Reinforcement of a Pipeline Right-of-Way in Eastern Kentucky: A Case Study

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ABSTRACT

Slope movements and instability present a costly geo-hazard throughout the State of Kentucky. Pipeline going through hilly areas are vulnerable to slope failure or ground movements due to natural features, slope geometry, and disturbance caused by the right-of-way construction. Slope movements may induce additional stresses on the pipeline, which increases the displacement demand that may endanger the pipeline integrity, causing pipe fractures, leakages, fires, and even explosions. Reinforcing the slope at the early signs of movement can reduce the risk of a costly pipeline failure. In this paper, a case history is presented for site assessments, geotechnical study, monitoring, and reinforcement of a slope within a natural gas pipeline right-of-way using steel plate piles. The reinforced slope was monitored using slope inclinometers for post-construction movement. A finite element analysis was conducted to analyze the performance of the reinforcement. The monitoring data were in agreement with the analytical results and design criteria.

INTRODUCTION

Ground movement including landslide and subsidence are the major geohazard that threaten the pipeline (oil and gas) integrity and associate public safety. It was reported that around an average of 200 incidents occurred to the liquid pipeline annually in the United States and an approximately $150 million was spent every year on the liquid pipeline incidents (excluding the cost for maintenance and minor repair), among which the majority was trigged by landslide and subsidence (Girgin and E. Krausmann, 2016).

Kentucky, West Virginia and Pennsylvania are known to civil engineers on landslides and ground subsidence caused by sinkholes because of the geological and geographical conditions. Oil and gas pipelines and other linear infrastructure traversing areas of sloping terrain in these states are vulnerable to these frequent and costly geologic hazards due to both natural and construction disturbance. A landslide may impact pipeline integrity if occurring near or within its right-of-way, causing bending strain, deformation, stress-cracking, leakage, and even ruptures, thus affecting nearby property and the environment. Mitigating the ground movement by reinforcing a slope when it shows early signs of movement can significantly reduce the risk of pipeline failure and minimize the effort and long-term cost to maintain the slope or remediate the failure.

Methods to remediate a landslide include conducting traditional earthwork repairs with removing existing slide mass/unstable material and replacing with engineered fill or performing in-place stabilization by installing structural elements into the slope which can provide additional resistance against the slide forces. For pipeline industry, operations are typically kept running even a landslide occurred within right-of-way, unless a structure failure occurred to the pipeline. The presence of pipelines often causes construction difficulties for traditional earthwork repair on landslides. Sometimes, earthwork requires benching on the slope which may worsen the site condition and induce more instability issues. On the contrary, in-place stabilization measures require minimal slope re-grading to reclaim slope configuration, thus generate less disturbance to the unstable slope.

There are quite a few in-place stabilization methods for slope reinforcement in the industry, including soil nailing, micro-piles, drilled shafts and all other steel elements that are designed to resist the sliding forces using structural capacity. In this paper, a case history is presented for reinforcing a moving slope within a natural gas pipeline right-of-way in eastern Kentucky using the steel plate piles. Analysis of the reinforced slope using limit equilibrium methods and finite element methods (Moudabel 2013, Yang 2005) were conducted and the results were compared and discussed.

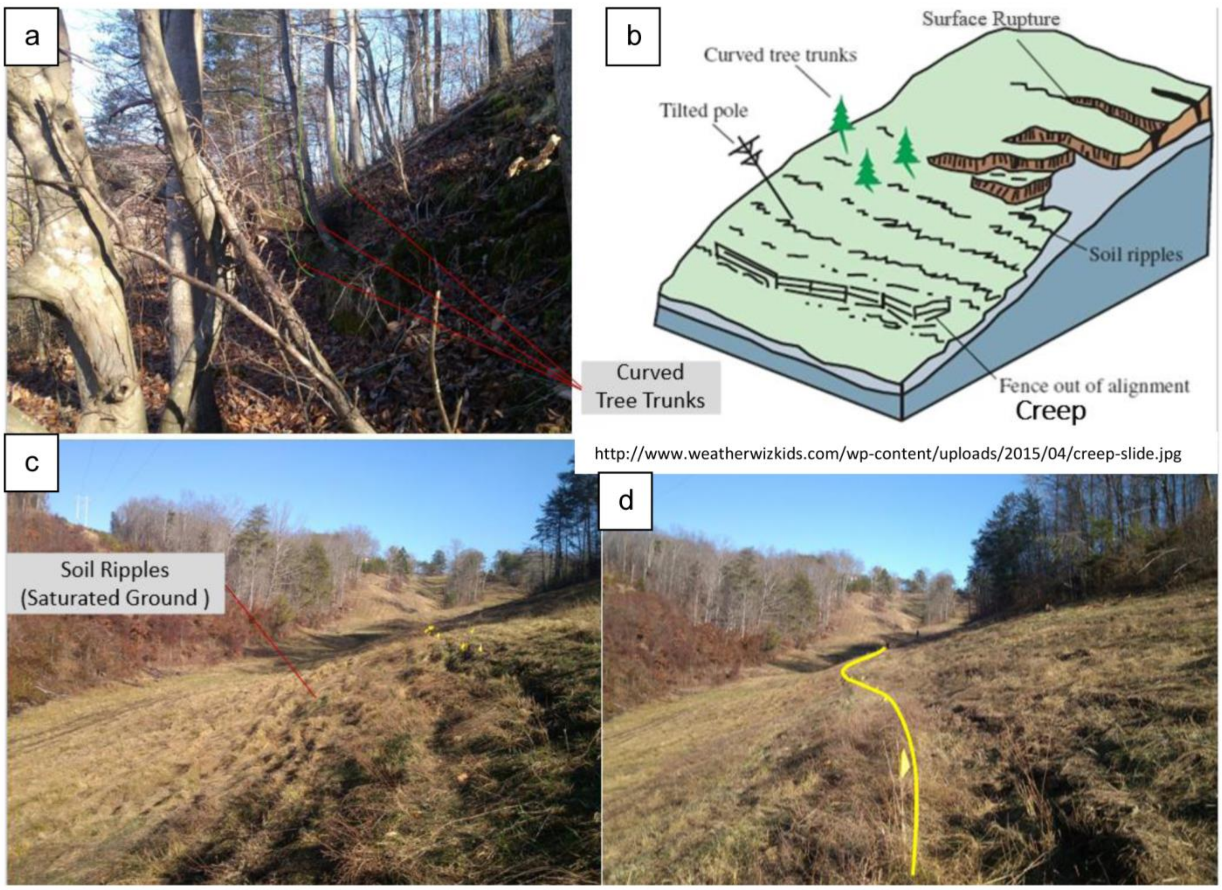


Figure 1. a) Curved tree trunks b) Schematic of creep-type slope movement c) Soil ripples of saturated ground on site d) Impacted pipeline alignment

PROJECT INFORMATION

The landslide site in eastern Kentucky was initially indicated to the natural gas pipeline personnel by a report of potential bending strain based on an analysis of in-line inspection data. In December 2012, a visual site assessment was conducted and creep-type slope movement was observed indicated by undulating (hummocky) ground surface (soil ripples) and curved tree-trunks near the top of the right-of-way. Additionally, a spring located at the top of the slope and an apparent landslide feature was observed near the indicated strain feature location. The marked pipeline alignment on the ground surface showed a clear curve in the downhill direction over an approximately 150 feet section and the maximum latera displacement from the original alignment is approximately 14 feet (shown in Figure 1 for general site conditions).

Subsequently, in June 2013 the impacted pipeline was excavated to allow direct assessment of pipeline strain to evaluate pipeline integrity. After the excavation, the curved pipeline was observed to bounce back approximately 6 feet and released some of the stresses and strains caused by the slope movement.

In November and December 2014, a partial pipe replacement was conducted, due to concerns regarding corrosion of the impacted pipeline section at this location (shown in Figure 2). Following the pipe replacement, the trench was backfilled and the slope was graded to provide protective pipe cover and temporarily mitigate ground movement. However, because the reclamation adopted at that time was temporary, the backfill was placed without compaction testing and the fill was not moisture conditioned or benched into the slope. The creep type of ground movement was still observed after the temporary reclamation. In the spring of 2015, the gas pipeline company sought slope reinforcement recommendations to permanently solve the problem. A geotechnical study of the slope and instrumentation monitoring were conducted during the year of 2015, and in-place stabilization option using steel plate pile system was eventually selected by the owner.

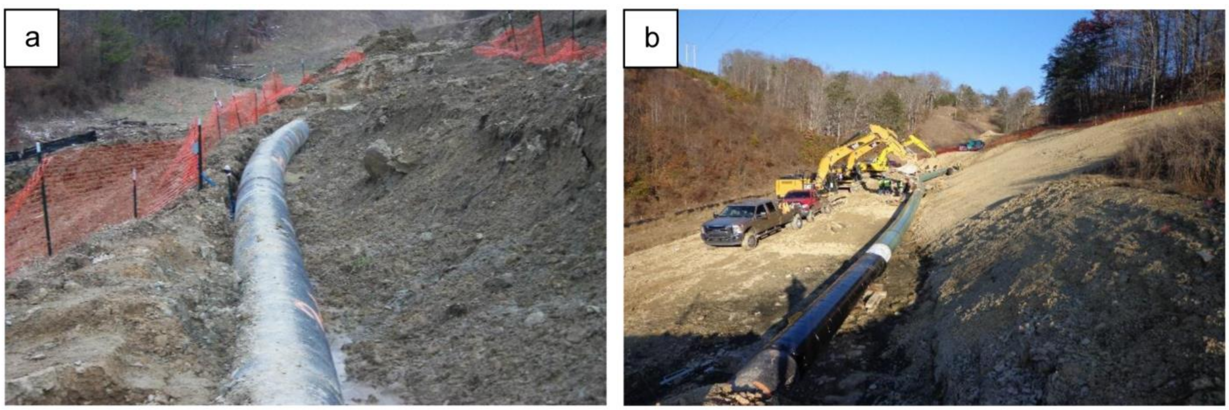


Figure 2. a) Exposed Old Curved Pipe 2) New Pipe Installation

SITE ASSESSMENT AND GEOTECHNCIAL STUDY

In August 2015, geotechnical drilling was conducted using the middle bench of the slope constructed to facilitate access on the right-of-way slope during the pipe replacement project in 2014. The side-hill slope was at a grade of approximately 2.5H: 1V to 3H:1V. During this study, an approximately 45-foot long tension crack was observed in the middle slope, between the access road at the toe of the slope and the middle bench along the right-of-way (Figure 3).

There are three pipelines running parallel within the right-of-way in the strike direction of the referenced slope. The one on the middle slope was the one impacted by the slope movement, experienced bending/strain features and was partially replaced in 2014. Saturated and soft ground with ponding water was observed over approximately 100 feet on the slope slightly above the middle pipeline alignment in the location where most slope movement was observed and the pipeline was curved towards downslope. Apparent spring water or underground seepage appear to be the cause of the soil saturation. During a walk on the upper slope outside the

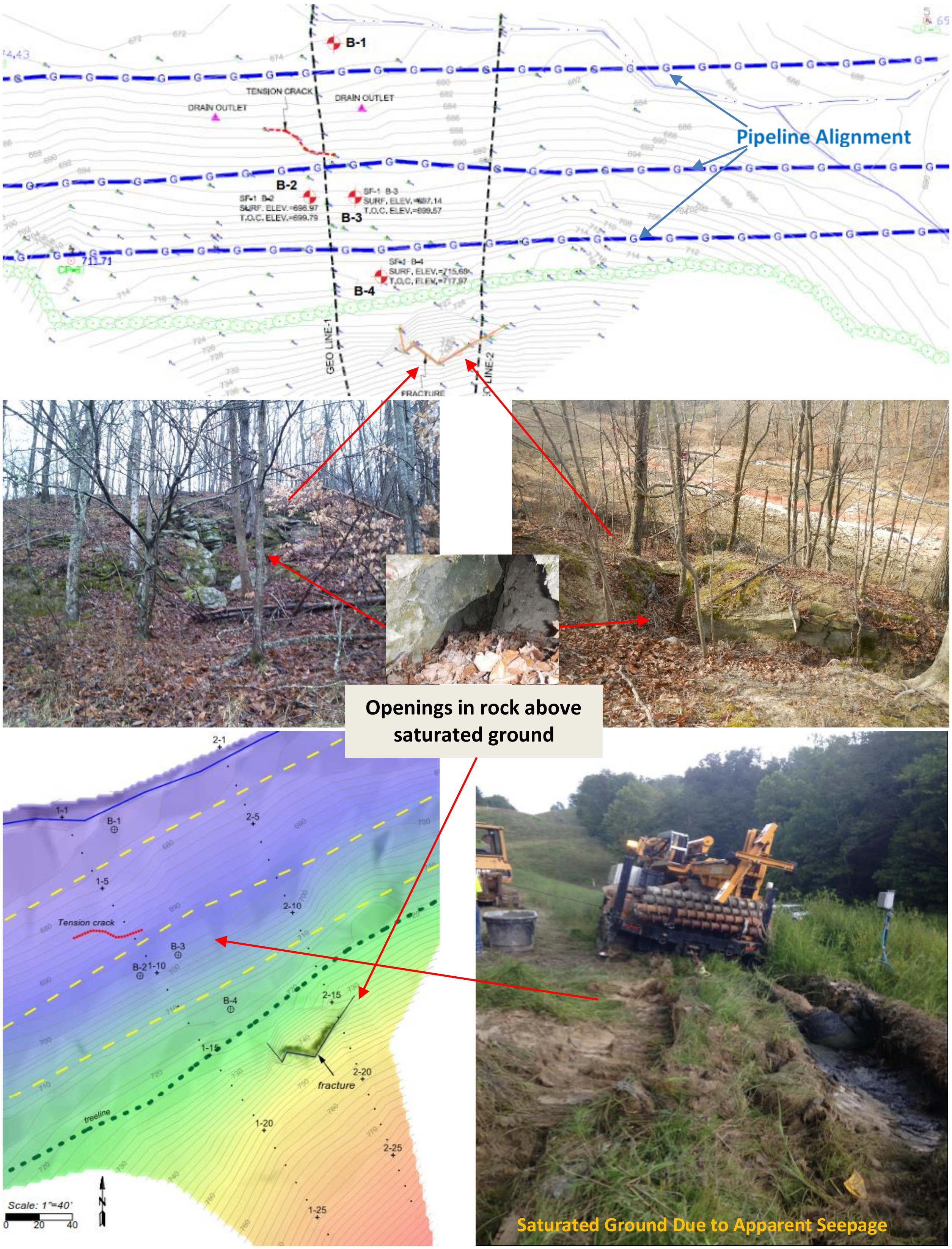


Figure 3. General Slope Conditions

right-of-way in the wooded area, a natural fractured gap of approximately 10 feet tall and 3 feet wide was observed into the rock (Figure 3). The opening is located approximately 120 feet away and above the saturated location in the right-of-way. The cut of sheet flow by the observed opening/fractured gap and inlet of water could be the source of underground seepage to the right-of-way (Figure 3).

Table 1. Generalized Subsurface Profile

Description	Approximate Depth to Bottom of Stratum (ft)	Material Encountered	Consistency/Density/ Rock Hardness
Stratum 1	5 to 8[1]	Fill – lean clay with sand, gravel and rock fragments (CL)	N/A
Stratum 2	5 B-1 only [2]	Residual soils – Clayey sand (SC)	Loose
Stratum 3	9 to 21	Sandy/clayey shale and sandstone	Highly weathered, soft
Stratum 4	Undetermined 3	Sandstone and clayey shale inter-bedding	Moderately to highly weathered, medium strong rock; Recovery: 87 to 100 % RQD: 40 to 70 %

1. Fill was encountered in borings B-2, B-3 and B-4.
2. Clayey sand residuum, encountered only at boring B-1, loose density based on SPT N-values of 8.
3. Borings were terminated in this stratum.

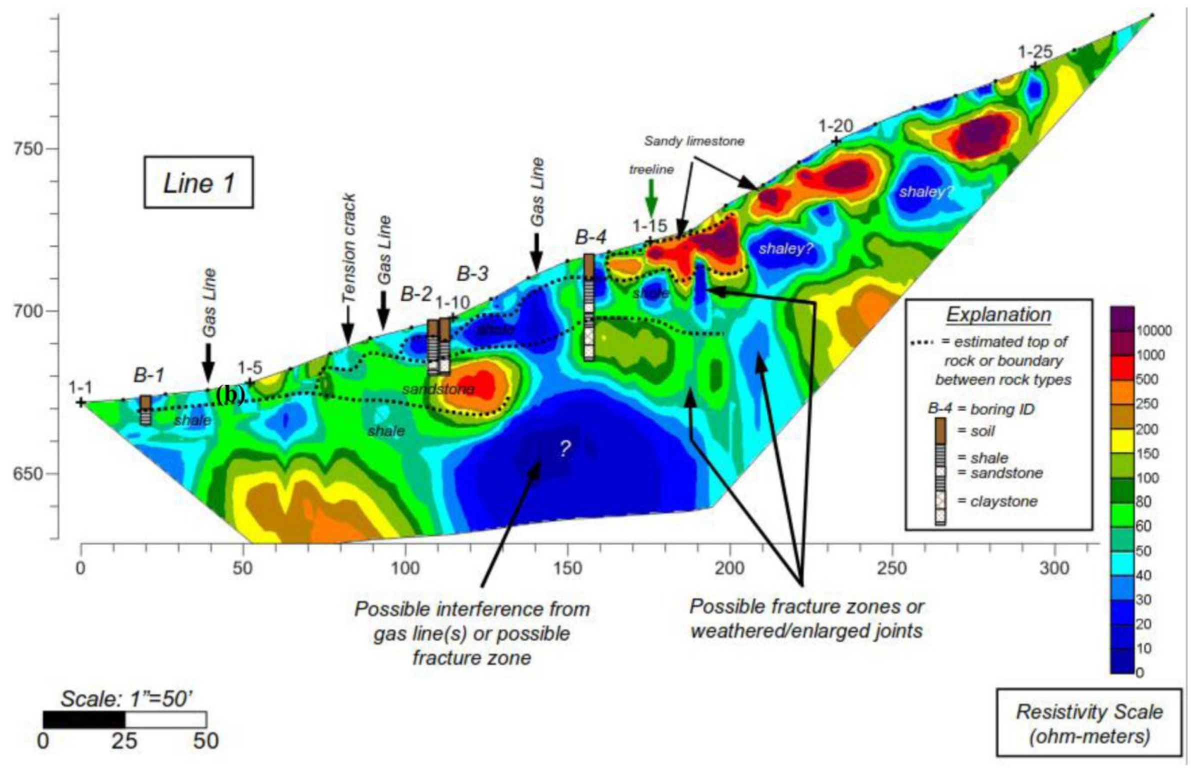


Figure 4. ERI test results

Figure 5. Inclinometer Locations and Its Surrounding Slope Conditions

A total of 4 geotechnical soil borings were advanced to depths ranging from 9 to 32.5 feet below the existing ground surface. A geophysical study consisting of Electrical Resistivity Imaging (ERI) test arrays was performed to supplement the boring data and obtain stratigraphy between boring locations and extending upslope from the right-of-way. Boring locations (B-1 through B-4) and ERI test arrays (Geo lines 1 and 2) are shown in Figure 3. Based on the results of this study, subsurface conditions at the site can be generalized in Table 1. Upon completion of the borings, three slope inclinometer casings were installed and grouted within boring locations B-2, B-3 and B-4. to monitor the slope movement.

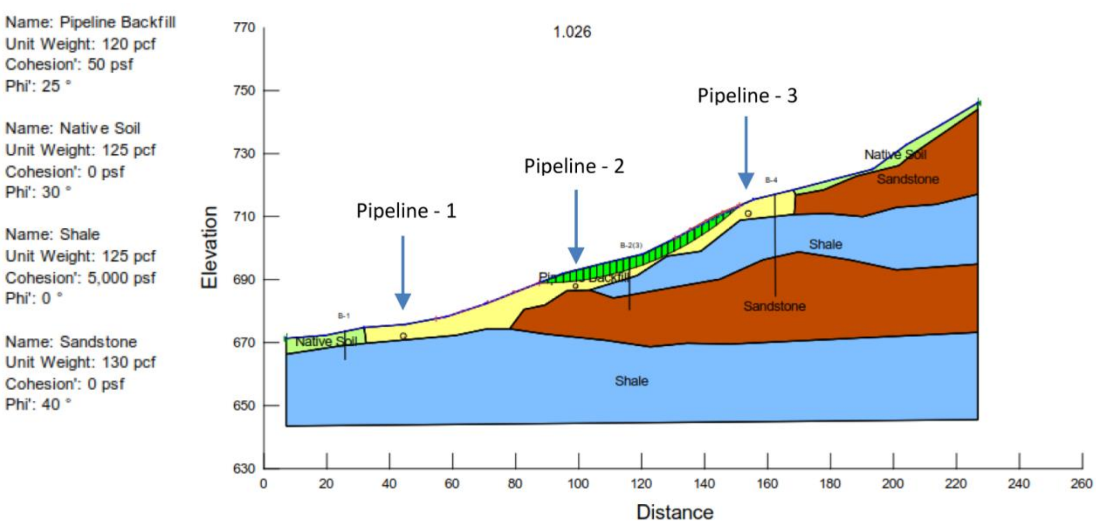


Figure 6. Inclinometer Locations and Its Surrounding Slope Conditions

Table 2 Back-calculated Soil Parameters

Material Description	Unit Weight (pcf)	c', Cohesion (psf)	Φ', Friction (degrees)
Pipe Backfill	120	50	25

The ERI tests taking into consideration the boring information resulted in two cross-sections of the referenced slope (Figure 4). Based on the test Line 2 which intersect the saturated zone in the right-of-way and the observed opening in rock face on the upper slope, it appears that fractured rock zone exists beneath these areas and water collected from the opening can be migrating through the fractures and seeping out into the right-of-way.

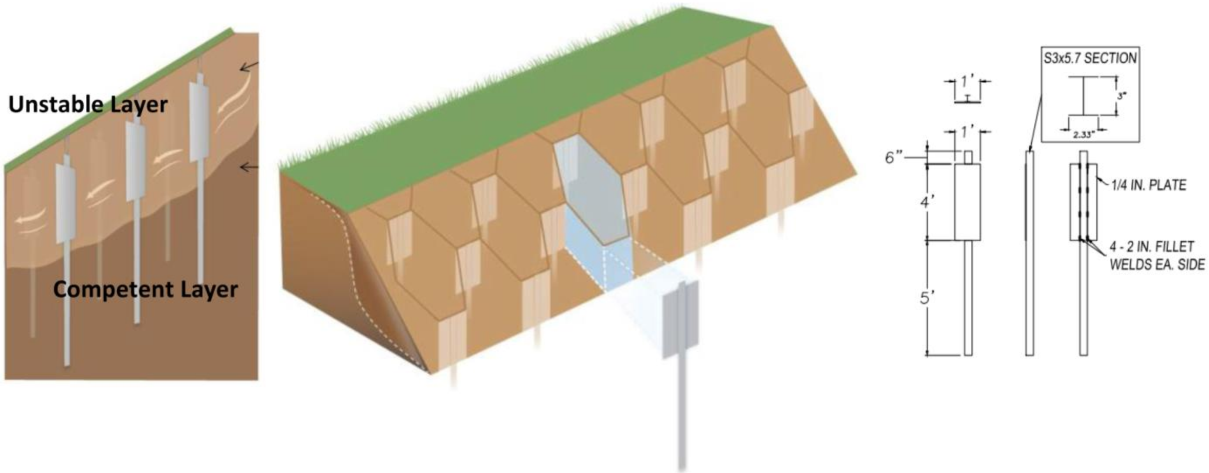


Figure 7. Repair Concept and Pile Dimensions (Geopier SRT)

SLOPE MONITORING AND STABILITY ANALYSIS

The slope was monitored using the three installed inclinometers from August 24, 2015 to April 27, 2016. The site photos of the inclinometer locations and their surrounding slope conditions are shown in Figure 5. Water can be observed seeping out of the ground in adjacent to inclinometer B-3 and above the curved pipe section. Based on interpretation of inclinometer monitoring results, the following assessments have been made:

- Inclinometer at B-2: A peak ground surface movement of approximately 0.25 inches was detected at a depth of 6 feet below ground surface, which is at the interface of existing fill material and underlying clayey shale bedrock.
- Inclinometer at B-3: A peak ground surface movement of approximately 0.25 inches was detected at a depth of 4 feet below ground surface, which is within the existing fill material.
- Inclinometer at B-4: No clearly defined zones of movement or failure surface was detected at this inclinometer location.

On-going ground movement is indicated by tension cracking observed at the lower side of the middle bench and inclinometer monitoring data. The observed ground movement was expected based on the temporary nature of the reclamation performed following the pipe replacement project in 2014 and is likely due to the uncontrolled backfill, poor drainage, and fluctuations in groundwater levels along the soil-bedrock interface.

Slope stability analysis was performed using the representative cross-section generated from the ERI test Line 2 (Figure 6). The purpose of the slope stability analysis was to model the observed tension-cracking and slope inclinometer monitoring results to back-analyze geotechnical parameters for design, evaluate the potential failure envelope, and facilitate conceptualization of remediation options. The slope stability analysis was carried out and the methodology of Morgenstern-Price was used in determination of the factors of safety. The selected two-dimensional section was analyzed assuming that the shear strengths of the materials along the potential failure surface are governed by linear (Mohr-Coulomb) relationships between shear strength and the normal stress on the failure surface. By assuming a factor of safety of 1.0

for a slope that is known to have failed, shear strength parameters for the slide mass can be back-calculated for use in the design of stabilization measures.

Based on the slope movement occurring during times of slope saturation, two groundwater conditions were considered in the analysis and a range of shear strength parameters is provided based on:

- groundwater condition with phreatic surface at the soil/bedrock boundary, and
- groundwater fluctuations with raised phreatic surface within the slope.

The back calculated effective stress shear strength parameters are listed in Table 2. A range is provided for the cohesion c' and friction angle φ (degrees), based on varying the phreatic conditions as described above.

Table 3 Steel Plate Pile (S3x5.7) Properties*

Area (in^2)	Moment of Inertia (in^4)	Section Modulus (ln^3)	Elastic Modulus (ksi)	Yield Strength (ksi)	Yield Bending Moment (kips-in)	Yield Shear (kips)
1.66	2.5	1.67	29,000	50	83.5	83

* According to AISC (2005)

REMEDIATION CONCEPT

Among in-place stabilization/reinforcement methods recommended to the gas pipeline company, lateral resistance steel plate piles were selected for this project. With this method, the disturbance to the slope was minimized and the pipeline operation was not interrupted during construction.

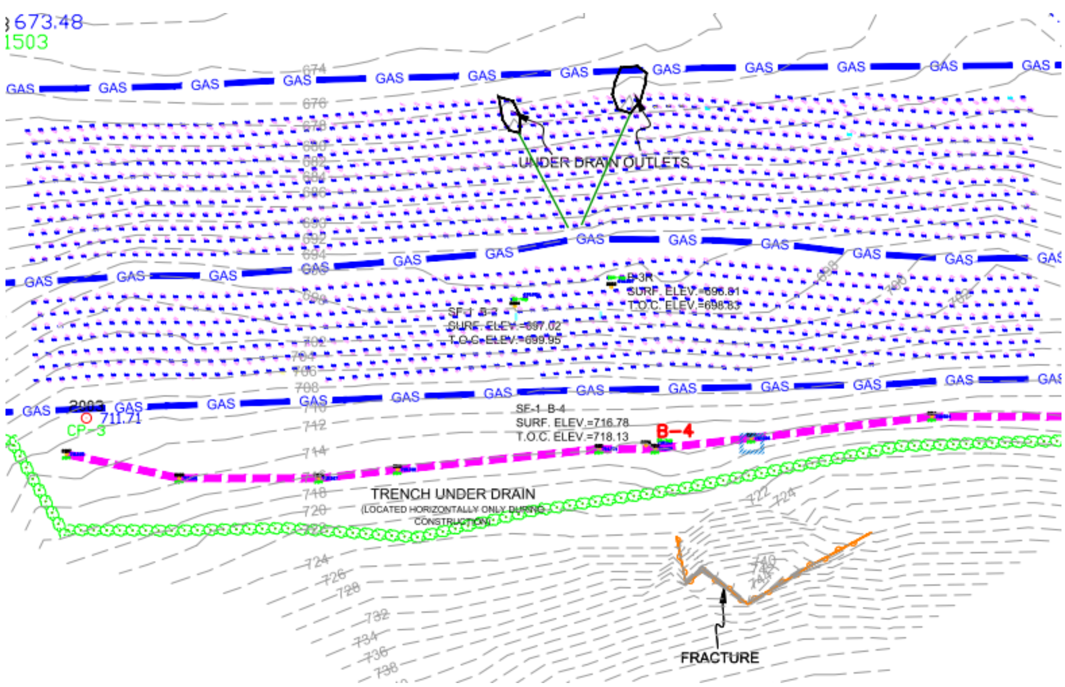
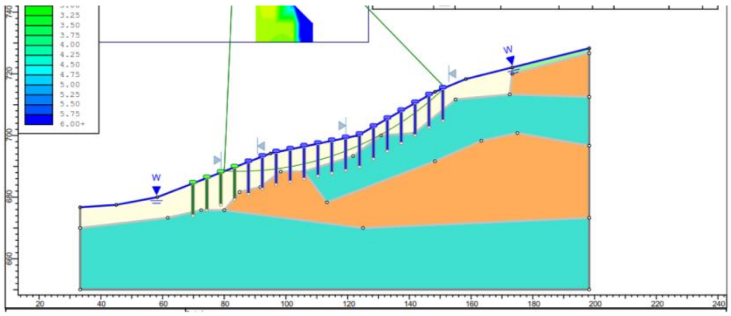
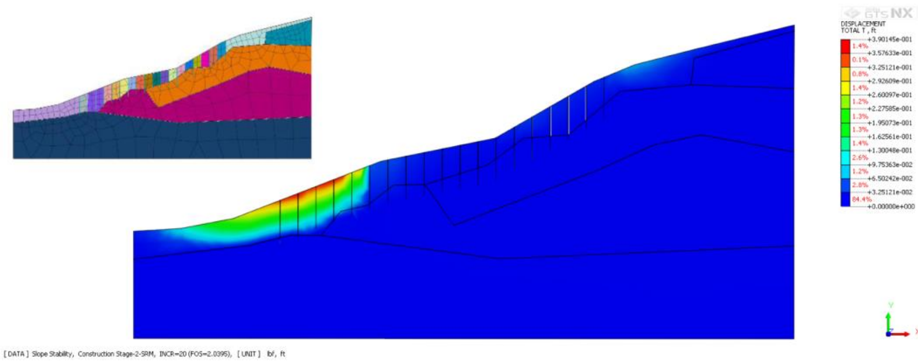


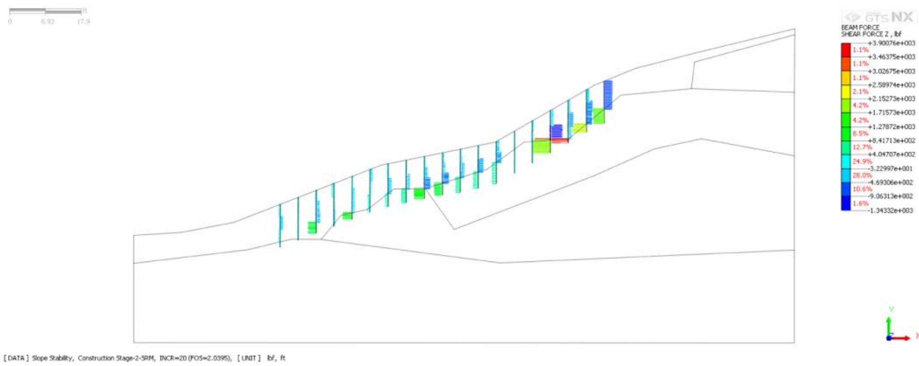
Figure 8. Inclinometer Locations and Its Surrounding Slope Conditions



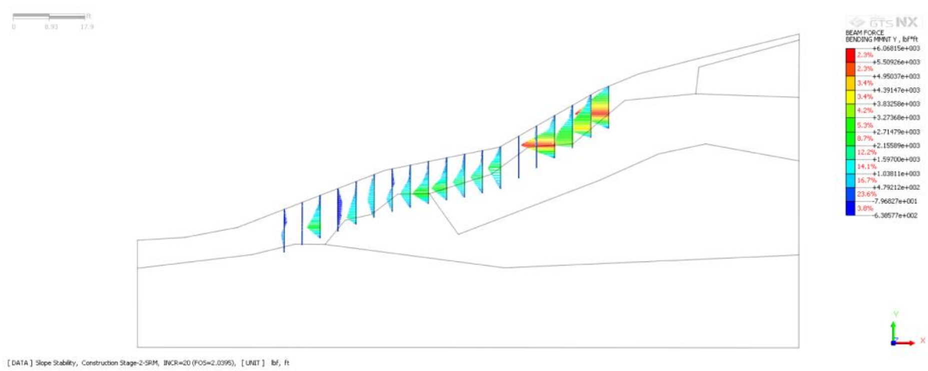
(a) Slope Stability Analysis (limit equilibrium)



(b) Total Displacement Plot (FEM)



(c) Shear Force on the SRT Plate Piles (FEM)



(d) Bending Moment on the SRT Plate Piles (FEM)

Figure 9. Slope Stability Analysis

To address the underground seepage issues of the slope, drainage improvement was designed and installed on the slope as well. To capture the potential seepage from upper slope, a trench drain down to bedrock (approximately 10 feet) was installed along the tree line above the right-of-way. To quickly drain the water trapped in the saturated zone and relieve the water pressures, two finger drains were installed on the down-slope side of the saturated zone and curved pipe location.

DESIGN AND ANALYSIS

Lateral resistance piles are typically installed on slopes to intercept the slope failure surface and stabilize the slope by pinning the unstable materials to more competent underlying layers. The system uses driven vertical steel “plate piles” as lateral-reinforcing elements. The plate piles consist of steel angles to which rectangular plates are welded. they are typically installed in a staggered grid pattern using impact or vibratory hammers. Plate piles mobilize the strength of the soil through arching and transmit forces to the underlying stiffer soil or bedrock. The Slope Reinforcement Technology (SRT) repair concept is shown in Figure 7.

The plate pile used was S3x5.7 steel section that was 9.5 feet in length and installed with the pile top being 12-18 inches below final grade. The plate piles were designed based on a 4.5-feet spacing in the up-slope direction and a 4-feet spacing center-to-center in the horizontal direction. Based on the site condition that bedrock is relatively shallow at around 10 feet below surface, slightly oversize pre-drilled holes were performed for plate pile installation and grouted to ensure penetrating into the underlying bedrock. An approximate area of 85 feet by 310 feet (26,250 ft^2) of slope was decided to be reinforced and a total of 1,440 plate piles were installed. As requested by the gas line company, the piles were not installed within 5 feet of the pipeline alignment. The installed pile locations are shown in pink in Figure 8. All plate piles were installed within three weeks and construction completed in six weeks.

The capacity of the S3x5.7 steel section plate piles subjected to lateral soil movements was evaluated. The properties are shown in Table 3.

The plate pile was designed by applying lateral soil movements over the depth of the slide plane until a limiting state is reached. The allowable soil movement was selected at 0.75 in. In the design, a sliding depth of 5.5 feet was considered by the slope stability analysis. The stability analysis for reinforced slope was also conducted as part of the design to ensure factor of safety is greater than 1.5.

For this case history study, a finite element analysis was performed to compare the results. The Analysis plots are shown in Figure 9 and the results of the analyses are compared in Table 4.

Table 4 Analysis Results Comparison

Method		Deflection [1] (in)	Max. Bending Moment (lb-ft)	Max. Shear Force (lb)	F.S.
LPile & Slide	Embedded in Rock	0.75	2242	1200	1.5
	Embedded in Soil		760	890	
FEM		4.5	6068	3901	2.0
Inclinometer		1.1	--	--	--

1. Deflection of the pile head

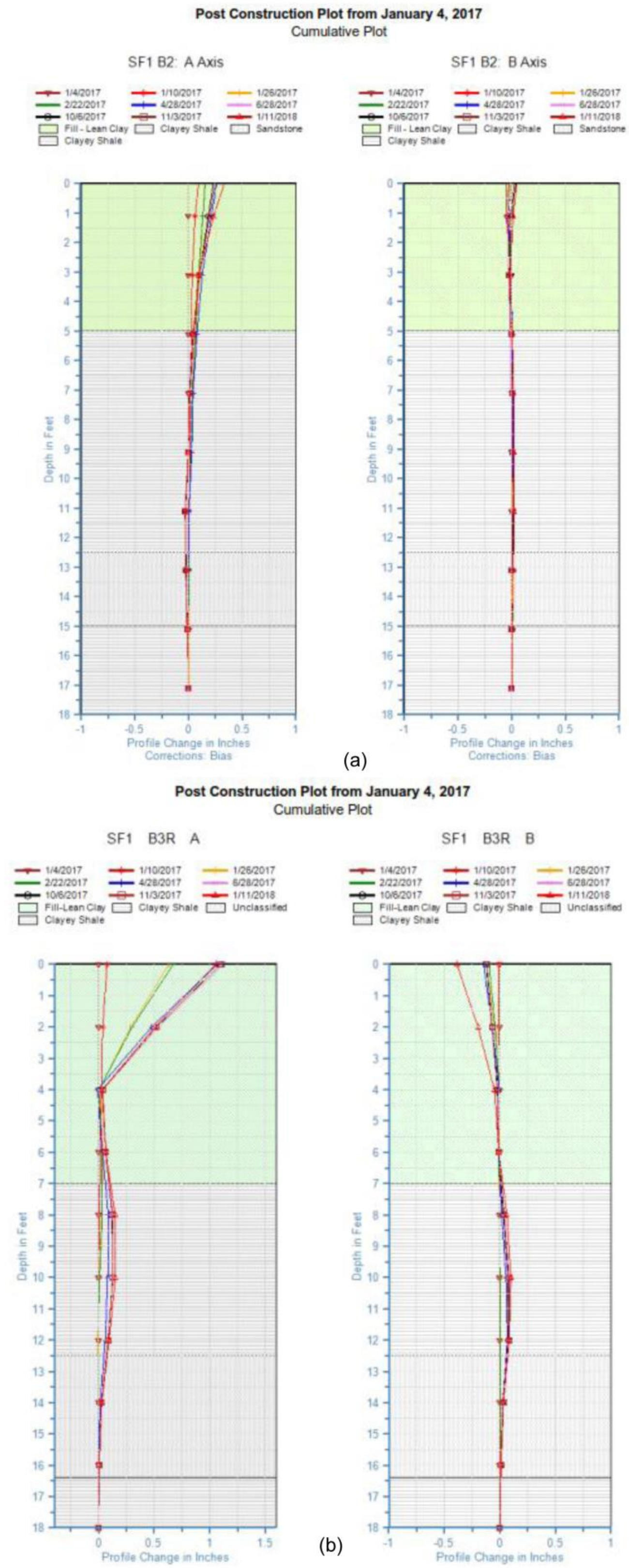


Figure 10. Inclinometer Monitoring Results a) Inclinometer B-2 b) Inclinometer B-3R

(a) Upper slope after 4 months of construction (b) Entire slope after 4 months of construction

(c) Water at drainage outlet 2 months after construction (d) Middle bench above curved pipeline

Figure 11. Inclinometer Locations and Its Surrounding Slope Conditions

COMPARISON AND CONCLUSION

The results from design analysis, finite element analysis and slope monitoring data are compared in Table 4.

Comparing to the design analysis performed using LPile and Slide, the FEM analysis results show larger maximum forces on the plate piles and larger deflections on the pile head. However, the maximum forces are still within the capacity of the piles. The FEM analysis also gives higher factor of safety for the slope based on the strength reduction method. Based on this case, the strength reduction method used in finite element analysis appears to result higher loading on the structural elements, which may push the design to too conservative.

The reinforced slope was monitored using the installed inclinometers for a one-year period from January 4, 2017 to January 11, 2018. The inclinometer reading plots for B-2 and B-3R (a replacement of inclinometer B-3 which was damaged during construction) are shown in Figure 10. Based on interpretation of inclinometer monitoring results, the following assessments have been made:

- **Inclinometer at B-2:** A gradual surface movement was recorded after construction being completed. A total surface movement of a magnitude of 0.3 inches can be observed on the plot during the 1-year monitoring period. The movement appeared to be along the soil bedrock interface at around 5 feet below ground surface. The rate of movement is relatively slow and the movement appeared to be stopped after April 2017.

- **Inclinometer at B-3R:** During the 1-year monitoring period, a total surface movement of a magnitude of 1.1 inches was recorded, in which approximately 0.6 inches of movement was detected within the first 3 weeks after construction from January 4, 2017 to January 26, 2017. After that, an approximately 0.5 inches of movement was detected from February 22, 2017 to April 28, 2017, with a relatively lower movement rate. The movement appeared to be at about 3-4 feet below grade within the lean clay fill layer. The movement appeared to be stopped after April 2017.

The inclinometer monitoring results show that after about four months, the installed steel plate piles appear to be fully engaged and the slope movement was stopped. The amount of recorded surface movement was slightly larger than the design criteria 0.75 inches, but the convergence of the movement generally indicated the well-functioning of the elements. Water was observed in the drainage outlet, which means the drainage system installed on site was also functioning (shown in Figure 11).

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