CASE HISTORY OF A REACTIVATION OF A LANDSLIDE DUE TO IRRIGATION ON UNSATURATED SOIL

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Abstract: An engineering investigation was conducted to determine mitigation methods for stabilizing an ancient landslide in Colorado, USA. A 0.71-km^2 golf course was constructed in 1973 on the ancient landslide, which was located in a semi-arid environment along the Front Range of the Rocky Mountains. As a result of the irrigation of the golf course, the landslide was reactivated in 1993, and additional movement occurred after a period of high precipitation in 1995. Seepage analyses were conducted to simulate long-term ground water conditions at the golf course. The results of the seepage analyses indicated that the increased infiltration rate, over the more than 20 year presence of the golf course, caused the increase in ground water elevations responsible for the reaction of the landslide. Slope stability analyses were conducted to simulate progressive slope movements observed in the field. This paper discusses the observations of the slope movement, the analyses that were conducted, and proposed remediation methods.

1. INTRODUCTION

An engineering investigation was conducted to determine mitigation methods for stabilizing a large ancient landslide in Colorado, USA. The ancient landslide was located in a semi-arid environment along the Front Range of the Rocky Mountains. In its natural state it was covered with scrub oak vegetation, and had a large depth of unsaturated soil over the groundwater level. A 0.71-km^2 golf course was constructed in 1973 on the ancient landslide. As a result of the irrigation of the golf course, the landslide was reactivated in 1993, and additional movement occurred after a period of high precipitation in 1995.

The construction of the golf course modified the site hydrology by adding irrigation water inflows and by changing the vegetation over 55 percent of the total area from native grass and scrub oak to turf grass. Seepage analyses were conducted that considered infiltration into the vadose zone due to irrigation and precipitation. Analyses of the irrigation and precipitation rates and the turf grass water consumption rates showed a relatively high infiltration rate in the turf grass areas compared to the unirrigated native areas that were covered with scrub oak. Long-term computer simulation of the ground water levels showed that the increased infiltration rate in the irrigated turf grass areas, over the more than 20 year presence of the golf course, caused the increase in ground water elevations responsible for the reaction of the landslide.

Field observations indicate that the slope movement occurred along the surface of the shale and that only sections of the ancient landslide failed rather than the entire ancient landslide. Therefore, the computer simulation was calibrated to the field observations considering progressive slope movements. The results of the progressive slope movement agree closely with the observed field movements recorded in the slope inclinometers installed at the site. The correlation of the results of the stability model with the field observations provides verification to the calibration of the stability simulation.

This paper discusses the observations of the slope movement, and the analyses that were conducted. It demonstrates the processes responsible for gradual long-term destabilization of the previously unsaturated slope and shows the importance of the depth of perched water over the failure plane. Potential remediation methods are discussed.

2. SITE BACKGROUND

The site is located on the east-facing slopes of Cheyenne Mountain in Colorado, USA and ranges in elevation from 1,881 to 2,012 meters. The site is incised by a moderately sized drainage that flows northeasterly. The natural vegetation consists of scrub oak, small shrubs, and grasses. The site is situated entirely within the boundaries of an ancient landslide. The landslide is believed to have originated in rapid flow from the side of the mountain west of the site.

A golf course was developed on the site in the 1970s. Development included excavating cuts and placing fills to generate planar surfaces for fairways and greens, constructing a club house, constructing a maintenance building, constructing golf cart paths and other hard scaping, installing drainage features and installing an irrigation system. Figure 1 shows the plan view of the golf course.

Slope movement at the site was first recorded in the spring of 1993. A local geotechnical consulting firm began geotechnical investigations shortly after the movement began and designed and installed a mitigation dewatering system by the late spring of 1994. Additional movement occurred after heavy spring rains in 1995. Our investigation of the causes of slope movement was performed in 1995.

3. FIELD INVESTIGATIONS

3.1 Ground Water Field Investigation

A ground water field investigation was conducted to collect data on ground water occurrence and ground water flow at the golf course. The field investigation included installing standpipe and pneumatic piezometers; conducting recovery, slug, and pumping tests; and inspecting horizontal drains. These tests provided information on aquifer parameters that were used in ground water modelling of the site.

3.2 Surface Water Field Investigation

A surface water analysis of the site consisted of flow measurements in the site drainages, infiltration measurements, surface soils characterization, and mapping of areas with standing water. These components of the analysis were necessary to quantify the response of the site to irrigation, precipitation, and storm flows.

3.3 Slope Movement Investigation

Fifty-six surface monuments were installed throughout the golf course to monitor the magnitude and direction of the ground surface movement. These surface monuments were installed in December 1995 and were surveyed in April 1996. The survey results indicated that the surface monuments did not show any movement from December 1995 to April 1996.

Eight slope inclinometers (INC-1 through INC-8) were originally installed on the golf course in July 1993 to monitor the movements of the landslide at depth. Inclinometers INC-1 through INC-4 and INC 6 were sheared off in April 1995 due to the movement. They were replaced, and subsequently were sheared off again. Therefore, no data was obtained from these inclinometers after April 1995. Inclinometers INC 5 and INC 7 were monitored through August 1996. Inclinometer INC 8 was monitored through July 1996. An example of the slope movements obtained from Inclinometer INC-8 is shown in Figure 2. The surface movement shown in Figure 2 for INC 8 is approximately 5.6 cm and indicates that the slope movement occurred at the interface between the shale and the weathered claystone.

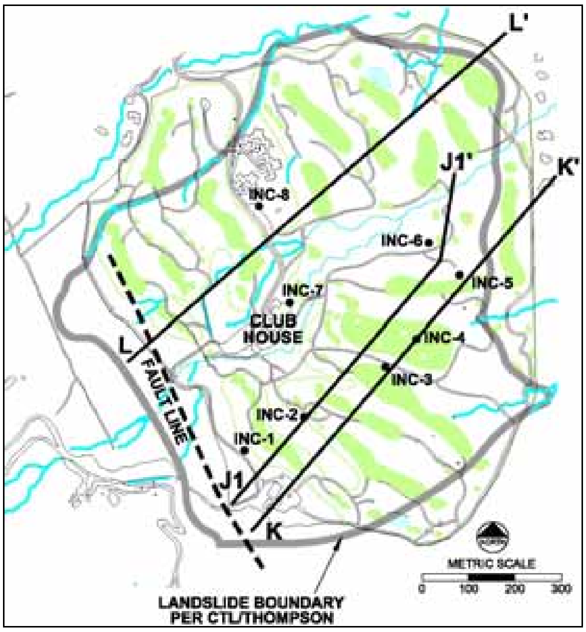

Figure 1. Plan View of the Golf Course and Locations of Cross Sections K-K', L-L', and J1-J1'.

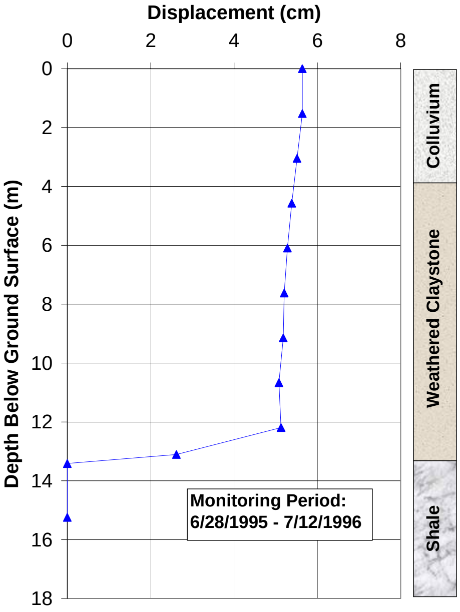

Figure 2. Displacement vs. Depth Below Ground Surface for Inclinometer INC-8.

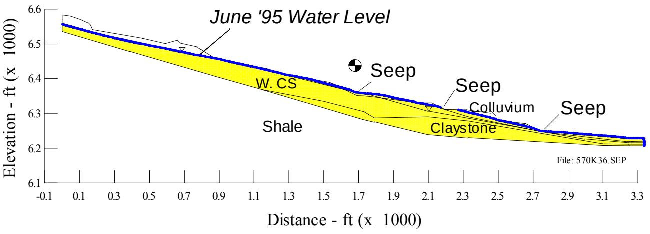

Figure 3. Steady-State Seepage Analysis - Model Calibration.

Table 1. Summary of Typical Soil Properties at the Golf Course.

Lab Testing	**Material Type**			
	Colluvium	**Weathered Claystone**	**Claystone**	**Shale**
Water Content (%)	15 – 22	21 – 33	15 – 26	8 – 15
Dry Density (g/cm^3)	1.50	1.38	1.55	1.62
Atterberg Limits LL/PL* (%)	49 – 64 / 17 – 21	-	-	-
Specific Gravity	2.66	-	2.74	-
Hydraulic Conductivity (cm/sec)	4.8×10^{-9}	3.1×10^{-8}	1.3×10^{-8}	-
Cohesion (kPa)	-	12.0	-	-
Internal Angle of Friction (%)	-	29	-	-

*LL/PL: Liquid Limit / Plastic Limit

4. LABORATORY TESTING

The soil samples obtained during the field exploration were tested for classification and material properties for use in evaluation of the subsurface conditions and to aid in the mitigation plan design for the project. The soil samples were identified as colluvium, weathered claystone, claystone, and shale. The tests performed on each of the different materials included water content, dry density, Atterberg Limits, flexible wall permeability tests, specific gravity, and direct shear tests. Results of the laboratory tests were summarized in Table 1.

5. GROUND WATER ANALYSES

Ground water modelling was performed to evaluate the effect of irrigation and horizontal drains, and to predict the effect of additional drains on the ground water level. Four scenarios that were modelled included:

1. Model calibration
2. Historic rise of water levels
3. Horizontal drain simulations
4. Vertical well simulations

The seepage analyses were analyzed using the finite element method computer program SEEP/W (GEO-SLOPE, 1995a). Analyses were conducted to predict ground water flow and the distribution of pore water pressure within the shallow aquifer (landslide mass). The seepage computations were conducted under both steady-state and transient conditions for cross sections K-K' and L-L'. The locations of the cross sections analyzed are shown in Figure 1. Cross sections K-K' and L-L' were determined to be critical sections based on the direction of the landslide and the maximum height of water above the shale. The transient analyses modelled the rise of the ground water level over time since the development of the golf course, and then predicted the time required to lower the water level after installation of drains.

5.1 Model Calibration

The hydraulic conductivity of the soil materials above the shale was calibrated by varying the hydraulic permeability until the computed phreatic surface matched the observed water level in June 1995. The calibrated permeability for the soils is 2.5×10^{-5} cm/sec for both cross sections. The calibrated permeability value of 2.5×10^{-5} cm/sec for the soils is higher than permeability values obtained from the recovery and slug tests. The calibrated permeability is expected to be higher due to the existence of gravel and sand pockets.

The predicted phreatic surface computed by SEEP/W for cross section K-K' with an infiltration rate of 2.5 cm/month is shown in Figure 3 as shaded areas, together with the June 1995 water level data obtained from the piezometers. The infiltration rate of 2.5 cm/month was obtained from our water balance analysis (SMI, 1997). Due to good matches between the computed piezometric surface and the field-measured water levels for cross sections K-K' and L-L', confidence can be placed in the seepage analysis models.

5.2 Historic Rise of Water Levels

Using the calibrated hydraulic conductivity and infiltration values obtained from the surface water study (SMI, 1997), the rise in elevation of the ground water level during the period from the construction of the golf course to the present was modelled. The initial condition of the phreatic surface before construction was determined by modelling the site with zero net infiltration. The predicted phreatic surface for cross section K-K' is presented in Figure 4. This figure indicates that the pre-construction water level was significantly lower than it is at the present time. Stability analysis results show that the slope with the pre-construction phreatic surface was stable.

The transient phreatic surface was modelled using an infiltration rate of 2.5 cm/month on the playing areas. Figure 5 shows the resulting rise of the phreatic surface at different time stages after the irrigation began at the golf course. Figure 5 indicates that the modelled phreatic surface took approximately 21 years to rise to levels matching the June 1995 field data. The time period for the rise of the phreatic surface to the present location closely matches the length of time since construction of the golf course.

5.3 Horizontal Drain Simulations

The fluctuation of the drawdown and the discharge from horizontal drains in different permeable materials were simulated using a 2-dimensional model. Horizontal drains were modelled using values of hydraulic conductivity of the landslide material ranging from 1.0×10^{-4} to 2.5×10^{-7} cm/sec. The calculated infiltration rates obtained from our water balance modelling at specific time periods were used as an input function in the SEEP/W modelling (SMI, 1997). The computer model predicted the resulting phreatic surface and the drain discharge for the specific infiltration rates. Results of our drain simulations indicate that for soil with a permeability of 2.5×10^{-5} cm/sec or higher, the drain would lower the phreatic surface to the depth of the drain out to a distance of about 30 meters or more. For soil with a permeability of 2.5×10^{-6} cm/sec, the drain is much less effective and for soil with a permeability of 2.5×10^{-7} cm/sec, the drain has no effect.

The modelled discharge was compared to measured drain flow rates from Drains CH-1 and DR-1, which are located in areas with similar permeabilities. The discharge at the simulated horizontal drain are plotted with respect to time in Figure 6 for permeability of 2.5×10^{-5} cm/sec. The model fits the measured data well and predicts the actual fluctuations in discharge that follow major precipitation events.

Figure 7 shows the predicted effect of hydraulic conductivity on the drawdown at the drains for the observed variation in precipitation. It can be seen that in the soil with a permeability of 2.5×10^{-5} cm/sec the drain is effective in maintaining a low phreatic surface, whereas in the soil with a permeability of 2.5×10^{-6} cm/sec the drain is not effective in lowering the phreatic line after large precipitation events. The horizontal drain will function well in soil with a permeability of approximately 2.5×10^{-5} cm/sec or higher, but is ineffective in soils with permeability values lower than that.

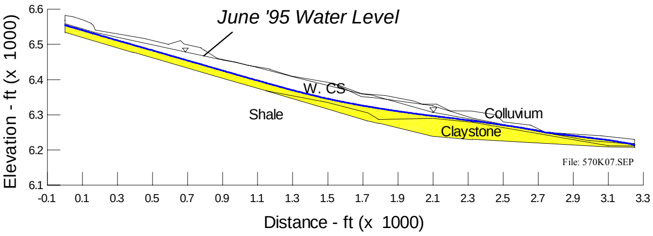

Figure 4. Steady-State Seepage Analysis – Historic Rise of Water Levels – Without Infiltration.

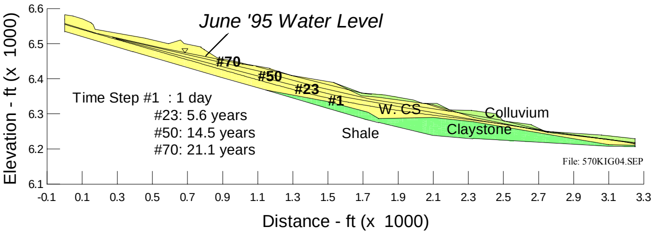

Figure 5. Transient Seepage Analysis - Historic Rise of Water Levels – With Infiltration.

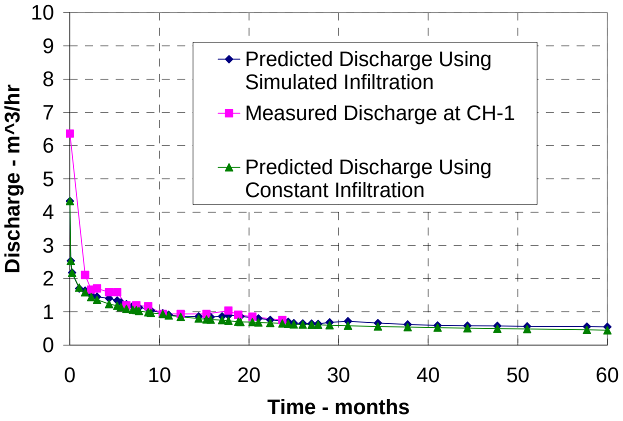

Figure 6. Plot of Predicted and Measured Discharge Rates.

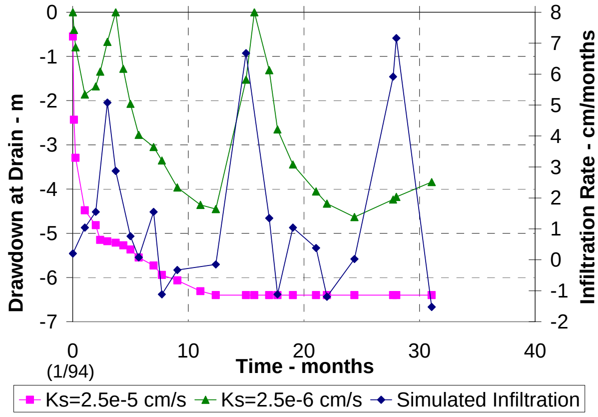

Figure 7. Plot of Drawdown at Horizontal Drain.

5.4 Vertical Well Simulations

The effect of a vertical well with a 1-foot radius on the phreatic surface was modelled using a 3-dimensional radial symmetry model. The soil in the model was assumed to be landslide material with a hydraulic conductivity of 2.5×10^{-5} cm/sec.

The predicted drawdown is 4.57 m at a radius of 3 m. The steady-state pumping rate calculated by SEEP/W was estimated to be 0.27 m^3/hr. The transient response of the phreatic surface surrounding the vertical well indicates that it would take approximately one year to reach steady-state conditions.

6. SLOPE STABILITY ANALYSES

Geotechnical slope stability analyses were performed utilizing the computer program SLOPE/W (GEO-SLOPE, 1995b) to analyze the active progressive movement and to assist in developing a mitigation plan to arrest slope movements. The analyses concentrated on two general cases: (1) progressive slope movement and (2) effect of irrigation.

Cross sections J1-J1', K-K', and L-L' were analyzed for the slope stability analyses. Locations of the cross sections are presented on Figure 1. Cross section J1-J1' was chosen for the analyses of the progressive slope movements, because it lies along the direction of primary movement. Cross sections K-K' and L-L' were considered to be the most critical cross sections based on the direction of the landslide and the maximum height of water above shale.

Based on the slope indicator data, the shear plane was determined to be the interface between the shale and the overlying materials (colluvium, weathered claystone, and claystone). The overlying material was modelled as one general material with no differentiation between zones. The total unit weight, γ, and angle of internal friction, φ, of the materials were determined from laboratory data (see Section 4). The cohesion, C, was assumed to be equal to zero for long-term stability analysis, since for large values of shear strain, the shear strength of a soil will decrease to the residual strength, in which case the cohesion is zero (Nelson and Thompson, 1977). For model simulations, a 3-meter-deep slide plane was assumed on top of the bedrock surface (shale). The parameters of the slide plane were determined by calibrating the model to the conditions on site (SMI, 1995). Table 2 summarizes the material shear strength parameters used in the analyses.

Table 2. Material Shear Strength Parameters Used in the Slope Stability Analyses.

Material Type	Total Unit Weight (g/cm^3)	Cohesion (kPa)	Angle of Internal Friction (degrees)
Coluvium	2.0	0	29
Weathered Claystone	2.0	0	29
Claystone	2.0	0	29
Slide Plane	2.0	0	11

6.1 Progressive Slope Movements

Field observations indicate that the slope movement occurred along the surface of the shale and that only sections of the slope failed rather than the entire slope. Therefore, the computer simulation was calibrated to the field observations considering progressive slope movements.

The results of the progressive slope movements, as summarized in Table 3, indicate that the lower region of cross section J1-J1' has the lowest factor of safety and, thus, would fail first. The region with the highest factor of safety is the middle region. This result agrees closely with the observed field movements recorded in the slope inclinometers. The correlation of the results of the stability model with the field observations provides verification to the calibration of the stability model.

Table 3. Summary of Min. Factors of Safety – Progressive Slope Movement for Cross Section J-J'.

Location of Failure Surface	Factor of Safety
Lower Region	1.00
Upper Region	1.14
Middle Region	1.19

6.2 Effect of Irrigation

Cross sections K-K' and L-L' were analyzed for slope stability analyses with respect to the effect of irrigation by using the calculated phreatic surfaces obtained from the ground water analyses.

The results of the slope stability analyses indicate that the factors of safety for cross sections K-K' and L-L' before construction of the golf course are 1.32 and 1.48, respectively, which implies that a stable slope existed before construction and irrigation of the golf course.

Figure 8 shows the steady decrease in factor of safety over time for an infiltration rate of 2.5 cm/month specified on the playing areas beginning in 1975 (beginning of golf course irrigation) for cross section K-K'. This figure indicates that the factor of safety remained greater than 1.0 until approximately 20 years had elapsed, which occurred in approximately 1995.

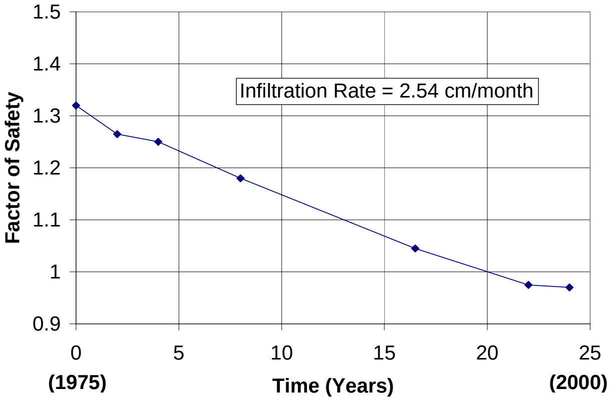

Figure 8. Factor of Safety vs. Time for Cross Section K-K' – Effect of Irrigation

7. MITIGATION PLAN DESIGN

The proposed mitigation plan design for the South Golf Course includes: (1) ground water dewatering systems, (2) surface water control systems, (3) collection of flow from horizontal drains, and (4) control of irrigation.

The proposed dewatering systems include dewatering drain that are intended to lower the ground water table, and interceptor drains that are intended to gather and divert surface water and to collect shallow ground water. Our analyses indicate that the long-term factor of safety of 1.3 or higher can be reached by constructing the proposed dewatering system.

Surface water mitigation includes minor regrading of the course to increase runoff and establishing drainage for sand traps, greens, and tees. The horizontal drain flow collection mitigation includes inspection and cleaning of drains, slip-lining, and improvements to flow collection and metering. Nine additional horizontal drains are also proposed. The control of irrigation includes the reduction of irrigated areas and optimization of the timing of irrigation episodes, irrigation rates, and distribution (SMI, 1997).

8. CONCLUSIONS

The construction of the golf course modified the site hydrology by adding irrigation water inflows and by changing the vegetation from native grass and scrub oak to turf grass over 55 percent of the total area. An analysis of the irrigation and precipitation rates and the turf grass water consumption rates showed a relatively high infiltration rate in the turf grass areas compared to the unirrigated native areas. Long-term computer simulation of the ground water levels showed that the increased infiltration rate in the irrigated turf grass areas, over the more than 20 year presence of the golf course, caused the increase in ground water elevations responsible for the unstable slopes.

The geotechnical slope stability analyses showed that the slope of the site remained stable until the ground water had risen significantly due to increased infiltration. The modelling showed that the slopes would become unstable after approximately 20 years of infiltration due to construction of the golf course. There is a good match between the stability analyses of the progressive movement and the field observations. The results of the slope stability analyses were used to develop an effective mitigation plan to arrest slope movement.

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