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IN-SITU MONITORING OF INFILTRATION-INDUCED INSTABILITY OF I-70 EMBANKMENT WEST OF THE EISENHOWER-JOHNSON MEMORIAL TUNNELS, PHASE II

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16. Abstract
Infiltration-induced landslides are common hazards to roads in Colorado. A new methodology that uses recent advances in unsaturated soil mechanics and hydrology was developed and tested. The approach consists on using soil suction and moisture content field information in the prediction of the likelihood of landslide movement. The testing ground was an active landslide on I-70 west of the Eisenhower/Johnson Memorial Tunnels. A joint effort between Colorado School of Mines, CDOT, and USGS performed detailed site characterization, set up and calibrated a hydrological model of the site based on three years of field data, and performed a preliminary stability analysis of the slope. Results indicate that the unique hydrology of the site is a key component in its stability and considering the whole water basin and not just the failure area is important.

Implementation
A third phase of this project is needed for completing a detailed parametric analysis of the slope stability so that sound recommendations for site remediation can be provided and coordinated with CDOT. In the meantime, continuous information on ground water location and discrete readings on site movement is obtained.

17. Keywords
landslides, unsaturated soils, saturated soils, slope stability, soil suction, hydrologic model

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In-situ Monitoring of Infiltration-induced Instability of I-70 Embankment West of the Eisenhower-Johnson Memorial Tunnels, Phase II

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EXECUTIVE SUMMARY

Landslides on highway embankments are common geologic hazards to transportation corridors in Colorado. Currently, the Colorado Department of Transportation (CDOT) has identified 124 such landslides, many of which move annually and are induced by infiltration of rainfall or snowmelt. Estimates of the costs to lower the risk to a moderate level often exceed tens of millions of dollars per landslide. When these slopes fail they threaten public safety and private property, block highway traffic, and damage transportation infrastructure. Despite the large and pervasive impact of this hazard, research into forecasting and prevention of infiltration-induced landslides is limited. Recent advances in unsaturated soils hydrology and mechanics allow us to take this challenge. An active landslide on I-70 west of the Eisenhower/Johnson Memorial Tunnels, mileposts 212.0 to 212.1 was identified as testing ground. Records indicate hillslope movement of more than 0.6 m of pavement settlement in two decades. This landslide is classified by CDOT as "large" and due to its location a permanent remedy cost is estimated to exceed $10 million and involve closing the highway for an extended period, which is not practical.

The research has three phases, two of which were completed and are reported in this document. The first phase was an effort to understand the environmental setting and triggering mechanism of the failure; this included: thorough literature review of previous work in the area, mapping of the failure zone, subsurface investigation through four new boreholes, laboratory testing of undisturbed samples to obtain hydrological and mechanical properties, and installation of sensors that continuously monitored groundwater behavior and ground movements in the slope. This information was analyzed through a preliminary conceptual model. From the data obtained during the first phase a unique phenomenon was observed; the fluctuation of groundwater table on the Westbound shoulder was 9 to 12 m while only 30 m across, in the eastbound shoulder the groundwater fluctuates only 4 to 5 m. Since the hydrology of the slope is critical in the stability analysis and therefore in the design for mitigation, phase II focused on developing a conceptual model and a numerical model that capture accurately the hydrological behavior of the slope. All historical information and data collected during the site characterization was used to create an extended geological cross section of the entire water shed area and then to implement a two-dimensional finite element model that analyzes the seasonal hydrology of the slope. The results of the hydrological model were then used in two preliminary slope stability analyses.

Results obtained during the first phase of the project provided additional information than expected. While the seasonal infiltration into the slope is directly related to the slope movement, field data highlighted the importance of considering the full watershed instead of just the landslide area and shed light on the fact that a large portion of water that infiltrates comes from the area north of I-70. Results obtained with the numerical model developed during the second phase are consistent with the refined conceptual model and with field observations. CDOT can use this calibrated model in a parametric analysis to examine different infiltration conditions (dry years, wet years, hydrologic history) in the landslide. In addition, the knowledge gained about how the stratigraphy and morphology of the site affect the stability of a slope can be translated to other slopes. Moreover, this methodology can be used to analyze other infiltration induced landslides.

Although records indicate that the movement near the crest of the slide slowed down after 2012, there is still the need to repair the highway periodically. In addition, the site stability under different infiltration conditions has not been characterized. A third phase of this project is needed for: 1) finishing fine tuning the hydrological model based on a longer data set from the field sensors and additional field observations, 2) performing a parametric slope stability analysis that implements the concepts of unsaturated flow, mechanical behavior of unsaturated soils, and a local factor of safety that does not need a predefined failure surface, and 4) providing recommendations for site remediation based on sound scientific basis. A lot of information has been obtained in this research; completing the objectives of the third phase would allow CDOT to design an effective remediation plan for the site and potentially using this methodology to examine other important landslides along key highways in Colorado.

IMPLEMENTATION

A third phase of this project is needed for completing a detailed parametric analysis of the slope stability so that sound recommendations for site remediation can be provided and coordinated with CDOT. In the meantime, continuous information on ground water location and discrete readings on site movement is obtained.

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1. INTRODUCTION AND BACKGROUND

Landslides on highway embankments and nearby hillslopes are common geologic hazards to transportation corridors in Colorado. Currently, the Colorado Department of Transportation (CDOT) has identified 124 such landslides, many of which move annually and are reportedly induced by infiltration of rainfall or snowmelt. Almost half of these landslides have been assigned a risk value of either “high” or “extreme” by CDOT; estimates of the costs to lower the risk to a moderate level often exceed tens of millions of dollars per landslide. When these slopes fail they threaten public safety and private property, block highway traffic, and damage transportation infrastructure. Instability of these slopes in many cases results from infiltration of snowmelt and rainfall into variably saturated hillslope soil and rock materials. As water infiltrates into the soil, the water content and suction of the soil change and the water table position varies leading to a change in effective stress throughout the slope. These changes then drive changes in the stability of the slope. Despite the large and pervasive impact of this geologic hazard, research into forecasting and prevention of infiltration-induced landslides is limited. Recent advances in unsaturated soils hydrology and mechanics allow us to obtain in-situ measurements of soil suction, and water content; thus, changes in effective stress can be monitored and forecasts of landslide movement and devising effective remedies are possible.

An active landslide was identified on I-70 west of the Eisenhower/Johnson Memorial Tunnels, mileposts 212.0 to 212.1. Records indicate that during the past forty years the hillslope in this area has moved episodically causing more than 0.6 m (2 ft) of pavement settlement in two decades. A temporary solution has been to level the road by adding asphalt to the area of settlement forcing to close at least partially the road on several occasions. The landslide is classified by CDOT as "large"; it has a width of greater than 152 m (499 ft) and a depth of greater than 15.2 m (50ft). Annual traffic records indicate that the average daily traffic exceeds 20,000 vehicles on that segment of I-70. Because it is located 3,240 m (10,630 ft) above sea level and surrounded by very steep terrain near the continental divide of the Rocky Mountains, the accessibility for heavy equipment is limited and the permanent remedy cost is estimated to exceed $10 million. Such a remedial fix would necessitate closing the highway for a extended period, which is not practical.

In 2010, a joint effort between CDOT, Colorado School of Mines (CSM), and the U.S. Geological Survey Landslide Hazards Program (USGS-LHP) was initiated to characterize the site conditions and understand the hydrological and mechanical behavior of this active landslide. This information will then be used in the design of future mitigation efforts that would prevent any further movement or catastrophic failure of the slope. CDOT, CSM, and USGS-LHP outlined a three phase collaboration that would include (I) Site Investigation, (II) Hydrological and Mechanical Analysis, and (III) Mitigation. Phase I, completed in 2014, was an effort to understand the environmental setting and triggering mechanism of the failure, which included mapping of the failure zone, a subsurface investigation, and installation of sensors that have continuously monitored groundwater behavior and ground movements in the slope since 2011. Phase II aimed to fully understand the seasonal hydrology that leads to mechanical instability. In order to accomplish this goal a complete historical review of the site was needed to collect all available information on stratigraphy, construction at this location, known water table levels, and previous investigations. This information was used to create an extended geological cross section of the entire water shed area and a conceptual model of the annual hydrology, which was incorporated into a 2-dimensional numerical model. The results of the hydrological model were then used in a preliminary slope stability analysis to assess the local factor of safety in the slope under the different hydrological conditions and confirm that movements in the slope are triggered by the large amount of infiltration into the slope during the spring season. Phase III proposed for 2016-2018 is needed for: 1) further characterization of the surrounding area of the landslide, 2) continue field monitoring and fine-tuning the hydrological model, 3) performing a detailed slope stability analysis using local factor of safety, and 4) providing recommendations for site remediation.

2. RESEARCH TASKS

Five main tasks were identified during Phase I and Phase II of this project:

1. Perform a detailed literature review of similar research including a national survey to state DOTs and a review of current CDOT/consultant methodologies.
2. Site characterization
3. Development of methodology and conceptual model
4. Characterization of seasonal site hydrology
5. Preliminary stability analysis
6. Draft report and final report

3. TASK 1: PERFORM LITERATURE REVIEW FOR SIMILAR RESEARCH

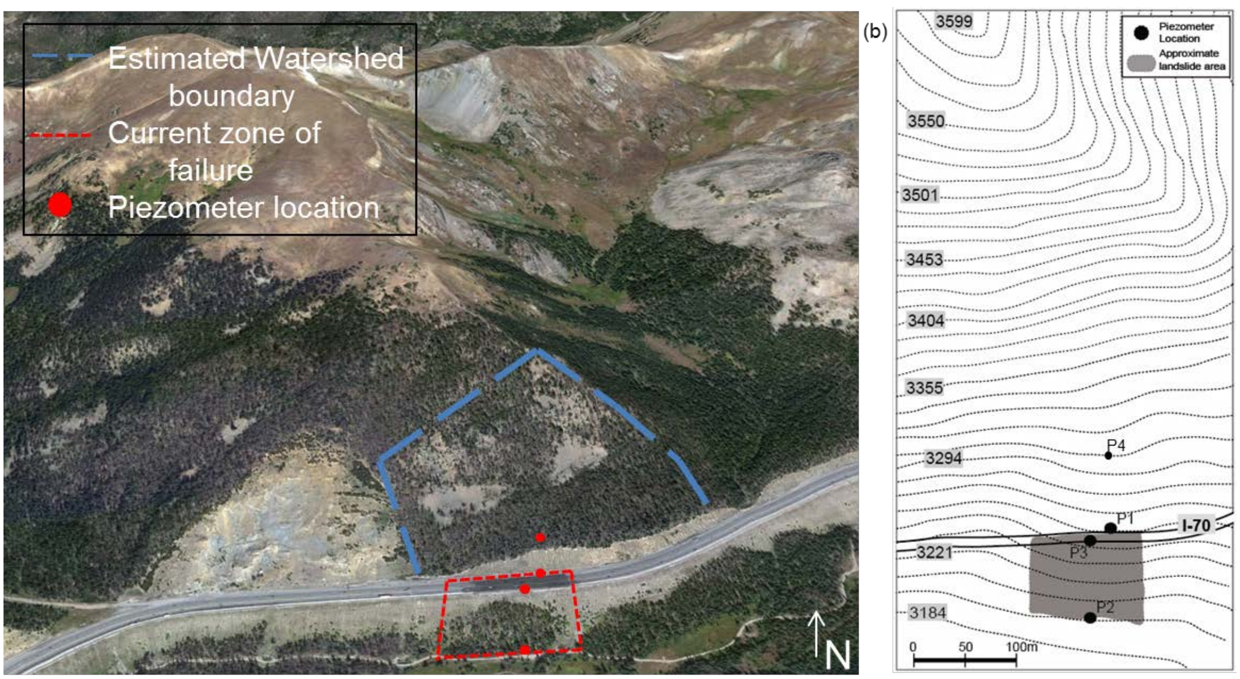
Snowmelt- and rainfall-induced landslides are major geologic hazards. In the U.S., landslides occur in all 50 states, they cause $1~2 billion in damages and average more than 25 fatalities each year (NRC, 2004). When landslides occur along highways they can impede travel, damage infrastructure, and threaten public safety. According to a recent survey, about half of the most destructive landslide disasters worldwide in the past century were infiltration induced (Sidle and Ochiai, 2006). The traditional approach to analyzing slope stability typically relies on limit-equilibrium methods, where the geometry of the potential failure surface in the slope is predetermined and the slope is discretized in vertical slices: the stability of each slice is then analyzed using principles of force and/or moment equilibrium (*e.g.*, Peterson, 1955; Duncan and Wright, 2005). A variety of techniques have been developed for assessing stability using the method of slices, depending on what equations of equilibrium are included and what assumptions are made on inter-slice forces (*e.g.* Fellenius, 1936; Janbu, 1954; Bishop, 1955; Morgenstern and Price, 1965; Spencer, 1967; Sarma, 1973, Duncan, 1996; Krahn, 2003). Recent advances in analyzing slope stability include the use of analytical and numerical methods such as the finite elements, where the global or overall factor of safety is calculated using either the "gravity increase method", or the "strength reduction method" (*e.g.* Duncan, 1996; Griffiths and Lane, 1999; Dawson et al., 1999). However, most of these numerical methods do not explicitly account for time-dependent changes in pore-water conditions and effective stress above the water table that accompany infiltration. In recent years, slope stability analysis has been expanded to include coupled hydro-mechanical processes under variably saturated conditions (*e.g.*, Griffiths and Lu, 2005; Lu and Godt, 2008; Borja and White, 2010). This study reports on the testing of a proposed methodology that accounts for such hydro-mechanical processes.

A survey to all DOTs regarding their methods to deal with landslides was performed during the first phase of this project, the main results were presented to CDOT and a summary is provided in Appendix A. Most DOTs recommend design drainage system to minimize infiltration, improve sub-drainage, and horizontal drainage. For landslides that are classified as small or medium (<152.4 m (500 ft) width, < 15.2 m (50 ft) depth) a common recommendation is to excavate the failed mass and replace with rock. States that report more than 30 landslides in the past 5 years generally work closely with the United States Geological Survey, they perform complete geotechnical investigations including ground water characterization. For large historical landslides where it is not economical or technically feasible to remediate them, the "balance approach" is used. This is a mitigation approach that seeks to slow the movement of the failure surface enough so that it can be managed through standard to heavy activities. Other typical recommended solutions include: Reinforce the soil slope, install french drains, soil replacement with geogrid, berms, rock buttresses, rock shear keys, construct pile walls with the pile tips embedded 3 m (10 ft) into bedrock, soil nails, and soil anchors.

4. TASK 2: SITE CHARACTERIZATION.

4.1 Site location and setting

The Straight Creek Landslide is located approximately 1.5 miles west of the Eisenhower/Johnson Memorial Tunnel, near the town of Silverthorne, CO (Figure 1) between mileposts 212.0 and 212.1. This landslide is situated on the southern facing slope of the Williams Fork Mountains in Summit County, CO. This range, and the surrounding area, is predominantly composed of Proterozoic age metasedimentary gneiss, schist, and pegmatite bedrocks with intrusive granite bands and surficial morainal deposits (Lovering, 1935). When exposed, the bedrocks were subjected to extreme erosion and weathering in the mountainous region which created a fairly thick, weathered bedrock layer beneath thin colluvium deposits along valley walls. Much of the area is forested presently although there are large outcroppings of exposed bedrock in the steepest slope sections and along the cut slopes just north of I-70.



(a) (b)

Figure 1. (a) Plan view of estimated watershed area, (b) contour map of area and piezometers location.

4.2 Work performed since construction of I-70

Construction on this section of the highway was initiated in the late 1960s along with the boring of the Eisenhower Tunnel. In 1970, highway construction triggered multiple landslides in the slopes just north of I-70 due to slope cutting operations. This prompted the first geological investigation into the immediate area, performed by Robinson & Associates in 1969. The report following this investigation provided interpretation of the geology of the area, surface geology maps, some borehole logs up to 36.6 m (120 ft) depth, and scattered water table position measurements. In addition, it included information concerning the construction of I-70 and the Eisenhower Tunnel, indicating that the embankment was constructed with tunnel cuttings from the boring of the Eisenhower Tunnel (Robinson, 1971.)

In 1973 a bulge in the eastbound lanes appeared directly above what is now known as the Straight Creek slide. The bulge eventually turned to downslope movement, although the Colorado Department of Highways (CDOH) initially assumed this was a settlement issue and continued to remediate the movements with asphalt caps to maintain a smooth road surface. In 1996 Kumar & Associates performed a geological investigation on the immediate area. The investigation efforts included mapping the extents of the failure mass and drilling 8 boreholes to determine the location of the failure plane and create a geological cross section of the landslide. This report only recorded water table position in select locations and at discreet times. Kumar & Associates were able to determine a failure plane about 29 m (95 ft) directly below the eastbound lanes' shoulder, confirming that the failures were due to landslide movement and not settlement.

Based on the findings of Kumar & Associates, CDOT installed three inclinometers along the east and westbound shoulders; the slide movements reached the instruments capacity in two years. In 2010 and 2012 light weight caissons were installed under the westbound and eastbound lanes, with the objective of decreasing the overburden in the slide. In 2012, with the caisson work, ten horizontal drains were installed at the toe of the slide, five of which produce water.

In 2010 CDOT had initiated the collaboration with CSM and USGS-LHP to perform the three phase research study on the landslide. In 2011 and 2012, CDOT and CSM drilled three new boreholes and installed three piezometers in the westbound shoulder, eastbound shoulder, and near the toe of the slide along with two inclinometers in the westbound and eastbound shoulder locations. These instruments showed the movements of the landslide were due to a large rise in the water table underneath I-70 during late spring and early summer months.

The data from Phase I and the early work for Phase II in characterizing the hydrology of the landslide showed the need to better characterize a larger area in the watershed. A new borehole and piezometer installation were done north of I-70 by CDOT and CSM in the fall of 2015.

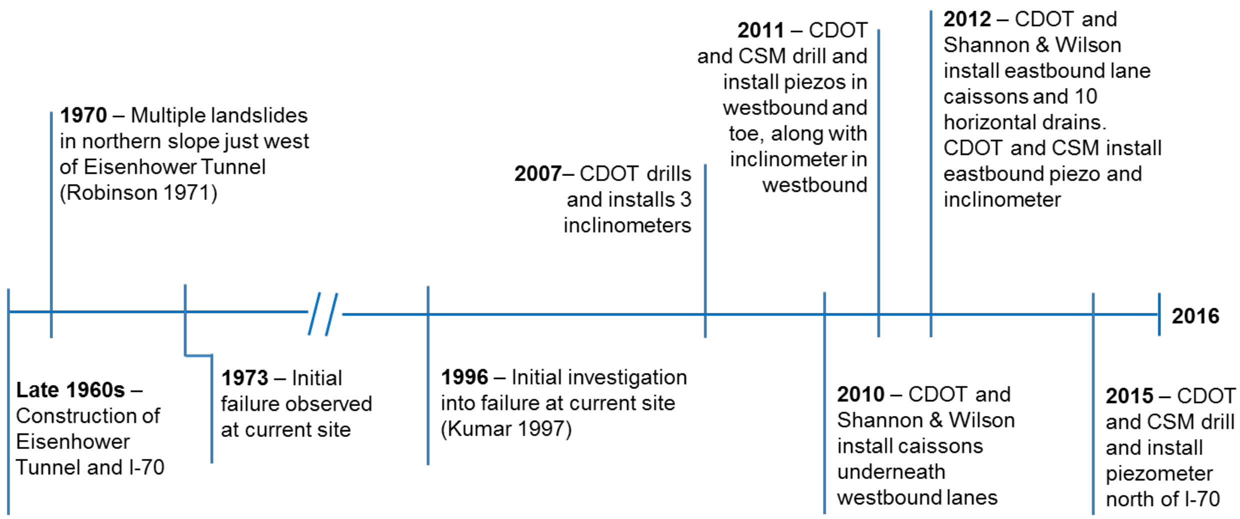


Figure 2. Timeline of work performed on site since construction of I-70.

4.3 Mapping of failure zone

In 2011, the slope was inspected on foot and GPS was used to mark definite areas of distortion in the slope face, damaged pavement, and rotated plant growth that marked the boundaries of the failure mass. Cracks in the pavement of I-70 mark the scarp of the failure plane, damage and displacement of the guardrail of I-70 eastbound lanes show the extent of the failure at the highway, and a 1.5 m (5 ft) tall vertical displacement downslope indicates the toe of a rotational failure. The mapping found the landslide was approximately 175 m (575 ft) wide and included over 120 m (400 ft) of the slope face south of I-70. The mapped landslide extents are seen in Figure 3.

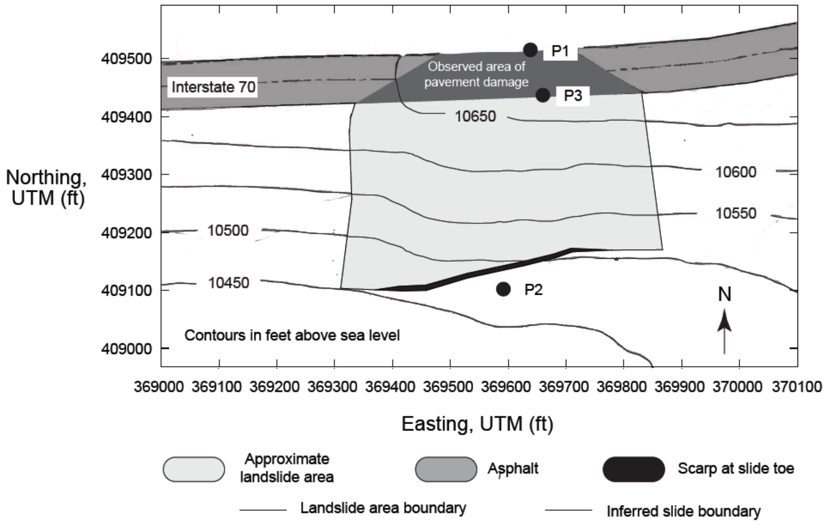


Figure 3. Plan view of GPS mapped landslide.

4.4 Subsurface investigation and characterization

4.4.1 Stratigraphic layers

From the boreholes drilled by CSM-CDOT and previous works in the area, the cross section area presented in Figure 4 was obtained. Asphalt pavement was found to be 3 inches (7.6 cm) thick along the westbound shoulder but up to 0.76 m (2.5 ft) thick in the eastbound lanes. Under the pavement, the highway embankment fill is encountered up to depths of 8.5 m (28 ft) below ground surface (bgs) in the westbound and 9.8 m (32 ft) bgs in the eastbound. Tunnel cuttings from the boring of the Eisenhower tunnel were used as the fill and this layer is composed of gravel and occasional boulders (up to 1.2 m or 4 ft in diameter) in a brown, clayey sand matrix. This "tunnel muck" is underlain by a 0.9–1.5 m (3-5 ft) thick layer of highly decomposed black and grey gneiss in 10-15 cm (4-6 inch) cobbles with slickensided, clay-filled joints. Clay deposits 0.3-0.6 m (1-2 ft) thick were also found in this layer in the eastbound shoulder location. Much thinner clay deposits were observed underneath the westbound lane shoulder. After the decomposed gneiss, bedrock was encountered at 12.2-14.3 m (40-47 ft) bgs along the westbound shoulder and 23.7-25.3 m (78-83 ft) bgs along the eastbound shoulder. These boreholes indicate the bedrock is less steep underneath I-70 than it is upslope.

Boreholes drilled near the toe of the slide, close to the valley floor, found native colluvium and alluvium soils at the surface up to 4 m (13 ft) deep underlain with 0.9-1.2 m (3-4 ft) of moderately weathered black gneiss bedrock. Competent gneiss bedrock was encountered at 5.2 m (17 ft) bgs. The rotational nature of the landslide also provides some information of the soil as almost 1.8 m (6 ft) of material has been displaced at the toe scarp. The scarp shows up to 0.3 m (1 ft) of organic matter followed by light brown sand with boulders up to 0.9 m (3 ft) deep and a dark brown, sandy silt layer up to 1.5 m (5 ft) bgs. At the bottom of the silty layer pulverized gravel and bedrock were found, indicating areas of failure and movement (Morse, 2011).

In the slope north of I-70, very thin colluvium deposits less than 0.9 m (3 ft) thick were encountered at the surface followed by a highly fractured rock layer extending to 12.2 m (40 ft) deep, where more competent gneiss bedrock was found. The highly fractured layer consists of pebble to small boulder size black gneiss and some granite with chaotic fracturing in all directions. Most fractures were clean, but traces of yellowish clay were found on some joints.

The failure plane of the landslide is believed to start near the centerline of I-70, and run along the decomposed gneiss-bedrock interface. This assumption is supported by the fact that there were significantly more paving efforts in the eastbound lanes, and that the movement observed in the westbound shoulder with the new inclinometer is small and does not show a clear failure surface. However, the possibility of the failure surface extending further north can be investigated in phase III. Borehole logs from CSM investigations can be found in Appendix B.

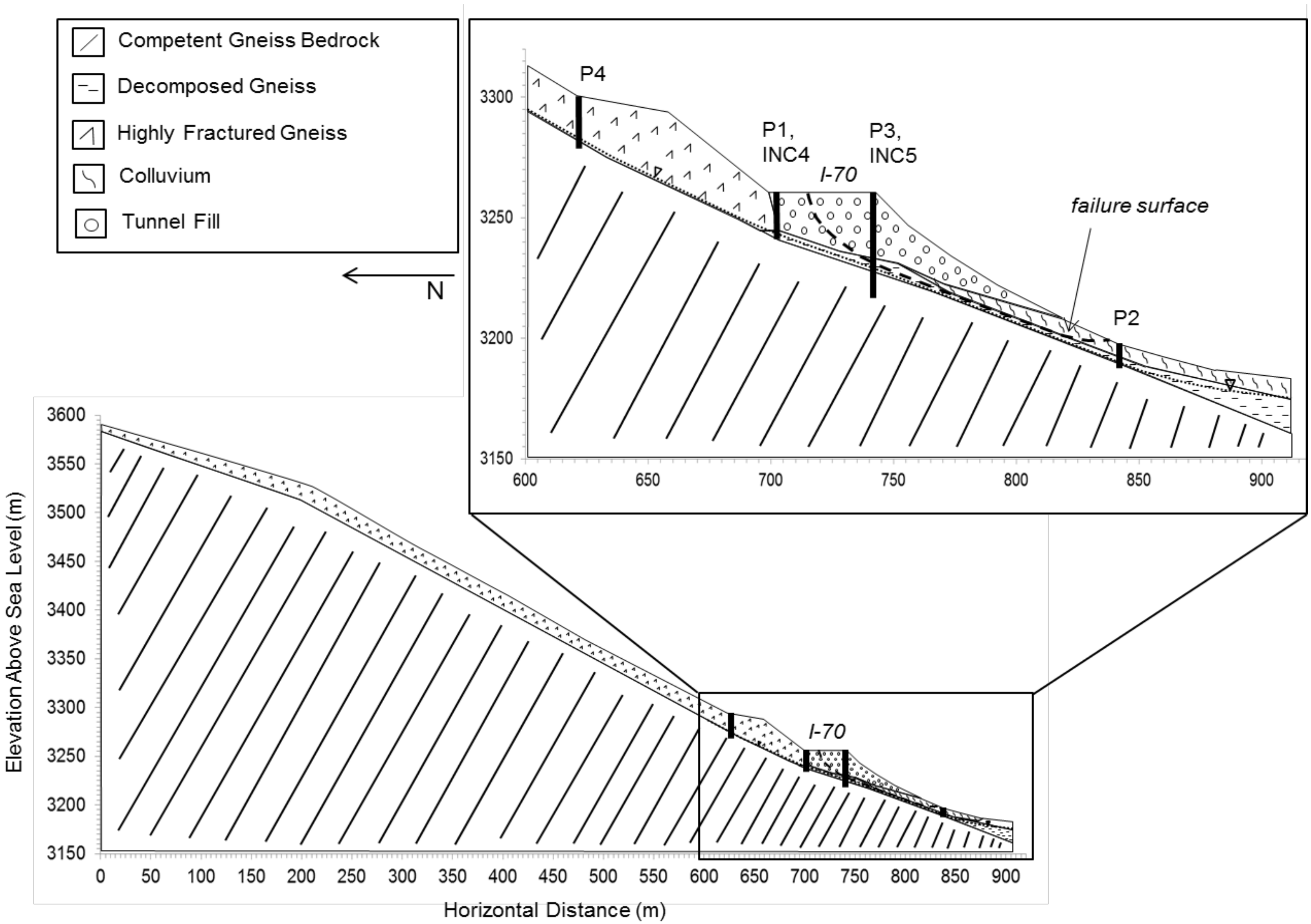


Figure 4. Geologic cross section with instrumentation, boreholes, and failure surface.

4.4.2 Hydrological and mechanical properties

Direct shear tests were performed on the samples to obtain strength properties. Transient Release and Imbibition Method (TRIM) was used to obtain the hydrological properties. A mini-disk infiltrometer test was performed in the colluvium near the toe of the landslide to establish a range of in situ hydraulic conductivity. A slug test in the borehole north of I-70 (P4) provided an estimate of the saturated hydraulic conductivity (k_s) of the highly fractured gneiss. These values are reported in Table 1 along with other properties that have been provided by CDOT.

Table 1 Material properties of soil layers

Material	θ_r	θ_s	α (m^{-1})	n	K_s (m/day)	γ (kN/m3)	c' (kPa)	φ' (deg)
Pavement	-	-	-	-		25	0	32
Bedrock	0.06	0.34	1.374	1.72	0.001	23	95	34
Decomposed Gneiss	0.065	0.41	7.5	1.89	1.06	21	1	23
Highly Frac.Gneiss	0.06	0.34	1.374	1.72	40	22	1	35
Colluvium	0.08	0.33	2.35	2.12	6	20	0	34
Tunnel Fill	0.08	0.33	2.35	2.12	0.5	21	0	30

4.4.3 Inclinometer data

The two inclinometers (INC4 and INC5) placed in 2011 and 2012 show displacements of 0.7 cm (0.3 inch) up to present (Figure 5). Details regarding the installation of inclinometers INC4 and INC5 can be found in Appendix C.

4.4.4 Piezometer data

Four Geokon vibrating-wire piezometers (P1-P4) were installed to record the variation of the water table position every 30 min (Figure 6.) P4 was installed in October 2015 and is located furthest north followed by P1 along the westbound lanes' shoulder, P3 along the eastbound lanes' shoulder, and finally P2 near the toe of the slide. P4 and P1 are connected to a Campbell Scientific CR10X datalogger with AVW200 sensor analyzer along westbound shoulder, while P2 and P3 are connected to a second CR10X/AVW200 set up near the valley floor. Details about the piezometers and datalogger set up are provided in Appendix C.

Figure 7 displays the recorded water table data from 2011-2015 along with atmospheric data. P1 along the westbound shoulder shows a very large and rapid response to heavy infiltration each spring when the water table rises 9 to 12 m (29 to 33 ft) over a period of 3-4 weeks. P3 along the eastbound shoulder, however, shows a much smaller response to infiltration and the water table only rises 4 to 5 m (13 to 16 ft) although the two instruments are only 30 m (100ft) apart across the highway. This behavior is rarely seen in a natural hillslope and was a large cause for concern when it was first observed. P2 is close to Straight Creek at the base of the valley, which controls the water table response to some degree and reduces the magnitude of response in the region. Hence, there are very minimal fluctuations observed in P2 and a rise of only 1 to 2 m (3.3-6.5 ft) is observed each year. P4 is recently installed and data from this piezometer is not available for these years.

Atmospheric data for this site is provided by a National Resource Conservation Service (NRCS) SNOTEL station at Grizzly Peak, approximately 14 km (8.7 miles) southwest of the landslide site. SNOTEL data provides information regarding daily precipitation, snowpack, temperature, etc. Snowpack information is reported in terms of snow water equivalents (SWE), which represents the total height of a water column the snowpack would reduce to if melted. Precipitation data includes both snowfall and rainfall in the area. The assumption that Grizzly Peak SNOTEL data is representative of the daily atmospheric conditions experienced by the Straight Creek landslide watershed is not entirely accurate. Grizzly Peak station was chosen because it is also a south-facing slope with similar terrain and the station is at a comparable elevation to I-70. However, the slope at Grizzly Peak is still a significant distance away and much more forested than the slopes near this site. More exposure at the Straight Creek landslide site most likely leads to faster snow melting that starts earlier in the year. This is evidenced by the piezometer data showing a water table response before Grizzly Peak reports any infiltration each year (Figure 7). Despite the discrepancy, SNOTEL data from Grizzly Peak is accepted to generally represent the seasonal atmospheric data of the Straight Creek landslide.

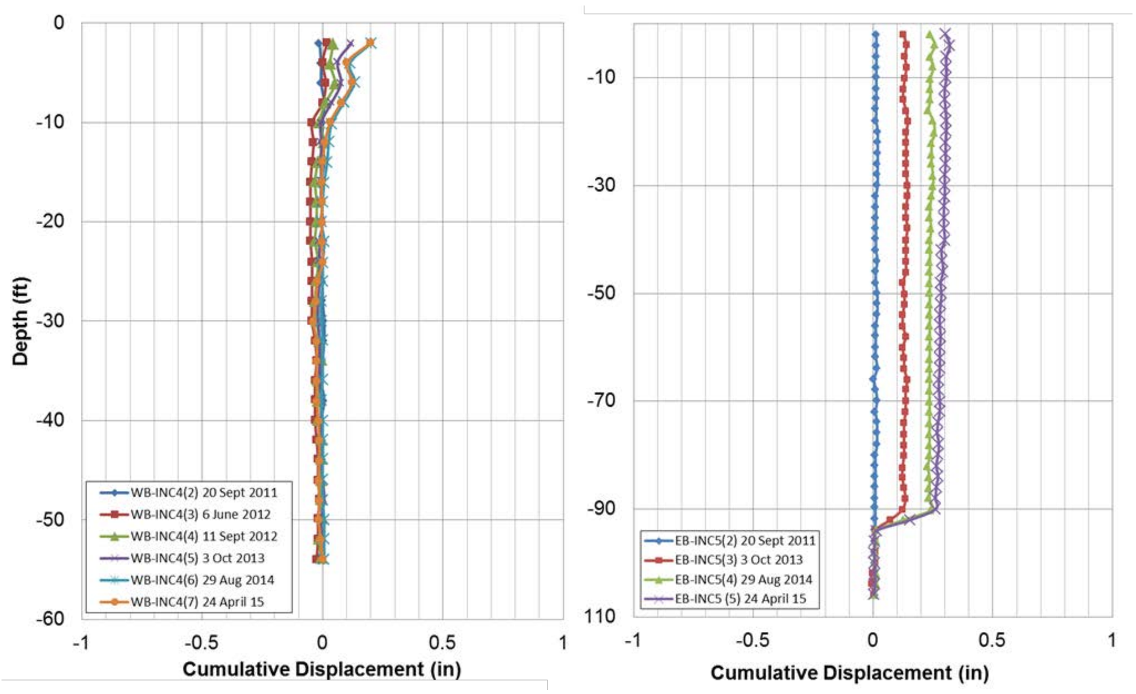


Figure 5. Inclinometer data from 2011-2015 (a) INC4 along the westbound shoulder and (b) INC5 along the eastbound shoulder (CDOT, 2015).

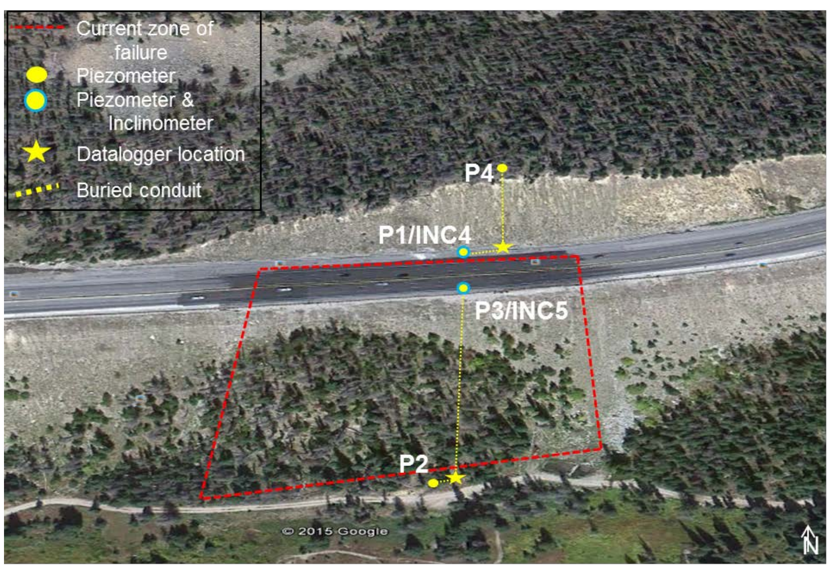


Figure 6. Site instrumentation locations.

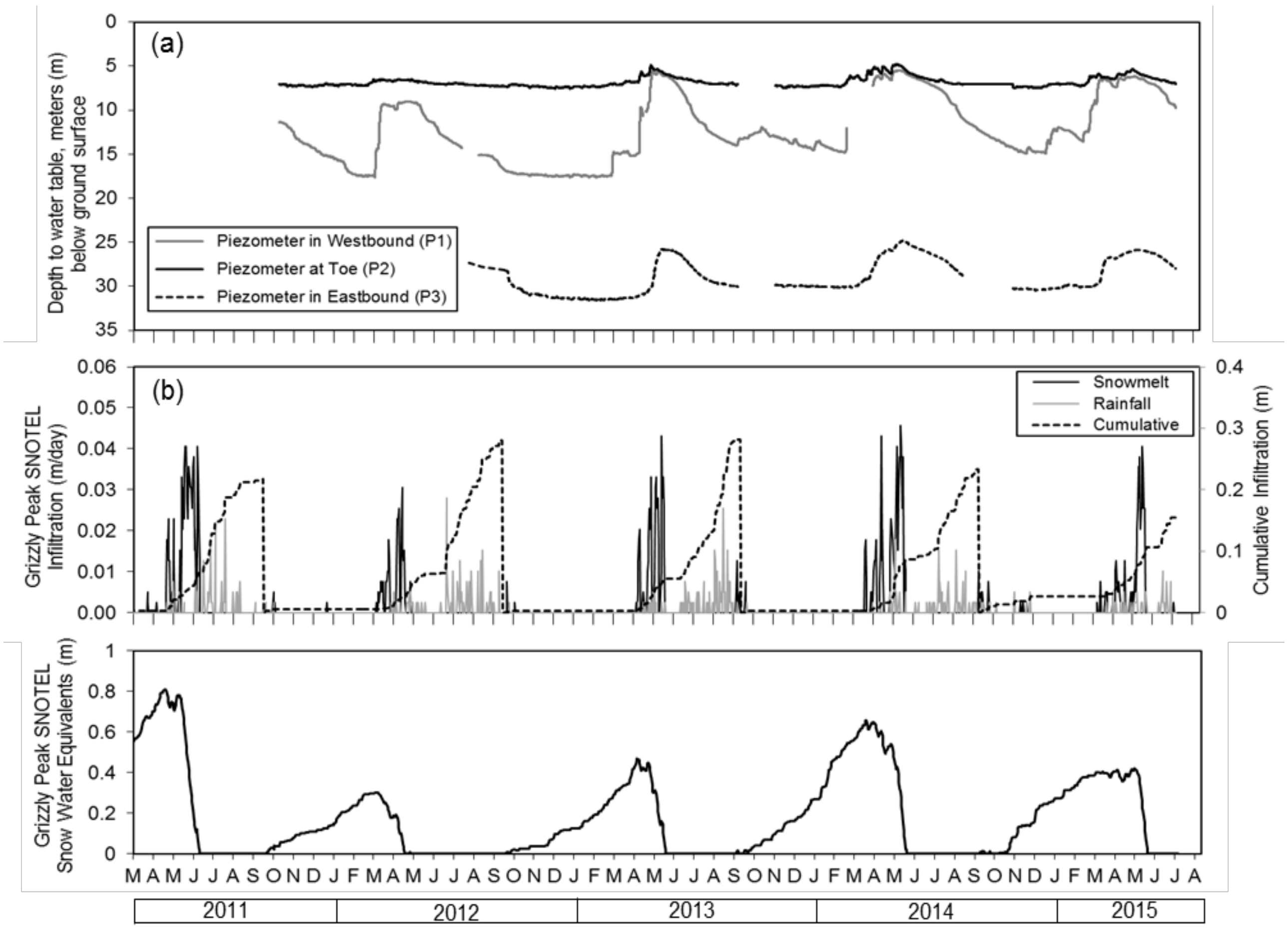


Figure 7. (a) 4 years of piezometer data, (b) infiltration data from Grizzly Peak, and (c) snow water equivalent data from Grizzly Peak.

5. TASK 3: METHODOLOGY AND CONCEPTUAL MODEL.

The methodology consists of obtaining soil suction and moisture content variation in the field and using the data to predict the likelihood of landslide occurrence. This can be accomplished by using a rigorous, yet simple coupled hydro-mechanical framework that accounts for the major physical processes in the slope: stress, deformation, and variably-saturated flow. In this framework, effective stress distributions used for the stability analysis are calculated throughout the slope by taking into account the slope's geomorphology, its hydrology, and the stress, strain, and deformation. The transient hydrological and mechanical behavior of the slope is analyzed by one-way coupling Richards' equation (1) with classical linear-elasticity equations.

$$\frac{\partial}{\partial x}\left[k_x(h_m)\frac{\partial h_m}{\partial x}\right]+\frac{\partial}{\partial y}\left[k_y(h_m)\frac{\partial h_m}{\partial y}\right]+\frac{\partial}{\partial z}\left[k_z(h_m)\left(\frac{\partial h_m}{\partial z}+1\right)\right]=C(h_m)\frac{\partial h_m}{\partial t} \qquad (1)$$

where h_m is head, $k(h_m)$ is the hydraulic conductivity function (HCF), t is time, $C(h_m)$ is the specific moisture capacity function, or the slope of the SWRC.

The effective stress for variably saturated porous materials is defined as (Lu and Likos, 2004):

$$\boldsymbol{\sigma}' = \boldsymbol{\sigma} - \left(u_a + \sigma^s\right)\mathbf{I} \qquad (2)$$

where $\mathbf{I}$ is the second-order identity tensor, σ^s is the suction stress that is a characteristic function of saturation or matric suction and is expressed in a closed form for all soils (Lu and Likos, 2004, 2006):

$$\sigma^s = -(u_a - u_w) \qquad u_a - u_w \le 0 \qquad (3a)$$

$$\sigma^s = -(u_a - u_w)S_e \qquad u_a - u_w \ge 0 \qquad (3b)$$

where $(u_a - u_w)$ is the matric suction and S_e is the equivalent degree of saturation. Using van Genuchten's model (1980) to describe the soil water retention curve, suction stress (equation (3b)) can be expressed as a sole function of matric suction (Lu and Likos, 2004; Lu et al. 2010):

$$\sigma^s = -\frac{(u_a - u_w)}{\left(1+\left[\alpha(u_a - u_w)\right]^n\right)^{(n-1)/n}} \qquad (3c)$$

where α and n are empirical fitting parameters in van Genuchten's soil water retention model. Once the total stress, matric suction, and suction stress distributions throughout the slope are known, effective stress is calculated, and the stability of the slope can be calculated by taking into account the shear strength properties of the soil combined with the effective stress distribution.

A conceptual model of the site considers four distinct stages that generally coincide with the annual seasons (Figure 8). An example of the ground water table variation for a year with reference to these stages is provided in Figure 9.

Stage I: Winter. The water table is observed at its deepest position with minimal fluctuation, resting just above the competent bedrock boundary and below the failure surface of the landslide. During this time, no water is entering the hillslope as snowfall accumulates along slopes rather than infiltrating. According to historical SNOTEL data from Grizzly Peak, the maximum annual snowpack in the area can range from approximately 0.3 to 0.8 m of snow water equivalents (SWE). (Figure 8a).

Stage II: Spring. With the warming temperatures the snowpack starts to melt. The soil near the surface is very dry at this time as no infiltration has occurred in the previous months, so there is large matric suction near the surface according to each soil's SWRC. This suction creates a gradient in total potential of the liquid water in the system that initiates shallow infiltration and water enters the hillslope perpendicular to the slope surface (Lu & Godt, 2013) (Figure 8b). No water infiltrates at the highway surface, however, as the snowfall is plowed off the road surface and the asphalt pavement is relatively impermeable.

Very little change is seen in the water table during this stage and the water table remains below the failure surface of the landslide. The dry conditions of the surface soils also mean a reduced hydraulic conductivity according to each soil's HCF so the wetting front moves slowly through the upper layers and has not yet reached the saturated zone of the hillslope. A small rise along the westbound shoulder (P1) is observed, possibly from plowed snow melting along the shoulder.

Stage III: Summer. During the late spring and early summer months, snowmelt and rainfall continue to infiltrate into the hillslope and the wetting front reaches the saturated zone near the bedrock boundary. The extreme contrast between the hydraulic conductivities of the highly fractured gneiss and the competent bedrock results in flow parallel to the bedrock (Lu & Godt, 2013). The bedrock in the northern slopes is steeply inclined (up to 60º), so large volumes of groundwater are able to travel downslope swiftly (Figure 8c).

When fast-moving groundwater reaches the highway, the lower hydraulic conductivities of the fill and decomposed gneiss, together with a shallower bedrock slope result in a backup of groundwater just north of I-70 and a significant rise in water table position occurs along the westbound shoulder, as seen in P1. The large volume and velocity of the infiltrated water in the northern slopes allows this backup to reach a maximum height in only 2-3 weeks.

Despite the reduction in elevation gradient, the large rise in water table eventually creates enough of a pressure gradient to drive a significant amount of flow under the highway that results in a rise of the water table south of I-70, as observed in P3 during this time. The response in this location is much less, however, only reaching a maximum rise of approximately half the height of the backup to the north because of the reduce flow volume through this region. Additionally, the response is delayed by as much as 30 days from the initial response in P1 as the excess groundwater flow is slowed by the lower conductivity soils under the highway.

Further south of I-70, the embankment fill stops and the decomposed gneiss layer becomes thinner. Instead, groundwater flow encounters native colluvium and alluvium soils with higher conductivities. Combined with the reduced flow rate of excess ground water caused by the soils at the highway, the increased conductivity of these soils and higher moisture content condition from infiltration at the surface enable the colluvium and alluvium to transmit the excess flow easily, with minimal fluctuations in the water table, as observed in P2. Additionally, the water table in this area is very close to Straight Creek, which acts as a relatively constant head condition in this system and helps to mute the already small response to excess groundwater.

The rise of the water table in P1 and P3 underneath I-70 is enough to saturate the majority of the landslide failure surface which result in positive pore water pressures and the reduction of effective stress and shear strength of the soils.

Stage IV: Fall. During late summer and fall there is minimum water infiltration. The water table returns to a deeper position below the failure surface (Figure 8d). The drainage of the water table occurs at a slower rate than the previous rise of the water table. Drier years were observed to drain completely in 3 months while a wetter year can take up to 5 months. Eventually, all excess groundwater is released from the hillslope and the water table reaches a steady state condition until the following spring season

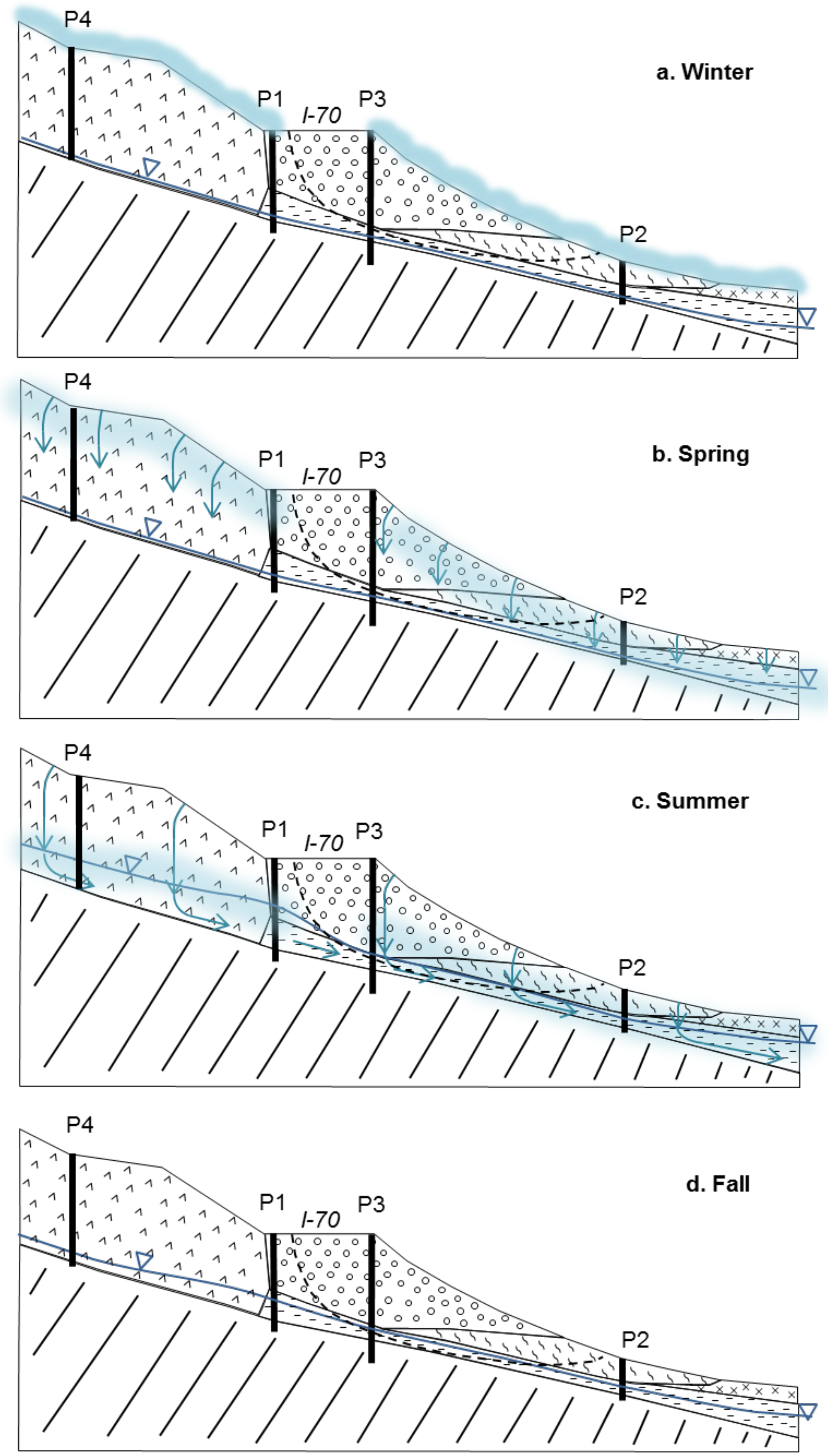


Figure 8. Conceptual model diagram

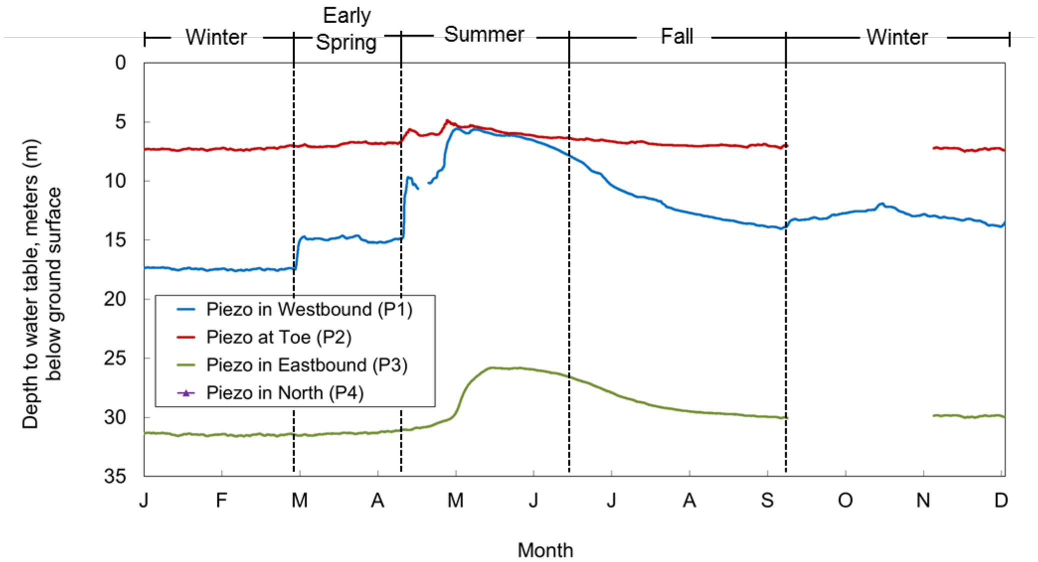


Figure 9. Water table variation for 2014.

6. TASK 4: NUMERICAL MODEL OF THE SITE HYDROLOGY.

A two-dimensional finite element numerical model of the Straight Creek landslide was set up to confirm the conceptual model and predict behavior of the site in order to simulate the hydrological conditions of the site. The model was calibrated using field data from piezometers P1-P3. Parametric analysis were performed to investigate the parameters that have larger effect on the site hydrology. The framework described in section 5 was implemented. The model domain, boundary conditions, and initial conditions are presented in Figure 10. Initial conditions were obtained at steady state with an infiltration of 0.001 m/day (0.003 ft/day). Boundary conditions are constant head near the toe (south side), no flow on the north end and on the bottom, and atmospheric conditions along the hillslope with the exception of the highway portion. Infiltration data for the model was obtained from NRCS, SNOTEL. The data used were the snow water equivalent (SWE) and rainfall measured in the Grizzly Peak station (Figure 11). It is important to note that although the Grizzly Peak station has similar conditions to the Straight Creek landslide, the later one is more exposed to the sun thus probably experiencing faster and earlier infiltration. Observation nodes were placed at locations coinciding with the piezometers in the field.

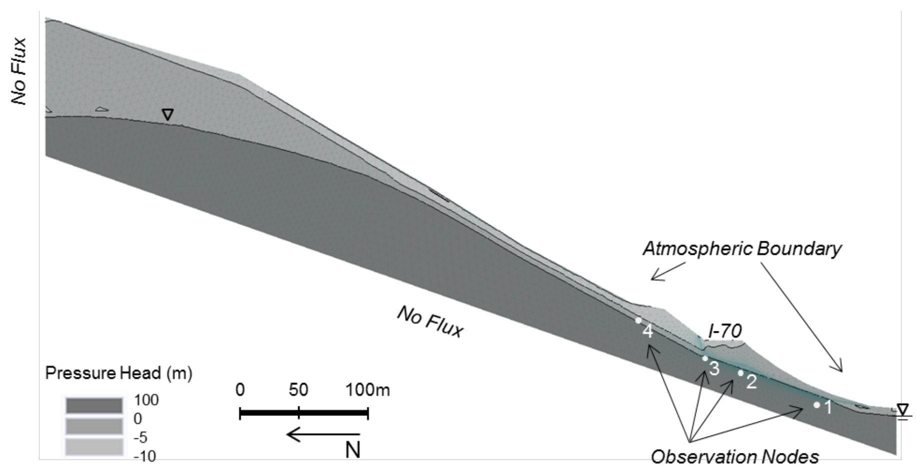


Figure 10. Numerical model domain: boundary conditions, initial conditions, and observation nodes.

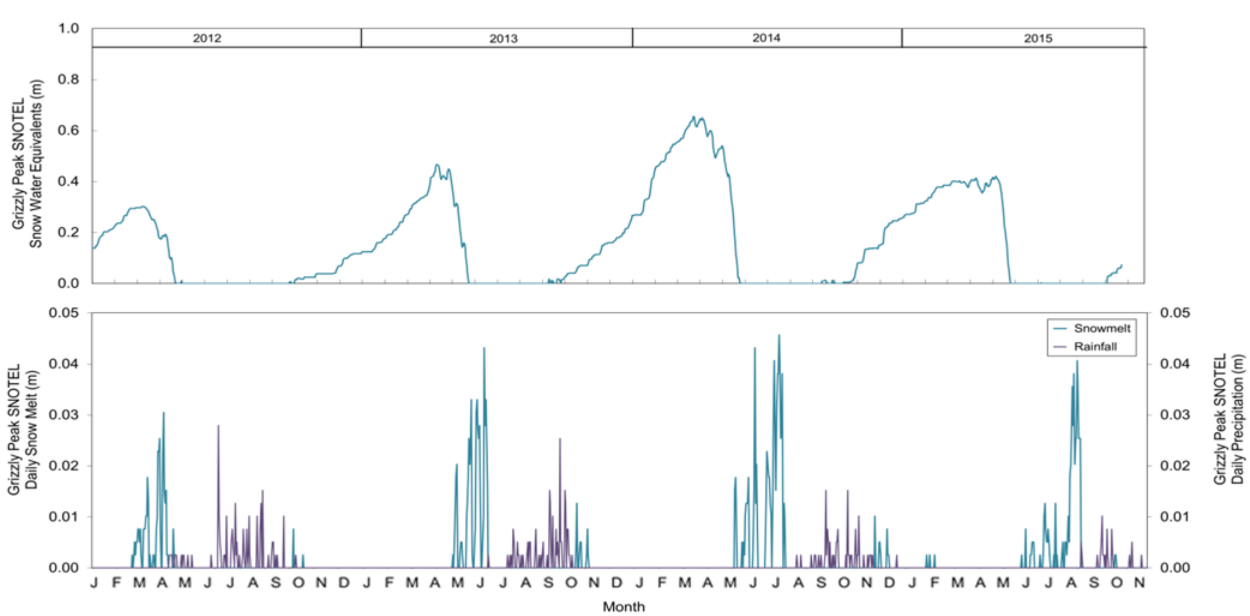


Figure 11. Snow Water Equivalent and infiltration data for Grizzly Peak, years 2012-2015.

A comparison between the field measured data and the numerical modeling results is presented in Figure 12. In the top portion (Figure 12a) the infiltration data from Grizzly Peak is used directly; a lag in time between the predicted and measured results is observed probably due to earlier infiltrations in the Straight Creek site. For example, in 2015 the monitored groundwater table increases before any infiltration was measured in Grizzly Peak. Adjusting the timing of the infiltration (Figure 12b) leads to a better comparison between the observed and simulated ground water response. The numerical model is able to reasonably capture the qualitative and quantitative seasonal ground water level changes, the fact that the water table in the westbound rises almost twice as much as the water table in the eastbound, and the effect of different infiltration rates and times throughout the years.

The pressure head distribution of the watershed throughout the year is provided in Figure 13. It is observed that the shallow bedrock in the north side of I-70 promotes larger water pressures along the bedrock interface; thus, more water flows to the landslide area. A comparison of the water table location measured in the field and obtained with the numerical model is shown in Figure 13b. The large rise in the westbound location during summer is seen in both observed and simulated data, although the simulation shows a slightly higher rise than what is observed in the field. In the fall, infiltration slows down and the simulation shows a decrease in pressure head and moisture content in the surface soils and a lowering of the water table throughout the water shed. The simulated water table in the westbound location drains faster than the observed water table in the field, and the opposite is seen in the eastbound location. Once again this is attributed to assuming atmospheric conditions that are similar to the study site, but not always exact. The numerical model captures qualitative behavior of the water table near the toe, but the simulated results show an overall water table shallower than the field observations. This difference is probably due to having a constant head boundary for the southern extent of the modeled watershed instead of a changing head with time.

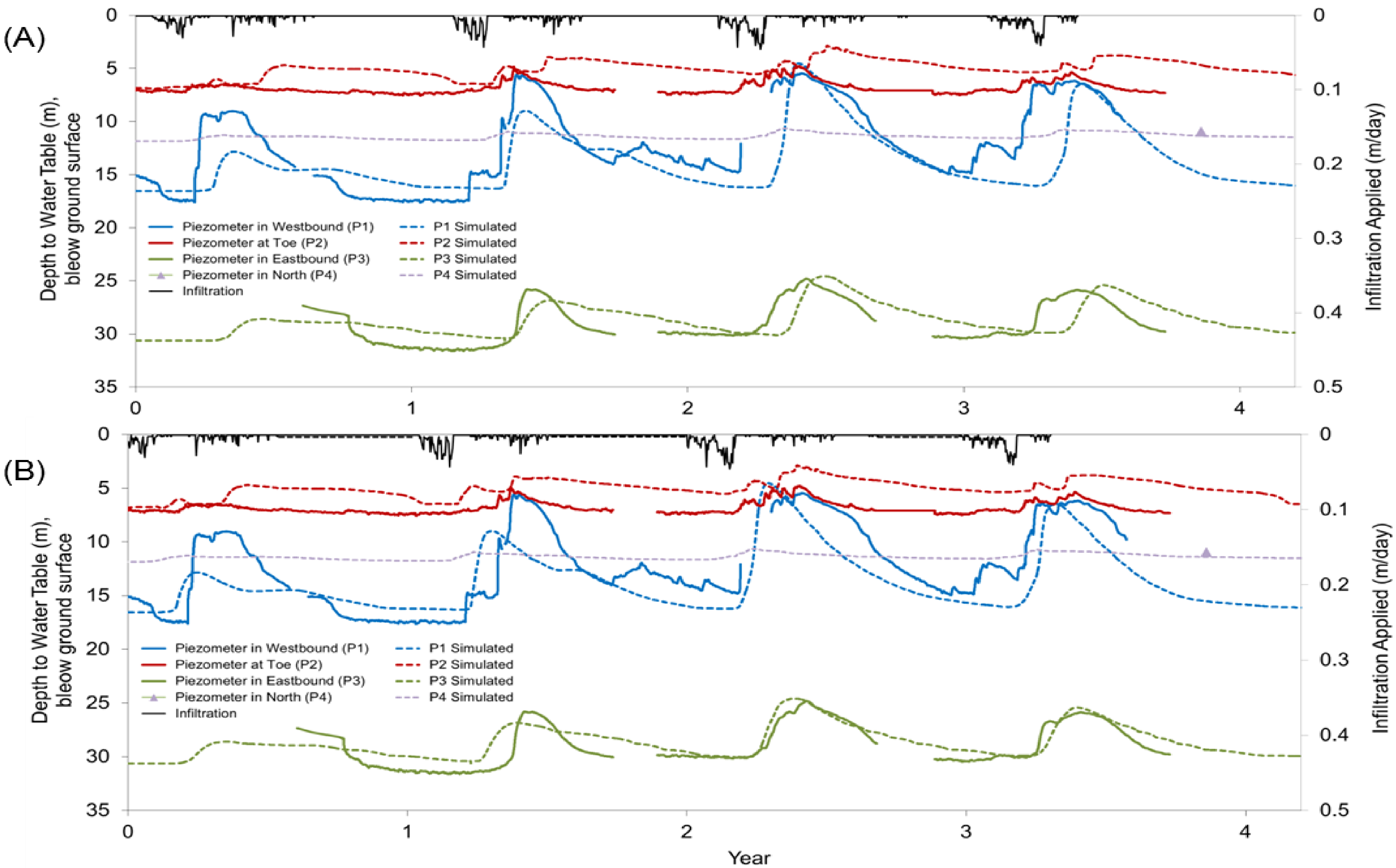
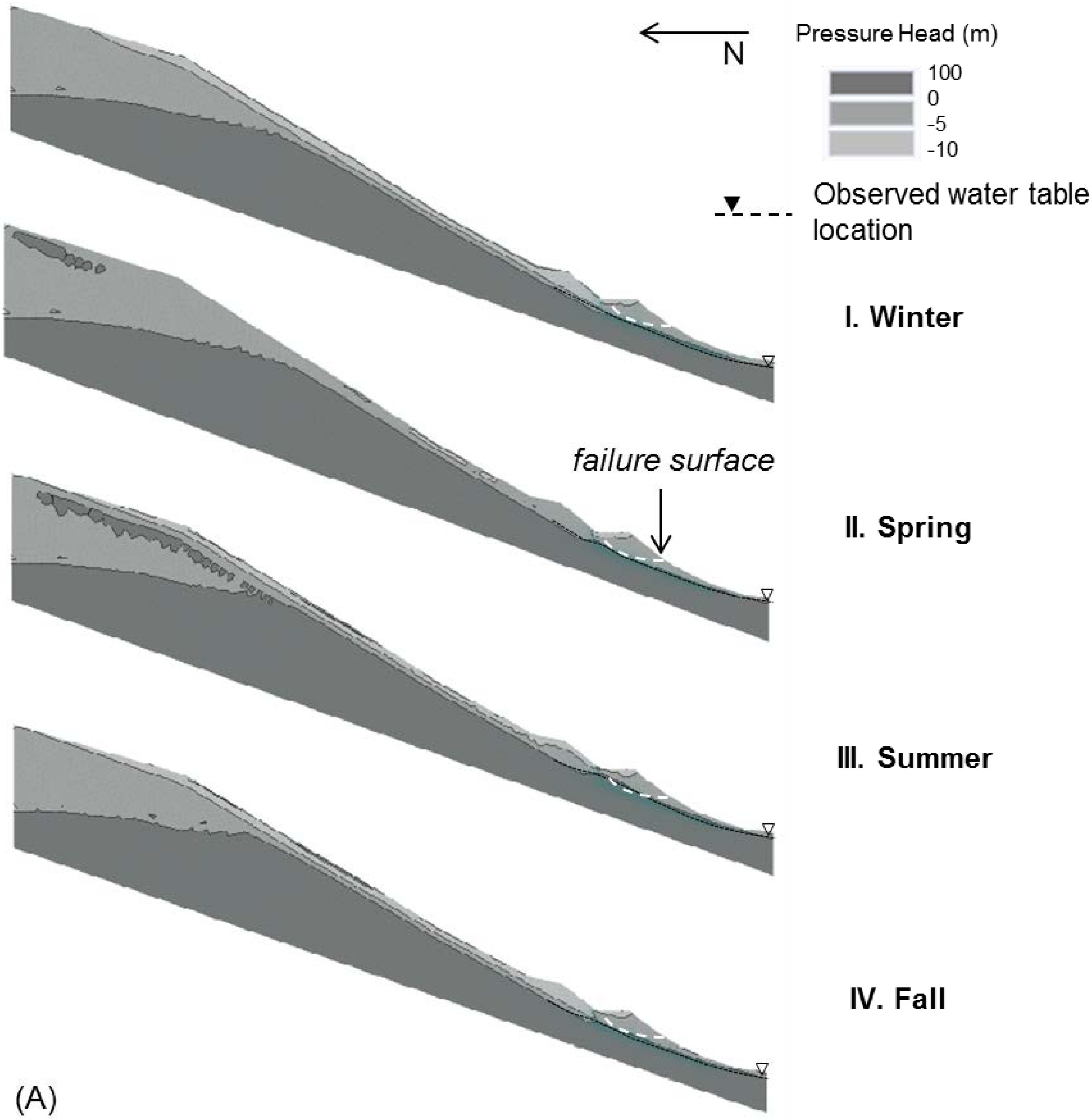


Figure 5.5 Comparison of field measurements and simulation results of ground water table elevations at observation points with (a) original infiltration data from Grizzily Peak SNOTEL and (b) infiltration data timing adjusted to match water table response in simulation to observe water table behavior


N
Pressure Head (m)
100
0
-5
-10
Observed water table location
I. Winter
failure surface
II. Spring
III. Summer
IV. Fall
(A)

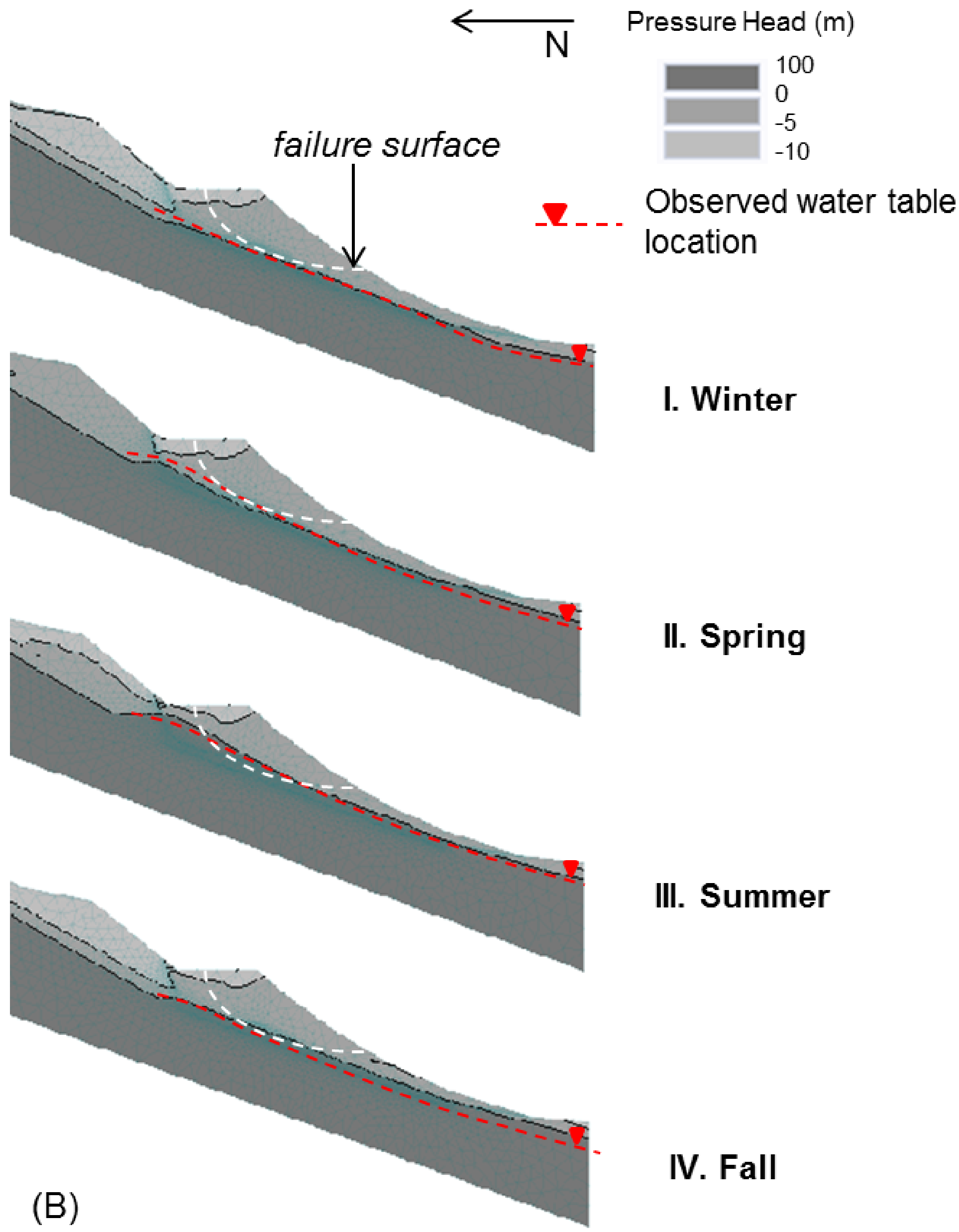


Figure 13. (a) Simulated pore water pressure distribution throughout the watershed and (b) simulated water table near I-70 compared with observed water table during each conceptual model stage over the course of one year.

7. TASK 5: PRELIMINARY STABILITY ANALYSIS

Two preliminary stability analyses of the site were performed. The first one used the traditional modified Bishop's method of slices and the second one analyzed the stability using Bishop's modified method of slices implementing suction stress.

7.1 Stability analysis using Bishop's modified method of slices

RocScience Slide 6.0 was used to perform a preliminary analysis of the site under the seasonal water table conditions in winter, spring, summer, and fall; the mechanical soil properties specified in Table 1 were used. Results from the analysis (Figure 14) indicate that the landslide is stable under lower water table conditions with a FS = 1.04-1.05 during fall, winter, and spring seasons and it is unstable under peak water table conditions during the summer with a FS = 0.95. The reduction in FS and loss of stability can be attributed to the decrease in effective normal stress (σ') caused by the increase of pore water pressures (u_w) along the failure surface from the water table migrating from a low position in winter to a peak position in the summer. A decrease in the effective normal stress leads to a decrease in the available shear strength of the materials, which is what triggers instability and the slope is susceptible to movement.

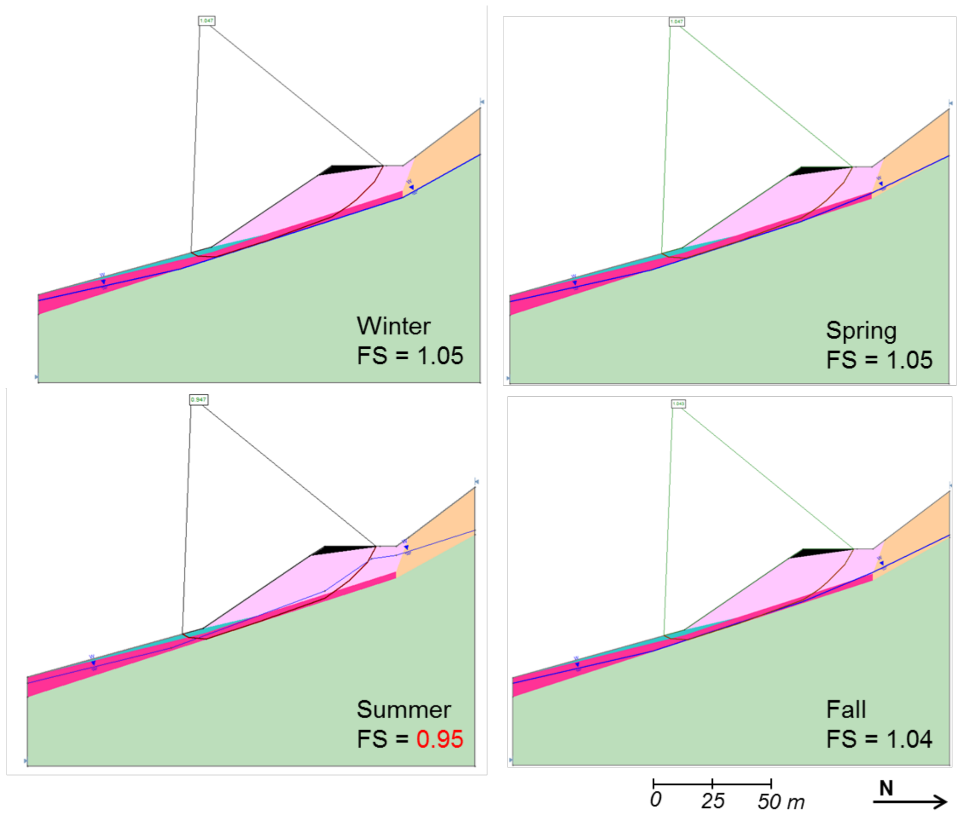


Figure 14. Slope stability results using Modified Bishop's method of slices

A quantitative look at the stresses that occur along the failure plane during low and peak water table positions is seen in Figure 15, where the magnitudes of pore water pressure, effective normal stress, and shear strength along the failure plane are displayed, from toe to scarp. According to these results, up to 40 kPa (835 psf) of pore water pressure is generated along the upper failure surface in the summer and the effective stress is reduced by the same value. This amount of pore water pressure decreases the shear strength by up to 22 kPa (459 psf).

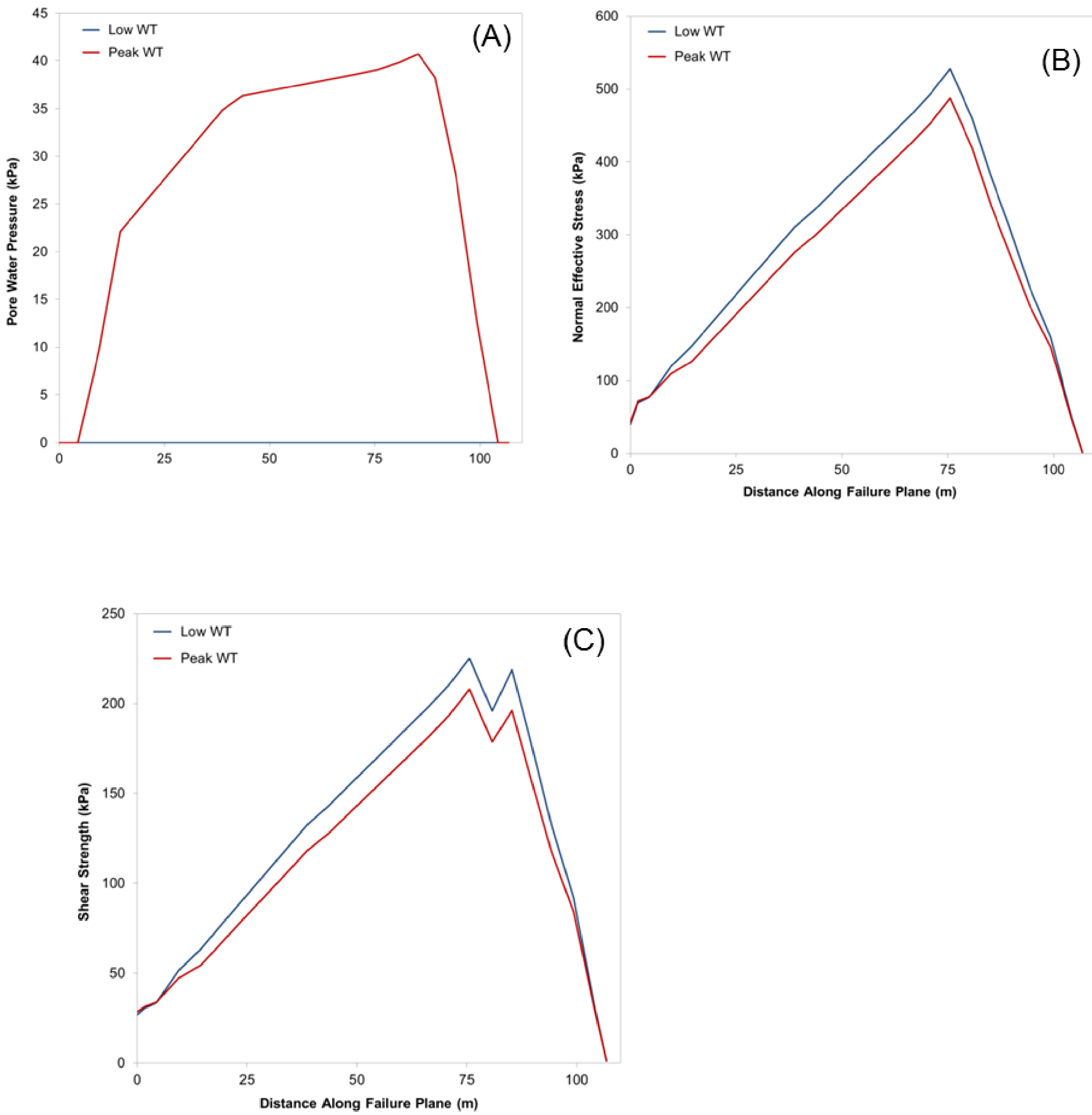


Figure 15. (a) pore water pressure, (b) effective stress, and (c) shear strength along the failure surface from toe to scarp during peak summer flow and low winter flow conditions

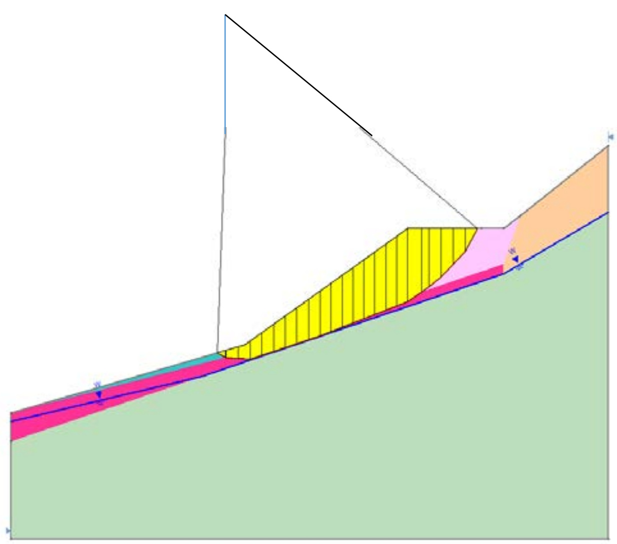
While the results indicate the landslide is stable when the water table is below the failure surface, the FS is only slightly greater than 1. The reduction of shear strength due to the change in groundwater table causes failure.

7.2 Stability analysis using the extended Bishop's method of slices and accounting for suction stress

The objective of this analysis was to understand the effect of the water table location and suction stress in the stability of the slope. A cross sectional area with the sliding surface, material properties, and slice discretization is presented in Figure 16. The failure surface was assumed based on observation of displacement near the highway divide, near the toe area, and previous inclinometer data. In addition, the water table is initially located slightly above bedrock. The factor of safety was calculated using an extended Bishop's method of slices, which accounts for the effect of suction stress in the soil (Lu and Godt, 2012):

$$FS_s = \frac{\sum_{n=1}^{m} \left(c' b_n + W_n \tan\phi' - \sigma_n^s b_n \tan\phi' \right) / I(\alpha_n, \phi', FS_s)}{\sum_{n=1}^{m} W_n \sin\alpha_n} \qquad (4a)$$

$$I = \cos\alpha_n + \frac{\tan\phi'}{FS_s} \sin\alpha_n \qquad (4b)$$



Season	Factor of Safety
Winter	1.052
Spring	1.052
Summer	0.939
Fall	1.049

Figure 16. Stability analysis using modified Bishop's method that accounts for suction stress

Stability of the slope was analyzed during the four seasons of the slope. Consistent with the results from section 7.1 the factor of safety in the slope is smaller than 1 (failure) during summer, when the water table rises and suction stress decreases. If the water table is at low conditions (fall, winter, and early spring) the soil has some suction stress that contributes to the strength of the material and the factor of safety is slightly greater than 1 (no more movement). It is important to note that decreasing the weight of the slope not only decreases the magnitude of the driving forces, it also decreases the shear strength of the soil.

A seasonal stability analysis accounting for the weight reduction due to the caissons installed underneath I-70 was also performed. Table 2 shows the calculated factors of safety. As it is observed, the factor of safety for the slope does not change, it is still less than 1 for the summer conditions, and slightly greater than 1 for the 3 other seasons. When looking at the difference in weight for each slice, the weight reduction is small compared to the total weight of the slice since this is a deep seated landslide. However, it is important to note that a factor of safety of 1 or less than 1 means failure whether it is for small movements or for large movements. Records indicate that after 2012 (the year when the caissons and horizontal drains were installed), the horizontal movement measured in the eastbound shoulder decreased significantly compared to the movement observed in 2008 and 2009. Looking at the yearly cumulative infiltration data provided by SNOTEL (Figure 17) it is observed that 2006 through 2009 were "wet years" (cumulative total infiltration was larger than the average), these years coincide with large movements measured by the inclinometers. In 2011 the cumulative infiltration was almost 50% larger than the average, but there is no inclinometer data to relate it to large or small slope movements. Finally, 2012, 2015 and 2016 were "dry years" and small movements were recorded. Therefore it is important to further investigate if the decrease in movement in the last years is due to dryer years or due to the horizontal drains maintaining a shallower water table.

Table 2. Factors of safety obtained for stability analysis with weight reduction due to caissons.

Season	Factor of Safety	
	No Caissons	With Caissons
Winter	1.05	1.06
Spring	1.05	1.06
Summer	0.95	0.95
Fall	1.04	1.05

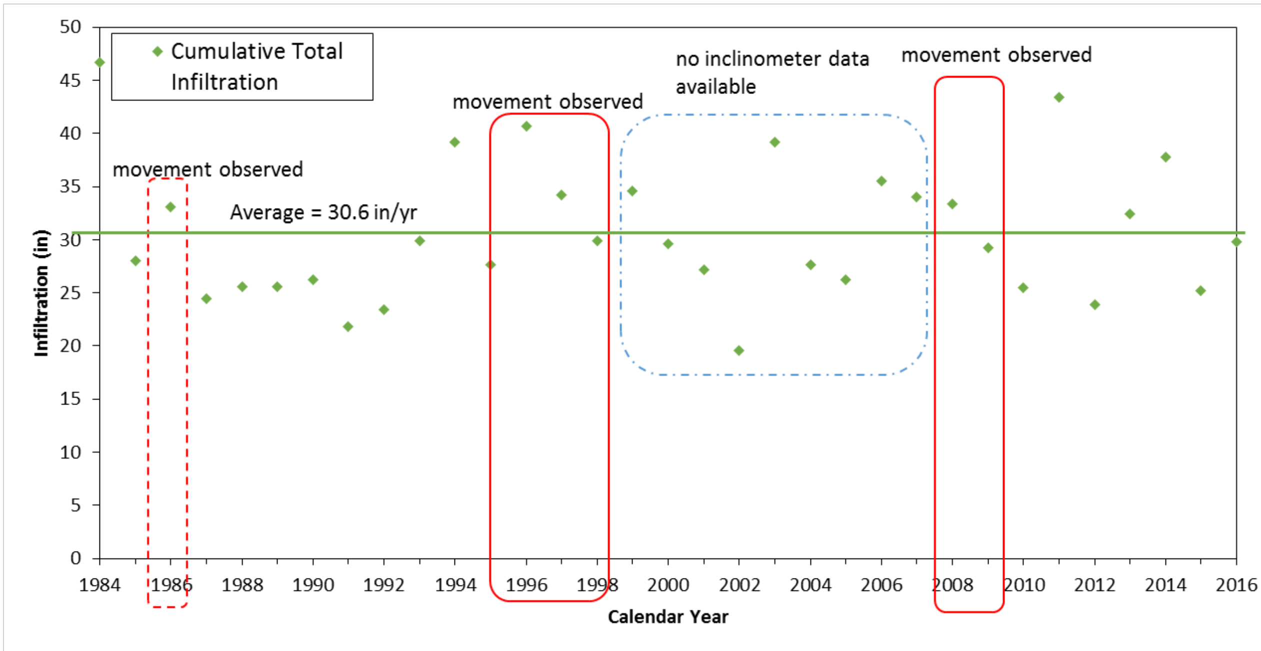


Figure 17. Cumulative total infiltration for 1984 - 2016 at Grizzly Peak (SNOTEL)

8. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Infiltration-induced landslides are a dangerous geological hazard in the United States and their occurrence result in expensive damages that often claim lives. Many of these landslides are triggered by a change in the hydrological conditions at the site. In Colorado, 124 landslides that affect roads have been identified. One site study (I-70 embankment) was identified to implement a novel approach that integrates field monitoring observations, laboratory testing, and a hydro-mechanical framework in the analysis and prediction of landslides.

During Phases I and II of this study, the site was characterized, the ground water table location was continuously monitored for over three years, displacement movements were monitored, hydrological and strength properties of the soil layers were obtained in the laboratory. All this information was then implemented in a numerical model that captured the hydrological behavior of the site and on a preliminary analysis of the slope stability under different conditions. From this study the following conclusions are obtained:

- An accurate characterization of the soil layers, stratigraphy, and atmospheric conditions is extremely important in the hydro-mechanical analysis of infiltration-induced landslides. These factors must be defined throughout the entire watershed, not only the immediate landslide area, to fully understand the hydrological conditions of the immediate landslide site.
- The unique hydrology of the Straight Creek landslide is a key factor in the stability of the site. The large difference in water table position in a relatively small distance is due to the large size of the watershed that allows a significant amount of infiltration into the hillslope, the contrast of hydrological properties of soils in the watershed that control the direction, speed, and amount of excess groundwater flow that can travel through the slope, and the steepness of the bedrock and flow boundary in the northern slopes of the watershed.
- Introducing soils to a slope with different engineering properties greatly affected the hydrology of the site.
- Focus of remediation options on the large water table rise north of I-70 may be most effective.
- A lot of information has been obtained on the site. A third phase is needed to perform a detailed slope stability analysis that includes a parametric study for different conditions. This is particularly important considering that hydrological history has an effect on the slope behavior and observing

SNOTEL records that indicate large variations in infiltration in different years. With that information recommendations for site remediation can be provided with sound scientific basis.

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APPENDIX A

ADDITIONAL INFORMATION ON NATIONAL SURVEY OF DEPARTMENTS OF TRANSPORTATION

A survey was sent to all 50 state Departments of Transportation (DOT) to determine if infiltration-induced landslides had occurred or impacted highways, and if so, we compiled their solutions and recommendations. We received 38 responses; the information obtained is summarized below.

A.1. Number of landslide events.

During the last 5 years along roadways, the number of landslide events was (Table A.1):

Table A.3. Reported number of landslides in the past 5 years

Number of landslides in past 5 years	States reporting
0-5	Arizona, Connecticut, Georgia, Florida, Hawaii, Maryland, Nevada, New Jersey
6-15	Illinois, Maine, Massachusetts, Michigan, Nebraska, New Hampshire, New Mexico, South Dakota,
16-30	Arkansas, Kansas, Louisiana, Mississippi, North Dakota, Utah, Wisconsin,
31-100	Alabama, Alaska, Kentucky, Minnesota, Missouri, Ohio, Oregon, Tennessee, West Virginia, Wyoming,
100 or more	Colorado, Iowa, Montana, North Carolina, Washington

States that reported 100 or more landslides include: Colorado, Iowa, Montana, North Carolina, and Washington. It should be noted that California, Pennsylvania, and New York did not respond to the survey; however they monitor hundreds of landslides each year along highways and corridors. The California Department of Transportation reports spending $22 million per year managing landslides along about 1200 miles of landslide-prone highways. Pennsylvania reported in May 2011 anticipating as much as $25 million cost to fix and prevent problems of infiltration induced landslides. New York reports high landslide incidence on the east area.

A.2. Infiltration induced landslides:

- <u>The States that reported 0 – 5 events</u>, indicated that only 1 or 2 of the events they had were infiltration induced, they report low frequency (no failures in the past 5 years) to moderate frequency (1 – 2 periods of movement in the last 5 years)
- <u>The States that reported 6 - 15 events,</u> indicated that most of their landslides are infiltration induced, the frequency reported is either moderate or annual, almost all are minor slides, at least 50% of those may be classified as shallow landslides. Shallow landslides were defined as a slide in which

the sliding surface is within the soil mantle or weathered bedrock and which has a depth of a few decimeters to a couple of meters.

- The States that reported 16 - 30 events, indicated that at least 50% of their landslides are infiltration induced, about half of them can be classified as shallow landslides, they report moderate and annual event frequency. Most of their solutions consist of replacing the failed mass with rock.
- The States that reported 31 - 100 events, indicated that most of their landslides are infiltration induced, in general, less than half can be classified as shallow landslides, they reported mostly annual and continuous frequency of events.
- The States that reported 100 or more events, indicated in most cases that 100 or more of the events were infiltration induced, 31 to 100 of the events can be classified as shallow landslides, and the frequency of the events ranges from moderate to continuous.

A.3. Risk value assigned to landslides:

The states of Washington, Oregon, Colorado, and Ohio assign a risk value to each landslide, which in some cases serves as a guidance for remediation. The Arizona and Nevada Departments of Transportation (ADOT and NDOT) use the Rockfall Hazard Rating System (RHRS) for cut slopes and some embankments. Wyoming uses a priority-based system that solely takes into account lane/road impedance. Maine is in the process of developing a system (Diaz et. al, 2008; Lowell and Morin, 2000; WSDOT 1995, WSDOT 2001, WSDOT 2002).

A.4. Monitoring instrumentation:

The instrumentation used by most DOTs include: visual inspection, conventional survey and slope inclinometers, rain gauges, piezometers, open air standpipes, observation wells, side scanning radar, survey positions and control points

A.5. Recommendations:

When dealing with infiltration induced landslides, most DOTs recommend design drainage system to minimize infiltration, improve sub-drainage, and horizontal drainage. For landslides that are classified as small or medium (<500 ft width, < 50ft depth) a common recommendation is to excavate the failed mass and replace with rock. States that report more than 30 landslides in the past 5 years generally work closely with the United States Geological Survey, they perform complete geotechnical investigations including ground water characterization. For large historical landslides where it is not economical or technically feasible to remediate them, the "balance approach" is used. This is a mitigation approach that seeks to slow the movement of the failure surface enough so that it can be managed through standard to heavy activities. Other typical recommended solutions include: Reinforce soil slope, french drains, soil replacement with geogrid, berms, rock buttresses, rock shear keys, construct pile walls with the pile tips embedded 10 ft into bedrock, soil nails, and soil anchors.

State DOT Contacts:

State	Contact name	Email	Phone	Answered
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Alaska	Dave Stanley	dave.stanley@alaska.gov	907-269-6236	y
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Wisconsin	Bob Arndorfer	robert.arndorfer@dot.wi.gov	608-246-7940	y
Wyoming	Jim Coffin	jim.coffin@wyo.gov	(307) 777-4205	y

Survey

Landslides Survey: http://www.surveymonkey.com/s/PVQDRFL

1. Name of Responder:

Name of Responder:

2. Phone Number of Responder:

Phone Number of Responder:

3. Email of Responder:

Email of Responder:

4. Please estimate the number of landslide events, within the last 5 years, along roadways within your state.

- 0 - 5
- 6 - 15
- 16 - 30
- 31 - 100
- 100 or more

5. About how many of the landslide events are either rainfall or snow-melt induced?

- 0 - 5
- 6 - 15
- 16 - 30
- 31 - 100
- 100 or more

6. About how many of the landslides can be classified as a shallow landslide?

- 0 - 5
- 6 - 15

- 16 - 30
- 31 - 100
- 100 or more

7. The failure frequency of most of the landslides monitored in your State is:

- Low: No failures in previous 5 years
- Moderate: 1 - 2 periods of movement in previous 5 years
- Annual: Movement observed on annual basis
- Continuous: Multiple movement episodes in one year

8. Has your department monitored soil moisture, displacement, and/or rainfall at the landslide sites? If so, please describe or list the monitoring techniques used.

9. Please list, or briefly describe any remediation measures your state's transportation department may use for rainfall-induced landslide damage scenarios. If you have documented cases, could you provide us a link to or the actual documentation?

10. Does your department assign a risk value to each landslide? If so, what are the variables that are taken into account for obtaining the risk value?

Determining the risk value for a slope

In 1993, WSDOT established the Unstable Slope Management System (USMS) to evaluate all unstable slopes, perform early project scope and cost estimation, perform cost-benefit analyses, and prioritize mitigation of unstable slopes (Lowell and Morin, WSDOT, 1995, WSDOT, 2001, WSDOT, 2002, Lowell et al., 2005, WSDOT 2010). WSDOT monitors about 3,100 unstable slopes which are scored using a numerical rating system based on 11 criteria (Table 1). WSDOT prioritize slope remediation based on 1) highway functional class, 2) USMS numerical rating, and 3) average daily traffic (Table 2). In addition, the field notes uploaded into USMS include at least 2 photos displaying both approaches, a typical cross section of the slope, impact of failure, rock mass characterization, types of instability, mitigation alternatives, and any additional notes pertinent to the site.

Table 4. USMS rating criteria (from WSDOT, 2010)

Category	Points = 3	Points = 9	Points = 27	Points = 81
Problem Type: Soil	Cut or Fill slope erosion	Settlement or piping	Slow moving landslides	Rapid landslides or debris flow
Problem Type: Rock	Minor rockfall Good catchment	Moderate rockfall Fair catchment	Major rockfall Limited catchment	Major rockfall No catchment
Average Daily Traffic	< 5,000	5,000 to 20,000	20,000 to 40,000	> 40,000
Decision Sight Distance	Adequate sight distance	Moderate sight distance	Limited sight distance	Very limited sight distance
Impact of Failure on Roadway	< 50 Feet	50 to 200 Feet	200 to 500 Feet	> 500 Feet
Roadway Impedence	Shoulder only	1/2 Roadway	3/4 Roadway	Full Roadway
Average Vehicle Risk	< 25% of the time	25% to 50% of the time	50% to 75% of the time	> 75% of the time
Pavement Damage	Minor - not noticeable	Moderate - driver must slow	Severe - driver must stop	Extreme - not traversable
Failure Frequency	No failures in last 5 years	One failure in last 5 years	One failure each year	More than one failure
Annual Maintenance Costs	< $5,000 per year	$5,000 to $10,000 per year	$10,000 to $50,000 per year	> $50,000 per year
Economic Factor	No detours required	Short detours < 3 Miles	Long detours > 3 Miles	Sole access - no detours
Accidents in Last 10 Years	0 to 1	2 to 3	4 to 5	> 5

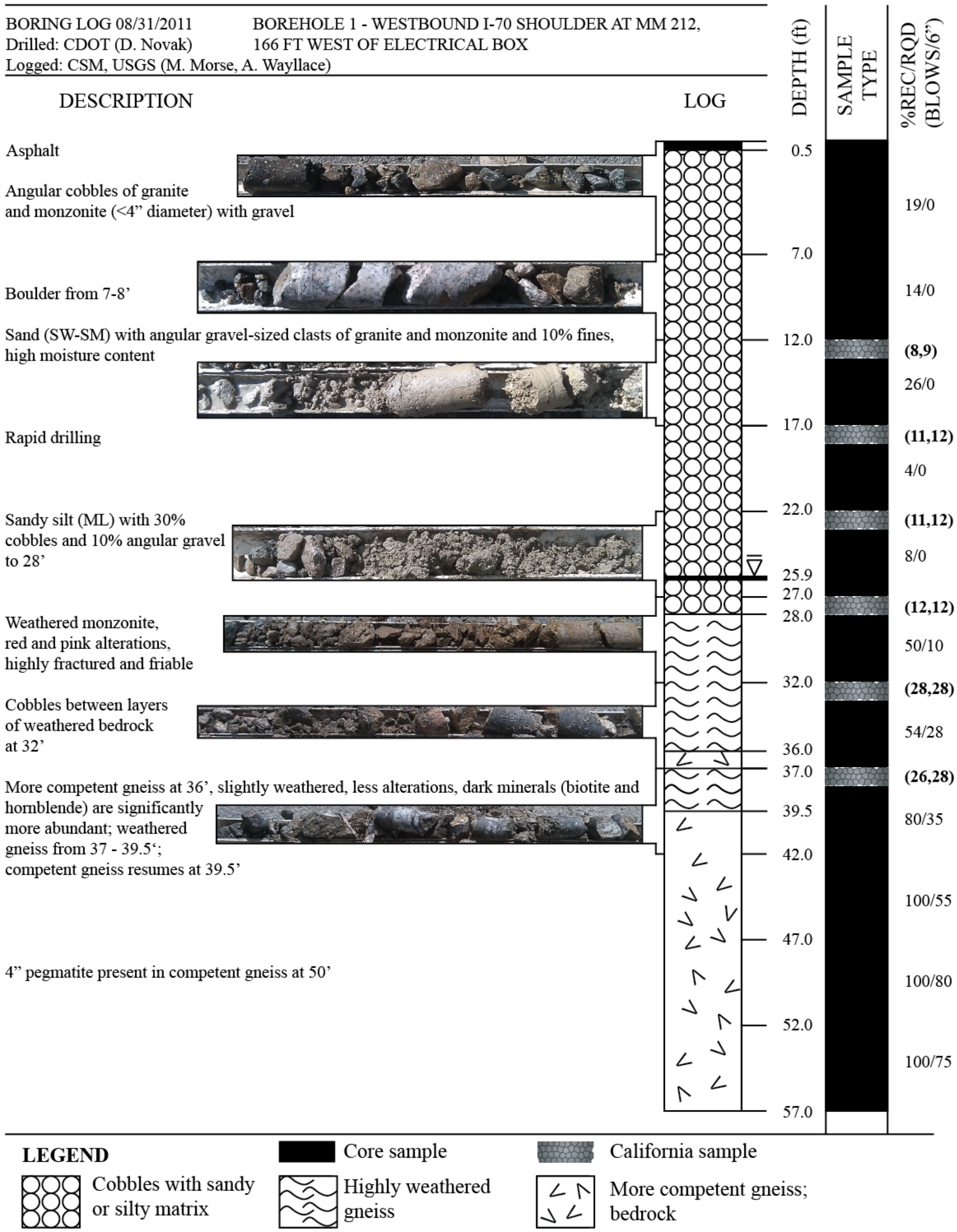
Table 5. Risk reduction rating criteria (from WSDOT, 2010)

Category	Points = 3	Points = 9	Points = 27	Points = 81
Slope Height	< 25 ft.	25 to 50 ft.	50 to 75 ft.	> 75 ft.
Ditch Effectiveness	Good catchment	Moderate catchment	Limited catchment	No catchment
Total Roadway Width	< 40 ft.	32 ft.	24 ft.	< 24 ft.
Rockfall History	Few falls	Occasional falls	Many falls	Constant falls
Number of Maintenance Calls per Year	< 1	1 to 3	4 to 5	> 5
Rockfall Block Size	< 1 ft.	1 to 2 ft.	2 to 3 ft.	> 3 ft.
Volume of Rockfall per Year	< 3 cyd.	3 to cyd.	6 to 10 cyd.	> 10 cyd.
Average Daily Traffic	< 500	500 to 2,750	2,751-5,000	> 5,000

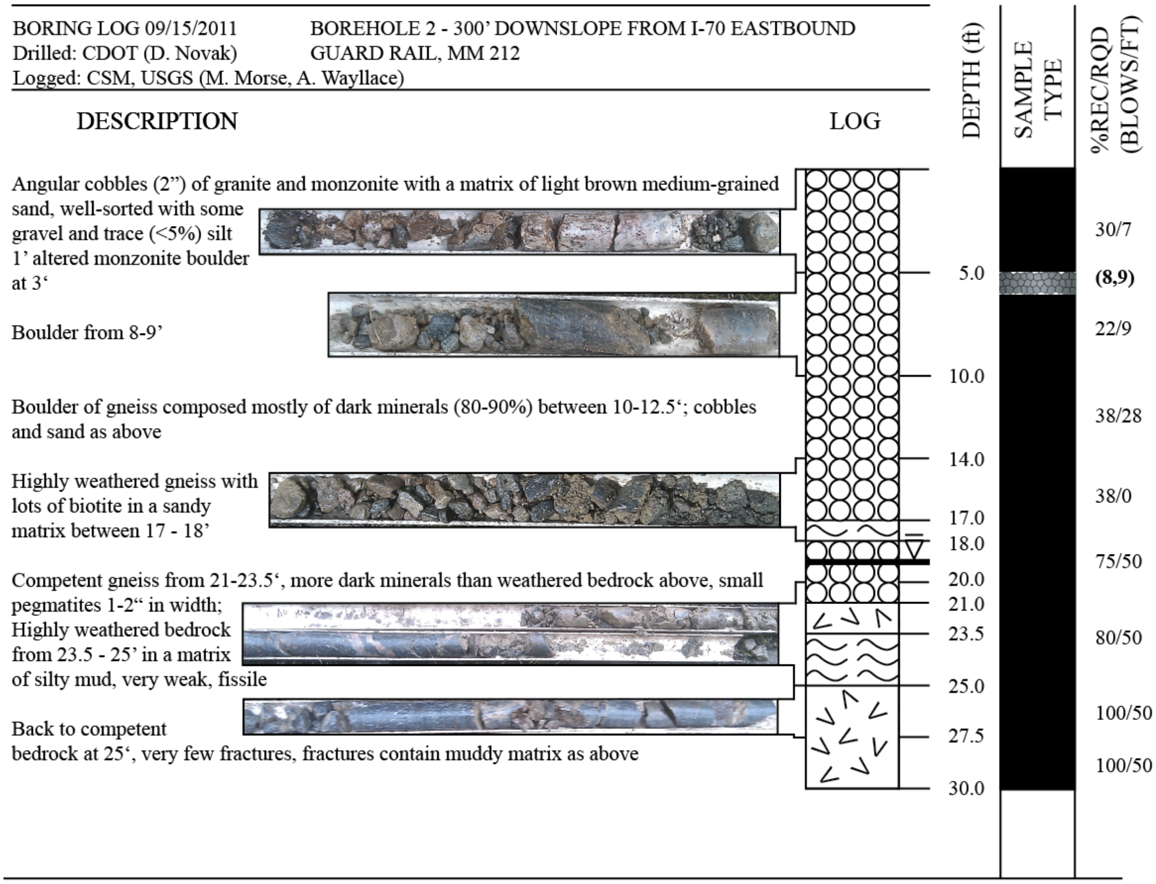
APPENDIX B

CSM BOREHOLE LOGS

Westbound (P1/INC4) – log by M. Morse, CSM (2011)



Toe (P2) – log by M. Morse, CSM (2011)



CALIFORNIA SAMPLE DESCRIPTION (5')

Liner 1:	No recovery
Liner 2:	No recovery
Liner 3:	No recovery
Liner 4:	**SW-SM** Brown silty sand with cm-scale gravel (10%)

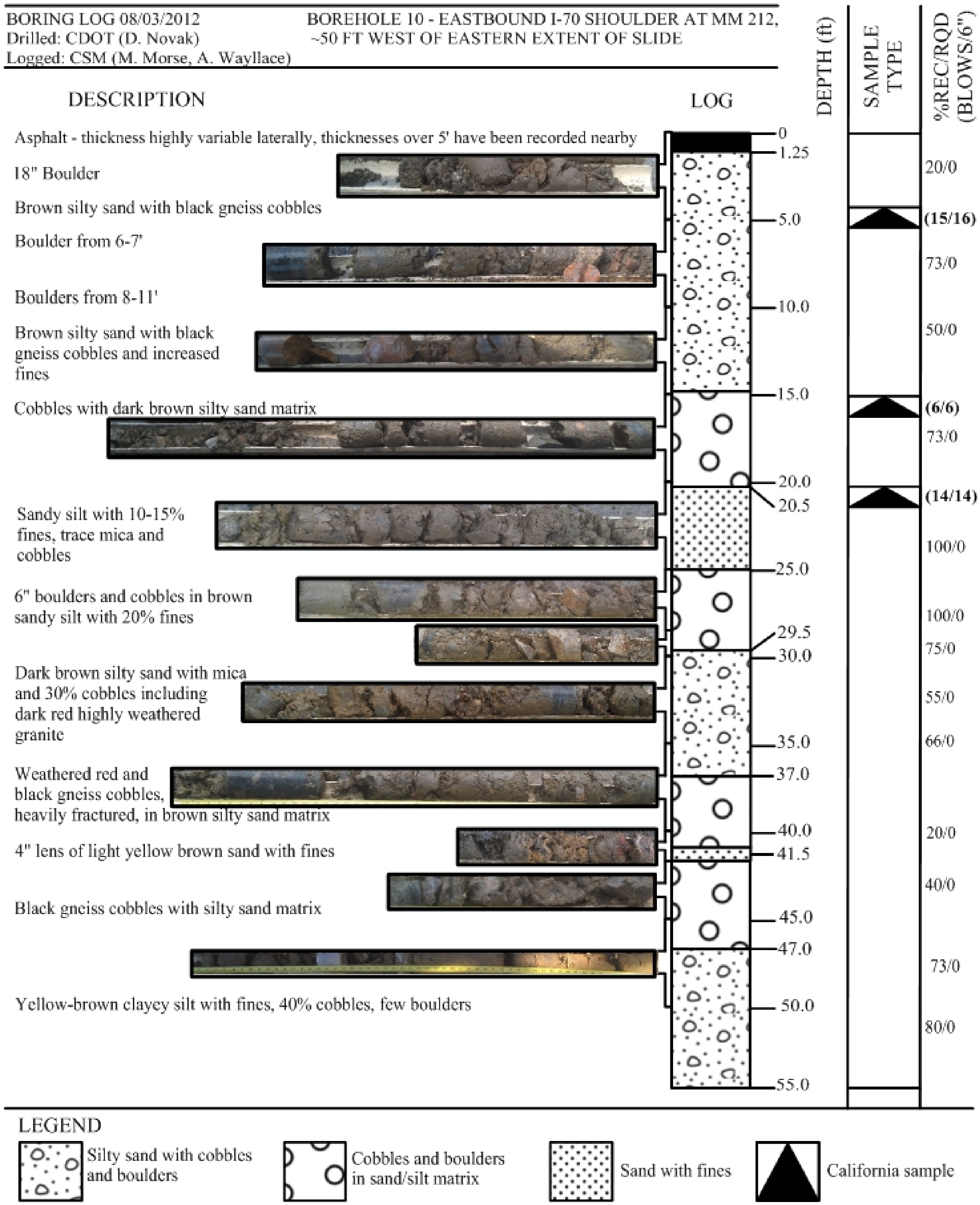
DRILL NOTES

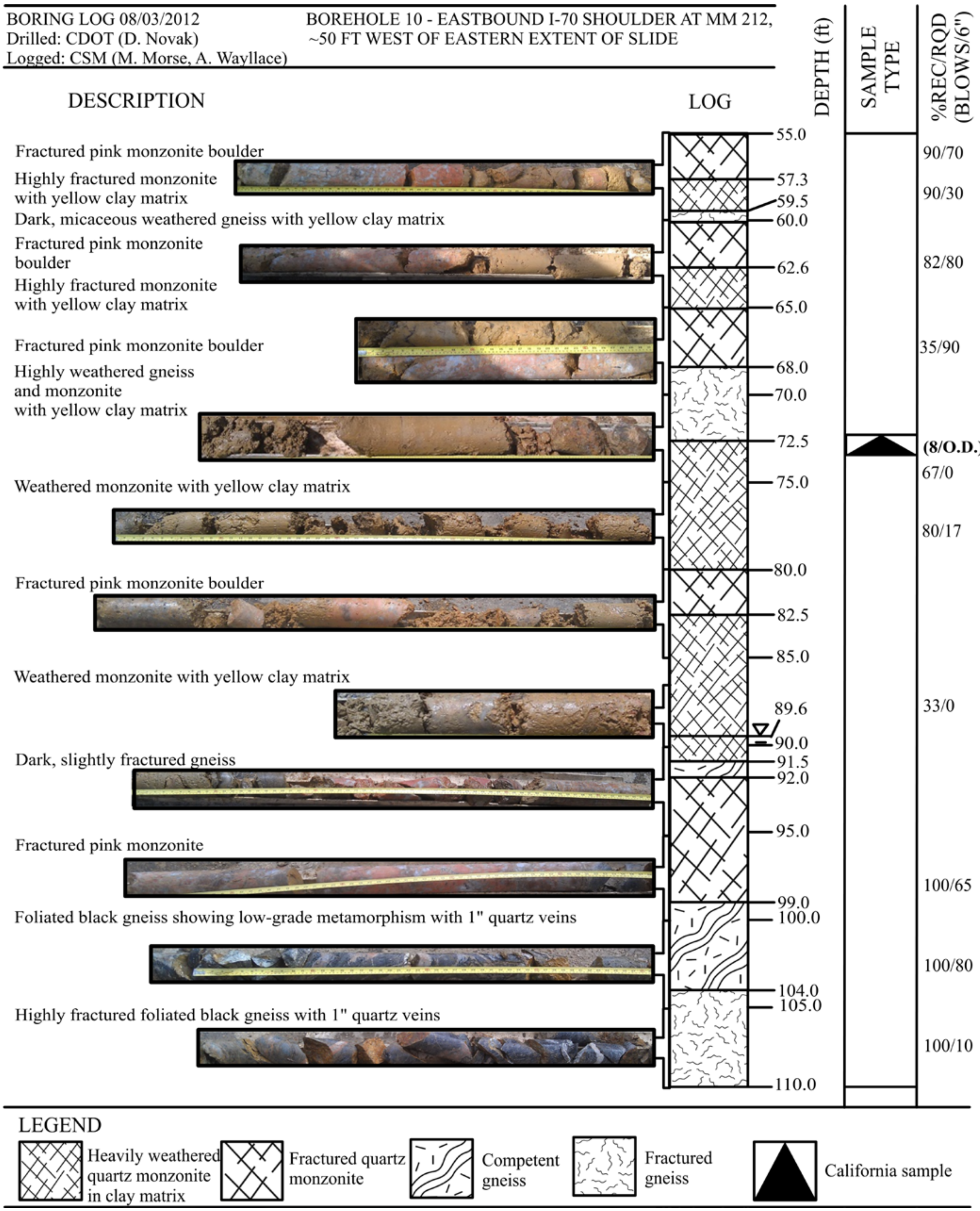
Drill rig: CME 55/300
Drill bit inner diameter = 2.5"
Total depth = 30.0'

LEGEND

- Cobbles with sandy or silty matrix
- Core sample
- Highly weathered gneiss
- California sample
- More competent gneiss; bedrock

Eastbound (P3/INC5) – log by M. Morse, CSM (2012)




BORING LOG 08/03/2012
Drilled: CDOT (D. Novak)
Logged: CSM (M. Morse, A. Wayllace)
BOREHOLE 10 - EASTBOUND I-70 SHOULDER AT MM 212,
~50 FT WEST OF EASTERN EXTENT OF SLIDE
DESCRIPTION
LOG
DEPTH (ft)
SAMPLE TYPE
%REC/RQD (BLOWS/6")
Fractured pink monzonite boulder
Highly fractured monzonite with yellow clay matrix
Dark, micaceous weathered gneiss with yellow clay matrix
Fractured pink monzonite boulder
Highly fractured monzonite with yellow clay matrix
Fractured pink monzonite boulder
Highly weathered gneiss and monzonite with yellow clay matrix
Weathered monzonite with yellow clay matrix
Fractured pink monzonite boulder
Weathered monzonite with yellow clay matrix
Dark, slightly fractured gneiss
Fractured pink monzonite
Foliated black gneiss showing low-grade metamorphism with 1" quartz veins
Highly fractured foliated black gneiss with 1" quartz veins
55.0
57.3
59.5
60.0
62.6
65.0
68.0
70.0
72.5
75.0
80.0
82.5
85.0
89.6
90.0
91.5
92.0
95.0
99.0
100.0
104.0
105.0
110.0
90/70
90/30
82/80
35/90
(8/O.D.)
67/0
80/17
33/0
100/65
100/80
100/10
LEGEND
Heavily weathered quartz monzonite in clay matrix
Fractured quartz monzonite
Competent gneiss
Fractured gneiss
California sample

North (P4) – B. Thunder (2015)

BORING LOG 11/10/2015
Drilled: CDOT (A. Moreno)
Logged: CDOT (J. Sieberg)

BOREHOLE 4 – 70M UPSLOPE OF I-70 SHOULDER, MM 212.0-212.1

Description	Log	Depth	RQD
Colluvium top soil		3’	
Highly fractured granite and gneiss, pebble size pieces, clean fractures			0%
		20’	18%
			0%
		33’	
Less fractured granite with muscovite veins			48%
		36’	
Heavily weathered granite with trace clay infilling			0%
		40’	
Off-white granite bedrock with some horizontal fractures and muscovite zones			95%
		50’	

APPENDIX C

FIELD INSTRUMENTATION SET UP

C.1. Installation of inclinometer casing and piezometer in I-70 East bound

A vibrating wire piezometer and a 7cm (2.75 inch) inclinometer casing were installed at (33.5 m) 110 ft of depth on the shoulder of the East bound of I-70. The drilling of the borehole was performed by Dave Novak (CDOT) using a CP drill system with a drill bit of 88 mm (0.26 ft) ID. During drilling, information was logged and California samples were obtained. The main observations from the drilling are as follows:

From 0 to 14.3 m (0 - 47.0 ft) below the pavement, the material extracted was mostly a dark brown silty sand with some fines (<10%), as well as cobbles of weathered black gneiss and weathered pink, quartz monzonite. Cobbles ranged in size from cm-scale to dm-scale boulders (Figure C.1). At 14.3 to 27.7 m (47 to 91.5 ft) depth the regolith became yellow-brown silty clay with about 30% gravel-sized grains, with a higher abundance of weathered quartz monzonite boulders and cobbles. Boulders of fractured quartz monzonite contained cm-scale inclusions of biotite. Fractures within the rock were filled with the yellow-brown silty clay. At 22 m (72.5 ft) depth the quartz monzonite became extremely weathered, breaking off in cm- and mm-scale grains with the slightest effort. The weathered rock was mixed in with the silty clay, and the rock increased in competence - measured by ease of breakability and occurrence of large clasts - with depth from 22.9 to 27.9 m (75 to 91.5 ft) (Figure C.2). After 27.9 m (91.5 ft) depth, a very competent quartz monzonite (100% recovery, ~80% RQD) was extracted from the borehole. The more competent quartz monzonite became a foliated, black (~70% dark minerals) gneiss at 30.2 m (99 ft) depth. The relatively competent (~100% recovery, ~80% RQD) gneiss was found until the base of the borehole drilled to 33.5 m (110 ft) (Figure C.3).

A reading with a watermeter of ground water table was obtained at 28.9 m (95 ft) of depth. The samples obtained from shallower depths were not saturated.

Figure C.1. Dark brown silty sand obtained from 0 to 47 ft

Figure C.2. Yellow brown silty clay obtained from 47 ft to 91.5 ft

Figure C.3. Sharp transition between clay/weathered bedrock layer and bedrock at 91.5 ft

Some of the steps during the piezometer and inclinometer casing installation are provided in Figures C.4. and C.5.

Figure C.4. Drilling on shoulder of East bound of I-70, August 3rd, 2012

Figure C.5. Piezometer installed in a cage on the tip of the inclinometer casing

Piezometer Equipment Information

Dataloggers:	(2) Campbell Scientific CR10X datalogger
	(2) Campbell Scientific AVW200 vibrating wire analyzer
Piezometers:	(4) Geokon 4500S Vibrating Wire Piezometer – 350 kPa capacity
Software:	PC200W - datalogger communication
	LoggerNet - datalogger program writing
	Microsoft Excel – piezometer calibration

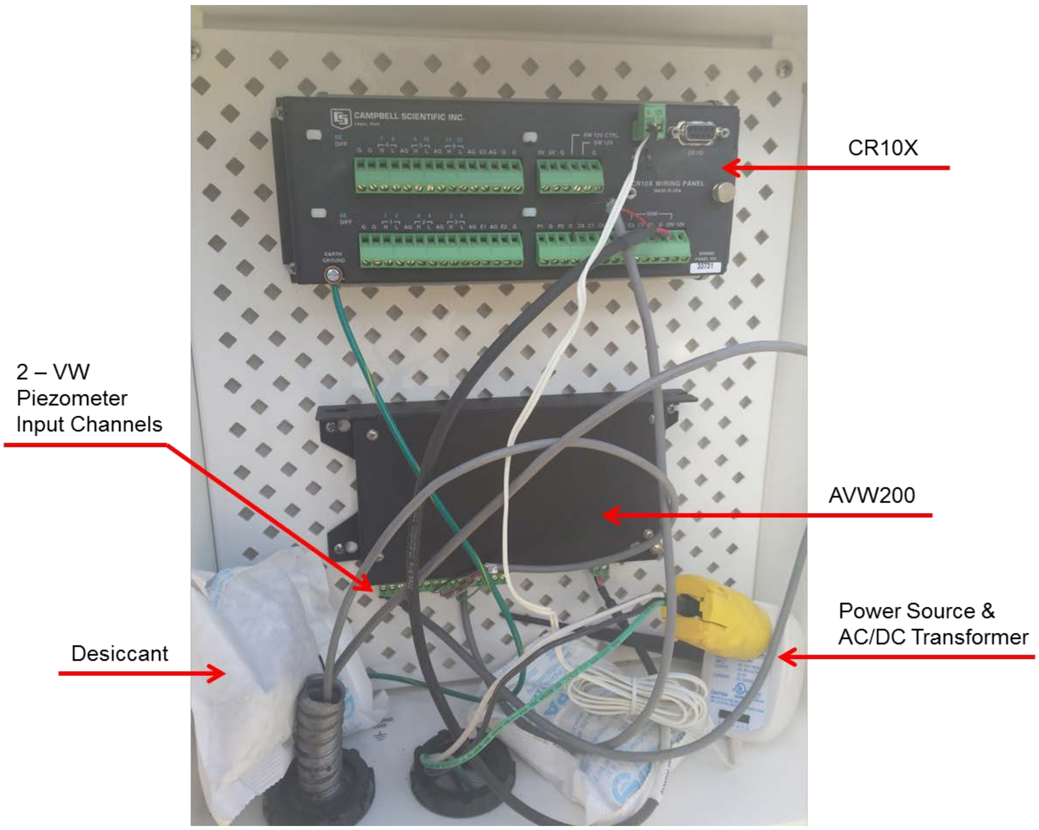


Figure C.6. Sample datalogger field setup with CR10X and AVW200 sampling 2 VW piezometers

Piezometer Installation

1. Calibrate piezometer in laboratory to obtain polynomial coefficients (see Geokon 4500S manual)

Figure C.7 Laboratory calibration of vibrating-wire piezometer

2. Splice cable to extend wire to datalogger box location.

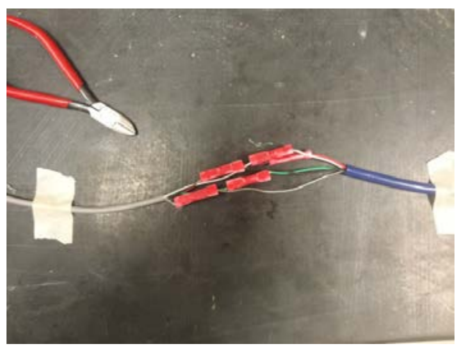


Figure C.8 Piezometer cable splicing in laboratory

3. Field calibrate piezometer in borehole prior to installation and test accuracy of instrument with simultaneous water table indicator or similar device.

Figure C.9 Field calibration and accuracy check of piezometer in borehole

4. Run piezometer cable through aluminum conduit to protect from animals, weather, etc.

Figure C.10 Piezometer cable protected with aluminum conduit

5. Install piezometer in borehole, tip up to allow any air to escape instrument.

Figure C.11 Piezometer oriented to stay tip-up when installed in borehole

Figure C.12 Piezometer placed in borehole

6. Measure installation depth of piezometer and place clean, coarse sand filter with bentonite seal.

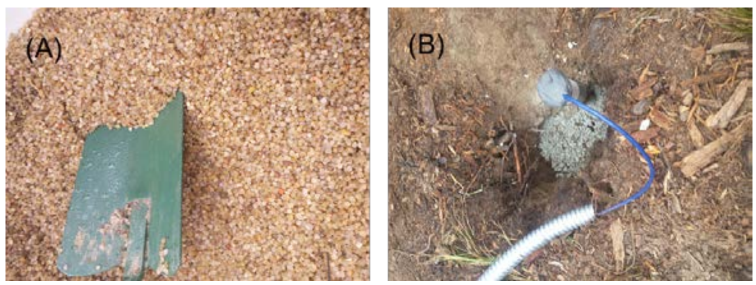


Figure C.13 Piezometer installation (a) coarse sand used for filter, (b) borehole sealed with bentonite

7. Wire piezometer to AVW200 and seal joints in conduit.