Seepage Monitoring Points

There are six seepage monitoring points for Fruitgrowers Dam. Seepage monitoring point SM-2 is the outfall for the original spillway drain system. Seepage from this point typically ranges from 10 to 30 gallons per minute. Seepage flows typically peak in the late summer and early fall and correlate with irrigation periods on Antelope Hill. It is likely that the source of the seepage from this drain is flow through joints, shears, and fractures in the Mancos Shale.

ESTIMATED SLIDE PLANE STRENGTHS

Laboratory Testing

Testing the slide plane material through traditional methods such as direct shear testing was found to be nearly impossible because the slide plane materials typically consist of saturated gravelly clay in the shear zones of the landslide. The samples that were chosen for testing disintegrated during the application of normal stress. Therefore, the residual shear strength of the slide plane material was estimated using torsional ring shear tests. Samples of the Mancos Shale in the slide plane were ground and remolded to sample size. The samples were then tested in the torsional ring shear test apparatus in conformance with ASTM D6467. Test results indicate a slide plane residual shear strength friction angle between 3.5 and 5 degrees.

The residual shear strength friction angle test results are lower than would be predicted using typical correlation relationships (such as Mitchell, 2005, Skempton, 1985 or Kenney, 1967). There is no direct explanation for the lower shear strengths at the Fruitgrowers site. However, the test results are similar to some residual shear strength tests of other claystones across the Front Range (Dewoolkar and Huzjak, 2005).

Landslide Back Analysis

The shear strength of the slide plane material was also estimated through back analysis, using the slide plane geometry from the inclinometer data. The landslide slope was analyzed using GeoStudios SLOPE/W stability analysis software (GEO-SLOPE, 2013). The piezometric surface in the slope was estimated using average ground water table readings from piezometers located across the site. A residual shear strength friction angle in the slide plane of 5 degrees resulted in a factor of safety of 1.0 for the slope.

RISK ANALYSIS

Estimates of dam failure risk using the expert elicitation and event tree methodology require a quantification of the probability of the loading, the potential structural response of the dam given the load, and a definition of the sequence of events which could lead to dam failure. The risk analysis focused on potential failure modes that could lead to dam failure caused by or exacerbated by movement of the left abutment landslide. Each potential failure mode was fully defined based on the likely progression of events from initiation to failure and was then deconstructed into separate events with a team estimate of event probability for each event. Probability estimates were gathered using the direct and scoring methods as described by Ayyub, 2000.

Potential Failure Modes

The potential failure modes considered in the risk analysis are as follows:

Landslide Movement Results in Internal Erosion of the Embankment

The landslide on the left abutment is moving into and around the left end of the embankment and is likely causing stress in the embankment. The stress due to movement of the landslide leads to cracking or other flaws developing in the embankment. Seepage initiates through the cracks and flaws in the embankment. The seepage gradient through the cracks is sufficient to initiate erosion through scour processes. In the event that a filtered exit, self-healing materials, or flow limits do not exist or are inadequate, continued flows enlarge the erosion paths. If the process is not detected, increased erosion leads to breach of the dam and release of the reservoir, through collapse or gross enlargement of the internal void.

The risk associated with this potential failure mode was estimated using the following events:

1. Landslide creep does not stop or slow in the year under consideration.
2. A continuous flaw or pattern of flaws develops in the embankment due to landslide movement.
3. Internal erosion initiates by scour.
4. An unfiltered exit exists.
5. Internal erosion progresses and does not self-heal.
6. Internal erosion progresses and there is no flow limiting or other limits to the progression.
7. Intervention fails.
8. The dam breaches through crest collapse and overtopping or gross enlargement of the seepage path.

The risk team considered the low, yet constant, movement rates of the landslide to be critical to this potential failure mode. However, the risk team felt that the relatively plastic materials in the embankment would likely resist cracking and deformation as shown by the successful performance of the dam since construction despite the landslide movement. The risk team also considered the potential evidence indicating a lack of direct landslide movement into the embankment from the inclinometers in and near the embankment as critical for estimating the risk. The risk from this potential failure mode was judged to be relatively low.

Internal Erosion through the Left Abutment Slide Mass

Geologic investigations at the site revealed several gravelly, shear zones in the Mancos Shale typically at slide plane elevations. The continued movement of the landslide results in a seepage path being formed upstream to downstream through the shear zones present at the left abutment landslide contact. Seepage initiates through the shear zones. The seepage gradient through the shear zones is sufficient to initiate erosion through scour processes. In the event that a filtered exit, self-healing materials, or flow limits do not exist or are inadequate, continued flows enlarge the erosion path. If the process is not detected, increased erosion leads to breach of the dam and release of the reservoir, through collapse or gross enlargement of the internal void.

The risk associated with this potential failure mode was estimated using the following events:

1. The reservoir reaches a critical pool elevation.
2. A relatively continuous flaw exists in the left abutment.
3. Internal erosion initiates by scour.
4. An unfiltered exit exists.
5. Internal erosion progresses and there is no flow limiting or self-healing.
6. Intervention fails.
7. The dam breaches through crest collapse and overtopping or gross enlargement of the seepage path.

The risk team observed that seepage emanating from the original spillway drain system indicates that seepage through the shears and joints in the Mancos Shale is probable. However, the landslide is located high enough on the left abutment that the risk team found it unlikely that seepage could be driven through the left abutment by the reservoir and judged the risk from this potential failure mode to be relatively low.

CONCLUSIONS

The extent of the landslide in the left abutment of Fruitgrowers Dam has been well defined through instrumentation monitoring. The landslide moves at a relatively constant rate of approximately 0.5 to 1.0 inches per year. There is evidence from the instrumentation data that the landslide is not moving directly into the embankment. The piezometric elevations in the left abutment do not appear to directly impact the landslide movement rates.

The landslide is well monitored through a robust instrumentation monitoring plan. Reclamation considers instrumentation to be a vital tool in an on-going review of dam safety (Bartholomew, Murray, and Goins, 1987). Inclinometer and piezometer data are collected monthly. Survey point data are collected annually. The embankment is visually inspected on a monthly basis with an emphasis on changes in structural or seepage conditions.

Any changes in instrumentation readings that are outside of anticipated operating conditions are reviewed by a team of engineers to determine if the instrumentation results impact the risks at the facility. The instrumentation data are discussed to determine if the monitoring indicates a potential change in the probability of any of the events related to potential failure modes. This relatively constant assessment of the performance of the dam is vital to the continued operation of the dam given the presence of the landslide in the left abutment.

The estimated risk to the facility is near Reclamation guideline values which indicates some justification for further actions regarding the risks. The risk analysis team concluded that structural modifications were not justified based on the historical performance of the embankment. The risk analysis team recommended continuing to monitor landslide activity including the placement of an additional inclinometer at the left end of the embankment.

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The Michigan Ditch Landslide and Tunnel Case History and Unique Delivery Method

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Abstract: The Michigan Ditch is located in Jackson County, Colorado and is part of the City of Fort Collins water supply system. It conveys water from the west side of the Never Summer Mountains to Joe Wright Reservoir on the east side of the range. The 5.2 mi long ditch was constructed in the early 1900's and has been plagued by slope instability since it was built. An area known as the "Mudslide" has experienced accelerated slope instability in recent years and the piped portion of the ditch through the landslide was destroyed in 2015 when the landslide moved over 20 ft horizontally and 7 ft vertically. Following an extensive geotechnical investigation utilizing vertical and horizontal borings at the site, several mitigation alternatives were evaluated, the associated risks were weighed, and a tunnel behind and under the landslide was selected. The project was designed and will be constructed utilizing the City of Fort Collins utilities alternative product delivery system (APDS). APDS is a collaborative approach unique to the City of Fort Collins utilities and focuses on delivering quality projects that are built on time and on or under budget. The total construction cost for this project is estimated at $8.5 million dollars and will be constructed between May 2016 and October 2016.

INTRODUCTION

Michigan Ditch is part of the City of Fort Collins (City) water supply system and conveys water from the west side of the Never Summer Mountain to Joe Wright Reservoir on the east side of the range. The ditch is located in Jackson County, Colorado, approximately 60 miles west of the intersection of State Highways 14 and

287 (Figure 1). Specifically, the project site known as the "Mudslide," is located approximately 1.7 miles southwest of State Highway 14 (SH 14) at Cameron Pass, along the Michigan Ditch access road. The roadway along the ditch is a heavily used hiking trail and is part of the Colorado State Forest system.

Michigan Ditch is approximately 5.2 miles long; starting at a diversion structure below Lake Agnes and terminating at the top of Cameron Pass. The ditch was constructed over a century ago and, since that time, has undergone numerous improvements that include several diversion structures that capture water from local drainages and piped portions of the ditch. The elevation of the ditch at the upstream end, near the Lake Agnes diversion structure, is approximately 10,484 ft, while the elevation of the ditch at the downstream end at SH 14 is approximately 10,274 ft. The average grade along the 5.2 mile alignment is approximately 0.7 percent. The ditch was purchased by the City approximately 40 years ago and the City has operated and maintained the ditch and ditch road since that time with a regular maintenance crew at the ditch during the spring, summer, and fall seasons.

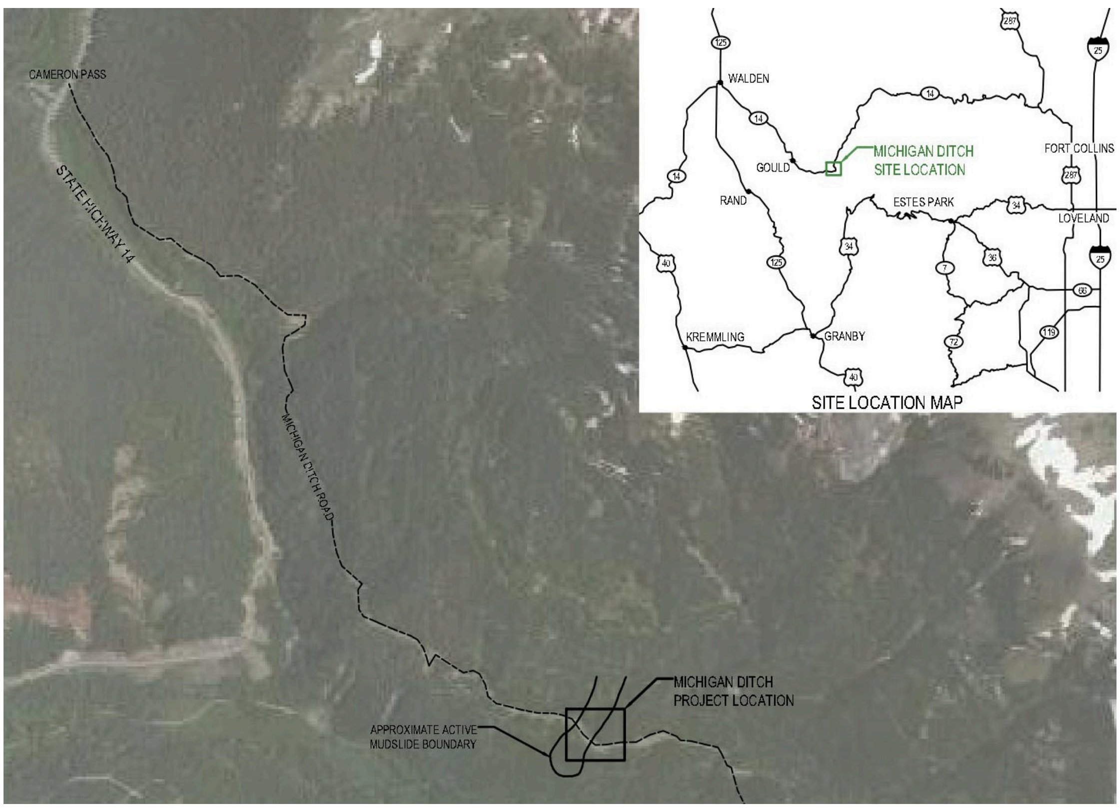


FIG. 1. Project Location.

The 5.2-mile alignment has been plagued with slope instability since the ditch was constructed. In 2011, members of the design team mapped and cataloged 50 individual landslides, 18 slump features, two areas of rockfall, and 12 avalanche chutes that cross the ditch. By far, the largest landslide along the alignment is the Mudslide. The geologic hazards mapped in 2011 are now part of an annual monitoring program that includes photographic documentation and survey monitoring points. To date, no significant movement has occurred on the majority of the mapped landslides or rockfall area. Several of the slumps have moved over the years, some of which have blocked the ditch but, thanks to the proactive approach of the City, were removed before significant overtopping of the ditch bank and roadway occurred and damaged the infrastructure.

In 2015, the Mudslide moved significantly and typical repairs that had been done in the past on the piped portion of the ditch across the landslide were deemed to be inadequate. The City and their design and construction team came together to discuss the project goals and constraints and evaluate mitigation alternatives, construction costs, and schedule. The City of Fort Collins Utilities (Utility) utilized a unique alternative delivery method known as the Alternative Product Delivery Method (APDS) to tackle the project. Numerous mitigation alternatives were evaluated and compared to the associated risks. The value of the water flowing through the ditch is estimated between $150 and $300 million and, as the bulk of the collected water passes through the pipe portion of the ditch on the Mudslide, the City could only afford to miss one season of collection and distribution. During the summer of 2015, a significant geologic and geotechnical investigation was conducted at the site and the design and construction team worked through potential mitigation alternatives. Ultimately, a tunnel alternative that conveys water behind the landslide was selected. Construction of the tunnel will begin in May, 2016 and is scheduled for completion by late October, 2016.

The subsequent sections of this paper describe the Mudslide, geologic and geotechnical investigations, alternatives evaluation, tunnel design, risk analysis, and how APDS will be utilized to successfully deliver this project.

THE MUDSLIDE

The Mudslide is a mapped geologic feature (Braddock and Cole, 1990) (Figure 2). The mapped or historic landslide is approximately 2,200 ft long, up to 800 ft wide, with approximately 660 ft of vertical relief. The total area of the historic mapped landslide is just over 32 acres. The average slope of the ground surface along the axis of the landslide is 30 percent; however, portions of the landslide are flat while other areas of the slide have near vertical slopes. Photograph 3 is of the Mudslide taken from across the valley, only the lower portion of the landslide can be seen.

Geologic mapping conducted in 2011 revealed approximately 60 percent of the historic landslide is active. Refined geologic mapping revealed the active landslide is 2,340 ft long, up to 565 ft wide, with approximately 730 ft of vertical relief. The area of the active landslide is estimated at 19 acres. Geologic mapping conducted in 2011 revealed the active portion of the landslide extends approximately 300 ft further downslope than the mapped historic slide (Figure 4). A geotechnical investigation conducted in 2011, revealed the landslide deposit was up to 90 ft thick and included woody debris at the base of the slide deposit. This was later confirmed during a second geotechnical investigation in 2015.

The Mudslide has experienced slope instability that has affected the ditch and roadway since the ditch was constructed. The ditch was piped through this area almost four decades ago in an attempt to mitigate the effects of the active landslide. Due to

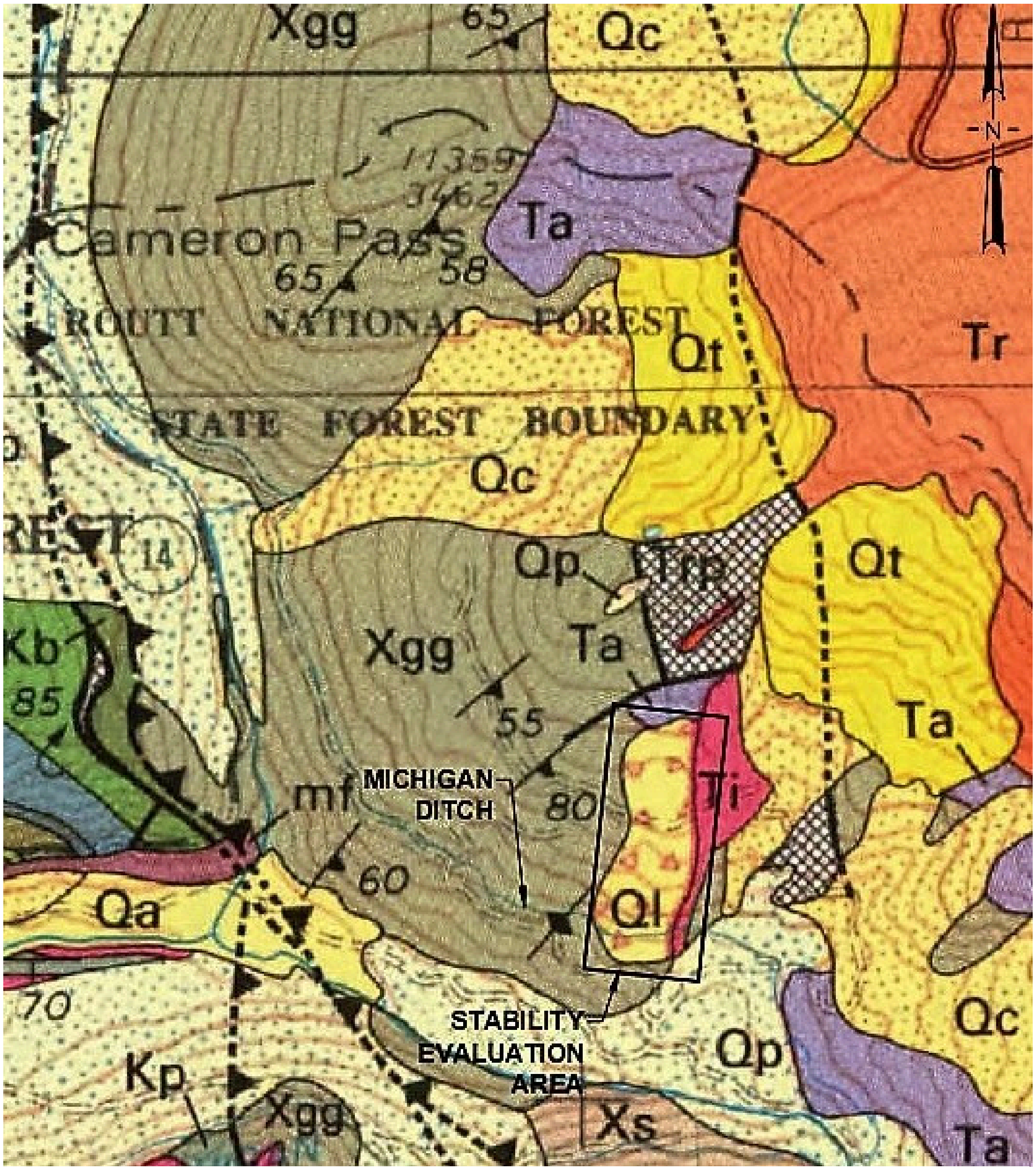
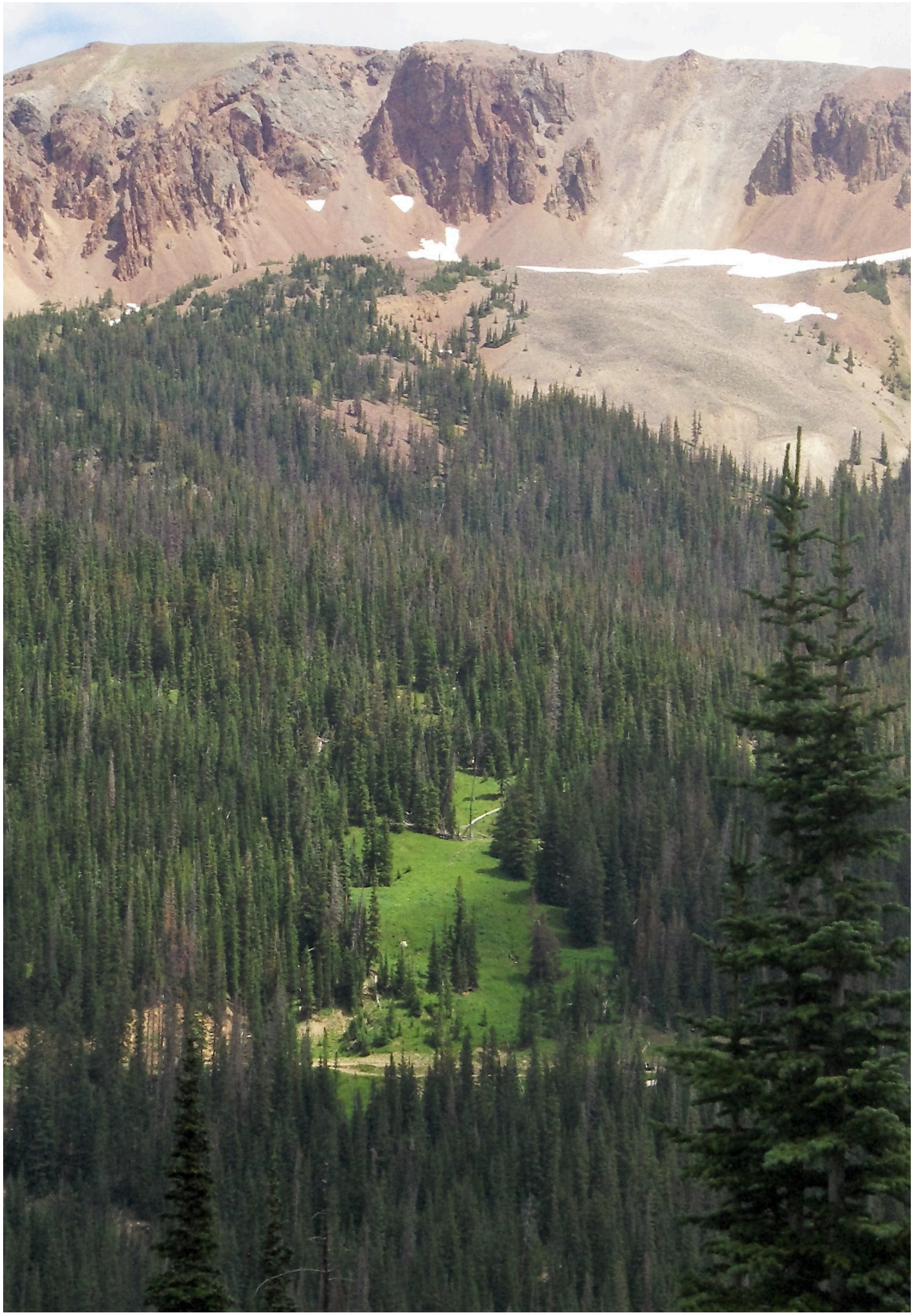


FIG. 2. Geologic Map Showing Mudslide (Ql) Area, Braddock and Cole, 1990.

continued movement of the landslide and damage to several corrugated metal pipes that were installed over the years, the piped portion of the ditch was reconstructed approximately 15 years ago utilizing a 54 inch diameter mortar lined, ductile iron pipe. Since the new pipe was installed, movement of the landslide has required the City to expose the pipe, push it back to its desired alignment, and re-cover it with onsite soils every three to five years. On average, the landslide movement ranged from less than a foot to 4 ft horizontally and up to 2 ft vertically. To date, the City has repaired the pipe in this manner approximately four times with a construction cost of $10,000 to $20,000 per event.



PHOTOGRAPH 1. Mudslide From Across the Valley.

In the Spring of 2014, the City maintenance crews started clearing snow from the ditch and roadway and noticed significant movement had occurred at the Mudslide and noted several transverse and lateral scarps that had resulted in approximately 4 ft of settlement along the ditch road. Inclinometers installed in 2011 were measured and