively high when the external water load is quickly removed or at least faster than the internal pore pressures can readily dissipate.

Seismic Loads - Earthquakes impart both horizontal and vertical accelerations unto a slide mass. Due to their cyclic nature, these dynamic forces briefly reduce stability when acting in an adverse direction to the slope surface. The vibrations may cause densification and the potential for elevated pore pressures and liquefaction.

4.2 St. Mary River Diversion Project Landslides

The sixteen landslides identified in the USBR Landslide Register can be placed into four different categories that describe their general mode of failure and triggering mechanism. These categories are described below along with the Project slides attributed to them.

Toe Erosion - This mode of failure consists of erosion of the slope toe as well as toe softening. Examples of this type of failure include the Camp Nine and Drop No. 2 Slides. Other factors may be contributing to their on-going instability but toe erosion is considered to be the predominant cause and triggering mechanism.

Weak Fill Slopes - Degradation of the downslope fill embankment is the primary reason for this mode of instability. Degradation consists of lateral displacements and settlement. These movements are facilitated by seepage from the canal. Seepage, whether naturally occuring or from the canal, softens the toe, adds hydrostatic and buoyant/artesian forces, and reduces the shear strength at the base of the fill. Seepage may also be progressive leading to internal piping failure. It is postulated that excessively steep fill slopes and inadequate earthwork techniques during original construction may have led to many of the observed seepage and instabilities in the fill embankments. Examples of these slides include, North 700, 4th of July and Gravel Road Bridge Slides.

Buoyant/Artesian Forces - In our opinion, there are at least two slides that have natural tendencies for failure and displacements regardless of canal operations. These are the St. Mary River Siphon and DeWolfe Ranch Slides. The south side of the Siphon Slide exhibits marginal soils, relatively steep terrain, and natural groundwater influxes that create excessive pore pressures. Similarly, the DeWolfe Ranch Slide exhibits marginal soils and destabilizing pore pressures from confined groundwater flows (Figure 18). Obviously the construction and operation of the St. Mary Canal has

contributed to the current instability of these slides, however, it is believed that the instability's existence and propensity may have proceeded the canal.

Cut Sections/Over-Steepened Backslopes - The remaining instabilities and the majority of the subject landslides are the result of over-steepened canal backslopes excavated in marginal soils and/or the removal of a stabilizing toe during original canal construction. Both of these are often exacerbated by saturation and softening of the slope base. Further reductions in the stability occur during years of above precipitation as groundwater impacts the slopes.

Table 6 below lists the landslides of the St. Mary River Diversion and Conveyance Project and their contributing slope stability factors. The table also summarizes the year the landslide was reportedly added to the Register, the year of the last noted slide movement and any slope remediation that may have been performed.

Table 6 - Contributing Slope Stability Factors Impacting Landslides of the St. Mary River Diversion Project

Slide Name (USBR)	Year Added to Register	Year Last Remediated	Year Last Moved	Failure Location	Excessively Steep Sill Slope	Contributing Slope Stability Factors								
						Over-Steepened Backslope	Marginal Natural Slope	Toe Erosion	Toe Softening	Canal Seepage	Marginal Soils	Surcharge Loads	Hydrostatic & Bouyant Forces	Groundwater Seepage
Camp Nine Slide	1995	1998	1995	NS	N	Y	N	Y	N	N	N	N	N	N
St. Mary River Siphon	1996	2006	2007	NS	N	N	Y	N	N	Y	Y	N	Y	Y
DeWolfe Ranch Slide	1995	2003	2005	BS	N	N	Y	N	Y	N	Y	N	Y	Y
DeWolfe Bridge Slide	1996	2003	2002	BS	N	Y	N	N	Y	N	Y	N	Y	Y
Mid-Section 22 Slide	1995	2003	2003	BS	N	Y	N	N	Y	N	Y	N	P	P
North Slope 700 Slide	2005	2007	2006	FILL	Y	N	N	N	Y	Y	Y	N	Y	N
East Section 22 Slide	1995	2002	2005	BS	N	Y	N	N	Y	N	Y	N	P	P
Grizzly Slide	2002	2002	2002	BS	N	Y	N	N	Y	N	Y	N	P	P
Big (Deep) Cut Slides	1995	2006	2006	BS	N	Y	N	N	Y	N	Y	Y	P	P
4th of July Slide	1995	1995	1995	FILL	Y	N	N	N	Y	Y	Y	N	Y	N
Hall Coulee Slide Complex	2002	2003	2004	BS	N	Y	N	N	Y	N	N	N	P	P
Gravel Road Bridge Slide	1997	1997	1997	FILL	Y	N	N	N	Y	Y	Y	N	Y	N
Martin Slides	1995	2003	2005	BS	N	Y	N	N	Y	N	Y	Y	P	P
Pipeline Slide	1995	2004	2002	BS	N	Y	N	N	Y	N	Y	N	P	P
Drop No. 2 Slide	1995	--	2006	NS	N	Y	Y	Y	Y	N	Y	N	N	N
Drop No. 5 Slide	2002	--	2002	BS	N	Y	Y	N	Y	N	Y	N	P	P

NS - Natural Slope
BS - Canal Backslope
N - No
Y - Yes
P- Probable

5.0 RECOMMENDATIONS

5.1 Overview

In 2003, the USBR estimated that approximately $21 million would be required to stabilize the Project landslides. This price included 280,220 cubic yards of excavation, 17,965 cubic yards of gravel cover, 65,764 cubic yards of riprap, a new 1,280 square foot bridge and 8,790 lineal feet of 8-inch perforated drainage pipe. This estimated dollar amount is independent of the final canal capacity (between 500 and 1000 cfs) and was estimated based on rehabilitating the existing canal prism along current alignment.

In 2001, USBR considered construction of an 84-foot high, earthen dam just east of Spider Lake as a means of avoiding the landslides along the existing canal between Spider Lake and Big Cut Slide. This dam would create a pass-through reservoir and as such, the reservoir could not provide any potential storage with respect to the St. Mary River Diversion and Conveyance Facilities. Potential, non-project benefits include recreation and fisheries at the proposed reservoir. This concept was originally conceived in 1912 during the construction of the canal. The approximate location of the right abutment is marked by a soil boring (DH01-SLR) at Sta. 737+50 on Figure 8.

An appraisal level design and cost estimate for this proposal was developed by USBR. The appraisal-level proposal did not assess the magnitude of year-round seepage or evaporative losses. The estimated cost for Spider Lake Dam was $9,600,000 in 2001, with a projected annual Operation, Maintenance and Inspection (OM&I) cost of $5,000. The proposed Spider Lake Dam did not appear cost effective as a slope stability mitigation measure and was not included in the final feasibility cost estimate prepared by USBR for the *North Central Regional Feasibility Study* (USBR, 2004).

Based on our past landslide experience and our observations for this project, it is our opinion that most of the backslope instability issues can be mitigated during canal rehabilitation. Flatter fill and backslopes will increase overall stability. Slope rounding over-steepened backslopes comprised predominantly of stable bedrock will facilitate gentler, overall slopes and reduce the magnitude of excavation in marginal soils. Canal lining in seepage-sensitive fill sections should be considered to

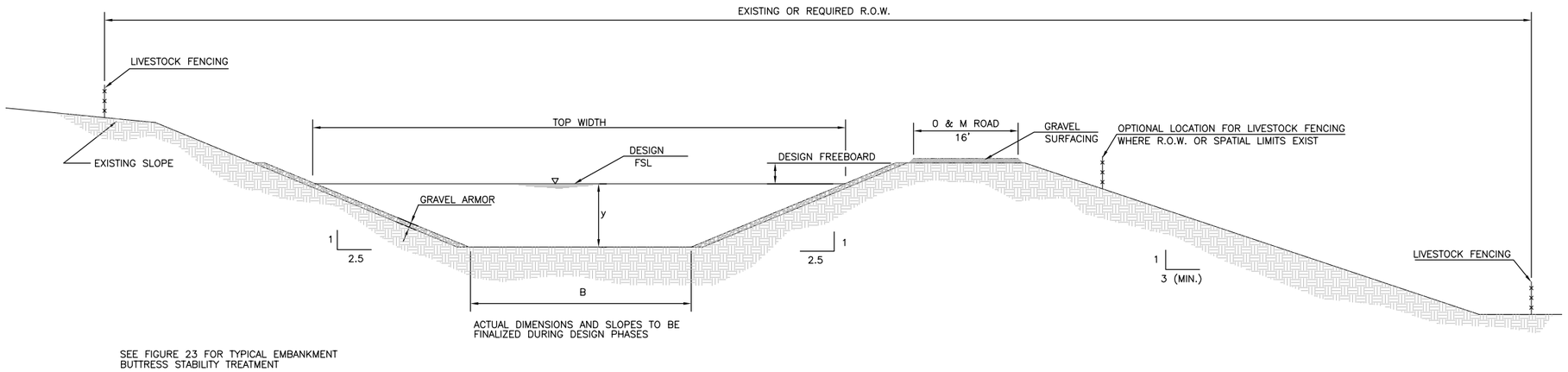
enhance the long-term stability of these downslope embankments. A gravel, gravity toe buttress can be employed where existing subsurface conditions warrant its use. Many of these potential improvements can be further enhanced if the alignment and grade can be shifted up and over onto the subject hillsides above the existing canal. Once a new alignment has been proposed, a detailed geotechnical investigation program is warranted to address these subsurface unknowns. Each of the proposed stability enhancements are discussed in further detail below.

5.2 New Typical Section

The original typical section reportedly consisted of 1½:1 (H:V) backslopes and 2:1 fill slopes. The use of horse-drawn fresnos and the magnitude of the original excavation most likely influenced the decision to steepen the cut and fill slopes thereby reducing the magnitude of excavation and expediting the construction progress. However, in our opinion, these slopes are too aggressive for the existing marginal soil shear strengths and the destabilizing nature of canal seepage. The original canal slopes have further degraded from periodic sloughing and toe erosion and no longer resemble original construction.

In our opinion, based on the native soils, typical inside canal slopes should be a maximum of 2½:1 except in sections that are proposed to be lined. In lined sections, inside canal slopes should be flatter at a maximum of 3:1. Outside fill slopes should be a maximum of 3:1 except in high fill areas which will be governed by the geotechnical analysis and design.

In addition, the new canal prism is proposed to be armored a minimum of 8 inches and 12 inches in lined areas. The armoring protects against sideslope deterioration and internal erosion. Also, the backslope service road acts as a mid-slope bench that improves stability of the lower canal sideslope. The new canal prism proposed in the *Feasibility and Preliminary Engineering Report* (TD&H 2006) is shown on Figure 19.

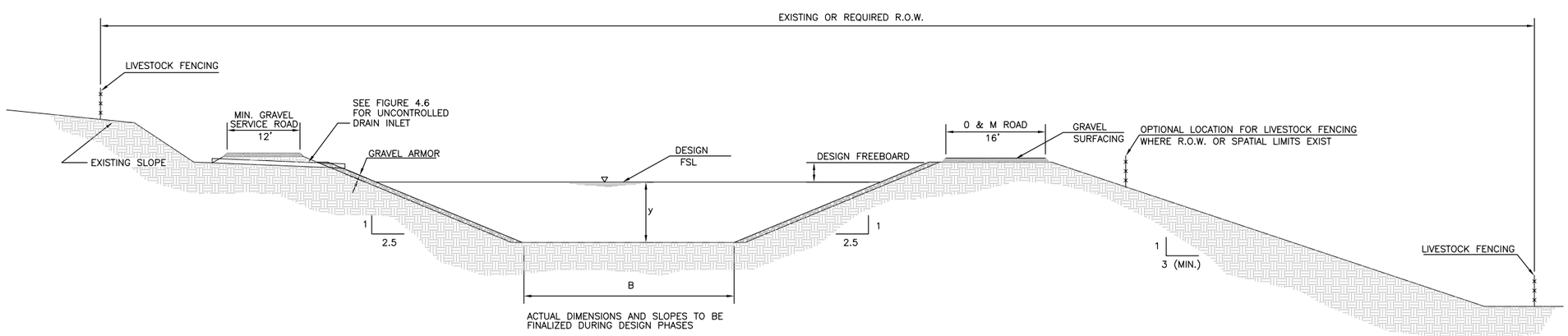


SEE FIGURE 23 FOR TYPICAL EMBANKMENT BUTTRESS STABILITY TREATMENT

ONE - BANK CANAL WITH ARMORING

TYPICAL (N.T.S.)

PRELIMINARY DESIGN CRITERIA	
DESIGN DISCHARGE	650 TO 1000 cfs
FLOW VELOCITY	1.5 TO 2.5 fps
GRADIENT	0.0001 TO 0.0002 ft/ft
MANNING'S ROUGHNESS	0.020 TO 0.025
INSIDE SIDE SLOPES	2.5:1 (MAX.)
OUTSIDE SIDE SLOPES	3:1 (MAX.)
ARMOR THICKNESS	6" TO 8"
B TO Y RATIO	3 TO 5
FREEBOARD	0.375y + 0.67
MIN. CURVATURE	5x TOP WIDTH



SEE FIGURE 23 FOR TYPICAL EMBANKMENT BUTTRESS STABILITY TREATMENT

TWO - BANK CANAL WITH ARMORING

TYPICAL (N.T.S.)

FIGURE 19

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PROPOSED TYPICAL CANAL SECTIONS

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The slope of the excavation backslopes will be a function of the existing terrain and the subsurface soil and groundwater conditions. As part of the preliminary design phase, these subsurface conditions need to be adequately identified during a comprehensive geotechnical program. The results of the Test Pits excavated for this study revealed the presence of shallow bedrock above the existing backslopes and that slope failure for the DeWolfe Ranch and Bridge Slides appears to be related to the soil-bedrock contact and, in some locations, to the potential pore pressures generated from the artesian groundwater as described earlier.

5.3 New Alignment

One major hurdle to canal rehabilitation along the same alignment is that the work must be performed in the winter (mid-September to mid-March) in order to avoid disruptions of water deliveries to the Milk River Basin. Another drawback is that any earthwork within the existing prism will most likely involve soft, wet soils that are difficult to place and compact during adverse winter weather conditions. Providing an alternate alignment outside the current prism allows summer construction to occur and reduces the quantity of undesirable wet earthwork. Both of these benefits will save considerable expense, time, and improve the quality of the work that is to be performed.

The troublesome stretch between the former Spider Lake outlet check structure and Big Cut Slide is situated on a relatively steep hillside. This affords little opportunity for canal realignment at the same grade. However, if the canal grade could be raised 5 to 15 feet, there would be additional lateral space to construct the new canal prism. Sufficient centerline offset would also permit summertime construction. In many cases, the shift in alignment would also result in an overall gentler footprint on the marginal slopes. The downslope embankment fill could be keyed into the existing prism. Excess excavation could be used to buttress existing fill slopes. This concept is presented in Figure 20.

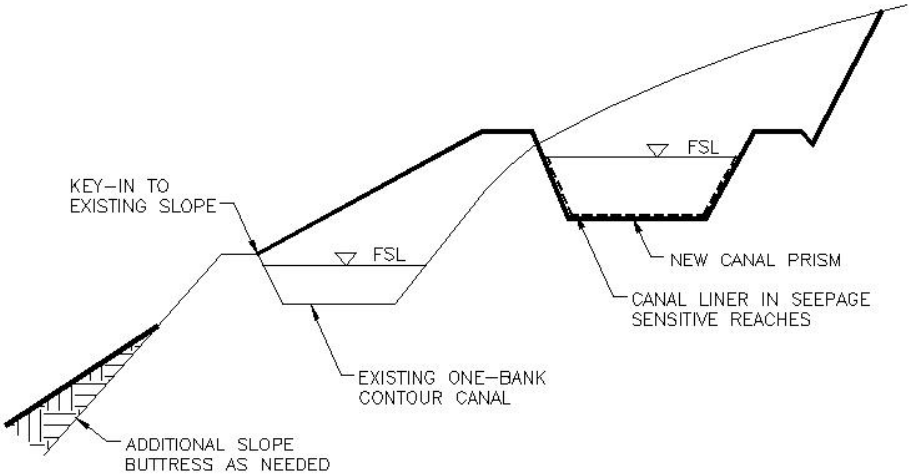

Figure 20. The advantages of being able to raise and realign the canal centerline.

The potential magnitude of canal realignment and grade raise has yet to be determined. This involves an overall grade and alignment study from the diversion dam to the North Fork of the Milk River. A proposed realignment was presented in the *Feasibility and PER* (TD&H, 2006) that reduced the overall length of the facilities by 9,450 feet (1.8 miles). This type of length reduction provides an opportunity to gain elevation. Also, improvements in the hydraulics of the St. Mary Siphon may also provide additional elevation gains. Additional head losses would be introduced with the new fish screens. This may be overcome by increasing the diversion dam crest elevation. These analyses are beyond the scope of this study.

A proposed alignment is shown on Figure 21. The feasibility of this alignment is assumed and is meant to illustrate a potential scenario should sufficient elevation gains be achieved prior to Spider Lake.

5.4 Slope Rounding

When bedrock is encountered in the backslope, compound backslopes with slope rounding is often implemented and designed. This earthwork technique is used frequently in highway and railroad design and minimizes the magnitude of excavation by utilizing the inherent strength of the bedrock. The excavation in the soil overlying the bedrock would typically be 2:1 or whatever is geotechnically permitted.

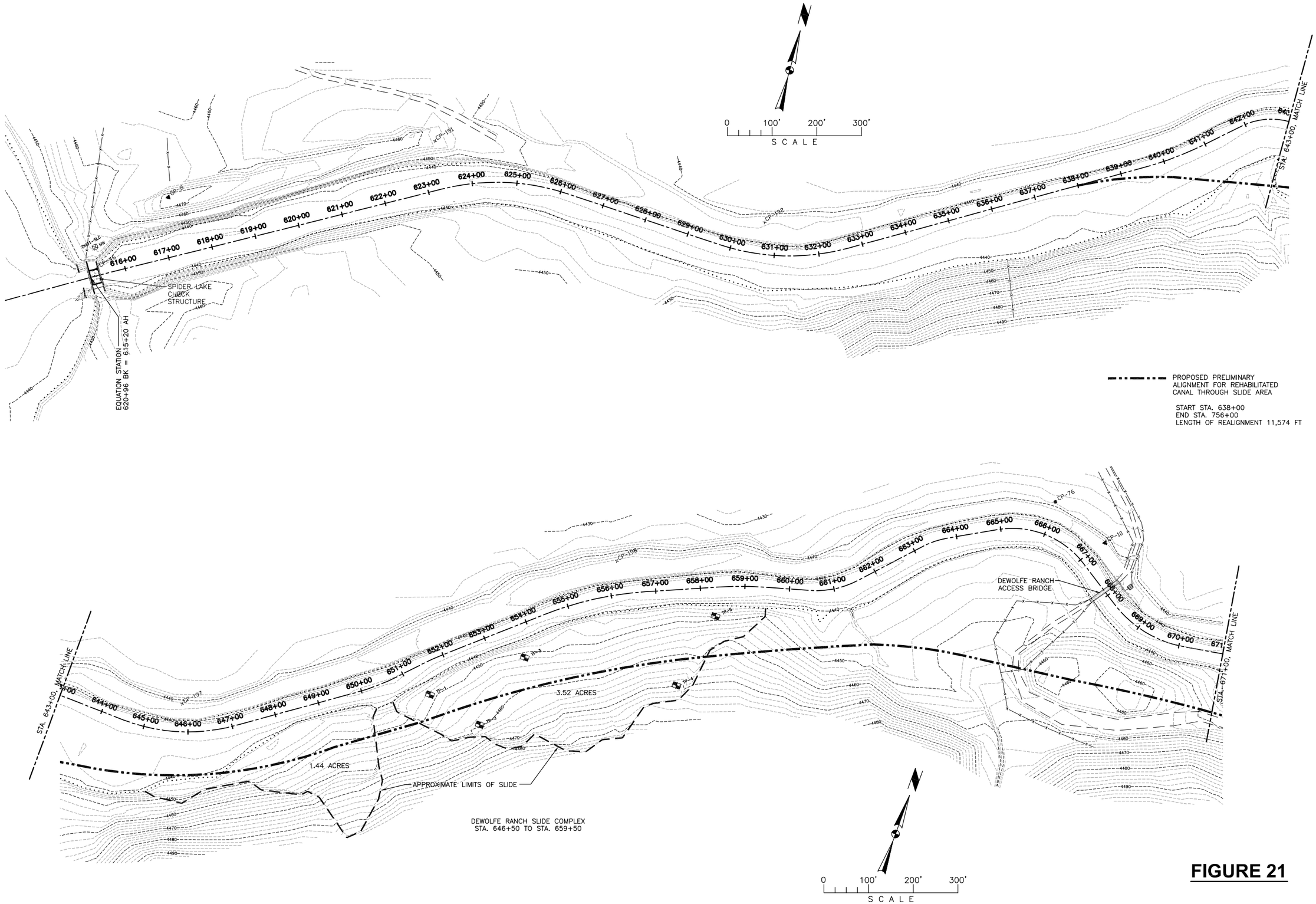

FIGURE 21

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PRELIMINARY ALIGNMENT
SPIDER LAKE TO BIG CUT SLIDE

CAD NO. 04167-9F21A .DWG

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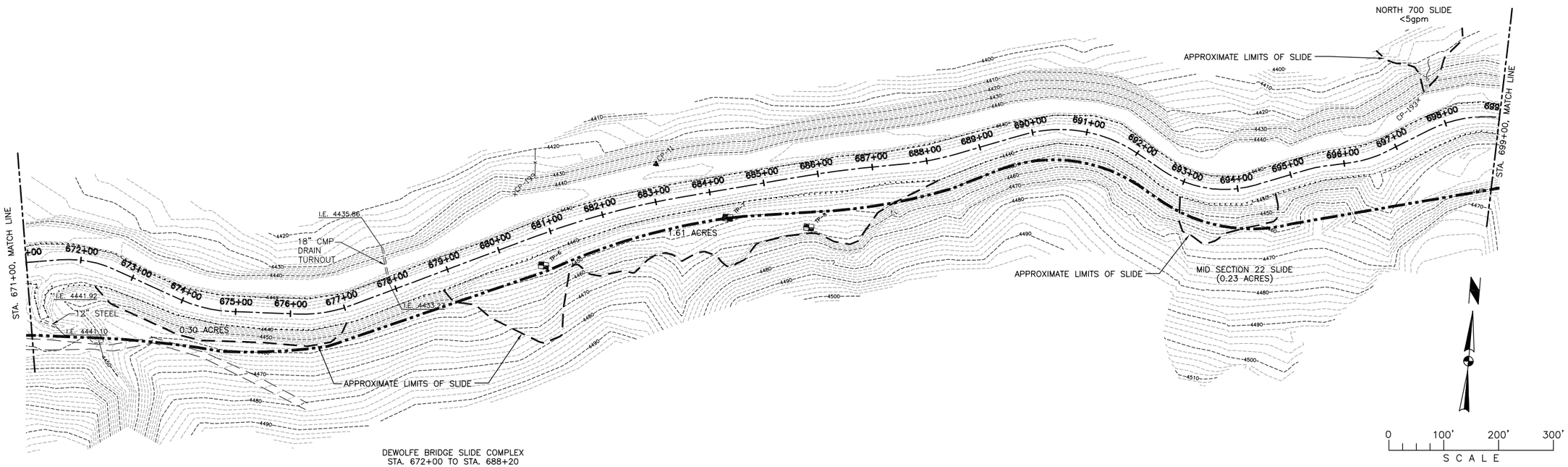
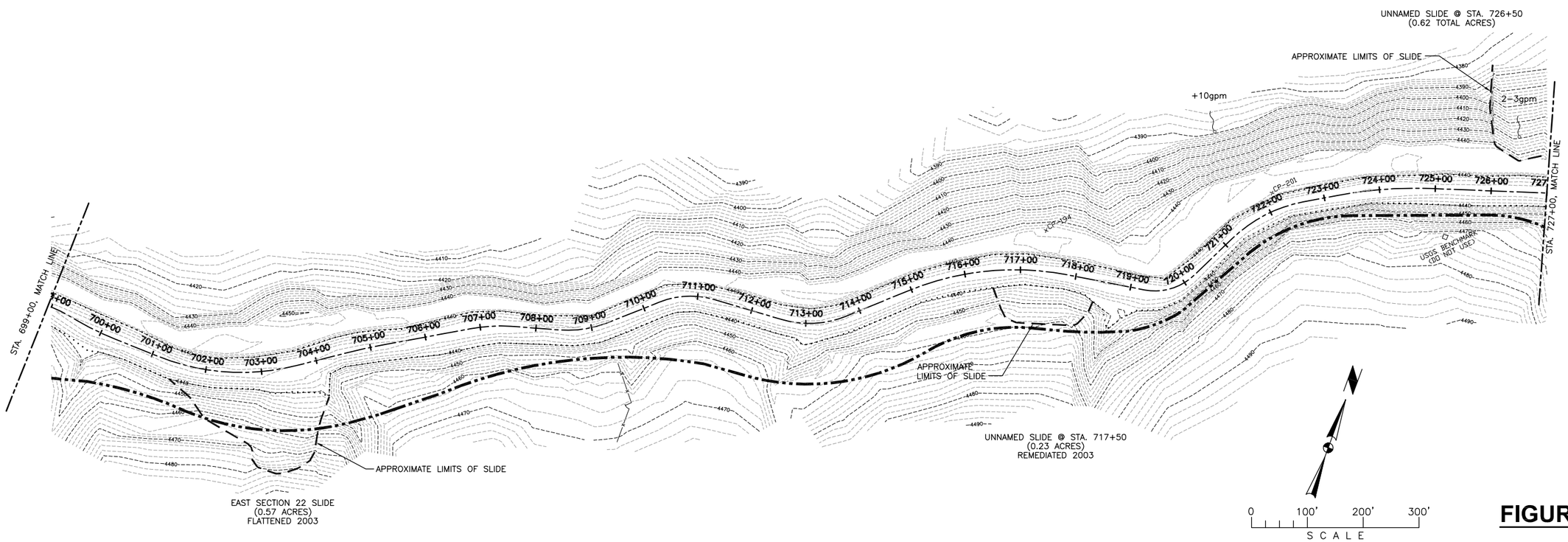


DEWOLFE BRIDGE SLIDE COMPLEX
STA. 672+00 TO STA. 688+20



UNNAMED SLIDE @ STA. 717+50
(0.23 ACRES)
REMEDIATED 2003

EAST SECTION 22 SLIDE
(0.57 ACRES)
FLATTENED 2003

FIGURE 21

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PRELIMINARY ALIGNMENT
SPIDER LAKE TO BIG CUT SLIDE

CAD NO. 04167-9F21B .DWG
SHEET 2 OF 3

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FIGURE 21

MONTANA
ERLING A. JUEL
10740PE
REGISTERED
PROFESSIONAL ENGINEER

REVISIONS

BY	DATE	DESCR
BY	DATE	DESCR
BY	DATE	DESCR

TD&H

THOMAS, DEAN & HOSKINS, INC.
ENGINEERING CONSULTANTS
GREAT FALLS–BOZEMAN–KALISPELL–HELENA
SPOKANE
LEWISTON
MONTANA
WASHINGTON
IDAHO

DRAWN BY:	WAB
DESIGNED BY:	EAJ
QUALITY CHECK:	
DATE:	01.17.08
JOB NO.	04-167
FIELDBOOK	

DNRC - CARDD
ST. MARY DIVERSION FACILITIES

PRELIMINARY ALIGNMENT
SPIDER LAKE TO BIG CUT SLIDE

CAD NO. 04167-9F21C .DWG
SHEET 3 OF 3

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Shallow soil layers are typically rounded over a short distance to match preexisting slopes. For excavations in marginal soil conditions, this approach minimizes overall slope steepening. This technique also minimizes Right-of-Way acquisition. A conceptual depiction of this approach is shown in Figure 22 below.

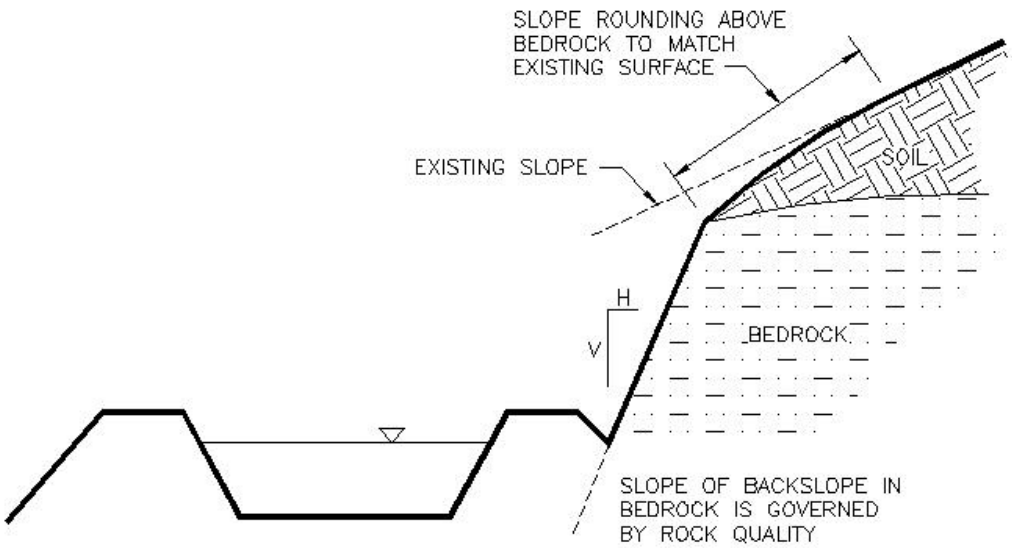
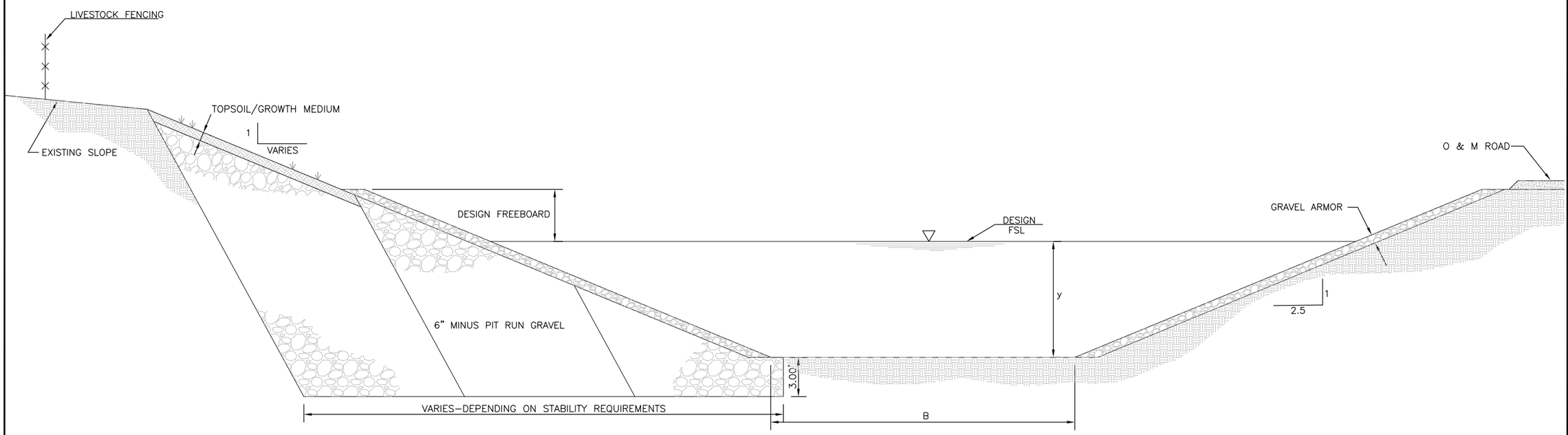


Figure 22. The Concept of a Compound Backslope and Slope Rounding.

5.5 Gravel Toe Buttress

For landslides and instabilities that intersect the canal prism, the excavation of the slide toe while creating the new canal further reduces stability. It is necessary to replace the excavated soil buttress with a gravel toe buttress. The volume of the gravity buttress would be determined during final design and would use the results of a comprehensive geotechnical investigation program. The gravel would provide more weight than the removed soil. Also, the gravel buttress would exhibit a higher shear strength and would be less susceptible to erosion, sloughing and rapid drawdown. A typical gravel toe buttress is shown on Figure 23.



ACTUAL BUTTRESS DIMENSIONS WILL BE A FUNCTION OF
GEOTECHNICAL PARAMETERS AND STABILITY REQUIREMENTS

EMBANKMENT BUTTRESS STABILITY TREATMENT

TYPICAL (N.T.S.)

FIGURE 23

MONTANA
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QUALITY CHECK:	
DATE:	01.16.08
JOB NO.	04-167
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TYPICAL BUTTRESS STABILITY TREATMENT

CAD NO. 04167-9F23 .DWG
SHEET 1 OF 1

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5.6 Canal Lining

The use of canal lining may be implemented in certain reaches where seepage losses could be detrimental to the downslope fill embankments. Linings may be placed only on the bottom when losses are predominantly in the vertical direction. Lining of the canal slope on the backslope side should be done with caution to avoid issues with upslope groundwater seepage. If fractured bedrock is observed in the canal prism, it may be necessary to over-excavate the bedrock and backfill with excess excavation of glacial till clay. This earthen liner would help control seepage and could be placed during original construction.

5.7 Summary

Sixteen landslides are currently listed in the USBR Landslide Register for the St. Mary River Diversion and Conveyance Facilities. Of these, fourteen are related to either the canal backslopes or the fill embankments. In 2003, the USBR estimated that $21 million would be required to remediate these instabilities within the rehabilitated prism following the current alignment. In our opinion, there are several approaches and techniques that could be implemented to increase bank stability during canal rehabilitation. Several of these could be applied along with a shift in canal alignment and a raise of the canal grade. A Preliminary Grade and Alignment (PGA) study is recommended to determine the effectiveness of this approach. This would be followed by a comprehensive geotechnical investigation along the new proposed canal centerline.

6.0 REFERENCES

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APPENDIX

LOG OF TEST PIT TP-1

Location:	Sta. 651+34
Offset:	69', Rt.
Elevation:	4448.5
Date:	10/22/07
Excavated by:	USBR using KOBELCO 914 Tracked Excavator
Logged by:	E.A. Juel, P.E., & B. Colenso, E.I.T. with TD&H

Depth	USC Soil Classification	Soil Description
0.0' - 0.8'	OL	**TOPSOIL**, organic material
0.8' - 4.5'	CL	**LEAN CLAY**, varying sand content, trace subrounded fine gravel, occasional cobbles, high plasticity, moist, grayish brown (Glacial Till). Gravel = 2% Sand = 23% Silt/Clay = 75% LL = 46% PI = 36% ω = 22% @ 3.25'
4.5' - 11.5'	CL	Sandy,**LEAN CLAY**, trace subrounded gravel, moist, grayish brown (Glacial Till). Gravel = 12% Sand = 35% Silt/Clay = 53% LL = 24% PI = 10% ω = 15%
11.5'		Groundwater seepage encountered
11.5' - 12.5'	GC	**CLAYEY GRAVEL** with sand, cobbles up to 10", subrounded to subangular, plastic fines, moist, grayish-brown. ω = 15% @ 9.5'
12.5' - 15.0'	---	**SANDSTONE**, medium-grained, moderately weathered, moderately fractured, harder with depth, olive to greenish brown.
15.0'		Bottom of Test Pit

LOG OF TEST PIT TP-2

Location:	Sta. 652+19
Offset:	176’, Rt.
Elevation:	4459.2
Date:	10/22/07
Excavated by:	USBR using KOBELCO 914 Tracked Excavator
Logged by:	E.A. Juel, P.E., & B. Colenso, E.I.T. with TD&H

Depth	USC Soil Classification	Soil Description
0.0’ - 0.3’	OL	**TOPSOIL**, organic material
0.3’ - 8.0’	---	**SANDSTONE**, medium-grained, slightly weathered, highly-fractured, olive to greenish brown.
8.0’		Bottom of Test Pit Groundwater Not Encountered

LOG OF TEST PIT TP-3

Location:	Sta. 653+68
Offset:	73', Rt.
Elevation:	4451.6
Date:	10/22/07
Excavated by:	USBR using KOBELCO 914 Tracked Excavator
Logged by:	E.A. Juel, P.E., & B. Colenso, E.I.T. with TD&H

Depth	USC Soil Classification	Soil Description
0.0' - 0.8'	OL	**TOPSOIL**, organic material
0.8' - 8.5'	CL	**LEAN CLAY**, varying sand content, trace subrounded fine gravel, high plasticity, moist, grayish brown (Glacial Till). Gravel = 2% Sand = 70% Silt/Clay = 28% LL = 38% PI = 23% ω = 18% @ 4.25'
8.5' - 19.5'	CH	**FAT CLAY**, relatively stiff, subrounded gravels, very moist grayish-brown (Glacial Till). Gravel = 1% Sand = 7% Silt/Clay = 92% LL = 62% PI = 42% ω = 32% @ 19.0'
19.5'		Bottom of Test Pit Groundwater Not Encountered

LOG OF TEST PIT TP-4

Location:	Sta. 657+33
Offset:	213', Rt.
Elevation:	4469.9
Date:	10/22/07
Excavated by:	USBR using KOBELCO 914 Tracked Excavator
Logged by:	E.A. Juel, P.E., & B. Colenso, E.I.T. with TD&H

Depth	USC Soil Classification	Soil Description
0.0' - 0.8'	OL	**TOPSOIL**, organic material
0.8' - 18.0'	CL	**FAT CLAY** with sand, trace subrounded fine gravel, high plasticity, moist, grayish brown (Glacial Till). Gravel = 8% Sand = 20% Silt/Clay = 75% LL = 51% PI = 35% ω = 21% @ 13.0'
18.0'		Bottom of Test Pit Groundwater Not Encountered

LOG OF TEST PIT TP-5

Location:	Sta. 658+30
Offset:	64', Rt.
Elevation:	4448.0
Date:	10/22/07
Excavated by:	USBR using KOBELCO 914 Tracked Excavator
Logged by:	E.A. Juel, P.E., & B. Colenso, E.I.T. with TD&H

Depth	USC Soil Classification	Soil Description
0.0' - 0.8'	OL	**TOPSOIL**, organic material
0.8 - 10.0'	CL	**LEAN CLAY**, trace subrounded fine gravel, high plasticity, moist to very moist, grayish brown (Glacial Till).
10.0' - 11.0'	GC	**CLAYEY GRAVEL** with sand, cobbles up to 10", subrounded to subangular, plastic fines, moist, light grayish-brown.
12.5' - 15.0'	---	**SANDSTONE**, medium-grained, moderately weathered, slightly fractured, olive to greenish brown.
12.0'		Bottom of Test Pit No Groundwater Encountered

LOG OF TEST PIT TP-6

Location:	Sta. 680+67
Offset:	55', Rt.
Elevation:	4445.8
Date:	10/22/07
Excavated by:	USBR using KOBELCO 914 Tracked Excavator
Logged by:	E.A. Juel, P.E., & B. Colenso, E.I.T. with TD&H

Depth	USC Soil Classification	Soil Description
0.0' - 0.8'	OL	**TOPSOIL**, organic material
0.8' - 5.0'	CL	**LEAN CLAY**, varying sand content, trace subrounded fine gravel, high plasticity, moist, grayish brown (Glacial Till).
5.0' - 8.0'	SM-SC	**SILTY, CLAYEY SAND** with gravel, cobbles up to 8", subrounded, plastic fines, moist, light grayish-brown. Gravel = 30% Sand = 32% Silt/Clay = 38% LL = 24% PI = 7% ω = 7% @ 7.5'
8.0' - 9.0'	---	**SANDSTONE**, medium-grained, moderately weathered, slightly fractured, olive to greenish brown.
9.0'		Bottom of Test Pit Groundwater Not Encountered

LOG OF TEST PIT TP-7

Location:	Sta. 684+24
Offset:	51', Rt.
Elevation:	4448.4
Date:	10/22/07
Excavated by:	USBR using KOBELCO 914 Tracked Excavator
Logged by:	E.A. Juel, P.E., & B. Colenso, E.I.T. with TD&H

Depth	USC Soil Classification	Soil Description
0.0' - 0.5'	OL	**TOPSOIL**, organic material
0.5' - 3.0'	CL	**LEAN CLAY**, varying sand content, trace subrounded fine gravel, high plasticity, moist, grayish brown (Glacial Till).
3.0' - 9.0'	GC	**SILTY, CLAYEY SAND** with gravel, subrounded to subangular, cobbles up to 10", moist, light brown.
9.0' - 9.5'	---	**SANDSTONE**, medium-grained, olive to greenish brown.
9.5'		Bottom of Test Pit Groundwater Not Encountered

LOG OF TEST PIT TP-8

Location:	Sta. 685+67
Offset:	93', Rt.
Elevation:	4457.4
Date:	10/22/07
Excavated by:	USBR using KOBELCO 914 Tracked Excavator
Logged by:	E.A. Juel, P.E., & B. Colenso, E.I.T. with TD&H

Depth	USC Soil Classification	Soil Description
0.0' - 0.3'	OL	**TOPSOIL**, organic material
0.3' - 4.0'	---	**SANDSTONE**, medium-grained, olive to greenish brown.
4.0'		Bottom of Test Pit Groundwater Not Encountered