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Landslide Best Practices HANDBOOK

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CENTER FOR IMPACTFUL RESILIENT INFRASTRUCTURE SCIENCE & ENGINEERING

Landslide Best Practices Handbook

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16. Abstract This handbook was written to produce region-specific guidance for practicing geotechnical engineers and geologists who are engaged to mitigate adverse impacts from an active landslide or reduce the risk of landslide movement for infrastructure. Best practice guidelines for the life cycle of a landslide mitigation response including approach, characterization, assessment and mitigation are presented. Best practice framework for slope maintenance and slope management systems are also included.		
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IRISE

The Impactful Resilient Infrastructure Science & Engineering consortium was established in the Department of Civil and Environmental Engineering in the Swanson School of Engineering at the University of Pittsburgh to address the challenges associated with aging transportation infrastructure. IRISE is addressing these challenges with a comprehensive approach that includes knowledge gathering, decision making, material durability and structural repair. It features a collaborative effort among the public agencies that own and operate the infrastructure, the private companies that design and build it and the academic community to develop creative solutions that can be implemented to meet the needs of its members. To learn more, visit:
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Disclaimer

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List of Abbreviations

AACE	Association for the Advancement of Cost Engineering
AASHTO	American Association of State Highway and Transportation Officials
AREMA	American Railway Engineering and Maintenance-of-Way Association
ASTM	American Society for Testing and Materials (ASTM International)
COOLR	Cooperative Open Online Landslide Repository
CD	Consolidated-Drained
CU	Consolidated-Undrained
DEM	Digital Elevation Model
DTM	Digital Terrain Model
FHWA	Federal Highway Administration
GCP	Ground Control Point
GIS	Geographic Information Systems
GLC	Global Landslide Catalog
GPS	Global Positioning Satellite
GRS	Geosynthetic Reinforced Soil
GSC	Geological Survey of Canada
GSD	Ground Sample Distance
IAEG	International Association of Engineering Geologists
IBC	International Building Code
InSAR	Interferometric Synthetic Aperture Rader
LHASA	Landslide Hazard Assessment for Situational Awareness
LiDAR	Light Detection and Ranging
NASA	National Aeronautics and Space Administration
NHCCI	FHWA’s National Highway Construction Cost Index
NDVI	Normalized Difference Vegetation Index
NIR	Near-Infrared
ODOT	Ohio Department of Transportation
OGC	Open Geospatial Consortium
OPCC	Opinion of Probable Construction Cost
PA	Pennsylvania
DCNR	Pennsylvania Department of Conservation and Natural Resources
PASDA	Pennsylvania Spatial Data Access
PennDOT	Pennsylvania Department of Transportation
PSU	Pennsylvania State University
TRB	Transportation Research Board
TRM	Turf Reinforcement Mat
UAS	Unmanned Aircraft Systems
UFC	Unified Facilities Criteria
USACE	United States Army Corps of Engineers

USCS	Unified Soil Classification System
USDA	United states Department of Agriculture
USGS	United States Geological Survey
UU	Unconsolidated-Undrained
VW	Vibrating Wire
WP/WLI	Working Party on the World Landslide Inventory
WSDOT	Washington State Department of Transportation

CHAPTER 1

Introduction

1.1 NEED

1.1.1 Unique Setting

Southwestern Pennsylvania is a region of extensive topographic relief, covered with slopes, many at their angle of repose. The last glacial episode was a driving force for many of the historic landslides in the region; these landslides are recognized by their extensive hummocky ground caused by earthflow and earth and rock slumps. These slopes often lack clear evidence of active sliding and are relatively stable in their natural, undisturbed state; however, the historic landslides can be re-activated by excavation, surcharge loading, and/or changes in groundwater and surface water conditions.

1.1.1.1 Regional Bedrock Geology

Beneath the surface features and soils, bedrock in southwestern Pennsylvania consists of flat-lying sedimentary formations generally consisting of the Pennsylvanian aged Dunkard, Monongahela, and Conemaugh Groups. These bedrock formations generally consist of a cyclic sequence of sandstone, limestone, coal, siltstone, shale, claystone, and clay shale. The strata cropping out within these slopes contain a high percentage of weak claystone and shales which are prone to accelerated weathering. Additionally, many slopes in the region have already experienced instability in the past, leaving many slopes founded within weak colluvial soils (e.g., soils that have been transported downslope by gravity).

1.1.1.2 Regional Hydrogeology

The regional hydrogeology is dominated by a dendritic pattern of tributaries that drain into the major rivers in the region. The cyclic sequence of bedrock has led to a series of perched water-bearing units. The alluvial deposits along the rivers, and to a lesser extent along the tributaries, are water-bearing. In addition, alluvial and glacial terrace deposits are resting on the former valley floor (i.e., Parker Strath) at an approximate elevation of 900 feet. The terrace deposits are reported to be more than 80 feet thick at some locations in southwestern Pennsylvania. Colluvial soil has spilled over and covered several water-bearing units, which has exacerbated landslide movement, particularly during periods of elevated precipitation (e.g., springtime).

1.1.2 Factors Driving Slope Movement

As the population in southwestern Pennsylvania has expanded, land development and civil works operations (roads, buildings, impervious cover, etc.) have escalated. These operations have covered and transformed the land surface, altering the natural topography and water drainage patterns. Additionally, southwestern Pennsylvania is known as a major center for energy development. As such, the abundant occurrence of both oil and gas and mining operations can, and have, disturbed the stability of surface slopes.

The increase in precipitation over the last decade has created conditions to further accelerate landslide movement on unstable slopes. Based on data provided by the National Weather Service, average rainfall per year has increased over the last decade. Average rainfall between 2010 and 2019 was 42.5 inches, an increase of 3 inches from the average 39.5 inches recorded between 2000-2009. Additionally, two of the top three wettest years on record were observed in 2018 and 2019, where over 52 and 57 inches of rainfall were recorded, respectively.

Considering these factors, the region contains very diverse forms of slope movement mechanisms, ranging from rotational features to debris flow. Bedrock failures such as rock fall and rock topple type movements are outside of the scope of this document. Due to the broad range of failure mechanisms, unique monitoring, and repair strategies should be prescribed for each situation; thus, there is no "one response‟ that fits all for landslide occurrences in the region.

1.2 SCOPE

This Handbook was written to produce region-specific guidance that applies to southwestern Pennsylvania including Greene, Washington, Beaver, Butler, Fayette, Indiana, Armstrong, Allegheny, and Westmoreland counties, along with the river valleys along the Monongahela River basin. The target audience is practicing geotechnical engineers and geologists who are engaged to mitigate adverse impacts from an active landslide or reduce the risk of landslide movement for infrastructure.

The scope of this document is to provide an overview of current practices used to characterize landslide hazards by utilizing published resources and focusing effort primarily on "best practice" to identify corrective action(s). The target audience will be equipped to:

- Classify the type and form of landslide, based on typical landslide movement and hazards in southwestern Pennsylvania.
- Identify proven/long-term or reliable design approach(es) as well as innovative construction methods and materials that will provide a more resilient infrastructure system. Corrective actions considered will include remediation measures to possibly slow the progression and rate of landslide movement, including, but not limited to, maintenance.
- Assess/develop a hazard rating & establish a threat priority, including, but not limited to, level of complexity, to arrive at a determination of "best practice".
- Differentiate between temporary and permanent mitigation response, including a rough cost comparison that can be used for planning use.
- Make distinctions about acceptable consequences to tailor solutions to the target audience (e.g., scaling for various types of clients, repair history and frequency, available funding, affected residents, vulnerability, consequence, and risk tolerance, control, and mitigation).
- Provide guidance and understanding on taking a proactive approach (resilience) versus a reactive approach (mitigation) and the cost implications of each.
- Form quasi "how-to" procedures to establish fundamental guidelines to approach, characterize, assess and take corrective action within the framework of "best practice."

It is understood the practitioner will conform to client specific requirements throughout the landslide mitigation. Some examples include, but are not limited to:

- Approved Materials – federally funded projects are likely to require domestic steel unless otherwise authorized; whereas material specified for PennDOT projects must be from an approved source as listed in Bulletin 15. Projects completed for private clients are not typically limited to these types of restrictions.

- Design Methodology – Department of Transportation projects are often in accordance with AASHTO LRFD (and/or the local DOT design manuals such as PennDOT DM-4); whereas, Class 1 railroad projects should be in accordance with AREMA which may require Allowable Stress Design (ASD).
- Codes - Projects completed for private clients typically are governed by the International Building Code (IBC); whereas federally funded projects are typically governed by the Unified Facilities Criteria (UFC).
- Specifications – Federal projects typically require specifications to be submitted through SpecsIntact; while PennDOT projects use standard specifications in accordance with PennDOT Publication 408.

1.3 RESEARCH ACCOMPLISHMENTS BY OTHERS

Extensive research has been conducted regarding landslide characterization and mitigation, both at the national and local levels. An extensive literature review was performed in preparation of this Handbook and the complete list of research reviewed is included in the references of each chapter. An overall summary of select research accomplishments is presented in the following sections.

1.3.1 National Papers

Comprehensive documents aimed at addressing the full range of landslide-related topics were generated by The Transportation Research Board (TRB) and The United States Geological Survey (USGS) in partnership with the Geological Survey of Canada (GSC). These research efforts created publicly accessible information which compiled the results of landslide related research at that time. The Transportation Research Board (TRB) produced Special Report 247 entitled *Landslides, Investigation and Mitigation* [128]; the document was written to cover the entire spectrum of issues related to landslides. Presented in this TRB document is a comprehensive and practical discussion regarding landslide topics aimed to provide a single-source reference for students, researchers, and practicing engineers and geologists. The United States Geological Survey (USGS) and the Geological Survey of Canada (GSC) compiled a document entitled *The Landslide Handbook – A Guide to Understanding Landslides* [75]. The handbook was aimed at helping homeowners, community and emergency managers, and decision-makers to take the positive step of encouraging awareness of available options and recourse regarding landslide hazards.

1.3.2 Digital Datasets

More recent research has focused on emerging technology with landslide risk management. This effort has been invigorated by the National Landslide Preparedness Act of 2021, which authorized a national landslide hazards reduction program involving multiple federal agencies. The National Aeronautics and Space Administration (NASA) and USGS have developed interactive web-based map viewers to help users identify landslide risk and access compiled landslide data by location. NASA launched the *Global Landslide Catalog* (GLC) to identify rainfall-triggered landslide events around the world, regardless of size, impact, or location; the model is comprised of the NASA landslide database, as well as data acquired as part of NASA's *Cooperative Open Online Landslide Repository* (COOLR). The USGS has a similar application in collaboration with state geological surveys and other federal agencies named the *U.S. Landslide Inventory Map*, which includes existing landslide data gathered from a range of federal, state, and local government agencies. In addition to compiling existing landslide data, NASA has developed the *Landslide Hazard Assessment for Situational Awareness* (LHASA) which aims to provide situational awareness of landslide hazards by combining near-real-time precipitation data with landslide susceptibility data to generate estimates of where and when rainfall-triggered landslides are likely to occur around the world.

1.3.3 Local Research

Extensive research has been performed in southwestern Pennsylvania to present data aimed at recognizing slope movement mechanisms and observations specific to the region.

Notable research, still utilized by many regional professionals, includes work that was performed by A.C. Ackenheil, W.R. Adams, R.P. Briggs, J.L. Craft, H.F. Ferguson, N.K. Flint, R.E. Gray, R.J. Hackman, J.V. Hamel, L. Heyman, S.S. Philbrick, J.S. Pomeroy, W.R. Wagner, and others; a large portion of the research performed was focused on producing landslide susceptibility maps. Additional contributions include:

- Shailer Philbrick participated as a member of the Committee of Landslide Investigations to publish the Landslides and Engineering Practice publication for the National Academy of Sciences in 1958, which included local case study information.
- Publications prepared by Reginald Briggs include recommendations and advice for the nontechnical reader, a discussion about geologic factors affecting susceptibility to landsliding, and a description of “selected landslide localities.”
- Research paper about valley stress relief prepared by Harry Ferguson which discusses geologic conditions that contribute to landslide susceptibility.
- Ph.D. Dissertations about landslides, such as the Glenfield Landslide along I-79 near the Ohio River by Dr. William Adams, P.E. Some have published internationally about landslide activity in the Pittsburgh Region, like Hamel and Adams’ Claystone Slides that was published in the proceedings of the International Symposium on Weak Rock that was convened in Tokyo, Japan, in 1981.
- Dr. Alfred Ackenheil published “Physical Capabilities Mapping” which was commissioned as part of an Urban Planning Grant by the U.S. Department of Housing and Urban Development

A full list of publications and references is presented in the bibliography.

Allegheny County has a Landslide Task Force with the goals of maintaining a repository of landslide information and furthering landslide-related research in the region. The Landslide Task Force created the Allegheny County Landslide Portal website, which serves as a central location for region-specific landslide data and resources. Similar to the web-based map viewers created by NASA and USGS, interactive mapping focused specifically to Allegheny County is included as part of the Landslide Portal. The Landslide Map Tool presents data regarding locations of reported landslides, areas susceptible to landslides, and the categorization of roads owned by the county and the state.

CHAPTER 2

Slope Movement Mechanisms and Common Triggers

2.1 CLASSIFICATION OF LANDSLIDES

Landslides may be simply defined as "the movement of a mass of rock, debris, or earth down a slope" [25] and by extension also the landform resulting from this process [75]. A classification system is necessary to describe the wide array of types of landslides and the processes driving their initiation, mechanics of movement, and ultimately planning and design for mitigation of the movement. The classification system for landslides originally presented in Varnes [138], expanded upon in Turner and Schuster [128], and further cited in The Landslide Handbook by the USGS [75] provides a simple criteria and clearly-defined definition of terms for the classification of landslides and is generally consistent with that terminology presented in USGS Professional Paper 1229 [109] specifically for the Greater Pittsburgh Region. This Varnes system, reproduced in Figure 2-1 below, utilizes a two-noun nomenclature, with the first term based on the primary type of material (i.e., earth, debris, or rock) involved in the displaced mass and second term describing the type of movement involved.

Instances of landslides throughout Pennsylvania that have occurred fall within most categories of movement type described in Table 2-1. However, as indicated in Pomeroy [109], the "three principal types of landslide movement are falling, sliding, or flowing, or a combination [thereof]". As a result, the classification of landslides by the practitioner in the southwestern Pennsylvania region should be expected to relate primarily to these types of movement, although other types and combinations may be encountered. Additionally, movement types such as "Fall" and "Topple" commonly occur involving bedrock material displacement from unstable natural and man-made rock slopes. Occurrences of bedrock displacement and considerations for rock slope stability are generally outside of the intended subject matter of this document. However, the reader is referred to the TRB publication on Rockfall Characterization and Control [129] for detailed treatment on rockfall investigation, characterization, prediction, and mitigation and PennDOT Publication 293, Chapter 8 – Rock Cut Slope and Catchment Design. Several landslide movement types are depicted in schematic view from the USGS as Figure 2-1.

Type of Movement	Type of Material		
		Engineering Soils	
	Bedrock	Predominantly Coarse	Predominantly Fine
Fall	Rock fall	Debris fall	Earth fall
Topple	Rock topple	Debris topple	Earth topple
Slide	Rock slide	Debris slide	Earth slide
Spread	Rock spread	Debris spread	Earth spread
Flow	Rock flow	Debris flow	Earth flow

Table 2-1 - Abbreviated Classification of Slope Movements [128]

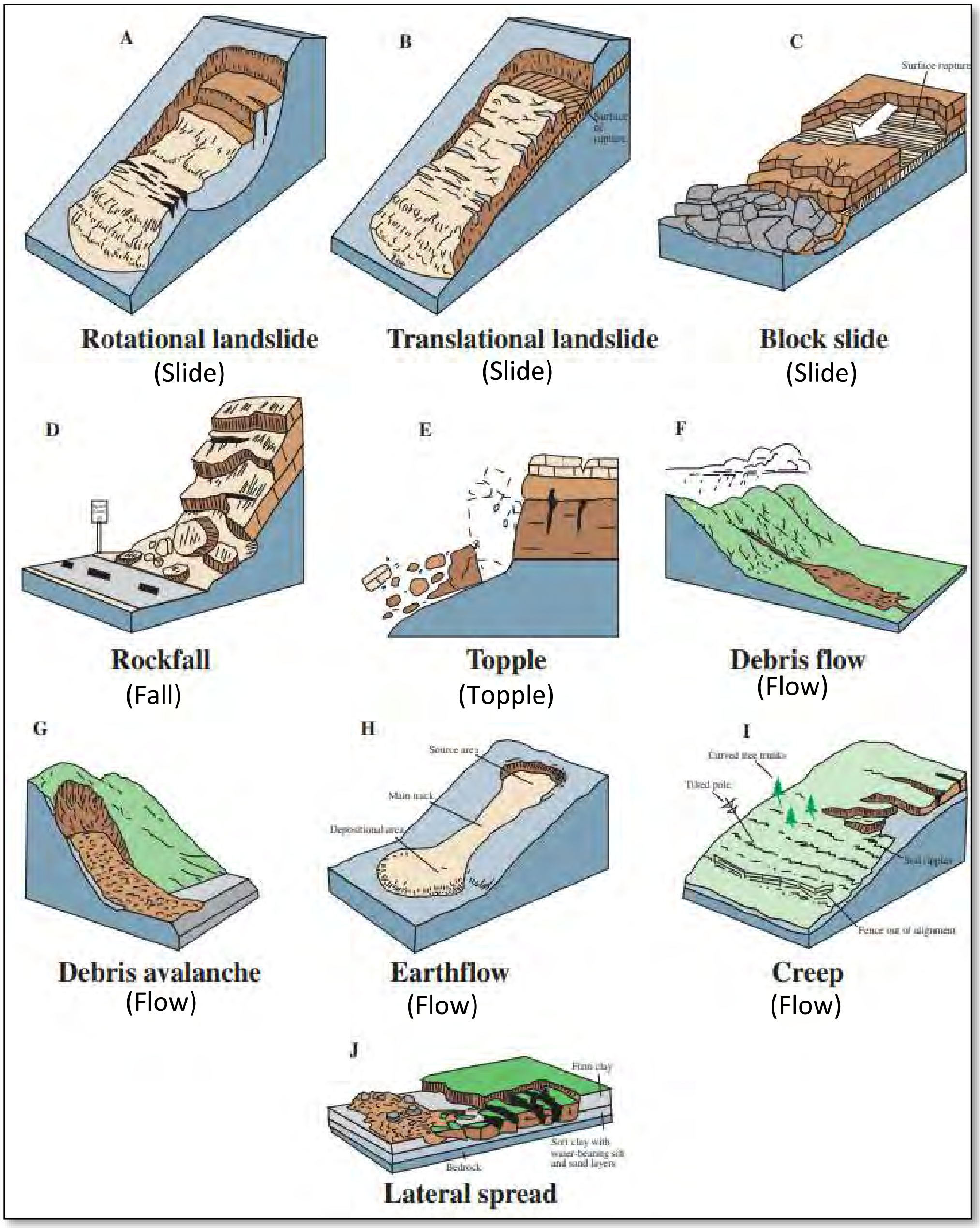


Figure 2-1 - Schematics of Landslide Movement Categories [76]

As indicated in Table 2-2, an additional glossary of adjective terms is available to provide additional description of landslide activity (state, distribution, and style). Landslide activity terms are broadly indicated by State of Activity (timing of movement), Distribution of Activity (where the landslide is moving), and Style of Activity (contribution of different movements to the landslide). Additional terms are also made available to provide a description of the rate of landslide movement and estimation of water content within the landslide mass.

Varnes [138] and Turner and Schuster [128] further emphasize that an important distinction should also be made for slide movement types that are rotational (movement along curved or concave rupture plan) or translational slides (movement along planar or undulating rupture plane). Recognition and characterization of this distinction is important in understanding the mechanism of movement for landslides occurring in southwestern PA.

ACTIVITY			
STATE	**DISTRIBUTION**	**STYLE**	
Active	Advancing	Complex	
Reactivated	Retrogressive	Composite	
Suspended	Widening	Multiple	
Inactive	Enlarging	Successive	
Dormant	Confined	Single	
Abandoned	Diminishing		
Stabilized	Moving		
Relict			
DESCRIPTION OF FIRST MOVEMENT			
RATE	**WATER CONTENT**	**MATERIAL**	**TYPE**
Extremely rapid	Dry	Rock	Fall
Very rapid	Moist	Soil	Topple
Rapid	Wet	Earth	Slide
Moderate	Very wet	Debris	Spread
Slow			Flow
Very slow			
Extremely slow			

NOTE: Subsequent movements may be described by repeating the above descriptors as many times as necessary.

Table 2-2 - Glossary of Forming Names of Landslides [128]

An important consideration in southwestern Pennsylvania is the natural process of soil creep, or creep. Creep is recognized in the Greater Pittsburgh Region in Briggs et al. [16] and Pomeroy [109], but is not considered a type of landslide movement [137] [109] nor recognized as a landslide movement type in Turner and Schuster [128] (Figure 2-1). The process of creep is slope movement that is "proceeding at an imperceptible rate…. Typical creep is a continuous movement which proceeds at an average rate of less than a foot per decade. Higher rates of creep movement are uncommon" [125] [128]. As indicated in Pomeroy [109] "…. soil creep can contribute heavily to damage in an area …. Obvious ground breakage, in the form of scarps and transverse and radial cracks, is lacking in an area of creep; however, creep can accelerate into landsliding. Sags or bulges along the slope may result from the slightest release of stress and are subject to greater movements."

It is additionally noteworthy that colloquial terms commonly used to describe landslides in southwestern Pennsylvania such as "Slip, "Slough", and "Slump" (commonly used to refer to a "Rotational Slide"), are deliberately excluded from use in this Handbook. The use of these and other primary and secondary terms diverging from this standard nomenclature should be discouraged or avoided when attempting to characterize landslide movement as a sufficient range of standardized, descriptive terms are available to the practicing community.

In summary, the Varnes Landslide Classification System as presented in Turner and Schuster [128] is suggested here for use as an industry-accepted landslide classification system that yields repeatable, reproducible results in characterizing landslide activity. The reader is referred to Turner and Schuster [128] for expanded discussion on use of this system and related terms that are applicable for practice in southwestern PA. Alongside the Varnes Landslide Classification System, extensive local studies on common landslide occurrence in the Greater Pittsburgh Region should also be consulted, such as those presented in Briggs et al. [16] and Pomeroy [109], as an integrated approach to classification of landslides in southwestern PA. Furthermore, consistent use of terminology in classifying landslides is encouraged to promote accurate understanding of landslide movement and common use of terms amongst those consultants and contractors, communities, and agencies and authorities contending with landslide activity. Lastly, gradual, slow-acting natural phenomena, such as soil creep, or creep, while prevalent in southwestern PA, should not be misrepresented as landslide movement, but rather carefully considered as a natural process with the potential to evolve into landslide movement.

2.2 DEFINITION OF LANDSLIDE FEATURES

Typical landslides in southwestern Pennsylvania may be characterized by field reconnaissance of the project site by personnel trained in field-identification of landslide features. Nomenclature governing the definition of field-observable landslide features has been presented in several publications by various authors over the years [128]. Figure 2-2 presents an idealized block diagram for a complex earth slide-earth flow type landslide, which is a common type of landslide found to occur within southwestern PA. This figure is frequently reproduced in various publications and is presented here as simplified means with which to denote landslide features.

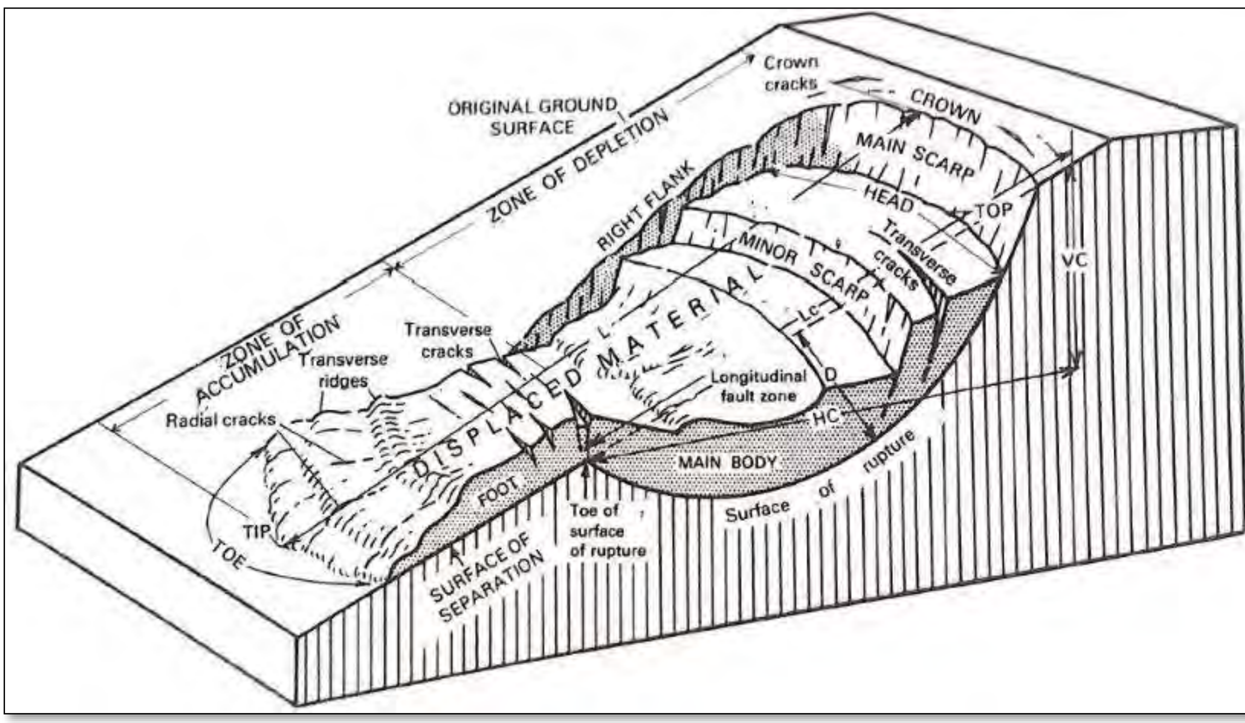


Figure 2-2 - Block Diagram - Idealized Earth Slide/Earth Flow [138]

An expanded body of landslide feature nomenclature is presented by IAEG. The nomenclature presented by IAEG is an adaptation of the nomenclature presented by Varnes [138] for landslide-related features but is presented in plan and section view with numbered definitions of individual landslide features. As indicated in this figure, hatching is used to represent undisturbed ground with stippling to represent displaced material. The numbered nomenclature and specific definitions presented in Figure 2-3 and Table 2-3 is suggested for use for characterization of landslide features in the field during detailed field studies.

In addition to the landslide feature nomenclature described above, the dimensions and approximate volumetric quantities of “Slide” and “Flow” type landslides may be quantified using the system of definitions presented by the IAEG Commission on Landslides [80] and Turner and Schuster [128]. For this system, the dimensions of the length and width of both the displaced landslide mass and original surface of rupture are measured or estimated in the field according to the plan and section diagram in Figure 2-3. Additionally, the depth of the displaced landslide mass and depth of original surface of rupture are measured or estimated. Definitions of the numerical dimension designations of the landslide mass are presented in Figure 2-3 and Table 2-4.

Based on the dimensions obtained for a given landslide, the volume of the landslide mass can be estimated using the methodology provided in Turner and Schuster [128]. The estimated volume can then be used for planning as well as cost estimating and design of mitigation work. The presented volumetric estimates of displaced landslide material are based on the principal that a typical “Slide” or “Flow” type of Movement can be approximated as a half ellipsoid in the “surface of rupture” area.

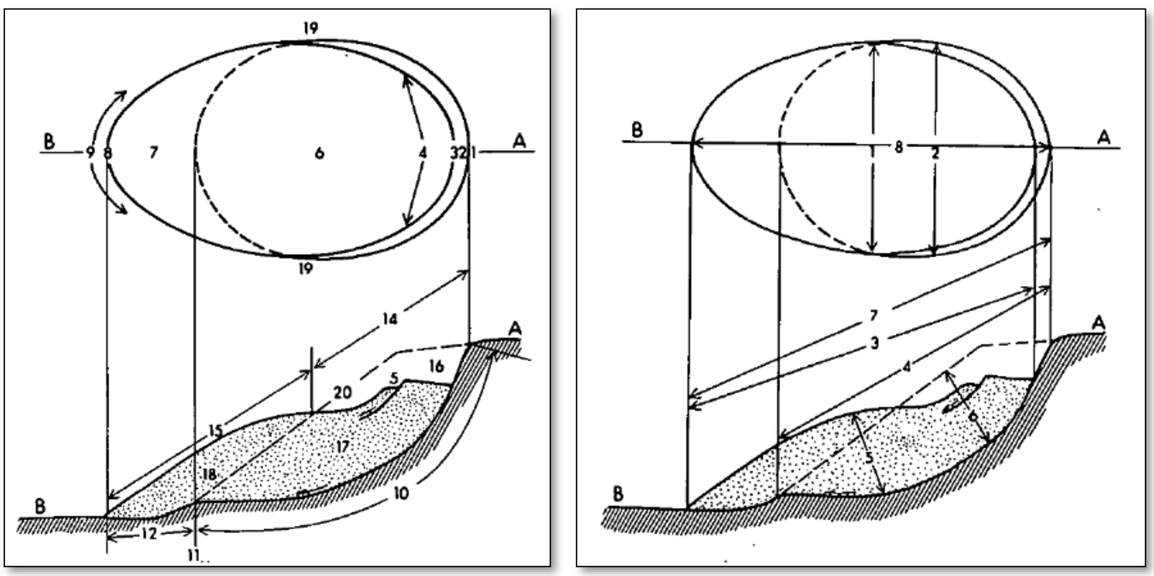


Figure 2-3 - Landslide Features (left) and Landslide Dimensions (right) (IAEG 1990)

As indicated in Turner and Schuster [128] and WP/WLI [144], the volumetric quantity of a landslide may be estimated based on Equation 2-1 (prior to landslide movement) or Equation 2-2 (after landslide movement) using the dimension terms depicted in Figure 2-4 and as defined in Table 2-4.

$VOL_{ls} = \frac{1}{6}\pi D_r W_r L_r$ (Equation 2-1)

$VOL_{ls} = \frac{1}{6}\pi D_d W_d L_d$ (Equation 2-2)

where:

D_r, W_r, L_r = Depth, Width, Length of Surface of Rupture

D_r, W_r, L_r = Depth, Width, Length of Displaced Mass

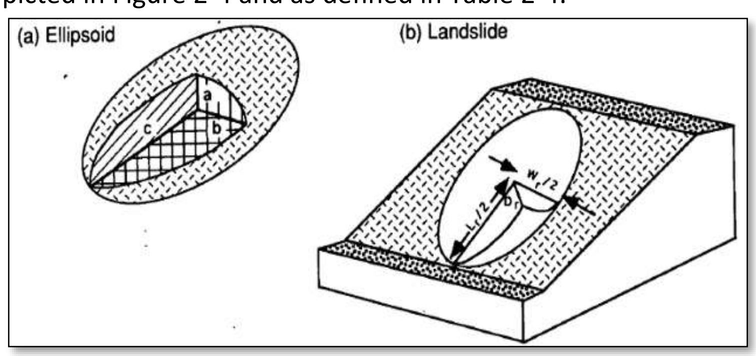


Figure 2-4 - Estimation of Landslide Volume [128]

Number	Name	Definition
1	Crown	Practically undisplaced material adjacent to highest parts of main scarp
2	Main scarp	Steep surface on undisturbed ground at upper edge of landslide caused by movement of displaced material (*13*, stippled area) away from undisturbed ground; it is visible part of surface of rupture (*10*)
3	Top	Highest point of contact between displaced material (*13*) and main scarp (*2*)
4	Head	Upper parts of landslide along contact between displaced material and main scarp (2)
5	Minor scarp	Steep surface on displaced material of landslide produced by differential movements within displaced material
6	Main body	Part of displaced material of landslide that overlies surface of rupture between main scarp (2) and toe of surface of rupture (*11*)
7	Foot	Portion of landslide that has moved beyond toe of surface of rupture (*11*) and overlies original ground surface (*20*)
8	Tip	Point on toe (9) farthest from top (*3*) of landslide
9	Toe	Lower, usually curved margin of displaced material of a landslide, most distant from main scarp (*2*)
10	Surface of rupture	Surface that forms (or that has formed) lower boundary of displaced material (*13*) below original ground surface (*20*); mechanical idealization of surface of rupture is called *slip surface* in Chapter 13
11	Toe of surface of rupture	Intersection (usually buried) between lower part of surface of rupture (*10*) of a landslide and original ground surface (*20*)
12	Surface of separation	Part of original ground surface (*20*) now overlain by foot (*7*) of landslide
13	Displaced material	Material displaced from its original position on slope by movement in landslide; forms both depleted mass (*17*) and accumulation (*18*); it is stippled in Figure 3-4
14	Zone of depletion	Area of landslide within which displaced material (*13*) lies below original ground surface (*20*)
15	Zone of accumulation	Area of landslide within which displaced material lies above original ground surface (*20*)
16	Depletion	Volume bounded by main scarp (*2*), depleted mass (*17*), and original ground surface (*20*)
17	Depleted mass	Volume of displaced material that overlies surface of rupture (*10*) but underlies original ground surface (*20*)
18	Accumulation	Volume of displaced material (*13*) that lies above original ground surface (*20*)
19	Flank	Undisplaced material adjacent to sides of surface of rupture; compass directions are preferable in describing flanks, but if left and right are used, they refer to flanks as viewed from crown
20	Original ground surface	Surface of slope that existed before landslide took place

Table 2-3 - Definitions of Landslide Features [128]

Number	Name	Definition
1	Width of displaced mass, W_d	Maximum breadth of displaced mass perpendicular to length, L_d
2	Width of surface of rupture, W_r	Maximum width between flanks of landslide perpendicular to length, L_r
3	Length of displaced mass, L_d	Minimum distance from tip to top
4	Length of surface of rupture, L_r	Minimum distance from toe of surface of rupture to crown
5	Depth of displaced mass, D_d	Maximum depth of displaced mass measured perpendicular to plane containing W_d and L_d
6	Depth of surface of rupture, D_r	Maximum depth of surface of rupture below original ground surface measured perpendicular to plane containing W_r and L_r
7	Total length, L	Minimum distance from tip of landslide to crown
8	Length of center line, L_{cl}	Distance from crown to tip of landslide through points on original ground surface equidistant from lateral margins of surface of rupture and displaced material

Table 2-4 - Definitions of Landslide Dimensions [128]

It is important to note that the estimation of volumetric quantities of landslides masses using the half ellipsoid method assumes a semi-circular arc beneath the surface of rupture area, which will not be the case for translational-type slides or complex, combination-type landslide types containing planar or semi-planar rupture planes common in southwestern PA. Additionally, the accuracy of the volumetric estimate will be governed in large part on the accuracy of the dimensions with respect to actual landslide dimensions, particularly the depth of the rupture plane and displaced mass, which may not be available without the aid of physical drilling and sampling methods. As a result, care should be exercised by the practitioner in the estimation of landslide volumetric quantities using the half ellipsoid method given the limitations of its application. Alternative means of estimating volumetric quantities of displaced landslide materials will need to be selected in cases where the half-ellipsoid method is not applicable.

2.3 CAUSES OF LANDSLIDES

Identifying and understanding the causal factors behind a landslide occurrence is of critical importance in understanding the most appropriate and cost-effective approach to landslide mitigation. A multitude of factors exist that contribute to landslide movement. For any given landslide, one or many factors may be contributing to the movement, each with varying degrees of influence on the imbalanced forces leading to the landslide. As indicated in Turner and Schuster [128], the three basic causes of landslide occurrence may be grouped as processes resulting in: 1) increase in shear stress, 2) low material shear strength, and 3) reduction in existing material strength. These three basic causes are summarized in the discussion below following the categories and presentation in Turner and Schuster [128].

2.3.1 Increase in Shear Stress

A sufficient increase in the driving forces beyond the capacity of counteracting resisting forces lead to an imbalanced condition, resulting in landslide movement. A major process resulting in increased shear stresses is the imposition of surcharges, typically at the upper portions of a slope. Examples of surcharge loading may include temporary stockpile placement, waste dumps, embankment regrading/widening, or the construction of new structures or modification to existing structures. Pomeroy [109] recognized surcharging by fill placement on slopes as a significant concern in the Greater Pittsburgh Region where "[m]ost fill failures are slumps and are of two types - those within the fill material itself that are largely independent of the materials on which the fill was placed and those that result from emplacement of fill materials on steep unstable slopes where both fill and underlying slope material move."

A second major process that increases shear stress is an increase of unit weight of slope materials. An increase in the unit weight of slope materials and resulting driving shear stresses can result from infiltration of surface water from precipitation and snow melting events, groundwater infiltration, malfunctioning/poorly maintained stormwater drainage systems, and leaking canals, irrigation systems, reservoirs, sewers, and septic tanks. Placement of new structures or modification of existing structures may also contribute driving surcharge to the existing shear stresses within a slope.

Corresponding to increases in shear stresses, loss of toe support, typically near the lower reaches of a slope, can lead to a sufficient reduction of resisting forces, resulting in imbalanced conditions and causing landslide movement. Processes resulting in loss of toe support may be natural or manmade. Natural processes common to southwestern Pennsylvania include erosion at the toe of slopes by scour action of water in rivers and streams passing in close proximity to the toe of a slope "….at the point of maximum curvature of the stream where the slope receives the greatest erosive force from the water…." [109]. This process indicates that "[l]andslides and flooding are closely associated because both are related to precipitation, runoff, and the saturation of groundwater" [75]. Similarly, discharge through unlined stormwater drainage channels during heavy precipitation events may cause a similar material loss and loss of lateral slope support. Lateral support loss may occur as a result of the removal of building and retaining structures, excavation earthwork, mining activity, and drawdown of lakes and reservoirs.

Lastly, transient forces resulting from natural processes, e.g., earthquakes, as well as human processes, e.g., pile driving, vehicular or rail traffic vibrations, and blasting may trigger landslide movement, particularly for soil materials sensitive to the adverse effects of transient loading conditions, e.g., sensitive clays, saturated silts and fine sands.

2.3.2 Low Material Shear Strength

Geologic processes such as the chemical weathering/alteration of soil particles to lower-strength clays, saturation-softening of fine-grained materials, decomposition of organic materials, or some combination thereof result in inherently low shear strength properties of the material. Geologic deposition or rearrangement of soil fabric into a structure sensitive to changes in stress may also contribute to landslide-susceptible conditions.

Similar to soil materials, geologic processes leading to the formation and weathering/alteration of bedrock materials result in adverse rock mass characteristics and inherently low shear strength. Particles constituting bedrock are subject to chemical weathering and alteration to lower strength clays, as well as loss of cementation in the rock fabric. Poorly indurated and/or fine-grained bedrock lithologies such as claystone, mudstone, siltstone, and clay and silt shales (all prevalent in southwestern PA) will typically exhibit lower rock mass shear strength characteristics because of the smaller particle sizes in their rock matrix and susceptibility to alteration by natural weathering processes. These geologic processes are particularly pertinent to southwestern Pennsylvania as observed in Pomeroy [109] where "[m]ost landslides observed in the Greater Pittsburgh area took place in colluvial or residual clayey to clayey silt soil and weathered rock derived from mudstone, claystone, and some shale."

Additionally, the formation of discontinuities within rock masses, including jointing, highly altered/weathered seams, faulting, and shear/gouge zones results from natural geologic processes acting on bedrock, which in turn contribute to low shear strength conditions. In particular for the southwestern Pennsylvania region, the preferential formation of stress relief joints in bedrock materials underneath valley bottoms and along valley sidewalls has been well-documented as a result of elastic response of the bedrock materials to the removal of vertical and lateral confining stress during valley formation in the region [39][65]. The presence of these and other types of rock mass discontinuities are significant contributing factors to low shear strength of rock masses.

2.3.3 Reduction in Existing Material Shear Strength

The principal contributing factors acting to reduce shear strength of the materials constituting slopes in southwestern Pennsylvania are elevated pore water pressure and moisture content. This condition contributes to a wide range of landslide activity in southwestern PA, and in particular severe cases of earthflow movement as observed in Pomeroy [109] where "[e]arthflows…. consist of colluvial (or fill) materials that move downslope as a viscous fluid. An earthflow has a scarp at its head and bulges and tension cracks at the toe. It grades into a mudflow in which water content is greater."

As previously stated, a variety of sources may contribute to introduction of water and elevation of the potentiometric groundwater surface within a slope. The presence of groundwater within a slope acts to elevate pore water pressure within the soil materials constituting the slope (fine-grained and coarse), producing buoyancy and uplift conditions, reducing intergranular effective stress, and thus frictional shear strength of the material. The reduced material shear strength condition is most severe when it occurs at the mid to lower portion of a slope, where the shear strength at these locations is acting to resist driving stresses. Additional effects of groundwater introduction into slope materials are to soften clay-bearing fine grained soils by hydration of the clay minerals constituting the soil, resulting in reduction of the cohesion component of the overall soil shear strength.

The effects of reduction in effective stress and cohesion on the soil strength are evident by inspection of the Mohr-Coulomb strength criterion, expressed as:

$\tau_f = c + \sigma' tan\varphi$ (Equation 2-3)

$\sigma' = \sigma - \mu$ (Equation 2-4)

where:

$\tau_f = Shear\ Strength\ of\ Soil$	$\sigma = Total\ Normal\ Stress$
$c = Cohesion$	$\mu = Pore\ Water\ Pressure$
$\sigma' = Effective\ Normal\ Stress$	$\varphi = Angle\ of\ Internal\ Friction$

In terms of rock mass shear strength, positive elevated pore water pressure within discrete rock mass discontinuities generates uplift conditions along the discontinuities, reducing effective stress, and thus acts to reduce sliding friction along potential rupture surfaces. This presence of groundwater may further act to soften the intact rock mass, soil infilling materials where present within the discontinuities, and other discontinuity types that may be present (e.g., highly weathered seams, gouge zones). Correspondingly, the presence of water-filled discontinuities in a vertical or inclined orientation may act to induce additional lateral water pressure loading on discrete rock mass segments within a slope, resulting in additional surcharge loading to the stress system within the slope.

As indicated in Turner and Schuster [128], secondary processes that reduce shear strength, such as weathering and physiochemical reactions, commonly affect fine-grained clay soils. These processes include fissuring of clays due to drying or release of vertical and lateral stress by erosion or excavation. This exposes the clays to the softening effects of water entering into the fissures as well as elevation of pore water pressure within the clay and lateral water pressure loading. The shear strength of some clays may be further affected by ion exchange between the clay minerals and water passing through fissures and pore spaces within the clay. Additionally, freeze-thaw and thermal expansion-contraction can have adverse effects to the shear strength of not only clays but also on rock masses, resulting in accelerated weathering, disintegration, and formation/exacerbation of discontinuities.

2.4 TRIGGERING MECHANISMS

Landslide triggering mechanisms as presented in this document are defined as an external stimuli that directly impact a slope and lead to an immediate or near-immediate slope response, i.e., initiation of new slope displacement or substantial acceleration of pre-existing movement (e.g., creep). As recognized in Turner and Schuster [128], in some cases landslides may occur without a definitive trigger, but rather as a result of one or several ongoing processes that gradually cause slope movement. A wide range of landslide triggering mechanisms exist, some of which are applicable regardless of region (e.g., intense rainfall, earthquakes), whereas others may only practically pertain to a specific region (volcanic eruption, wildfire activity). Those natural and man-made landslide triggers common to the southwestern Pennsylvania region are presented herein, broadly categorized and summarized as indicated below.

2.4.1 Precipitation

A strong correlation exists between precipitation and triggering of landslide activity. Precipitation as a landslide trigger relates to those rainfall events of a sufficient intensity and duration to result in landslide activity. Correspondingly, melting of a snowpack accumulated from previous snowfall events via rainfall and/or rapid temperature increases are considered as an equivalent to rainfall so far as an equivalence between snowfall accumulation and rainfall can be made. Landslide triggering effects of precipitation in the Greater Pittsburgh Region are noted by Pomeroy [109], where "[l]ate winter and early spring rains in combination with the thawing of partially to completely frozen ground create unstable conditions along the slopes. Consequently, more slides take place at this time of the year….".

The commonly accepted means by which precipitation can trigger landslides is by the hydrologic process of infiltration. The rainwater infiltrates into soil pore spaces (primary porosity) and/or open fissures in soil and bedrock (secondary porosity). This infiltration process works to temporarily increase the density of soil and rock materials, reduce the cohesion of fine-grained materials (i.e., clays), and result in elevated levels of the potentiometric groundwater surface. No specific magnitude or threshold of rainfall intensity/duration and/or amount of snowmelt can be quantified as sufficient to cause a landslide in southwestern PA. The slope geometry, pre-existing soil, bedrock geologic, and groundwater conditions, and past stress history will govern the amount of rainfall/snowmelt necessary to trigger movement for any particular slope. Additionally, while indirectly tied to precipitation events and infiltration, seasonally variable seep and spring locations serve as point source concentrations of slope saturation with corresponding destabilizing effects.

2.4.2 Seismic Loading

Similar to precipitation events, seismic loading, referred to here as earthquakes, is another landslide triggering mechanism, particularly for steep-sloped areas that are already landslide-prone [75]. The source of earthquake loading in southwestern Pennsylvania may result from natural sources of earthquake energy due to sudden displacements in the Earth's crust from tectonic stresses. In the case of seismic-loading, soil slope movement may be triggered where transient earthquake loads transmitted as seismic waves through the subsurface act to alter the pre-existing stress field within a slope. Depending on the orientation with which these loads pass through the slope, the resulting effects may involve either a temporary increase in the driving shear stresses within the slope, reduction in effective normal stress and resulting shear strength of materials providing resisting forces, or some combination thereof.

As a further effect of transient seismic loading, certain soil material types including saturated loose silts and fine-grained granular materials found near rivers, lakes, and terrace deposits, may undergo a liquefaction response (quick state) depending on the duration and frequency of the seismic loading. In the liquefaction process, susceptible soil materials experience a significant or complete loss of material shear strength. The material responds to the seismic loading by densifying during shear, but due to the short duration of the loading is unable to drain during shear, resulting in temporary spiking of pore water pressure and loss of effective stress.

Similar to the effects of precipitation in southwestern PA, the ability of seismic loading to trigger landslide activity is dependent on the inherent characteristics of individual slopes. As a result, the frequency and magnitude of seismic loading required to trigger landslide movement varies for individual slopes.

2.4.3 Rapid Water Level Change

As presented in Turner and Schuster [128], rapid changes, particularly lowering, of water levels within slopes adjoining natural waterways (e.g., lakes, rivers, and streams) or man-made features (e.g., reservoirs, canals) can trigger landslides. These rapid changes may result from natural processes such as dissipation of high-water flood stages or deliberate (man-made) water level drops such as the case of a reservoir lowering by draining at a dam control structure or controlled dam breach.

In these cases of rapid lowering of water levels, the stability of adjoining soil slopes may be adversely affected where the rate of water level drawdown exceeds the rate with which the potentiometric surface within the slope can respond and dissipate down to an equilibrium condition with the new water level within the waterway. The resulting effects where this condition occurs are a temporarily elevated pore water pressure within the slope (with respect to the new water level) and loss of lateral water pressure against the slope. The reduction in soil shear strength within the slope combined with the loss of resisting stress against the lower reaches of the slope face can trigger landslide movement before the potentiometric surface within the slope can dissipate (drain) down to equilibrium with the new water level within the waterway. This process is depicted in Figure 2-5 from Turner and Schuster [128] (originally presented in Lambe and Whitman [83]).

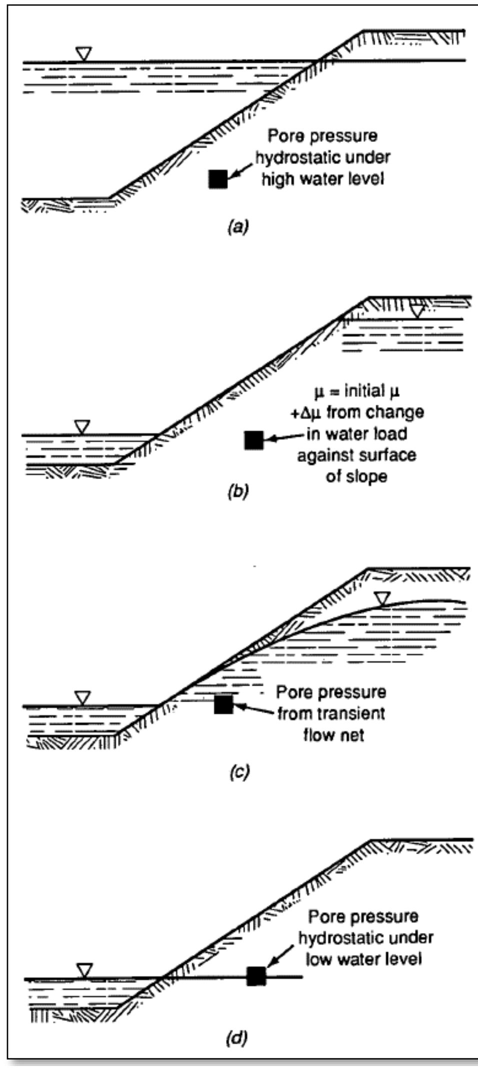


Figure 2-5 – Rapid Drawdown [83]

2.4.4 Human Activity

The effects of human activities are a frequent and recurring landslide triggering mechanism in southwestern PA, and result from a large assortment of activities including but not limited to poor practices in siting and executing civil (and private) constructions operations on or in close proximity to slopes, incidental effects and after-effects of mining activities in bituminous coal seams, aging or inadequate maintenance of infrastructure such as stormwater and sewer pipes, and improper installation and maintenance of erosion and sedimentation control measures. The effect of common human activities in triggering landslides is explored with respect to their relationship to several landslide causes described herein. For more information on this topic, refer to the Greater Pittsburgh Region in Briggs et al. 1975 and Pomeroy [109].

Excavations into the toe of slopes and placement of fill on slopes are both recognized in Briggs et al. 1975 and Pomeroy [109] as prevalent landslide activity triggers in southwestern PA. In the case of civil works and construction operations, earthwork activities may provide the most prevalent recurrence of landslide triggering primarily where excavations are made into the toe and lower reaches of slopes or where previously constructed retaining structures are removed, both of which result in loss of lateral resisting support and imbalanced stress conditions within the slope. Correspondingly, placement of fill for site grading purposes, temporary or permanent material stockpiling, and construction of new shallow-bearing structures along the upper reaches and tops of slopes result in application of static surcharge loads to the slope system, increasing driving shear stresses to a magnitude sufficient to create imbalanced conditions and initiate slope movement. Placement of fill with poor strength characteristics and improper compaction for private property modifications (e.g., driveways, yard space, parking areas) may result in similar surcharge loading. Recurring transient surcharge loading due to vehicular or railroad traffic on alignments constructed at the top or along upper reaches of slopes can have similar triggering effects to static surcharge loading. The triggering effects of both excavation activities at the toe of slopes and surcharging loading at the top of slopes are exacerbated where they take place in areas where landslide movement has previously taken place but since equilibrated, or where creep or landslide activity is already taking place at a very low rate of movement. As a noted practice commonplace for the Greater Pittsburgh Region, "[excavation at the foot of the slope to make more flat land in the narrow valley areas is a popular practice. If the cut is in the toe of an unidentified old landslide deposit, slippage or accelerated creep…. might take place" [109].

Improper design, installation, inspection, and/or maintenance of temporary and permanent erosion and sedimentation control measures commonly act to trigger landslides associated with civil construction work. For temporary erosion control measures where recent site grading has resulted in unvegetated slope areas, inadequate cleanout of typical stormwater control features such as water bars, silt fences, compost filter socks, and rock-lined sumps can result in malfunction of these features during rain precipitation events, leading to uncontrolled stormwater runoff to susceptible slope areas, and further resulting in excessive erosion, ponding, and/or infiltration of water into the slope. Open channel stormwater conveyance channels functioning post-site restoration that are not routinely inspected and maintained may similarly permit uncontrolled discharge of stormwater to landslide-susceptible slope areas. Improper siting of well-functioning erosion and sedimentation control measures may still result in landslide triggering where site design allows for the construction of stormwater runoff to landslide-susceptible areas of a slope.

Aging water, stormwater, and sewer conveyance infrastructure, for which inadequate funding exists for maintenance and replacement work (a common problem throughout southwestern PA) provides a large source of triggering modes for landslide activity. In particular, inlet and conveyance structures constructed from brick masonry may suffer the deleterious effects of mortar aging and breakdown/strength loss, and subsequent loss of integrity, leading to backup of stormwater and sewage effluent. Where this condition occurs in the vicinity of landslide-susceptible slopes, the affected areas may serve as point sources for infiltration into and saturation of surrounding slope materials. Infiltration through deteriorated brick-masonry lined open stormwater channels can serve as similar sources of water infiltration into slopes. Similarly, cracking of ceramic piping or corroded ductile iron piping may provide similar point sources of water infiltration into slope materials, as well as drainage tile from abandoned or developed agricultural areas. Additional point sources of water infiltration into slopes may result from unregulated outlet of private residential, commercial, or industrial building structure downspouts, liquid wastes, and sewage effluents onto slopes. Clogging of otherwise properly functioning stormwater inlets by vegetative or soil debris may contribute to conveyance and resulting infiltration of water onto slopes in a similar fashion. Lastly, undocumented or unregulated mine drainage discharge locations with seasonally fluctuating discharge rates can serve as significant point sources for slope saturation, along with leaking swimming pools and decorative pools and ponds.

The effect of past and present bituminous coal extraction (i.e., mining) in southwestern Pennsylvania may also contribute to the triggering of landslides. As specifically pointed out for the Greater Pittsburgh region, "[a]ny landsliding that is as much as several hundred feet above a mined-out horizon might be due, at least in part, to subsidence" and "[e]mplacement of fill over a mined-out area could lead to a collapse of the underlying strata and induce movement of the over-lying earth materials" [109]. As further pointed out in Pomeroy [109], loose placement of mine spoils can contribute surcharge loading to a slope, resulting in landsliding.

CHAPTER 3

Identification of Failure Prone Areas and Features

3.1 VULNERABLE LOCATIONS AND FEATURES

The nine counties constituting the southwestern Pennsylvania area for the purposes of this document (Allegheny, Armstrong, Beaver, Butler, Fayette, Greene, Indiana, Washington, and Westmoreland) and their inherent topographic and geologic features have been recognized as a landslide-prone region by many writers and agencies. The regional geology as it relates to landslide susceptibility has been discussed by numerous previous works including Ackenheil [6][7], Briggs [15][16], Delano and Wilshusen [31], Hamel [66], Myers [90], Philbrick [107], Pomeroy [109][111][113], Wagner et al. [140], and Winters [143].

Identification of landslide-prone areas and slopes can serve as a proactive tool to avoid or maintain at-risk slopes before any major slope movement occurs. Alternatively, where slope movement has occurred, identification of landslide-prone areas can serve to characterize existing conditions that may have led to landslide movement and inform the subsequent landslide investigation.

3.2 LANDSLIDE MAPPING

Mapping and related literature about landslide susceptibility, and past landslide occurrences, are available to the practitioner from a variety of sources (see Section 3.1); these works were often prepared using air photo interpretation combined with field reconnaissance verification and other site-specific studies.

General landslide susceptible areas for southwestern PA, depicted in Figure 3-1, are broadly categorized on a scale ranging from high landslide susceptibility to generally low susceptibility (note that landslide susceptibility varies by location within each area). Southwestern Pennsylvania falls entirely within the Appalachian Plateaus physiographic province and spans three of its Sections – Pittsburgh Low Plateau, Waynesburg Hills, and the Allegheny Mountain. Approximately half of the southwestern Pennsylvania region is within the Pittsburgh Low Plateau section, which covers portions of all of the counties within the region and represents an area of generally high to moderate landslide susceptibility. The Waynesburg Hills section, covering the majority of Greene and Washington counties and extending into Allegheny, Westmoreland, and Fayette counties, is broadly categorized as having the highest landslide susceptibility in the region. The Allegheny Mountain section is limited to the eastern end of the region covering the eastern portions of Indiana, Westmoreland, and Fayette counties, and is represented as generally low landslide susceptibility with localized high to moderate susceptibility.

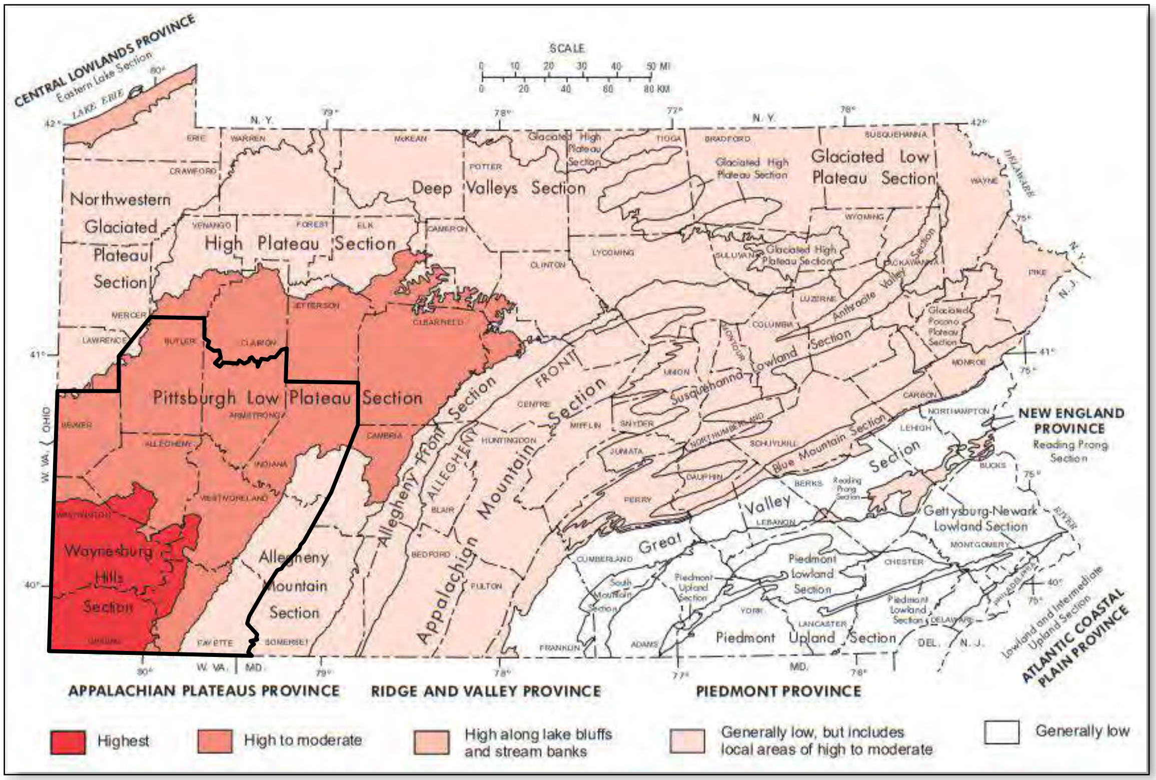


Figure 3-1 - General Landslide Susceptibility in Pennsylvania [31]

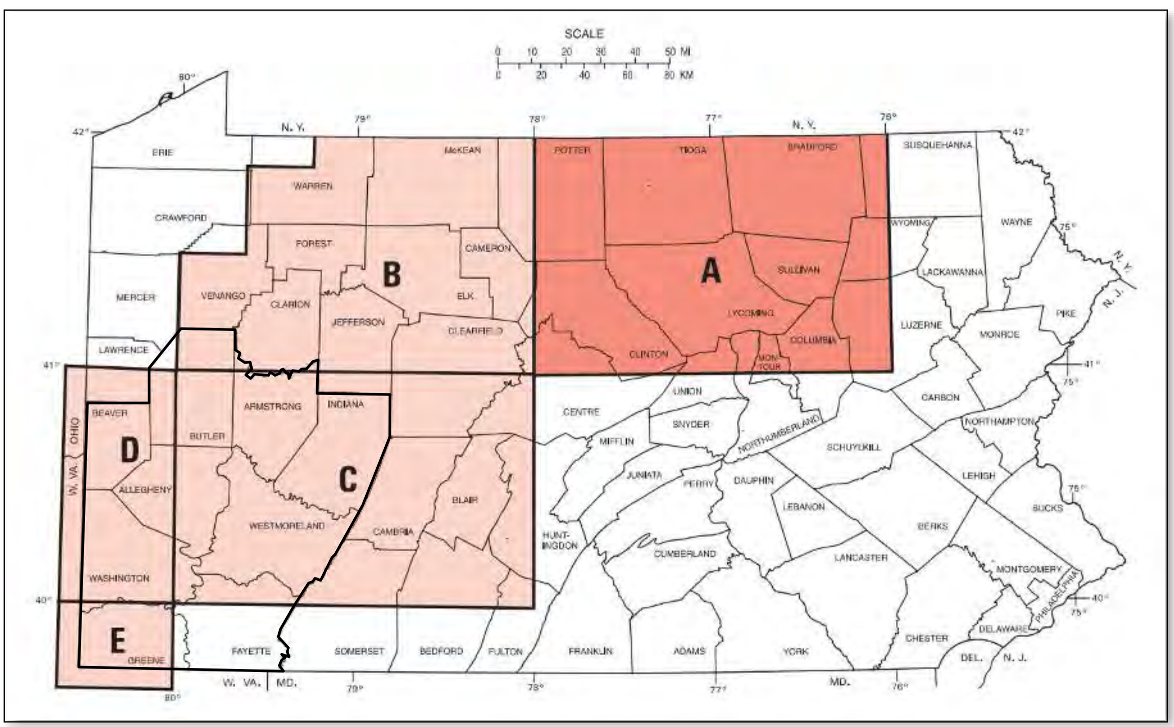


Figure 3-2 - Landslide Inventory Mapping Coverage in Pennsylvania [31]

3.2.1 Landslide Susceptibility Maps

Landslide susceptibility mapping illustrates areas that have the potential for a landslide to occur and areas of historic instability. These maps are created by correlating principal factors that contribute to landslides (such as steep slopes, weak geologic units, and poorly drained rock or soil) with the past distribution of landslides.

Specific mapping of landslide inventory which can be used to assess landslide susceptibility is readily available from the U.S. Geological Survey (USGS) for nearly the entire southwestern Pennsylvania region, as indicated in Figure 3-2. The available mapping presents landslide data overlain onto topographic maps including but not limited to active or recently active landslides, old-inactive landslides, overdip slopes, colluvial slopes and areas susceptible to debris flows, and topographic features (e.g., concave slopes and u-shaped valleys) with soil and groundwater conditions predisposed to landslide activity. Typically, the resolution of mapping available in southwestern Pennsylvania is limited to the 7.5-minute quadrangle sheets (Figure 3-3) which make up larger 1x2-degree map areas.

Mapping of landslides and related features for Armstrong, Indiana, Westmoreland, eastern portions of Allegheny, Butler, and Washington counties, and northern Fayette County is covered by the Pittsburgh 1x2 degree sheet, indicated by the shaded area labeled C [114]. Beaver and the western portions of Allegheny and Washington counties are covered by the Canton 1x2 degree sheet, labeled as shaded area D [30]. Shaded area E from the Clarksburg 1x2 degree sheet covers the majority of Greene County [64].

Resources for landslide susceptibility mapping are listed below:

- Allegheny County Landslide Map Tools – presents landslide susceptibility within Allegheny County by highlighting areas with recent or historic landslides, slopes with observed creep, “Red Bed’ outcrops, and areas of highly variable slope conditions.
- Landslide Susceptibility Open File Reports – landslide susceptibility mapping developed by Pomeroy for southwest Pennsylvania developed by Pomeroy with the degree of susceptibility ranging from little, slight to moderate, and moderate to severe
- USGS Landslide Inventory Mapping (PA) – presents links to the USGS landslide inventory mapping for all 7.5-minute quadrangles in Pennsylvania.

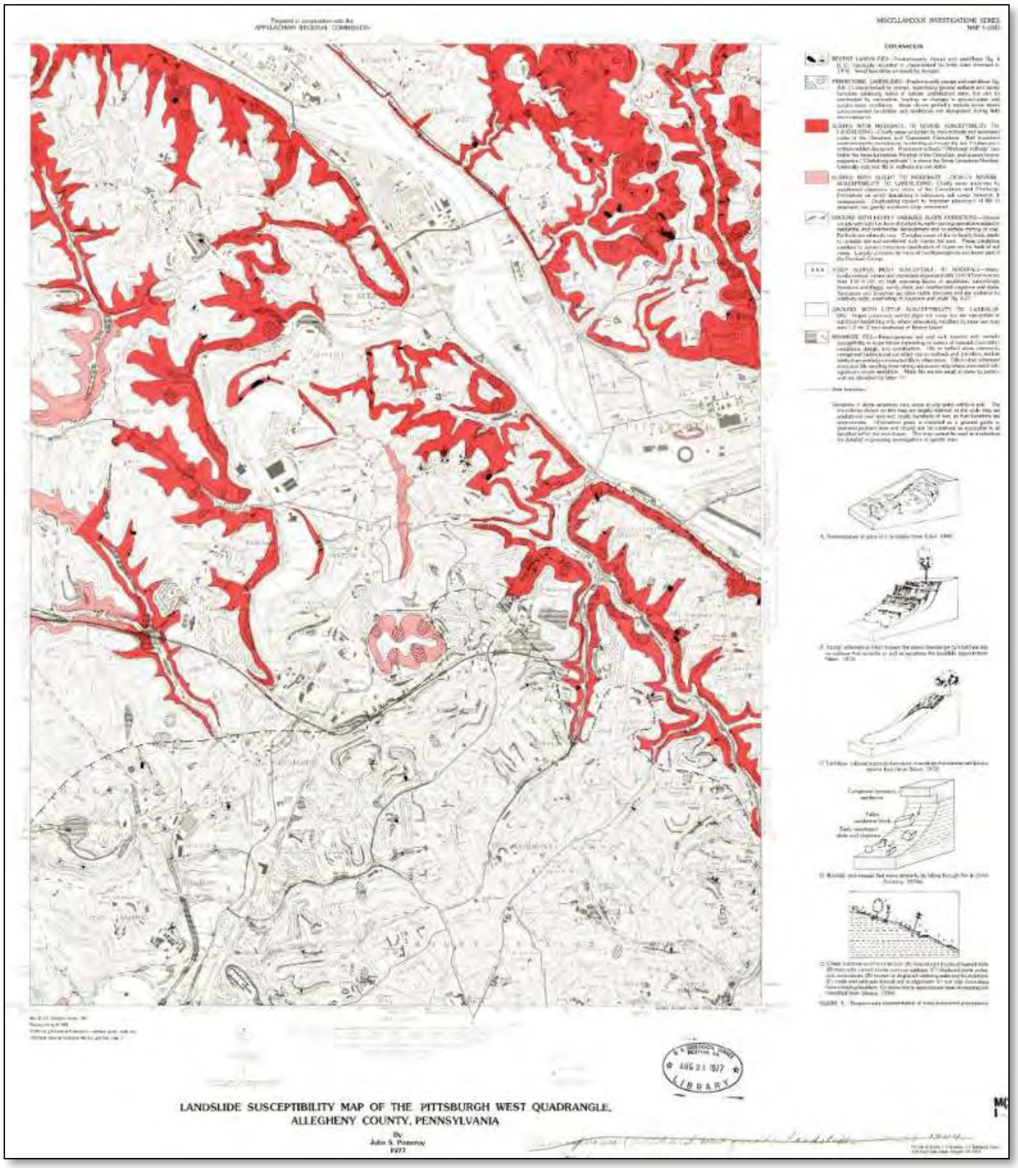


Figure 3-3 - Landslide Susceptibility Map of the Pittsburgh West Quadrangle, Allegheny County, Pennsylvania [112]

3.2.2 Landslide Inventory Maps

While landslide susceptibility mapping presents areas of historic slope movement or other risk factors to slope stability, landslide inventory mapping provides more detailed data for areas that have failed by a landslide process which may include dates, times, notes, and photographs. These maps are useful to inform the practitioner with a general assessment of the site by understanding the details of recorded landslide processes occurring in an area. Figure 3-4 shows an example of a landslide inventory map.

Resources for landslide inventory mapping are listed below.

- Allegheny County Landslide Map Tools – presents recorded landslides in Allegheny County including the potential information (where available) regarding location, traffic restrictions, remediation (Yes/No), damage, and photos.

- NASA Global Landslide Catalog – presents recorded landslides globally including potential information (where available) regarding location, date, trigger, and information source links.
- USGS Landslide Inventory - GIS-based inventory of landslides covering the United States. This platform includes landslide inventoried by the USGS for southwestern PA, including spatial mapping of the counties within the subject southwestern Pennsylvania area, and point nodes of landslide locations documented therein. Landslide location nodes allow for access to information about the date and time of occurrence, probability (or confidence) in the extent or nature of the landslide, general notes about the landslide occurrence, and web page locations for data sources and additional information.

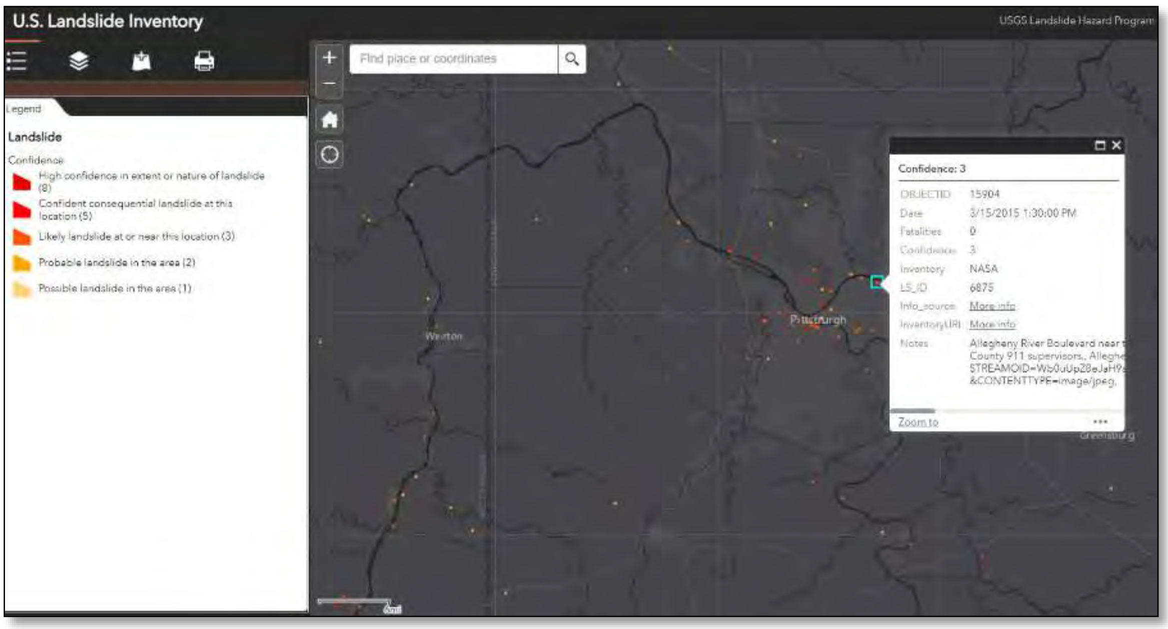


Figure 3-4 - Example of the USGS landslide inventory mapping showing the location of reported landslides [131]

3.3 TOPOGRAPHIC AND HILLSHADE MAPPING

Topographic mapping and hillshades are great sources to gain a three-dimensional perspective of the terrain to help identify landslides, changes in topography, and the history of site development. These maps can help the practitioner to identify terrain features indicative of landslide activity and human activity that may result in increased landslide susceptibility.

Resources for topographic and hillshade mapping are listed below:

- Topographic Mapping – Interactive web mapping application which allows access to current and historic topographic mapping for all 7.5-minute quadrangles within the United States.
- PASDA Pennsylvania Imagery Navigator – Online interactive mapping tool; statewide LiDAR hillshade mapping layer available.
- USGS The National Map - Online interactive mapping tool; USGS shaded relief mapping layer available. The USGS 3D Elevation Program (3DEP) Bare Earth DEM Dynamic service is based on multi-resolution USGS DEM sources and provides dynamic functions for visualization. These functions include: Hillshade, Aspect Map, Hillshade Stretched, Multi-directional Hillshade, Slope Map, Elevation Tinted Hillshade, Contour. In addition, the Open Geospatial Consortium (OGC)

Web Map Service (WMS) and Web Coverage Service (WCS) interfaces are enabled. Data available in this map service reflects all 3DEP DEM data published as of December 21, 2022.

3.4 AERIAL ORTHOPHOTOS

Recent and past aerial photographs of an area may be reviewed since older slides might not be apparent in more recent photographs. Features visible on aerial photographs can help identify landslide type and develop a reasonable assessment of overburden characteristics which ultimately support postulation about landslide hazards.

Valuable information on site topography, geomorphological characteristics, previous landslide activity, historical land use, and their corresponding influence on landslide vulnerability for discrete locations and areas, may be gained by review of current and historic aerial photography. The Pennsylvania Imagery Navigator, maintained by PSU on the Pennsylvania Spatial Data Access (PASDA) website, provides readily available recent and historic aerial photographic imagery that can be retrieved for all areas of Pennsylvania, including the entire southwestern Pennsylvania region. LiDAR hillshade imagery is also available for review and retrieval from the Pennsylvania Imagery Navigator for use in identifying landslides and landslide-vulnerable locations.

Resources for historic imagery applications are listed below:

- Historic Aerial Image Viewer - Archive of historic aerial photos available between the 1940s and the 1970s.
- PASDA Pennsylvania Imagery Navigator – Archive of historic aerial photos available between the 1990s and 2019.
- Pittsburgh Historic Maps - Historic City Maps circa 1800's and 1900's; aerial photos from 1939, 1957, 1967, and 1993.
- Google Earth – 3D representation of earth based primarily on satellite imagery with capabilities to assess topographic features and access historic aerial views globally.

3.5 GEOLOGIC MAPPING

3.5.1 Soils

General soils data for the site is available through the USDA Web Soil Survey. This tool can provide useful information on the recorded soil properties including parent material, landforms, estimated engineering properties, and general drainage properties of the soil units. For more detail regarding the mapped soil units obtained using the Web Soil Survey, it is important to reference the complete soil survey reports.

Resources for soils data are listed below:

- USDA Web Soil Survey – Web-based mapping application.
- Soil Survey for Allegheny, Armstrong, Beaver/Lawrence, Butler, Fayette, Greene/Washington, Indiana, and Westmoreland counties – Complete Soil Survey Reports.

3.5.2 Bedrock Geology

The USGS provides various types of maps that can be used in landslide analysis such as maps of bedrock and surficial geology, topography, soils, and geomorphology. These maps are useful to obtain a general knowledge of the geologic units and the soil origins within the limits of the site. Based on the extensive research performed by Pomeroy and Hamel (see Chapter 1 for further discussion), bedrock geology is of particular interest when assessing landslide susceptibility in southwestern Pennsylvania.

An additional resource available through USGS includes structure contour maps which provide the approximate outcrop elevations of common marker beds in the area, namely the Pittsburgh coal, Ames limestone, the Upper Freeport coal, the Middle Kittanning coal, and several other coal beds. From these maps, the approximate location in the stratigraphic column at the site can be determined as well as the approximate dip of the bedrock.

Resources for geologic mapping applications are listed below:

- Stratigraphic Column – Stratigraphic columns for western Pennsylvania (see Plate 3).
- Atlas A27 (Pittsburgh 15' quadrangle) – Resource to identify the crop line for the Ames limestone and view structure contours in portions of Allegheny, Washington, and Westmoreland Counties.
- Coal Resources of Allegheny, Fayette, Greene, Indiana, Washington, and Westmoreland Counties - Structure contour mapping for the 7.5-minute quadrangles in Allegheny County.
- Geologic Atlas of the United States (15' maps); Amity, Beaver, Brownsville-Connellsville, Burgettstown-Carnegie, Claysville, Masontown-Uniontown, Rogersville, Sewickley, and Waynesburg Folios – Structure contour maps with a stratigraphic column, geology description, and topographic mapping from the early part of the 20th century.
- Bedrock Map for Allegheny County – Bedrock mapping for Allegheny County with stratigraphic column and tabular summary of the rock units including notable properties and sources of published information.

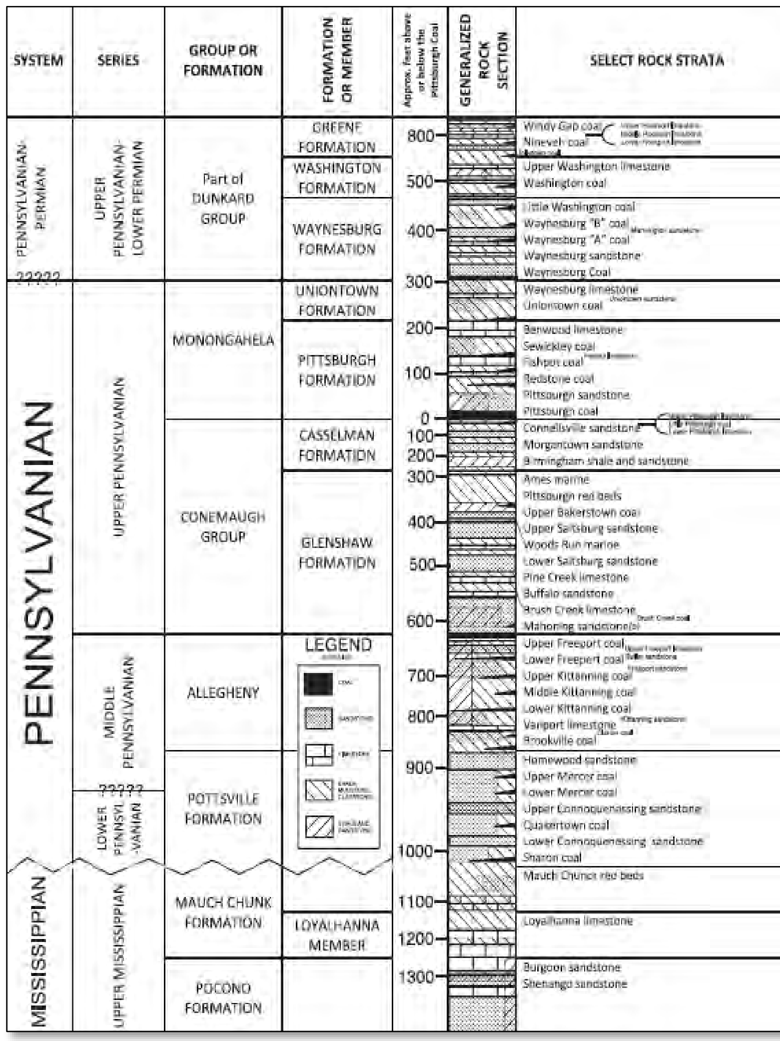


Figure 3-5 - Generalized Stratigraphic Column of the Pittsburgh Region [72]

- Bedrock Geology Mapping – DCNR web portal with links to multiple bedrock geology resources for Pennsylvania.
- Geology of the Dunkard Group - Bulletin posted by ODNR presenting a compilation of the knowledge of the Dunkard Group.

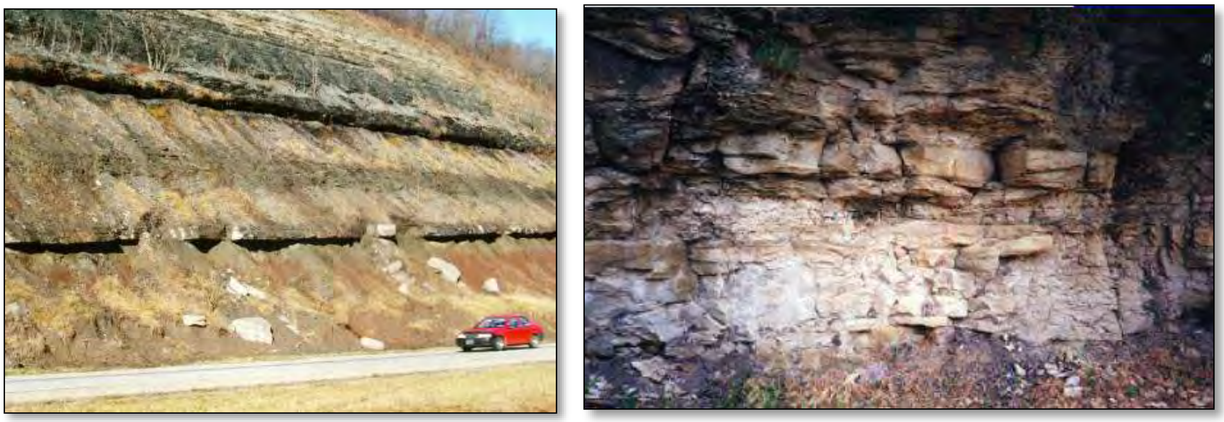


Figure 3-6 - Outcrops of the Ames Limestone (left) and the Morgantown Sandstone (right) [136]

3.5.3 Coal Maps

Resources for geologic mapping applications are listed below:

- Mine Map Atlas – Interactive map viewer that presented downloadable mappings for selected coordinates. Mapping is available to be separately downloaded or viewed on the mapping application.
- PASDA – Detailed data points related to mining operations able to be imported into ArcMap or viewed in Google Earth as a KMZ file.

3.5.4 Interactive Mapping

A comprehensive GIS-based web platform, designated Pennsylvania Geologic Data Exploration or PAGEODE, is maintained by the Pennsylvania Geological Survey. This online tool provides an interactive map to the user to query point locations and areas within Pennsylvania and return publications and mapping on geology and natural resources in Pennsylvania, including those publications and mapping about geologic hazards such as landslides and landslide susceptible areas. Geologic spatial (GIS) data may also be downloaded by the user for use and evaluation in identifying locations and areas predisposed to landslides.

3.6 ASSESSMENT OF GEOLOGIC FEATURES

Similar to the location of features representative of landslide vulnerability, comprehensive coverage of the geologic features associated with landslides in the southwestern Pennsylvania region is provided by several authors of previous notable works, including but not limited to Gray et al. [62], Myers [90], Philbrick [107], Pomeroy [109], Wagner [140], and Winters [143]. The reader is referred to these and other previous works for in-depth, comprehensive coverage of geological features impacting landslides in the southwestern Pennsylvania region. See Table 3-1 for a summary of the geologic units prone to landsliding in southwestern PA. Further discussion of those unique geologic features frequently and directly impacting landslide activity in southwestern Pennsylvania is presented in the following sections.

Geologic Formation	**Soil Units and Geologic Members Prone to Landslides**	**General Remarks**
--	Alluvial and Glacial Terrace Deposits (on Parker Strath)	Up to 80 ft thick; high plasticity
--	Upland Silt Loams	Comprised of silty loam soils and perched water tables found on hillslopes and valleys.
--	Colluvial Deposits	Soils are indicative of historic slides (i.e., unstable slopes); soils exhibit very low shear strength due to previous shearing; residual strength values shall be assigned to this material
--	Strip Mine Spoils	Soils typically end dumped and heterogenous; these soils will likely exhibit low shear strengths and perched water tables.
Dunkard Group; Washington and Waynesburg Formations	Dunkard Group	Variable claystone interbeds; known for “carpet slides”
Conemaugh Group; Casselman Formation	Pittsburgh Limestone	Includes up to nine separate limestone beds: potential water-bearing formation
	Upper Clarksburg limestone underlain by the Clarksburg Redbeds	Shaley redbeds with clayey shale interbeds
	Duquesne Coal and limestone underlain by the Grafton sandstone and deeper Schenley (Birmingham) Redbeds	Pale red to greenish claystone and shale
Conemaugh Group; Casselman and Glenshaw Formations	Unnamed Redbeds underlain by the Ames limestone and the Pittsburgh Redbeds	Marine limestone distinguishable by an abundance of marine fossils including crinoid stems between pale green and pale red interbedded claystones and shales

Table 3-1 - Summary of Landslide Prone Geologic Units

3.6.1 Adverse Soil Units

Landslide prone soils in southwestern Pennsylvania include alluvial and terrace deposits, colluvial deposits, strip mine spoils, and upland silt loams. See Table 3-2 for the list of these soils and the extent of their occurrences throughout the region. For reference the stratigraphic interval of the upland silt loam deposits has been listed in parentheticals.

Soils	**Allegheny**	**Armstrong**	**Beaver**	**Butler**	**Fayette**	**Greene**	**Indiana**	**Washington**	**Westmoreland**
Alluvial and Terrace Deposits (Pleistocene)	X				X	X		X	X
Colluvium	X	X	X	X	X	X	X	X	X
Strip Mine Spoils	X	X	X	X	X	X	X	X	X
Upland Silt Loams									
Brooke (Monongahela-Dunkard Interval)					X	X		X	
Dormont (Conemaugh-Monongahela-Dunkard)	X					X		X	
Gilpin (Allegheny-Conemaugh)	X		X	X					X
Guernsey (Conemaugh-Monongahela-Dunkard)			X		X	X	X	X	X
Library (Monongahela-Dunkard)						X		X	
Upshur (Conemaugh)	X	X	X	X	X	X	X	X	X
Westmoreland							X		X
Wharton (Allegheny-Conemaugh)					X				
Vandergrift (Conemaugh)	X	X	X	X					

Table 3-2 - Landslide Susceptible Soils, modified from Pomeroy [109]


Figure 3-7 - Typical Carpet Slide (left) and colluvial slope (right) in Greene County

3.6.1.1 Alluvial and Terrace Deposits

Occurrences of alluvial and terrace deposits contribute to landslides in the southwestern Pennsylvania region [16]. Alluvial deposits, or alluvium, refer to unconsolidated fine to coarse-grained sediment, ranging from clay and silt up to boulders (>12” diameter) in size (though often in the sand and gravel size range), with variable gradation and often rounded particles. The alluvium is present along the river and stream valleys and floodplains throughout the southwestern Pennsylvania region, very often present at the toe of the slopes forming the terrain (see Figure 3-8). Notably, in several instances throughout the region, recent alluvial deposits in valleys and floodplains may be further underlain by similar deposits of glacial origin, often referred to as glacial outwash, or in certain cases the glacial outwash deposits may be present without any overlying alluvium.

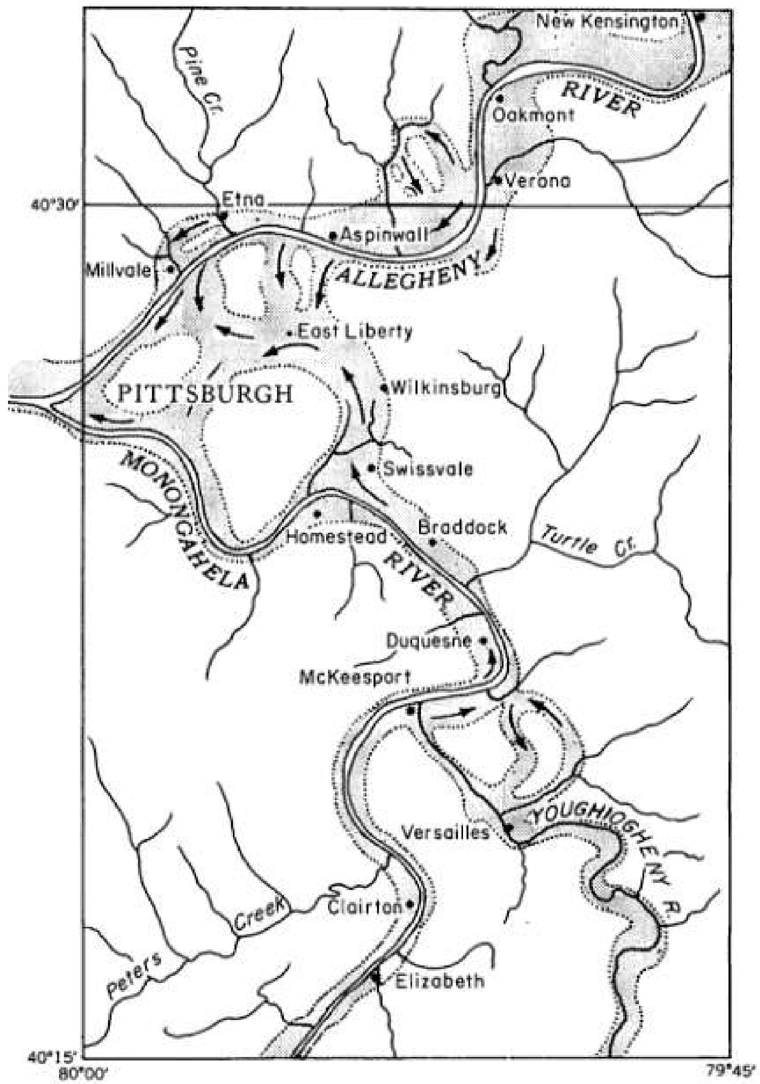


Figure 3-8 - Old Channels and high-level terraces of the lower Monongahela, Youghiogheny, and Allegheny valleys [140]

Corresponding terrace deposits are broadly regarded as alluvial-type deposits on flat benches above the elevation of the present river and stream networks and may be found throughout southwestern PA, often in abandoned channels and meanders of previous river and stream networks [62]. The terrace deposits in southwestern Pennsylvania are broadly termed the Parker Strath, consisting of glacial outwash (typically sand, gravel, and larger-sized material) along the Allegheny and Ohio River systems and the Carmichaels Formation along the Monongahela and Youghiogheny River systems, with the Carmichaels Formation notably comprised of a lacustrine clay, silt, and sand matrix containing subangular to well-rounded, cobble- to boulder-sized, typically sandstone clasts. Figure 3-9 provides a depiction of the Parker Strath terrace levels with respect to the present river levels and associated alluvial and glacial deposits in the region. Note that the elevations presented on this figure are specific to Allegheny County, and may vary in other areas of southwestern PA.

By inspection of Figure 3-9, terrace deposits often reside on the upper and intermediate portions of the slopes adjoining waterways in southwestern PA. The presence of these deposits along slopes in the area, which often consists of water-bearing unconsolidated soil materials, frequently provides conditions of low shear strength and elevated pore water pressure of the soils forming the slope, leading to potential landslide activity, especially where excavated. Correspondingly, alluvial soil deposits comprising the lower portion of slopes in floodplains and riverbeds of the region often provide negligible resistance to the stability of the soil materials on the slopes above the floodplain. In select instances, water-bearing terrace deposits with elevated hydraulic conductivity may also serve to drain water onto colluvial soils and further downslope, leading to unstable slope conditions. In these and other scenarios, alluvial soil and terrace deposits serve as a key contributing geologic feature for landslide susceptibility in southwestern PA.

The type and distribution of alluvial and glacial outwash deposits in the southwestern Pennsylvania region are available from a variety of sources with often overlapping study areas, notably in O'Neill, Jr. [94] and Wagner et al. [139], but also specifically for Washington County in Newport [92] and for Greene County in Stoner et al. [123].

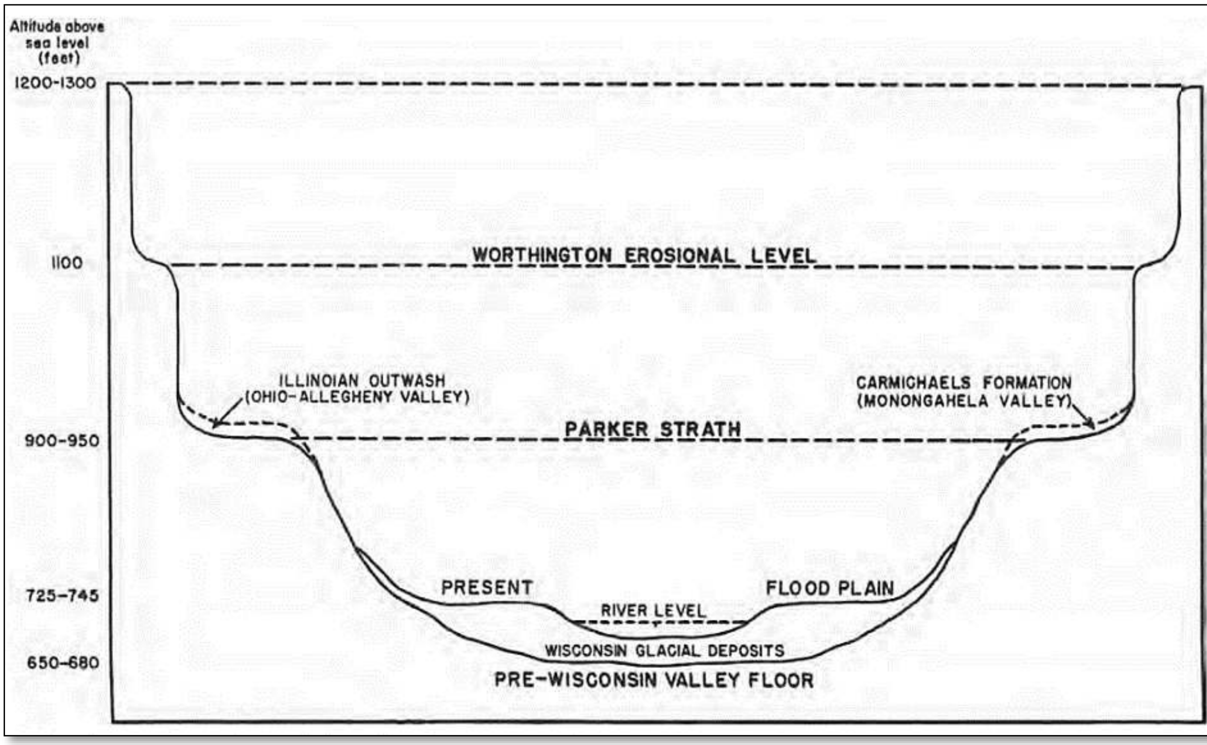


Figure 3-9 - Erosional Levels and Position of Valley Fill Deposits in Allegheny County [9]

As alluvial soils commonly occur as a surficial layer throughout the region, the areal distribution of alluvial soils may also be identified from available soils mapping (see Section 3.5.1). The reader is further referred to notable sources describing the complex geologic processes leading to the formation, distribution, and types of alluvial and terrace deposits present throughout the southwestern Pennsylvania region in Leverett [84], Piper [108], Leverett [85], Adamson et al. [9], Wagner et al. [140], Harper [73], Harper [74], and Gray et al. [62].

3.6.1.2 Colluvium

Colluvium is formed by the in-place physical and chemical weathering of bedrock into disaggregated particles followed by downslope movement of the particles under gravity into an accumulated deposit along the lower portion of slopes. Colluvial deposits generally consist of a heterogeneous mixture of fine to coarse-grained soil and rock fragments, are generally marginally stable in place, and have characteristics greatly dependent upon the nature of the source bedrock and climate conditions under which the weathering and transport have taken place [128].

Colluvium is present along slopes throughout the southwestern Pennsylvania region (sometimes more than 20 feet thick in Allegheny County [16]), having formed from the deep in-place weathering of claystone, clay shale, and other bedrock materials prevalent in the area along with downcutting of the stream and river valleys to form steep slopes along which gravity-transport and deposition have evolved as shown in Figure 3-10. Key to the characteristics of colluvium in the southwestern Pennsylvania region is the inherently low shear strength as well as the strain-softening behavior of the material [119]. As indicated in works by others summarized in Gray et al. [62], the residual (large-displacement) shear strength of claystone-derived colluvium is typically less than half of the peak (small displacement) shear strength at a given effective normal stress. The reader is referred to studies in D'Appolonia et al. [29], Gray et al. [58], Gray and Donovan [61], Hamel [66][67][68], and Hamel and Flint [69][70] for expanded reading on colluvium and the low peak and residual shear strength of claystone-derived colluvium that is common to the southwestern Pennsylvania region.

Additionally, similar to the characteristics of the parent claystone and clay shale from which they are derived, the colluvial soils are highly subject to absorption and retention of water due to infiltration, groundwater seepage, and other sources (e.g., utilities). As a result of these inherently adverse characteristics and behavior of colluvium, combined with its prevalence across the southwest Pennsylvania region, colluvium represents a significant geologic feature impacting landslide formation in the area.

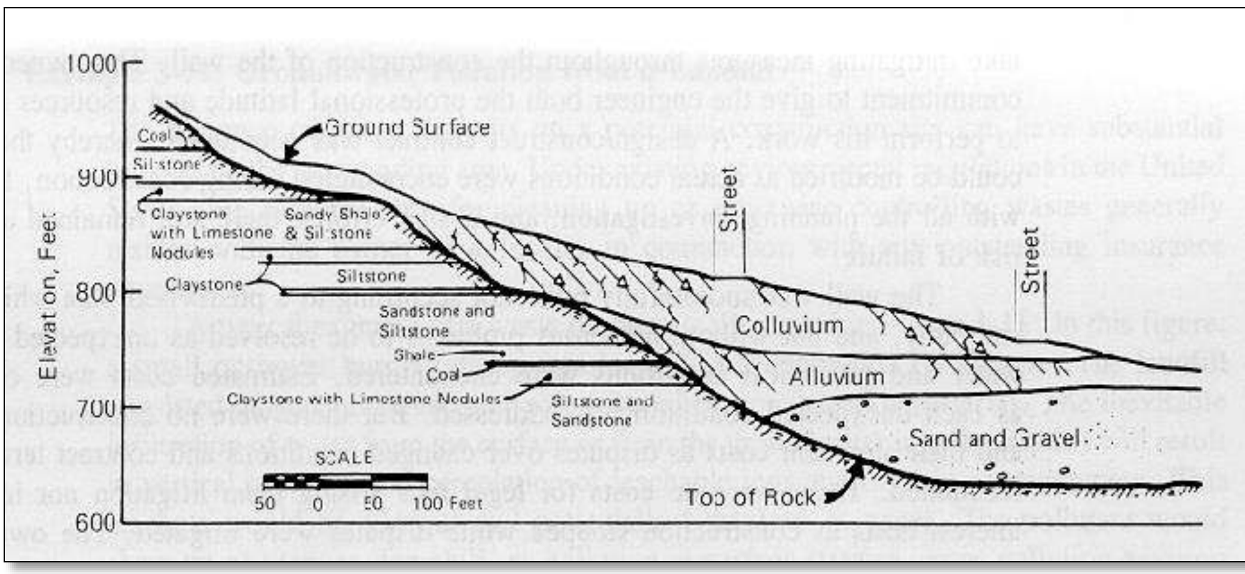


Figure 3-10 - Profile of Typical Colluvial Soil Cover of the Pittsburgh Region [29]

3.6.1.3 Upland Silt Loams

Upland soils comprised of silt loams in southwestern Pennsylvania are typically prone to instability due to percentage of silt and clays in combinations with perched water tables characteristic of the region. See Figure 3-11 through Figure 3-13 for typical depositional environments for these soils across several southern Pennsylvania counties.

3.6.1.4 Strip Mine Spoils

Strip mine spoils are often heterogenous in nature and randomly placed, thereby exhibiting typical weak shear strengths. Additionally, the mining operations likely contributed to alternating drainage patterns which could cause further instability. Landslides in strip mine spoils are typically associated with spoil banks rather than reclaimed lands; however, reclaimed land failures are not uncommon. Stability issues with strip mine spoils are most prevalent in Armstrong and Butler counties along with localized areas of Washington County [109].

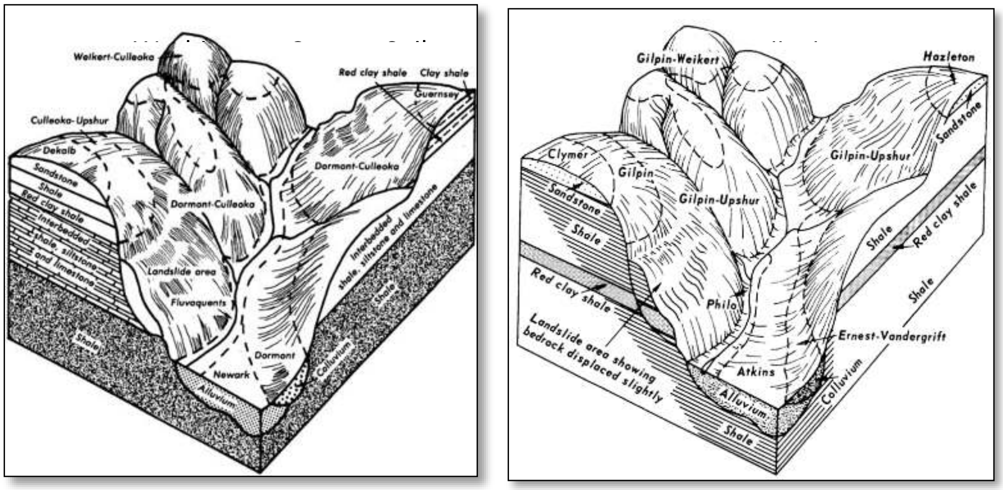


Figure 3-11 - Typical pattern of soils and underlying material in the Dormont association (left) and Gilpin association (right) (Soil Survey)

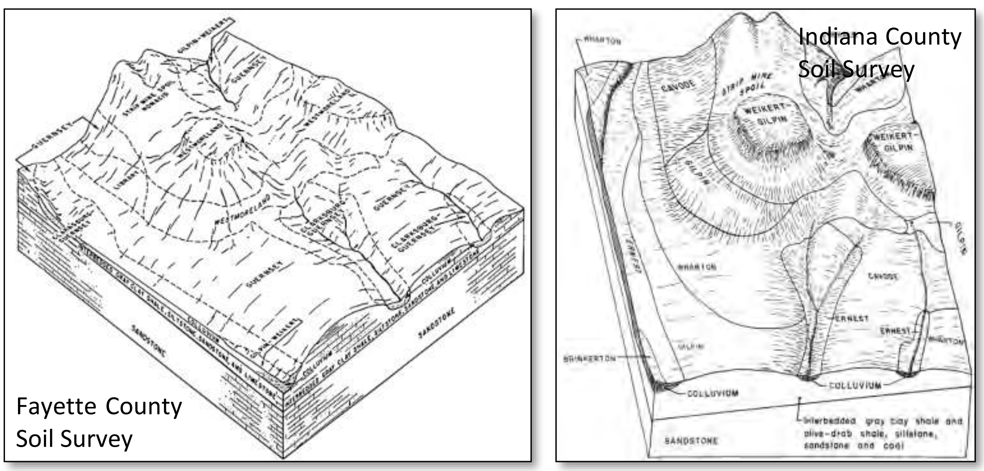


Figure 3-12 - Typical pattern of soils and underlying material in the Guernsey-Westmoreland-Clarksburg association (left) and Gilpin-Wharton-Cavode association (right) (Soil Survey)

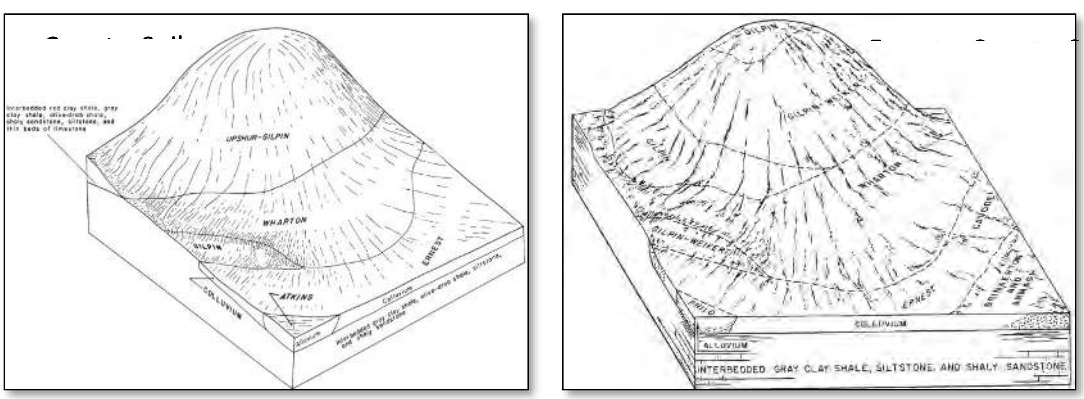


Figure 3-13 - Typical pattern of soils and underlying material in the Gilpin-Wharton-Ernest association (left) and Gilpin-Wharton-Upshur association (right) (Soil Survey)

3.6.2 Adverse Bedrock Units

As indicated in the stratigraphic column provided in Figure 3-5, bedrock stratigraphy in the southwestern Pennsylvania region is comprised of a cyclic sequence of sedimentary rock generally consisting of sandstone, siltstone, claystone, and shale, with notable seams of bituminous coal and limestone with associated underlying claystone (underclay) units. The claystone units and clay shales in the region generally exhibit inherently low shear strength and permeability characteristics, and are prone to accelerated deep weathering and disintegration under exposure to groundwater infiltration or repeated freeze-thaw cycles, volumetric expansion upon exposure to water, and retention of excess pore water pressure. As a result of these characteristics, claystone and clay shale represent a primary geologic feature contributing to landslide susceptibility where they occur within the slopes of the region.

While any claystone and clay shale may contribute to landslide vulnerability in southwestern PA, some of the most noteworthy occurrences are present in the stratigraphic sequence in the greater Pittsburgh region. The Pittsburgh Red Beds constitute an approximate 40- to 60-foot thick sequence of claystone and clay shale near the top of the Glenshaw Formation. The adverse geotechnical characteristics of the Pittsburgh Red Beds and the soils resulting from their in-place weathering or downslope movement, combined with their comparatively large thickness and prevalence along the slopes within and around the City of Pittsburgh, have resulted in a prevalence of past landslide activity and high susceptibility to landslide activity where they occur. At the base of the Casselman Formation, an additional approximate 20 to 25 feet thick sequence of Red Beds often occurs above the Ames limestone (a consistent marker bed throughout the Pittsburgh Region) bounding the top of the Pittsburgh Red Beds [7] and are sometimes colloquially referred to as the 'Unnamed Red Beds'. Where these occur, it results in an even greater thickness of adverse claystone and clay shale sequence in association with the Pittsburgh Red Beds. Similarly, a 5- to 15-foot thick sequence of landslide-susceptible claystone, referred to as the Birmingham, or Schenley, Red Beds may occur between the major sandstone units of the Morgantown sandstone and Birmingham shale and sandstone. At some locations, an unconformity in the Morgantown can lead to a much thicker section of the Schenley red beds, such as that encountered along I-376 to the west of the Pittsburgh International Airport. Due to their depositional history, the thickness of these and other claystone units in the sedimentary sequence of southwestern Pennsylvania may fluctuate over a short horizontal distance, resulting in laterally discontinuous claystone bedrock units occurring within a limited, localized area.

Claystone units immediately underlying bituminous coal and limestone units throughout the stratigraphic sequence (e.g., underclays), while typically thin (less than 5 feet), provide a distinct contribution to landslide vulnerability of slopes where they occur. In these instances, the underclays frequently act as a hydraulic barrier (aquitard) to the downward movement of groundwater through overlying rock units (overburden). Where this condition prevails within slopes, a concentration of groundwater in the immediate overlying bituminous coal or limestone unit may occur, resulting in localized perched groundwater conditions discharging laterally towards the slope face, formation of seeps and springs on the slope face, and thereby saturating the soil mantle and/or rock mass constituting the slope itself.

Claystones of the Dunkard Group, consisting of the Greene, Washington, and Waynesburg formations, represent a key geologic feature contributing to landslides in southwestern PA, and are generally limited to the areas of Washington and Greene counties, although portions of the Dunkard Group occur to a lesser extent in Allegheny, Westmoreland, and Fayette counties. The areal extent of the Dunkard Group closely coincides with the area designated as having the highest landslide susceptibility (Figure 3-1), which is indicative of its propensity for landslide activity in the distinctly rugged, steep-sloped, and exaggerated relief terrain in Greene and Washington counties. The Dunkard Group generally contains similar sedimentary bedrock unit types as other areas in southwestern PA, but with an increased propensity for interbeds of sandstone and claystone and clay shale that are subject to deep in-place weathering and formation of colluvium, and fewer, less persistent bituminous coal seams. The reader is referred to Newport [92], Piper [108], Stoner et al. [123], and Wagner et al. [139] for detailed mapping and discussion of Dunkard Group occurrences in southwestern PA.

CHAPTER 4

Landslide Investigation

4.1 CLARIFICATION OF INVESTIGATION OBJECTIVES

Landslide investigation covers a wide range of topics. But at its core, the investigation of a landslide starts first with a clear definition of the purpose and objective of the investigation. The investigation can then be adapted to meet site-specific needs ranging from a simple repair to a detailed investigation of a complex landslide that must accommodate multiple design constraints. A clear definition of the purpose of the investigation and what questions need answered should be identified. The investigation scope, the areal limit of the investigation, and the required depth of the subsurface investigation are aspects that need to be identified; if not, the investigation may be inconclusive, cause delay, and or escalate cost to complete.

Landslide investigations are typically classified into one of two approaches:

1) Pro-active approach to reduce and/or mitigate the risk of potential landslide activity.
2) Reactive approach to arrest slope movement at an existing active or former landslide.

The pro-active approach focuses on the identification of facets to mitigate against potential future movement. Landslide-prone areas usually show indicator(s) of past movement that need to be identified by reviewing existing data, which is typically supplemented by site-specific site reconnaissance and investigation, and then is followed by engineering assessment to support design. Results from the investigation will help identify what mitigation measures should be taken to minimize the risk of future movement or suggest an alternate route, location, or feature that is less prone to landslide activity.

The reactive approach focuses on the characterization of existing conditions that may have led to landslide movement, desktop study, site-specific reconnaissance and investigation, and then is followed by engineering assessment to support mitigation design. Once a landslide has occurred, the investigation is undertaken to diagnose the factors affecting the movements, and support back-analysis to calibrate solutions to match existing conditions. Often these investigations are urgent due to the threat to property or public safety in which corrective measures need to be identified to arrest or minimize further movement.

It is important for the investigation to recognize actual and/or potential slope movement and identify the type and cause(s) of movement. A major component of investigation includes the identification of site subsurface characterization to allow the prediction of a material's strength, deformation, and permeability properties in response to changes over time due to stress or other environmental conditions [128]. A successful investigation is critical as it produces the information that serves as the basis for analysis and design.

4.2 STEPS TO INVESTIGATE

Five primary steps are involved to complete a landslide investigation:

1) Initial Site Visit and Scoping – identify the purpose and objective(s) of the landslide investigation.
2) Desktop Study – collect and interpret existing data and available geologic information.
3) Site Reconnaissance – gather data on the current site conditions; make site observations critical to the evaluation of potential instability of vulnerable slopes.
4) Subsurface Investigation – based on the interpretation of the desktop study and site reconnaissance efforts, collect soils and rock data to provide site-specific subsurface information and obtain samples.
5) Derivation of Subsurface Sections – generate a profile to present all the pertinent data gathered during the landslide investigation.

The results of each step should further be integrated with the design process to identify the unknowns that should be discovered in the next step [32]. The goal of the investigation process is to identify the slope movement trigger(s), potential failure plane(s), and potential design constraints; the sum of all the steps performed should tell a story that will serve as the basis of the decision-making process performed during analysis and design.

Since no two landslides are the same, not every field investigation will be the same; however, these steps should be the foundation for planning. A general checklist of items to be considered when planning a landslide investigation including the source of information and the details to be considered are presented in Table 4-1.

4.3 INITIAL SITE VISIT AND SCOPING

The first step is to conduct an initial site visit. This initial site visit is critical to help clarify and affirm the purpose and objective(s) of the landslide investigation. The initial scoping visit should focus on high-level assessments of the site and the implications of the observed slope movement. The main objectives of this visit should include a preliminary assessment of the characteristics of the slide mass, water flow/patterns, and the proximity of infrastructure. See Appendix A.2 for an example abbreviated site visit checklist that can advise the practitioner of pertinent information to be collected. This is the time to mold the scope of the landslide investigation and assess the urgency of the slope movement.

The areal extent of landslide investigation is controlled by the size of the project and the extent of topographic and geologic features that are involved in the landslide activity [128]. It is much easier to define the area of a project site after a landslide has occurred compared to a location that has signs indicating potential movement. Best practice when defining an area for the landslide investigation is to increase the size by two or three times wider and longer than the suspected area. Known areas of major slope angle changes, groundwater present, or geologic structures that align with the area of instability should be included in the overall investigation area.

Keeping and maintaining good records, as well as documented written correspondence and photos during this time frame is important. This may be especially beneficial when the slide area is known to affect adjacent landowners or impact a large population.

□ Topography	*CONTOUR MAPPING, HILLSHADE MAPPING, HISTORIC TOPOS*	• Anomolous patterns such as bulges and scarps • Potential drainage patterns • Surface elevations at the site to correlate with the stratigraohic column • Changes in topography over time
□ Weather	*METEOROLOGICAL RECORDS*	• Precipitation rates (annual, daily, monthly) • Snow melt • Extreme temperature changes (rapid freeze/thaw)
□ Geology	*BEDROCK GEOLOGY & STRUCTURE CONTOUR MAPPING, STRATIGRAPHIC COLUMN*	• Stratification • Bedrock dip in relation to the slope • Proximity to landslide prone geologic and soil units (see Ch. 3) • Proximty to water-bearing formations (see Ch. 3) • Observation of outcrops in the vicinity of the site
□ Movement Characteristics	*SITE RECONNAISANCE, SUBSURFACE INVESTIGATION, AERIAL PHOTOGRAPHY, LANDSLIDE MAPPING*	• Rate of Movement • Feature measurements • Evidence of past movement or adjacent signs of movement • Documented historic landslides • Correlate movement to previous observations and data collected • Location of failure plane
□ Water	*SITE RECON, SUBSURFACE INVESTIGATION*	• Perched water and relation to geologic formations • Seasonal fluctuations • Piezometric levels within slope • Seeps, springs, wet areas, vegetation unique to saturated areas • Adjacent waterways
□ Site Development	*SITE RECONNAISANCE*	• Site constraints such as utilities, roadway, or structures adjacent to the slide mass • Evidence of previous earthwork causing deficient stability • Failing infrastructure ncluding structures, pavement, and drainage features • Manmade influence to groundwater patterns and/or drainage

Table 4-1 - Checklist for Planning a Landslide Investigation, modified from Sowers [121]

4.4 PRELIMINARY DESKTOP STUDY

The next step in the investigation process is to research, collect and document existing data and available information. A thorough desktop study will serve to inform the practitioner of the site history and provide a foundation for the site reconnaissance and planning of the subsurface investigation.

4.4.1 Available Mapping

There are numerous sources of existing data available that include but are not limited to geologic, landslide, mining, and topographic information. See Chapter 3 for a detailed discussion of the mapping resources to be considered as part of a complete desktop study; the discussion presented in this section will focus on additional resources and the application of the data obtained from the available mapping for a landslide investigation.

4.4.1.1 Surficial and Bedrock Geology

Determination of where the project site is geologically can alert the practitioner to geologic units and factors that could be contributing to low shear strength at the project site. A detailed assessment of the potential geologic triggers can help determine potential modes of failure and serve as a valuable tool when planning the site reconnaissance and subsurface investigations. See Chapter 3 for a summary table of the soils and geologic members prone to landslides in southwestern Pennsylvania.

In addition to the landslide-prone units, the practitioner should also be sensitive to the water-bearing formations in the region that may be contributing to elevated pressure head or perched groundwater conditions at the project site. Water-bearing units in southwestern Pennsylvania are listed below; these units generally consist of highly fractured geologic units and limestone members.

- Coal seams.
- Limestone members.
- Fractured sandstone members, such as the Waynesburg, Connellsville, Morgantown, Birmingham, and Saltsburg sandstones.

Figure 4-1 – Mitigation effort for a rotational slide caused by excess pore water pressure at the toe of slope; toe key elevation, as shown, intercepted the water bearing Uniontown limestone member

4.4.1.2 Landslide Susceptibility

Landslide susceptibility mapping can alert the practitioner to areas with documented concerns regarding long-term stability. This mapping may also inform an assessment of whether the landslide is localized (likely manmade or surficial trigger) or if the landslide is part of a larger regional instability (likely geologic trigger).

4.4.1.3 Aerial Orthophoto Interpretation

Aerial photography is a resource to gain a three-dimensional perspective of the terrain to help identify landslides, changes in topography, and the history of site development. Not only can these mapping and aerial photos help identify terrain features indicative of landslide activity, but also human activity that may result in increased landslide susceptibility.

Recent and past aerial photographs of an area should be reviewed since older slides may not be apparent in more recent photographs. Features visible on aerial photographs can help identify landslide type and develop a reasonable assessment of overburden characteristics which ultimately provide estimates for landslide hazards.

4.4.1.4 Topographic and Hillshade Mapping

Topographic past landslide features generally include:

- Concave (amphitheater) contours as zones of displacement.
- Convex (downslope/ lobate nose) contours as zones of accumulation.
- Flat wide bench areas (widely spaced contours) as part of former head scarps.
- Crenulated contours (shallow, rounded finely notched and scalloped projections) as signs of past overlapping superimposed landslides.

See Figure 4-2 and Figure 4-3 for examples on interpreting topographic mapping.

Figure 4-2 - Cut slope constructed within a topographically anomalous area resulted in a slope failure

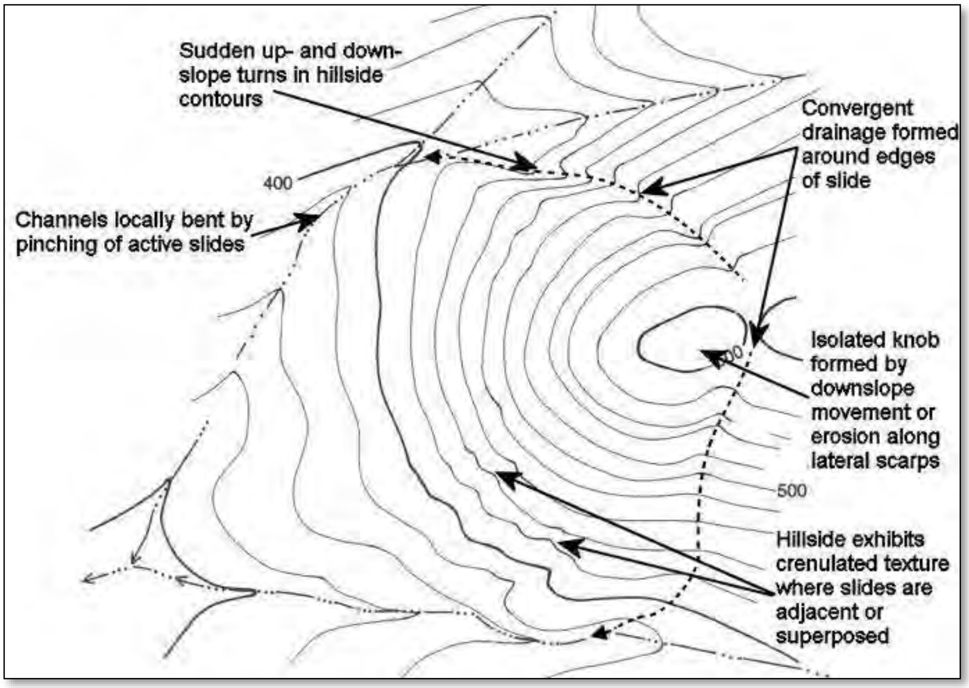


Figure 4-3 - Use of drainage and topographic keys to recognize anomalous site characteristics typical of landslides

Comparing historic topographic maps, identification of changes in topography is useful since excess stockpiles near the slope crest, poorly constructed fill, and poorly executed cuts that compromise the toe of slope or over-steepen the slope are all major contributors to slope movement.

Hillshade mapping is especially useful to identify previous slide features including the head scarp and zone of accumulation as the topography is more easily discernable through the three-dimensional imaging. Drainage patterns within the slope can also be more easily identified using hillshade maps.

4.4.2 Prior Land Use and Construction Records

Prior land use and construction records are valuable sources of information to frame the subsurface and environmental setting. These records may inform the practitioner of any previous earthwork at the site and potential infrastructure. Mining records should also be considered when evaluating prior land use in order to identify any potential subsidence issues or past strip-mining operations which may negatively impact global stability at the site.

4.4.3 Meteorological Records

In the southwestern Pennsylvania region, water is typically at least one of the driving factors in slope movement. Perched groundwater tables within the regional clayey soils can result in a pressure head and/or decreased shear strengths. Additionally, the spring wet season in the region also typically coincides with large temperature variations leading to continuous freeze/thaw conditions. During freezing temperature, the ice expands and loosens the soils which are then saturated during the thaw cycle, this can create weak planes within the soil mass.

Meteorological records are useful to alert the practitioner of weather patterns leading up to the slope movement for example increased rainfall or freeze/thaw conditions.

4.4.4 Emerging Technology

4.4.4.1 Interferometric Synthetic Aperture Rader (InSAR)

Satellite, infrared, radar Interferometric Synthetic Aperture Rader, or InSAR, uses active sensors to record a pulse of energy emitted from a satellite to the ground, back to the sensor. This process of bouncing signals from a radar satellite off the ground creates a digital elevation model (DEM) that shows ground terrain. An interferogram map is a map that merges two images of the same place, taken at different times. The merging of the two images shows ground displacement that has occurred over time and would indicate the movement of an area, see Chapter 6 for further discussion.

4.4.4.2 Hi-Resolution Near-Infrared (NIR) 4-band Satellite Imagery

More often than not, landslides occur more frequently in slope areas where groundwater seeps out of the ground surface. Multispectral color infrared (4-band) red/green/blue/NIR satellite imagery is typically available from commercial sources with 6-inch ground sample distance (GSD) pixel spacing. Data can be post-processed to compute a Normalized Difference Vegetation Index (NDVI) to accentuate surface water features and free-water seepage at the ground surface. NDVI is a ratio that equals (NIR +RED) / (NIR – RED). NDVI values typically range from -1.0 to +1.0. Negative NDVI values are typically indicative of free water, which exhibits almost no reflectance of NIR and low reflectance of red light. Positive NDVI ratios are indicative of healthy vegetation, which tend to have a strong NIR reflectance and low reflectance of RED light. NDVI ratios are then density-sliced to target areas where the probability of groundwater seepage is more prevalent.

4.5 DETAILED SITE RECONNAISSANCE

A detailed site reconnaissance is important to understand the geotechnical, topographic, and geological features of the site [89]. Not all signs of slope movement can be identified by using maps or photographs; additionally, past maps and photographs might not accurately depict current site conditions. Thus, detailed site reconnaissance is needed to verify or detect landslide features, and to critically evaluate the potential instability of vulnerable slopes.

4.5.1 Field Reconnaissance

Landslide types, processes, and triggering mechanisms are explained in Chapter 2. Detailed notes and sketches of features observed on the ground should be made to help classify the age and type of movement. A few key features that may indicate landslide movement mechanisms or triggers to previous movement include, but are not limited to:

- Springs, seeps, or wet areas that were previously dry.
- Damaged or deficient drainage infrastructure (see Figure 4-4).
- Broken underground utilities.
- Stress cracks.
- Sidewalks or slabs pulling away from structures.
- Unusual bulges, or elevation changes in the ground.
- Tilting telephone poles, trees, retaining walls, or fences.
- Sunken or down-dropped roads or paths.

For example, cracks in pavement or foundations give evidence to stress produced by the movement of a landslide. Since landslide features become modified with age, older landslides that have been stable for thousands of years are less defined than active landslides [128]. In addition, with landslides constantly changing, the correlation between on-the-ground observations and maps may not perfectly line up. These differences should be expected and are particularly useful in understanding landslide deformation.

See Appendix A.1 for an example slope movement field visit checklist which can serve as a template of the pertinent information to be collected. It is helpful for the desktop study be completed before the detailed field reconnaissance so that the information collected during the desktop study may be used inform the practitioner of targeted features to include in the site assessment.

After completion of the detailed site reconnaissance, the practitioner should have a preliminary hypothesis of the type of movement, depth to rupture plane, possible triggers, and viable mitigation strategies. This information is key to planning the next step of the landslide investigation, the subsurface investigation.

Figure 4-4 - Landslide triggered by saturated soils and erosion gullies; drainage pipe concentrating water on the slope was observed during reconnaissance

4.5.1.1 Determination of Physical Features

Field evaluation of site physical features can help the practitioner assess likely triggers at the site. Physical features at the site pertinent to slope stability include:

- Surface features such as bulges, or tension cracks that can indicate possible historic movement and the potential presence of colluvium.
- Observation of human influence that can negatively affect slope stability such as earthwork activity, surcharge loading, lake dredging or drawdown operations, undercutting, infrastructure loading, changes to drainage patterns, mining, etc.
- Evaluation of natural topography and outcrops (if visible).
- Drainage patterns and flows through the site.

It can be helpful for the practitioner to record observed features on the orthophotos collected during site reconnaissance to provide a base map for site sketches, see Figure 4-5.

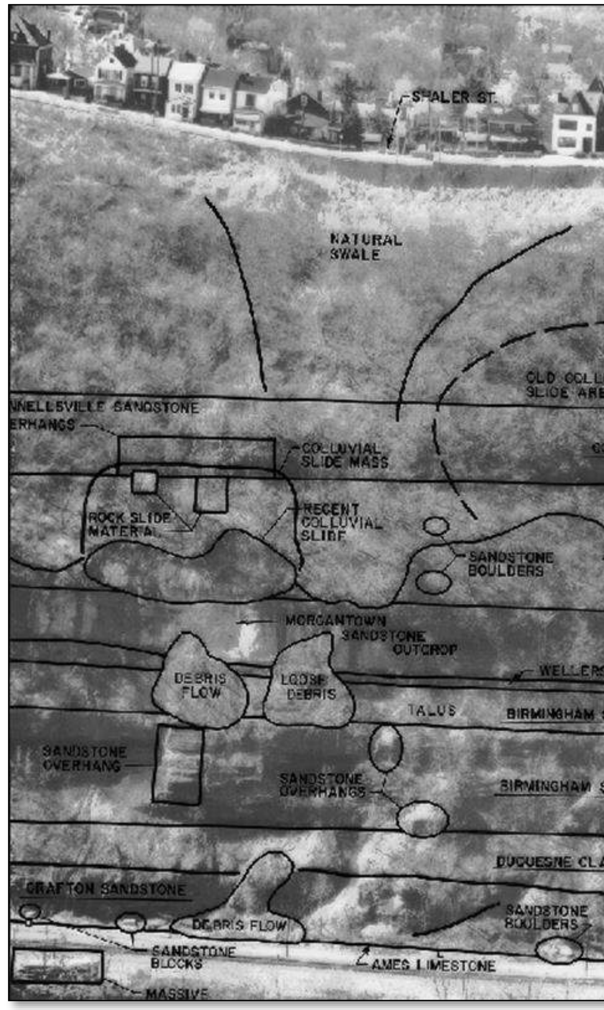


Figure 4-5 - Site reconnaissance notes overlain on aerial photo

4.5.1.2 Evaluation of the Extent of Slope Movement

Field evaluation of the slope movement can help the practitioner continue to refine their hypothesis with the suspected mode (type) of failure and the volume of displaced material. At a minimum, the measurement to be recorded should include the length of the slide mass (zone of depletion and zone of accumulation), the width of the crown, the height of the head scarp, and the estimated depth to surface rupture. These measurements will serve as a baseline to assess future movement and rate of movement, as well as provided an approximate estimate of the volume of displaced material. This data will also serve to inform the subsurface investigation and the potential mitigation strategies.

Complete evaluation of the slope movement not only includes evaluation of landslide type and measurement of features but also potential effects of slope movement on the adjacent infrastructure. For example, can further slope movement cause damage or compromise adjacent parcel(s), or is there potential for disruption of sanitary sewer service that can threaten public safety, health, and welfare. These evaluations can directly impact actions to address public safety and inform the urgency and timeline of the landslide mitigation.

4.5.1.3 Evaluation of Site Constraints

The landslide investigation needs to consider the proximity of pertinent physical features and infrastructure. This is particularly important concerning proximity to sensitive infrastructure, both above and below ground. For instance, it may or may not be possible to relocate underground and/or overhead utilities. Proximity to environmentally sensitive areas, like wetland and high-quality waterways, will also require special consideration to avoid adverse impact.

In addition to physical and infrastructure-related site constraints, topography and accessibility at the site may also play a role. The practitioner should evaluate the site concerning the accessibility of equipment or the practicality of construction methods for the project site.

The site constraints will be a major factor when assessing available mitigation strategies; accessibility, available area of disturbance, and vulnerability of infrastructure to further movement are key observations that are needed to assess viable mitigation strategies.

4.5.2 Emerging Technology

4.5.2.1 Light Detection and Ranging (LiDAR)

LiDAR technology is used to collect data points at a high density to process and create a three-dimensional (3D) rendering of the ground surface (i.e., digital elevation model, DEM). LiDAR survey can be conducted using three methods that allow for flexibility where access is limited; static LiDAR (stationary tripod), mobile LiDAR (mounted on a vehicle or all-terrain vehicle), and aerial LiDAR (mounted on an airplane or helicopter).

LiDAR accuracies are determined by the stability of the platform upon which the LiDAR apparatus is mounted (i.e., method) and the due diligence of the survey team that is engaged to collect the LiDAR data (i.e., point cloud). Tripod-mounted and mobile-based LiDAR can usually be collected to an accuracy of about 0.25 inches. Verification of the accuracy of the data should be done by using the same methods as stated in the ASPRS Positional Accuracy Standards for Digital Geospatial Data.

Collection of topographic data through LiDAR can be performed more efficiently than conventional survey considering:

- The rate of return of the collection (millions of data points per second).
- The different platforms that can be used to collect data.
- Emerging technology that facilitates improved penetration of LiDAR scans through vegetation canopies.

The following are some of the advantages of using LiDAR compared to conventional data collection methods (field measurement or survey):

- A large data set can be collected within a short period; many surveys using LiDAR can be completed in the field in a matter of hours as compared to days for a conventional survey; however, time in the office is needed to process digital data that was obtained from the LiDAR scans to prepare mapping in a useable form. For smaller landslide areas (e.g., less than 5 acres), LiDAR data that is received from the field can typically be processed in 1 or 2 days in the office.
- Simultaneous collection of LiDAR data and imagery is available and can be linked via georeferencing; which could allow for a pseudo "field survey" from the computer.
- There is a wide variety of deliverables and applications available for the data collected. Of particular interest to a landslide investigation, is topographic relief which could help identify the slide mass and gather measurements to inform the subsurface investigation and subsequent geotechnical analysis.
- Data can be used to develop final plan sets.
- Safety hazards of employees are reduced by eliminating the amount of field survey needed near potentially unstable slope areas.

Although LiDAR does provide a host of advantages to site reconnaissance efforts, some considerations need to be made including the cost of using the application and the likely need to engage a specialty firm that provides these services.

4.5.2.2 Unmanned Aircraft Systems (UAS)

UAS, also known as drones or unmanned aerial vehicles (UAV), can collect imagery and datasets for landslide investigations. UAS make it possible to perform time-effective and potentially cost-effective digital photogrammetric surveys to obtain high-definition point clouds and orthophotos using Structure from Motion (SfM) software using image triangulation and bundle adjustment to reconstruct an accurate 3D representation of the ground surface [117]. These data sets allow for detailed geomorphological mapping and the ability to obtain quantitative measurements of surface deformations [37]. The extended aerial view of the project site (see Figure 4-6) also allows the practitioner to view the slide mass from a global perspective which can be useful when identifying the extent of the mitigation and making known evidence of possible large-scale slope movement.

Figure 4-6 - 3D Ortho-Photos projected over the DTM generated from the point cloud [37]

4.6 SUBSURFACE INVESTIGATION

A site-specific subsurface investigation is completed to characterize existing soil, rock, and groundwater conditions. A subsurface investigation typically commences with the development of an exploration plan, which defines the purpose, type, number, location, and depth of the proposed exploration. The exploration plan should be purpose-driven and executed to further refine the hypothesis derived during the desktop study and field reconnaissance. The practitioner should have a general notion of the areal extent of the slide mass, potential remediation methods, and depth of failure plane to determine boring locations, termination depths, and other pertinent data to be collected for analysis and design.

Both direct and indirect methods of exploration are available. Direct methods to investigate landslides typically involve test borings and test pits, which include physical sampling. Direct methods are particularly useful to identify thin weak layers that may be present, which may not be detected by other means. Indirect methods, such as ground penetrating radar and geophysical methods, are often used to fill in gaps between representative borings to characterize subsurface conditions such as delineating variation in lithology (e.g., an irregular top of rock surface) and variation in moisture content within the landslide mass itself. The types of investigation discussed in this chapter include geophysical investigations, test boring and test pits, and in-situ testing.

4.6.1 Test Borings and Test Pits

Test borings and test pits provide detailed information about the type and condition of subsurface conditions encountered at the location(s) investigated. Within the southwestern Pennsylvania geologic region, the Pennsylvania Department of Transportation Publication 222 – Geotechnical Investigation Manual [101] is referenced frequently to specify subsurface investigation procedures such as completing test borings and test pits, installing monitoring instrumentation within the borehole, and preparing boring logs. Additionally, the Subsurface Investigations – Geotechnical Site Characterization Reference Manual by the U.S. Department of Transportation Federal Highway Administration (FHWA) can be used as a supplemental reference. This manual provides detail about subsurface investigation planning, drilling and sampling methods and equipment, and boring log preparation.

The desktop study, field observations, extent of the landslide, and client specific requirements will dictate the extent of subsurface investigation needed. For example, for localized surficial slides where a clear hypothesis has been formed prior to the subsurface investigation, test pits through the slide mass may be adequate. For other cases, such as large slope failures where multiple defining factors of the slope movement are still undetermined, a large-scale test boring investigation may be warranted.

The depth of subsurface investigation needs to be based on site-specific conditions and needs. Test boring depths should be planned to a depth sufficient to extend past material that has moved or previously moved and penetrate into underlying stable material. These depths may change throughout the investigation based on field observations and improved understanding of the site conditions. An investigation should allow for flexibility in case initial data suggests deeper (or shallower) slope movement within the area. Typical layouts for boring investigations would include borings at the crown, at the toe and within the slide mass where accessible. Consideration should be made to potential subsurface section locations when determining the boring layout. See Figure 4-7 for a sample boring layout for an investigation at an active landslide; see Chapter 6 for instrumentation details corresponding to the same project site.

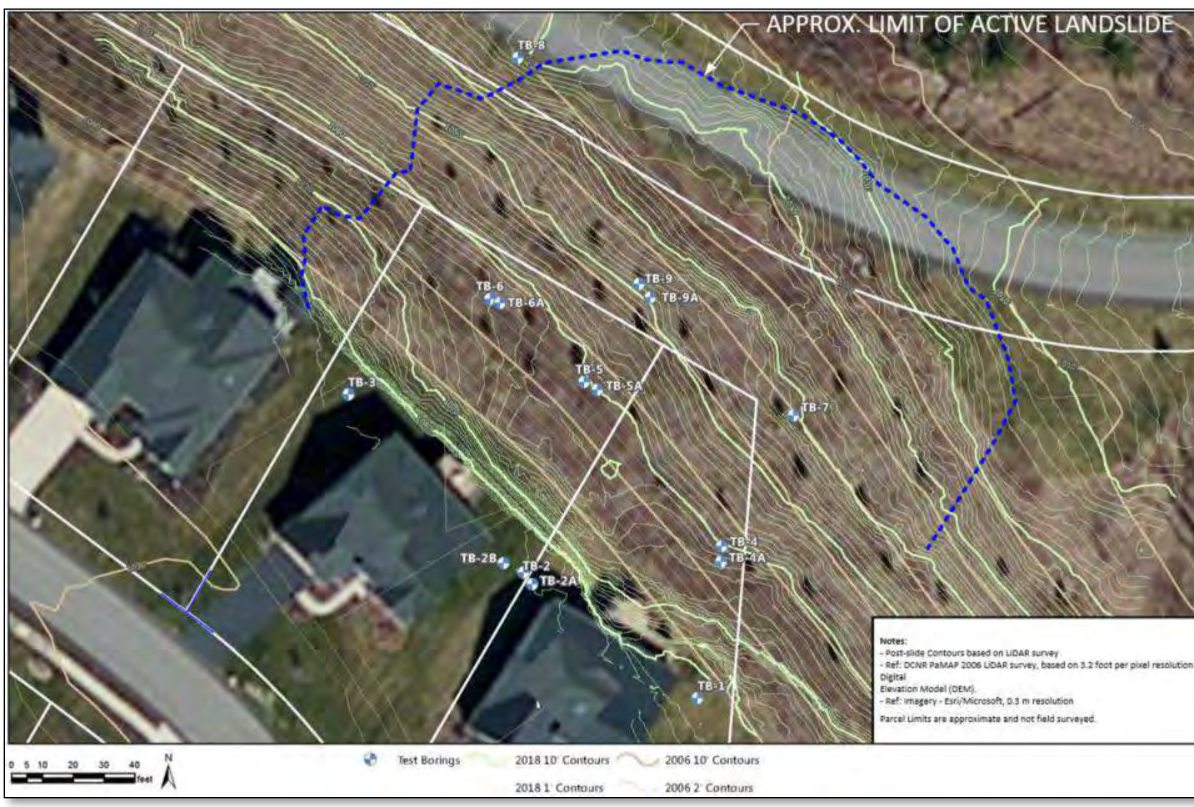


Figure 4-7 - Sample boring plan including offset borings for undisturbed sample collected as well as subsequent inclinometer and piezometers installations

4.6.1.1 Test Pits

Test pits may be performed as the sole extent of the subsurface investigation or as a supplement to collected test boring data. Test pits are useful to give the practitioner a holistic view of the soil profile and can be especially helpful in determining zones of perched water. Test pits are typically only applicable to surficial slides as the reach of the equipment is limited (typically maximum of 10 feet below ground surface) and excavations cannot penetrate hard soil or rock layers.

The practitioner should use caution when performing test pit activities within an active slide area. In order to minimize disturbance, test pits for landslide investigation may be conducted as slot excavations parallel to the slope movement to minimize overall disturbance to the slope (Figure 4-8). Additionally, the practitioner may avoid any perpendicular excavation or excessive excavation at the toe of slope which may cause further slope stability issues.

Figure 4-8 - Test pit investigation performed to confirm presence of limestone bed and wet clay soils at toe of slope identified in the desktop study; bag samples were collected for laboratory testing

4.6.1.2 Test Borings

Continuous sampling is recommended for landslide investigations since the practitioner can identify the presence of localized weak seams that otherwise may have been overlooked. Representative rock core samples are typically obtained to define the depth, quality, and type of bedrock present, including but not limited to characterization of rock discontinuities, degree of brokenness and weathering, and identification of possible slickensides. Both double and triple tube core barrels with diamond impregnated and carbide tip bits are commonly used to obtain rock core samples.

Landslide investigation typically requires both disturbed and relatively undisturbed soil sampling. Disturbed sampling methods are useful to log the soils and obtain samples for index testing. Undisturbed sampling is needed to obtain samples for strength testing, which is generally critical to a landslide investigation. The representative samples are typically retrieved and stored for subsequent classification and testing in the laboratory. Typically, in the southwestern Pennsylvania region test borings are performed using hollow stem augers within the soil strata; sampling is then advanced through the augers with a split barrel sampler to obtain disturbed samples and conduct Standard Penetration Tests (SPTs) (Figure 4-9) or a Shelby tube to obtain undisturbed samples. For rock sampling, NQ/NX core barrels advanced with water or air rotary methods are typically used.

Water level readings are typically recorded during the boring investigation to assess groundwater conditions. It is typical practice for the practitioner record water level readings at the completion of drilling (0-hour) and a day after the completion of drilling (24-hour), when possible. Where rock coring is being performed, it is also advantageous to record the water level prior to rock coring since the core water used can oftentimes interfere with the water levels at the 0-hour and 24-hour readings. When practical, the practitioner should consider installation of piezometers or extended water level readings (after 24-hour) to more accurately assess the steady state groundwater conditions.

Boreholes can also be used to install monitoring equipment such as piezometers and inclinometers for subsequent monitoring without the need for a separate mobilization of equipment. Therefore, before the commencement of a subsurface investigation, it would be advantageous for the practitioner to determine the preferred monitoring equipment and data needed for analysis and design.

Figure 4-9 - Test boring investigation using hollow stem augers and split spoons

4.6.2 Geophysical Investigation

Three types of active geophysical methods that are used to investigate landslides include electrical resistivity, seismic refraction, and ground penetrating radar. Geophysical methods and their applications and limitations are briefly explored in Table 4-2.

Electrical resistivity measurements can be used to detect changes in lithology, zones of saturation, and depth to the groundwater table [75]. Geophysical surveys are particularly useful to correlate information among widely spaced exploration areas and greatly reduce the cost and time that would normally be associated with large-scale drilling programs [128].

Seismic refraction surveys use shock waves to propagate through the body or surface of the earth to a detection point; when encountering different materials, the waves refract and reflect which helps identify the depth to different strata [128]. Since colluvium or landslide debris is typically less dense, problematic areas can be identified where there are large density contrasts [128].

Lastly, ground penetrating radar surveys are useful in creating a surface profile by constantly emitting and receiving signals along the ground surface.

Surface-Based Geophysical Methods

Type of Survey	Applications	Limitations
Electrical and electromagnetic		
Electrical resistivity	Locates boundaries between clean granular and clay strata, groundwater table, and soil-rock interface	Difficult to interpret and subject to correctness of the hypothesized subsurface conditions; does not provide engineering strength properties
Electromagnetic conductivity profiling	Locates boundaries between clean granular and clay strata; groundwater table, and rock-mass quality; offers even more rapid reconnaissance than electrical resistivity	Difficult to interpret and subject to correctness of hypothesized subsurface conditions; does not provide engineering strength properties
Seismic		
Seismic refraction profiling	Determines depths to strata and their characteristic seismic velocities	May be unreliable unless strata are thicker than a minimum thickness, velocities increase with depth, and boundaries are regular. Information is indirect and represents average values
Direct seismic (uphole, downhole, and crosshole surveys)	Obtains velocities for particular strata, their dynamic properties, and rock-mass quality	Data are indirect and represent averages; may be affected by mass characteristics
Microgravity	Extremely precise; locates small volumes of low-density materials utilizing very sensitive instruments	Use of expensive and sensitive instruments in rugged terrain typical of many landslides may be impractical; requires precise leveling and elevation data; results must be corrected for local topographic features; requires detailed information on topography and material variations; not recommended for most landslide investigations
Ground-penetrating radar	Provides a subsurface profile; locates buried objects (such as utility lines), boulders, and soil-bedrock interface	Has limited penetration in clay materials

Table 4-2 - Surface-Based Geophysical Methods [128]

4.7 SUBSURFACE SECTION(S)

Subsurface section(s) are used to present a graphic image that present a comprehensive understanding of the relevant data collected during the landslide investigation. Although typically shown as a two-dimensional (2D) image that is drawn to scale, subsurface sections are sometimes drawn as an oblique projection to depict subsurface conditions in three dimensions. Subsurface sections provide a means by which to convey an interpretation of subsurface conditions and highlight relevant information obtained. Multiple 2D subsurface sections are typically needed to illustrate the subsurface setting.

It is important to develop subsurface sections that a) are succinct to characterize subsurface conditions, b) depict key site constraints, and c) are directly relevant to the purpose, objective, and conclusions of the landslide investigation. Subsurface sections can and should be used to highlight key information to characterize subsurface conditions including:

- Correlations of bedrock encountered to the stratigraphic column.
- Presence of colluvium and/or high plasticity clays.

- Approximate breaks between soil and rock strata between borings.
- Presence of weak geologic units or coal beds.
- Design water table.
- Perched water and/or wet soils.
- Evidence of slickensides.

No one style or format is better than another as to how one should prepare a subsurface section. However, the subsurface section should be cut generally perpendicular to the contour lines so that the slope of the ground surface is accurately depicted, and it is preferred the sections be drawn to scale both horizontally and vertically. Based on experience, a horizontal to vertical scale ratio of no more than five to one (5H:1V) is preferred to prepare subsurface sections where detailed information needs to be presented. If needed, match lines can and have been used to portray subsurface sections on multiple pages (Figure 4-10).

Occasionally subsurface data is graphically presented to scale vertically with no apparent horizontal scale (e.g., fence diagram) (Figure 4-11). At sites with flat terrain, fence diagrams are often adequate. However, landslides typically involve sloping terrain, fence diagrams should supplement the subsurface sections (e.g., not be used stand-alone and instead of subsurface sections that are drawn to scale horizontally and vertically). Fence diagrams that include all the borings drilled at the site correlated to the stratigraphic column can be particularly useful to identify possible geologic hazards and unconformities.

The subsurface sections are developed to convey and confirm the practitioner's hypothesis of the site (Figure 4-12); the information presented in these sections is key to the success of the project. More often than not, subsurface sections form the cornerstone upon which subsequent engineering considerations and analysis are based.

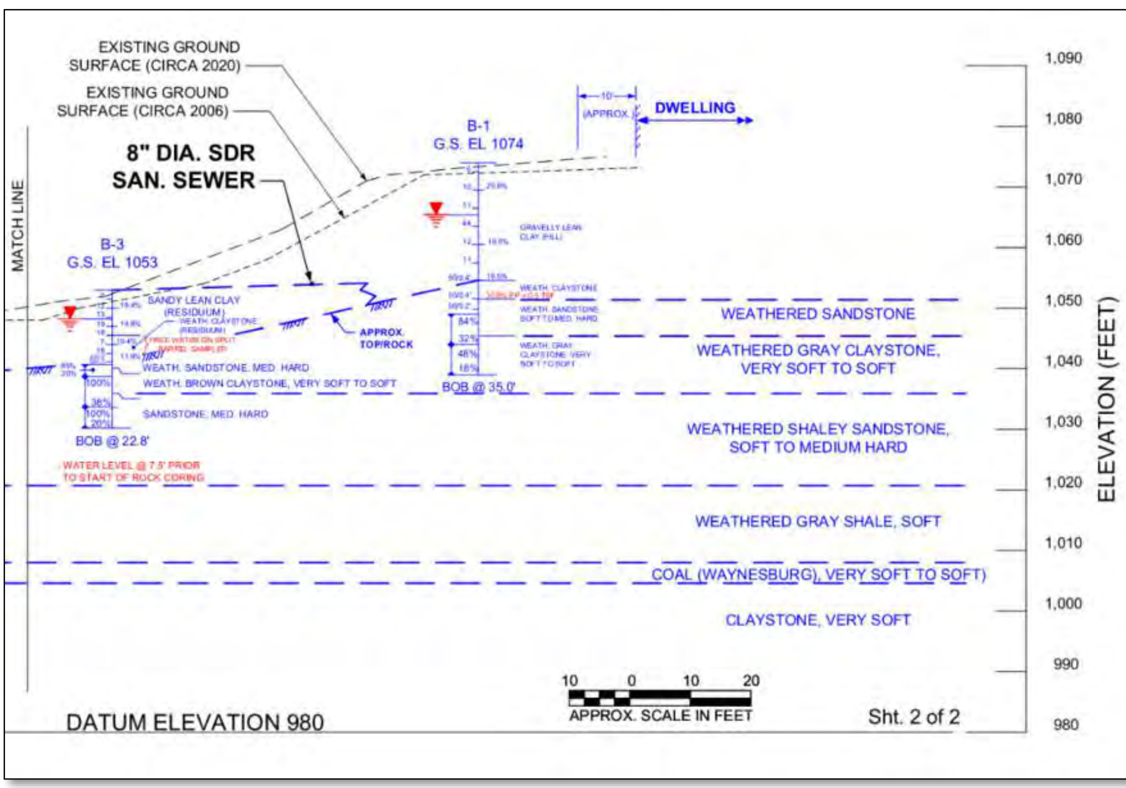


Figure 4-10 - Subsurface Section, Example 1

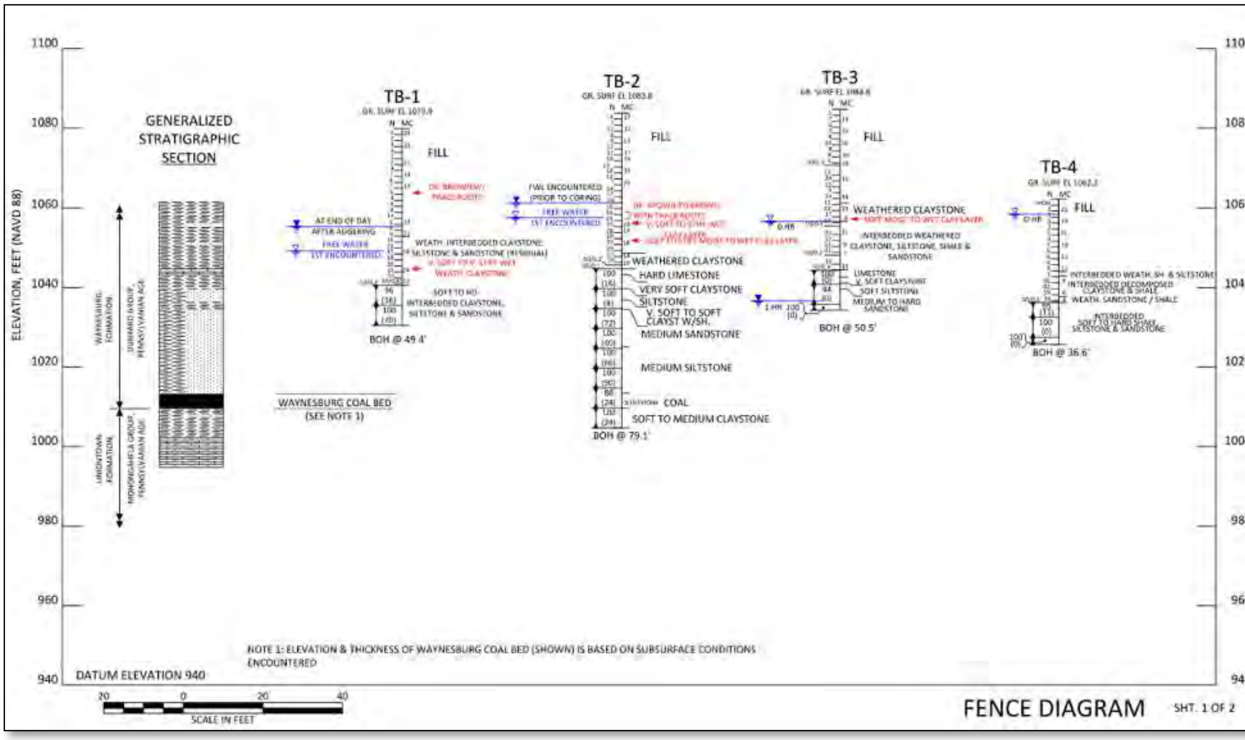


Figure 4-11 - Fence Diagram, Example

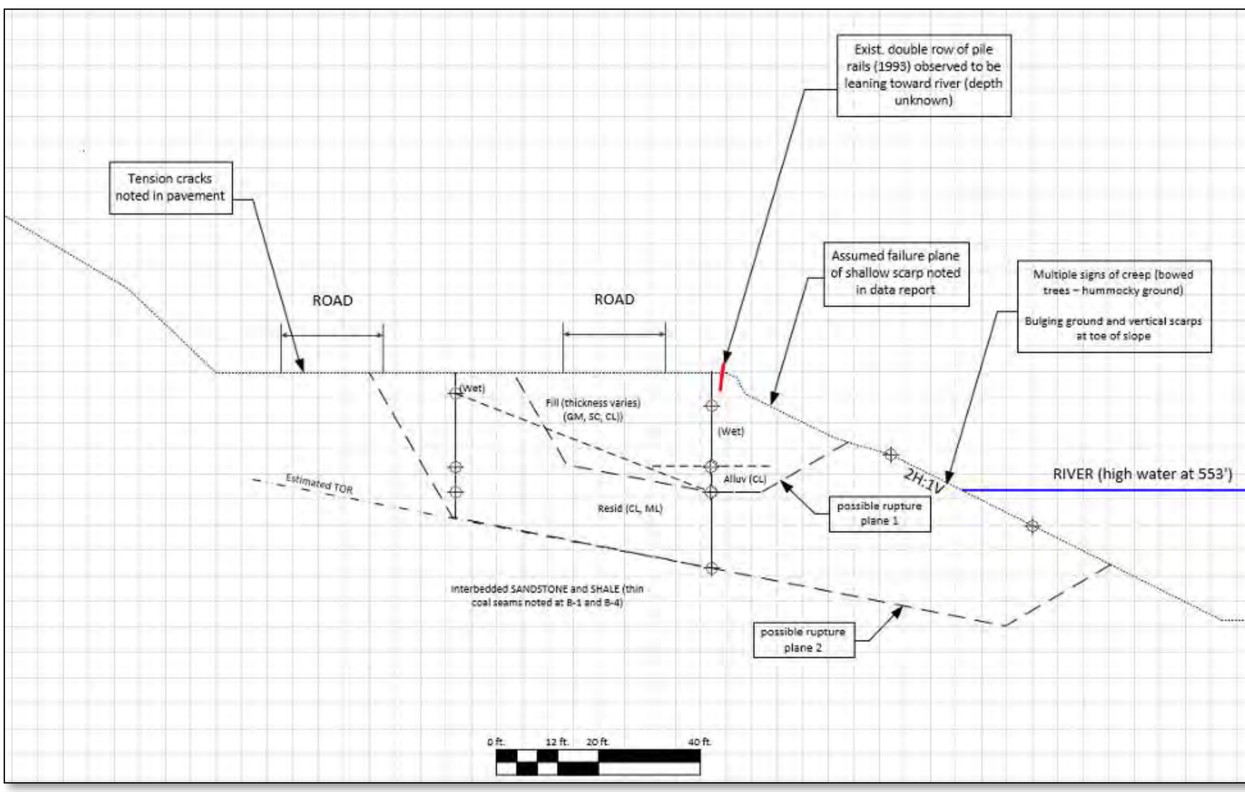


Figure 4-12 - Subsurface Section, Example 2

CHAPTER 5

Problem Definition

5.1 POSSIBLE MODES OF FAILURE

Problem definition begins with understanding possible mode(s) of failure. Modes of failure are a function of the landslide classification, loading mechanism, material properties, and triggering cause of landslide movement.

Landslide classification falls into the primary categories of slide, fall, topple, spread and flow. Another key consideration is the predominant type of material that is moving, whether it be fine grained soil, coarse grained soil or bedrock. See Section 2.1 of this Handbook for further discussion on landslide classification.

Slope creep that is moving at an imperceptible rate is recognized, but is not considered a type of landslide movement. However, creep can lead to a landslide with accelerated rate of movement.

Modes of failure are linked to the triggering cause(s) of landslide movement. Landslides in southwestern Pennsylvania are frequently triggered by an increase in precipitation, human activity (e.g., excavation and land disturbance), differential weathering, reduction in shear strength, and disruption of a natural seepage pathways such as when a water bearing limestone is buried beneath an earthflow of colluvium or fill with subsequent buildup of pore water pressure. Multiple causes contributing to instability may exist, but acceleration of landslide movement is frequently attributed to a single triggering cause. Further discussion about triggering causes is presented in Section 2.5 of this Handbook. It is important to recognize that multiple modes of failure can occur, especially when several smaller landslide masses combine to form a compound landslide mass.

At the conclusion of the landslide investigation, the practitioner should have a well-defined hypothesis of potential triggering factors. Additional monitoring, laboratory testing, and back-analysis to develop an appreciation of the sensitivity of potential factors can help to the practitioner to pinpoint a triggering cause and arrive at a working theory. This working theory will allow the practitioner to identify preferred mitigation measures and serve as the basis of design.

5.1.1 Porewater Pressure

Characterizing groundwater conditions and porewater pressures at landslides are key factors in defining the problem and assessing potential triggering causes. This is particularly true with respect to the prominent seasonal changes in surface water infiltration and fluctuation in the groundwater table characteristic of the region.

Seasonal increases in precipitation can lead to intermittent seepage that emanates at the ground surface. Perennial seepage typically flows year-round; whereas intermittent seepage typically emanates only certain times of the year at the ground surface when there is an increase in percent saturation due to precipitation or snowmelt. An increase in seepage flow rate may indicate a buildup of excess pore pressure. A buildup in excess pore pressure can result in a reduction in available shear strength and induce and/or accelerate landslide movement.

Both perched and static groundwater conditions may or may not exist in combination. Evidence of wet areas, seeps and springs that were observed during the detailed site reconnaissance and subsurface investigation may be considered in combination with the groundwater reading recorded at test borings, test pits, and/or piezometer locations. Moisture content of existing soil conditions can be further refined as part of the laboratory testing program; continuous or frequent measurement of soil moisture in the laboratory will allow the practitioner to develop a subsurface moisture profile at test borings drilled and sampled. See Chapter 7 for additional discussion on laboratory testing.

Where elevated porewater pressures or saturated soils are of concern; piezometers may be used to supplement groundwater observations and measure pore water pressure. Further discussion about instrumentation and monitoring is presented in Chapter 6.

5.2 EXTENT OF SLIDE MASS

The extent of the slide mass and the proximity to infrastructure is a function of the purpose and objective of the landslide investigation, including but not limited to need and timeliness of response.

Identification of the areal extent and depth of the slide mass is a crucial component to an accurate analysis and identification of available mitigation options. Large and deep slide masses will require more robust mitigation measures as compared to shallow, surficial failures. The extent and robustness of the mitigation measures required will coincide with significant cost implications. Alternatively, an inaccurate assessment of the extent and depth of the slide mass could result in insufficient mitigation measures. Therefore, performing a detailed landslide investigation and subsequent monitoring and laboratory testing program, to provide a well informed and accurate assessment of the extent of slide mass, is quintessential to providing best value solutions and the overall success of the mitigation response.

5.3 MAGNITUDE AND RATE OF SLOPE MOVEMENT

The magnitude and rate of slope movement is requisite to understanding the mitigation options and timeline of response. Small rapid landslides have been known to cause a large amount of damage, whereas large slow-moving landslides can be mitigated to minimize adverse impact.

Rate of slope movement can provide a measure by which to isolate and identify a triggering cause and determine when slope movement is arrested. Acceleration in the rate of movement can serve as a warning indicator to implement emergency action measures, such as vacating structures in immediate proximity to a landslide mass.

Both direct and indirect methods are available to measure slope movement. These methods range from comparison of aerial orthophotos and Light Detection and Ranging (LiDAR) data, to time lapse photography, to slope inclinometers, to traditional field survey of surface monitoring points, to visual observation of change in topography and physical features, such as leaning trees. Further discussion about instrumentation and monitoring is presented in Chapter 6.

CHAPTER 6

Instrumentation and Monitoring

6.1 SCALE AND PURPOSE OF INSTRUMENTATION

Answering the question of what should be measured when it pertains to landslides can be made overly complicated due to the sheer scale of the problem. On the contrary, instrumentation and monitoring should be kept as simple as possible. A site-specific instrumentation plan should be developed to achieve specific goals with measurable and repeatable data, make observations to enhance understanding of the landslide mechanism(s), and produce targeted and actionable results.

Measurement typically requires information ranging from a large scale, such as field observation by a geo-professional that is trained to observe, assess, and identify key features and mechanisms (see Chapter 4), down to a smaller scale utilizing multiple techniques that will be further discussed herein. Small-scale information, such as movement that cannot be observed by the naked eye, is vital information that can only be collected by installing and monitoring instrumentation. The main goal is to identify the most sensitive parameters that will change significantly during a landslide event [128]. However, not all parameters can be measured due to possible physical or economic constraints.

Landslide instrumentation is most commonly used at landslides that have previously shown signs of movement [128]; monitoring active landslides serves the purpose of providing either a periodic or real-time hazard warning to reduce the risk of adverse impacts on property and safety. In addition, monitoring can support the data collected and hypothesis formed during the landslide investigation. Alternatively, monitoring may also be pro-actively installed on sensitive slopes for which no significant slope movement is anticipated, but for which early forewarning is desired.

Instrumentation is utilized to supplement the landslide investigation, as an aid to warn of impending major events [128], and/or to evaluate the effectiveness of mitigation measures. Monitoring data can be used to create models, improve and/or monitor mitigation measures, inform decision-making for mitigation and public-notification measures, and assist in refining landslide prediction capabilities. The following presents a few common conclusions able to be made with targeted use of instrumentation:

- Prediction of the depth and areal extent of the sliding mass in a developed landslide.
- Determination of lateral and vertical movement within a sliding mass.
- Determination of the rate of landslide movement (i.e., velocity).
- Identification of effects of construction activity or precipitation.
- Monitoring of groundwater levels or pore pressure normally associated with landslide activity.
- Provision of remote digital readout or information to a remote alarm system that would warn of possible danger.
- Monitoring and evaluating the effectiveness of various control measures.

The duration of monitoring program should account for site-specific needs. Landslides are constantly evolving and should be monitored over a period of time that is adequate to evaluate environmental factors and rate of active slope movement. Climatic changes are a major influence on landslides and should be investigated for one seasonal change in precipitation, when possible. It is important to note that a monitoring program completed during a period when the precipitation rate is more severe will often be associated with accelerated rates of slope movement.

6.2 MONITORING APPROACH

6.2.1 Field Monitoring

As presented in Dunicliff and Green, slope monitoring may be broadly categorized as being undertaken to monitor the deformation of surficial slope features and groundwater pressure (both categorized as routine monitoring), and also monitor subsurface deformations (special applications).

Figure 6-1 - Survey Station for Surficial Deformation Monitoring

Surficial deformation monitoring may consist of successive survey measurements of control points established on the surface of a slope (e.g., closed level loop survey, see Figure 6-1). Surficial deformation monitoring consists of measurements that are limited to the surface of a slope; thus, these measurements are unable to provide additional information about deformations at greater depths, location of the failure plane, or related groundwater regimes. However, general movement magnitude (horizontally and vertically) and direction of movement may be derived from this method to inform the practitioner of overall trends in slope movement. See Figure 6-2 for example output data able to be collected using surficial deformation monitoring techniques.

Instrumentation (crack meters) may also be applied to measure minor changes in the aperture of cracks forming on the sloped surface that may not be readily apparent to the naked eye or within the limits of manual measurements.

Additionally, the use of optical fiber to measure surficial surface strains in slopes is an emerging technology [120].

Monitoring of groundwater and pore water pressure usually takes place through the installation of piezometers of varying types (see Section 6.5.3) including observation wells and open standpipe piezometers. Information acquired from the piezometers may be further supplemented by visual inspection of slope features such as seeps and springs, ponded water, wet terrain, and hydrophilic vegetation, which may provide important context in the interpretation of the data that is acquired from groundwater monitoring instrumentation.

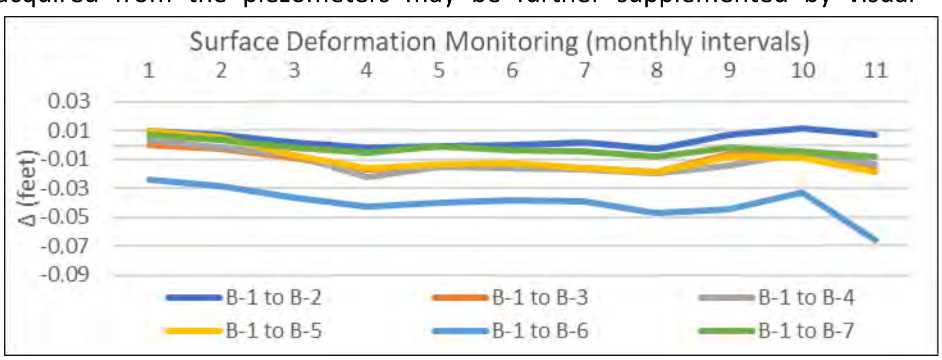


Figure 6-2 - Data Received from Surficial Monitoring

Special applications in the form of subsurface deformation monitoring may be performed with the use of instrumentation that is designed to detect and measure displacement at a very small scale. For typical landslide practice in southwestern PA, inclinometers are most commonly used for subsurface deformation monitoring and are available from a wide range of manufacturers. However, other instrument types, such as shear plane indicators and extensometers, may also be utilized in limited circumstances.

However, before an instrument can be selected, locating where to place the monitoring instrumentation needs to be identified by the practitioner. An instrumentation and monitoring program is of little use if installed at the wrong location, wrong depth (above the zone of movement), or incorrectly. In order to gain best value, the monitoring program should be purposeful and reliable. See Figure 6-3 for an example instrumentation layout for monitoring of an active landslide.

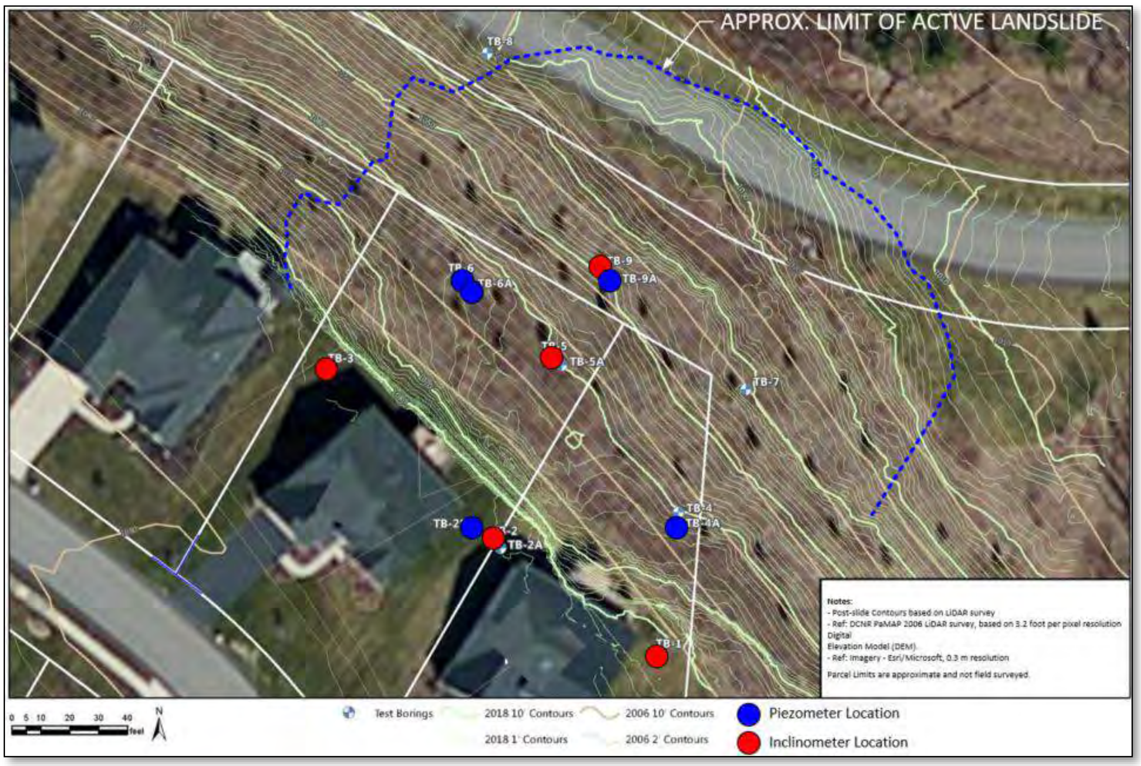


Figure 6-3 - Sample Instrumentation Layout

6.2.2 Remote Monitoring

Remote sensing methods, including the use of LiDAR surveys, may be utilized where readings are acquired and compared over a time interval (selected by the practitioner) to reveal locations and magnitude of movement over various parts of a landslide. Leveraging Unmanned Aircraft Systems (UAS) also provides a useful means of acquiring photographic and remote sensing data for surficial monitoring, particularly in difficult-to-access locations.

6.2.2.1 LiDAR

As discussed in Chapter 4, LiDAR technology is able available to collect data points at a high density to process and create 3D renderings of the ground surface with an accuracy of up to 0.25 inches; therefore, LiDAR creates potential applications beyond the initial site reconnaissance to be utilized as a remote monitoring method.

Given the accuracy able to be provided by LiDAR survey, a wide range of monitoring applications are available including:

- monitoring of an active landslide where slight movement is of interest; or
- monitoring of slopes throughout an entire region or district where larger movements are of interest to inform slope management systems.
- UAS with LiDAR sensors can be used to develop bare-earth surface models, due to the LiDAR's ability to penetrate vegetation canopy.

The processing and presentation of the data collected can be tailored to fit the project specific needs. In landslide applications, digital elevation models (DEMs) are often the most useful to alert the practitioner of slope movement exceeding specified thresholds. See Figure 6-4 for example output data for LiDAR surveys as applied to landslide monitoring.

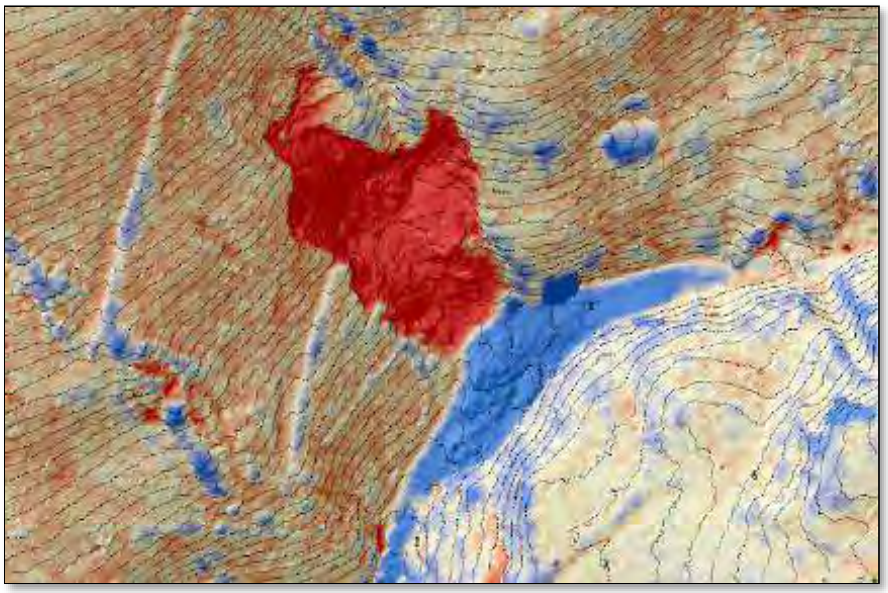


Figure 6-4 - Landslide Monitoring using aerial LiDAR
Heat Mapping Depicting Ground Elevation Loss (Red) and Gain (Blue) [86]

6.2.2.2 Unmanned Aircraft Systems (UAS)

As discussed in Chapter 4, UAS can be used to perform photogrammetric surveys to obtain orthomosaics, develop elevation models and high-resolution point clouds, and produce 3D representations of the surface features. Considering a vertical accuracy potential within several centimeters [37], UAS can typically provide the ability to efficiently monitor for possible landslide movement with significantly less field effort in comparison to traditional survey and geotechnical instrumentation installations. According to the study performed by Rossi [118], the UAS were able to survey an approximate 215,000 ft^2 area in about 40 minutes including flight planning and Ground Control Point (GCP) acquisition with Global Positioning Satellites (GPS). Once the survey is completed, the data is available for processing; for this site, the time to process the point cloud was approximately 30 minutes followed by a few hours of post-processing which included vegetation removal, mesh generation, mesh refinement, and Digital Terrain Model (DTM) generation [118]. See Figure 6-5 and Figure 6-6 for example output data gathered as part of a case study performed by Rossi [118].

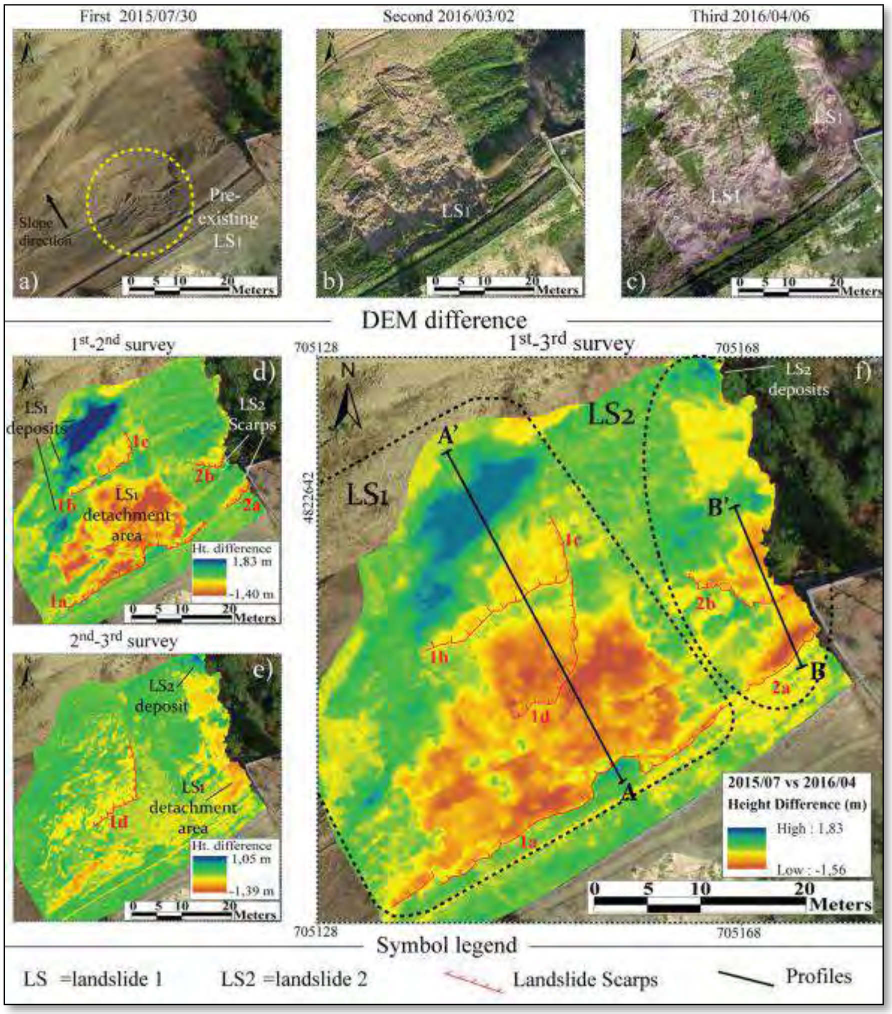


Figure 6-5 - Orthophotos of the area affected by the landslides (a, b, c) and DEM differences among different acquisitions (d, e, f) [118]

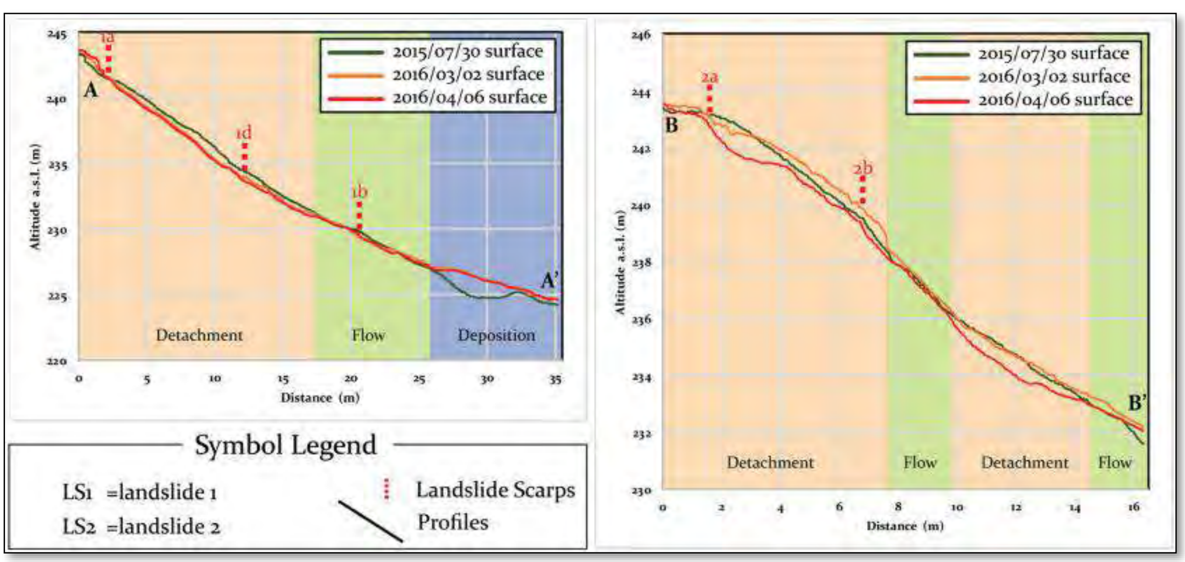


Figure 6-6 – Topographic profiles obtained from the three raster surfaces [presented in Figure 6-5] [118]

Advantages for remote monitoring using UAS include:

- The ability to survey areas too small for efficient manned aircraft and too large for efficient manual collection.
- Increased safety for field staff and ability to monitor non-traversable areas.
- High density of measurement data (see Figure 6-5).
- Increased repeatability in short periods [118] which allows for increased monitoring frequency.

Disadvantages of remote monitoring using UAS include:

- Startup time to obtain the technology, learning curve to use it accurately, and establish a workflow for flight planning and processing data.
- Filtering (removal of) vegetation points to obtain an accurate representation of ground surface when using photogrammetric methods [118].
- Decreased accuracy when compared to some conventional field monitoring techniques.

It is important to note that the practitioner should perform UAS flights in accordance with FAA Part 107 Regulations as well as any client-specific policy such as the PennDOT UAS Guidelines.

6.2.2.3 Synthetic Aperture Radar Interferometry (InSAR)

The use of InSAR is an emerging technology to identify and monitor unstable slopes remotely. InSAR compares complex satellite radar image datasets and measures slight changes in topography that occurred between the two acquisition dates. The practitioner is referred to Liu et. al [87] and Fobert et. al [52] for further information regarding the data processing and technical aspects of using InSAR for landslide monitoring. See Figure 6-7 and Figure 6-8 for example output data gathered as part of these case studies.

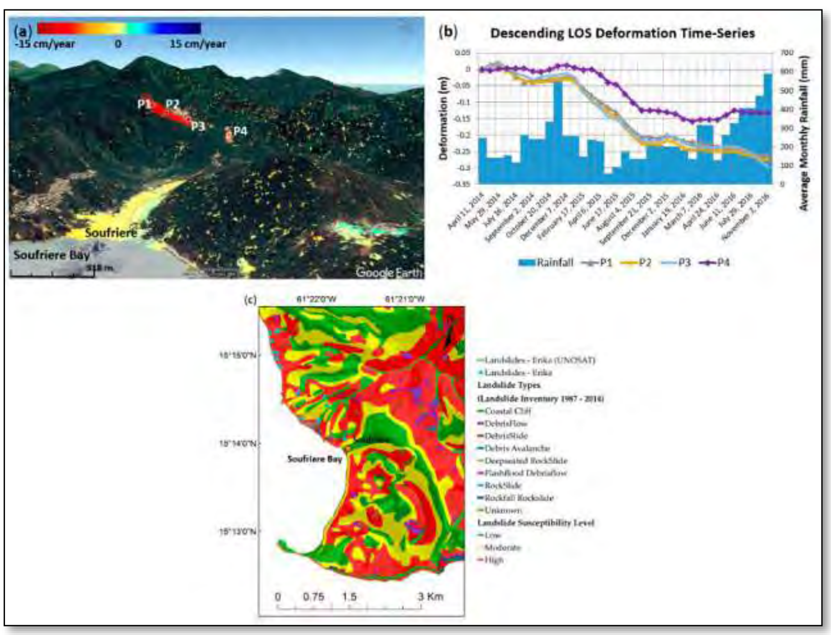


Figure 6-7 - Line-of-sight (LOS) descending deformation-rate map over Soufriere Village, overlain on Google Earth. (b) InSAR time-series overlaid on monthly rainfall measurements showing differential rates of landslide motion. (c) Susceptibility map overlain on a DEM with landslide inventory data over the same area [52]

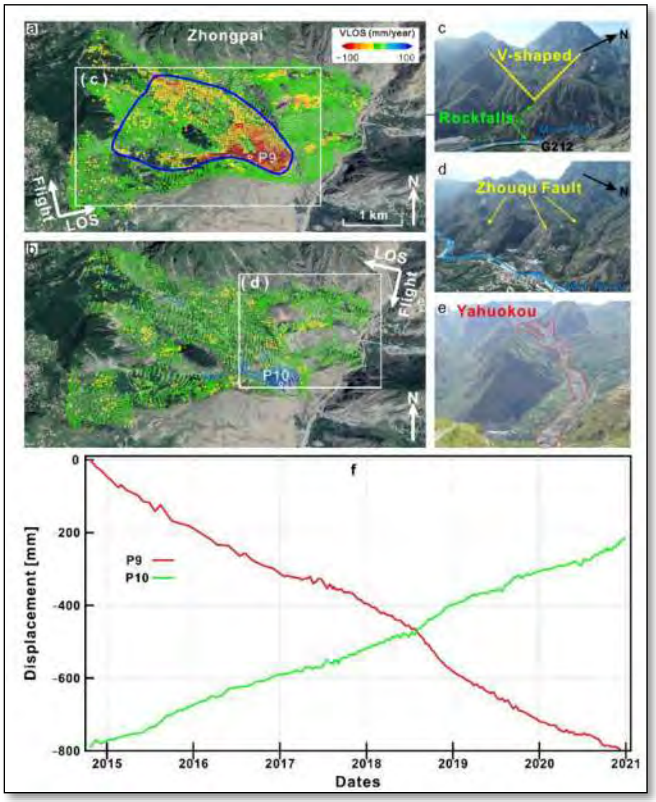


Figure 6-8 - Zhongpai landslide. LOS deformation rate of the MPs from the (a) ascending track and (b) descending track superimposed on Google EarthTM; (c) and (d) photos acquired using a UAS on 13 July 2021; (e) photos of site e taken on 13 July 2021; and (f) time-series displacement of points P9 and P11 [87]

6.3 TIMING: PRO-ACTIVE VERSUS REACTIVE APPROACH

A landslide-prone hillside can have protective or avoidance measures in place that help prevent a landslide from occurring. Other methods include monitoring and warning systems used not necessarily to prevent landslides but to alert downslope citizens, and provide data for engineers and owners to evaluate, inform decision-making, or some combination thereof. Pro-active measures to protect from landslides are preffered; however, sometimes incidents occur that need a reactive approach.

Pro-active approaches apply to areas that are known to be susceptible to landslides. An example of a pro-active approach is installing a landslide-warning fence at the base of a hill near a railroad track. If the hillside were to experience any movement and debris would fall towards the track, a warning system would trigger and alert the railroad of potentially blocked or distorted tracks. Typically a pro-active approach is cheaper long-term, as early warning and subsequent mitigation efforts may be implemented before substantial damage occurs.

Conversely, a reactive approach occurs during the time of failure and thereafter with little to no advanced planning [141]. A reactive approach typically occurs at an inconvenient time and place and is usually more costly to execute. One of the major problems with a reactive response is the indirect and direct costs/impacts that are more difficult to manage. With the example above, if no warning system is installed at the base of the hill near the track, a simple clearing of the track for debris could potentially become a major train derailment.

6.4 INSTRUMENTATION TYPES

The magnitude, rate, and distribution/direction of movement are the fundamental measurements to be achieved with instrumentation. In addition, pore water pressures are equally important in characterizing, resolving, and determining the causation of landslide issues. Many types and models of instruments are available for measuring the changing conditions of a landslide with varying degrees of readout capabilities.

Table 6-1 presents a summary of common landslide instrumentation methodologies that are applied to landslide characterization, mitigation, and monitoring practice in southwestern PA. Additional discussion of these methods are summarized in the following subsections.

Instrumentation	Data Obtained	Benefit	Relative Cost
Fixed Mount Tiltmeter	Data to evaluate structure rotation from slope movement (e.g., rotational slide) and cause for distress or loss of function	Evaluate hazard(s) for large permanent structures such as a bridge pier or bearing wall that is near a landslide or where slope movement is a concern.	$1,500 per tilt plate plus $1.50 per foot of cable
Probe Inclinometer	Data to define the rupture plane with a limited number of reading sets; able to survey the entire length of the casing (i.e., depth to rupture plane does not have to be pre-defined)	Define the location of the rupture plane at an active landslide	$10,000 for manual probe and 100’ cable plus $6.50 per foot of casing. Less front-end cost than in place inclinometers but more field labor is required
In-Place Inclinometer	Long-term data on high-value projects; can serve as an early warning alert system set up to warn of an acceleration in the rate of movement (e.g., when there is a fluctuation in rainfall); failure plane should be defined prior to deployment; not typical for routine landslide projects	Monitor long-term change in the rate of movement (e.g., rotation) with a higher volume of readings compared to manual methods	$900 per 2-foot segment; $1,000 per 5-foot segment; $1,200 per 10-foot segment; autonomous readings decrease the cost of labor
Crack Gauge	Data to discern differential movement at structure locations	Demonstrate measured movement at a wall with an existing crack to evaluate the influence of slope movement on existing structures	$30 for gauge plus cost of labor to install

Instrumentation	Data Obtained	Benefit	Relative Cost
Observation Well	Water level data over an extended period	Assess variability in water levels with time	$25 per foot plus $150 for slotted screen
Standpipe Piezometer	Water level and/or pore pressure data; uses to evaluate static water level or, with prolonged readings, seasonal fluctuation in precipitation; use over multiple seasons is preferable	Assess static water table where levels have stabilized to a constant reading (not so common in Pittsburgh geologic region, except for alluvial and terrace deposits)	$600/each plus an optional $50-$100 for Casagrande tip; $900/each datalogger cost separate for continuous monitoring
Vibrating Wire (VW) Piezometer	Water level and/or pore pressure data; assess influence of variable precipitation to build confidence about the risk of local fluctuation	Assess transient fluctuating water table (e.g., perched water influenced by surface water infiltration); correlate to rainfall/snowmelt (precipitation data)	$600 per sensor plus $1 per foot; $900/each datalogger cost separate for continuous monitoring

Table 6-1 - Summary of Manual Instrumentation and Monitoring Methods

6.4.1 Tiltmeter

6.4.1.1 Portable Tiltmeter

6.4.1.1.1 Uses and Limitations

A tiltmeter (Figure 6-9) is used to detect the tilt (rotation) of a surface point.

In place, inclinometers are fixed in place, attached with epoxy and screws, and used to measure structure rotation from slope movement (e.g., rotational slide) that would cause distress or loss of function. The operating range for these devices is typically on the order of +/- 15 degrees.

Figure 6-9 - In-Place Tiltmeter [56]

Portable tiltmeters are also available and typically used in highway and railway cuts or areas that may experience rotational mode failures. The tiltmeter is portable and lightweight which makes it an appealing low-cost option for instrumentation that is readily deployable in the field. However portable tiltmeters are not common in landslide investigations.

6.4.1.1.2 Deployment and Data Collection

Typically, in-place tiltmeters are deployed on large permanent structures such as bridge piers or bearing walls in proximity to a landslide to monitor displacement where slope movement is a concern. Upon deployment, the practitioner will typically identify thresholds beyond which there is reason for concern;

think of these thresholds as indicative of when we initiate a green light (go), yellow light (caution), and red light (stop) response.

Readings should be obtained in combination with a second alternate source of measurement (e.g., field survey, plumb line, etc.) to validate results. Additionally, it is important that the practitioner be sensitive to thermal and humidity changes, and surfaces that are exposed to direct sunlight due to potential for thermal movements. It is best to obtain instrument readings before sunrise, if possible, at a consistent time of day.

Data post-processing will be site-specific. However, generally, the data should be reduced into the most direct form of presentation, typically graphical, with distorted scale when possible. There will be apparent noise in the results, but definitive trends should be observed. When in doubt, complete a supplemental set of readings at the site to validate the data set.

6.4.2 Inclinometer

The development of the inclinometer has significantly contributed to the analysis and detection of landslide movement. The use of inclinometers has been so successful that it has gained widespread use as a monitoring instrument for other projects such as dams, bulkheads, and other earth-retaining structures [128]. The inclinometer sensor uses a closed-loop, servo-accelerometer circuit that measures inclination in one plane.

See below for a summary of probe and in-place type inclinometers and their applications in monitoring slope movement. The practitioner can be further directed to Green and Mikkelsen [63] for additional discussion on obtaining deformation measurements with inclinometers.

6.4.2.1 Probe Inclinometers

6.4.2.1.1 Uses and Limitations

The probe inclinometer (Figure 6-10) is a commonly used inclinometer that contains a servo-accelerometer, which is fitted with guide wheels and lowered by an electrical cable down casing with machined grooves. The cable is connected to a readout unit so data can be recorded automatically. There are four main components to a probe inclinometer system. The first component is a casing that is permanently installed in a vertical borehole in the ground; the casing is made of circular sections typically out of plastic, steel, or aluminum. The second component is the probe sensor unit that is mounted in a carriage designed for operation in the guide casing. The third is a control cable that raises and lowers the sensor unit within the casing and transmits electrical signals to the surface. And lastly, a portable control and readout unit located at the surface that obtains instrument readings, and often stores and processes the data obtained.

Figure 6-10 - Probe Inclinometer [56]

Probe inclinometers are typically used to define or verify the location of the rupture plane and characterize the slide mass at an active landslide. Probe inclinometers can also be used to monitor slope movement. The typical operating range for these instruments is a typical maximum out of plumb tilt of +/- 30 degrees.

6.4.2.1.2 Deployment and Data Collection

It is advantageous to the practitioner to schedule inclinometer installation with the subsurface investigation as the inclinometers can be installed in the existing boreholes rather than having to drill additional boreholes. The bottom of the inclinometer casing should be socketed in a stable soil or rock unit (i.e., stratum) below the slide mass; the practitioner should be able to identify this stratum based on the results of the landslide investigation. As an alternate, survey the top of the inclinometer to provide data upon which a lateral and rotational transformation can be rationalized to normalize the data sets.

Figure 6-11 - Inclinometer Reading at Crown of Landslide

During installation, the practitioner should be particularly sensitive to and aware of how the borehole was backfilled when the inclinometer was installed. Poor backfill placement has been a recurrent problem and resulted in discrediting the results for a substantial number of inclinometers that were installed under inadequate quality control, for example where air pockets or excess voids can form. See Figure 6-12 for a typical probe inclinometer schematic.

When possible, combine the inclinometer readings with a surface monitoring program (conventional survey), which can complement each other to build more confidence. Ideally, the instrumentation should show/document the zone of depletion and zone of accumulation limits, verify the depth to the rupture plane, clarify the type of slide (rotational or translational), validate the toe of the slide, and quantify the rate of movement including trends in the variability of rate of movement. Ideally, these readings would be correlated to precipitation data to enhance the interpretation of results.

Where probe inclinometers are deployed to monitor movement, the practitioner will typically identify thresholds beyond which there is a reason for concern; think of these thresholds as indicative of when we initiate green light (go), yellow light (caution), and red light (stop) conditions.

Refer to [PennDOT Publication 222] [101] Section 207 for additional guidance about installing inclinometers.

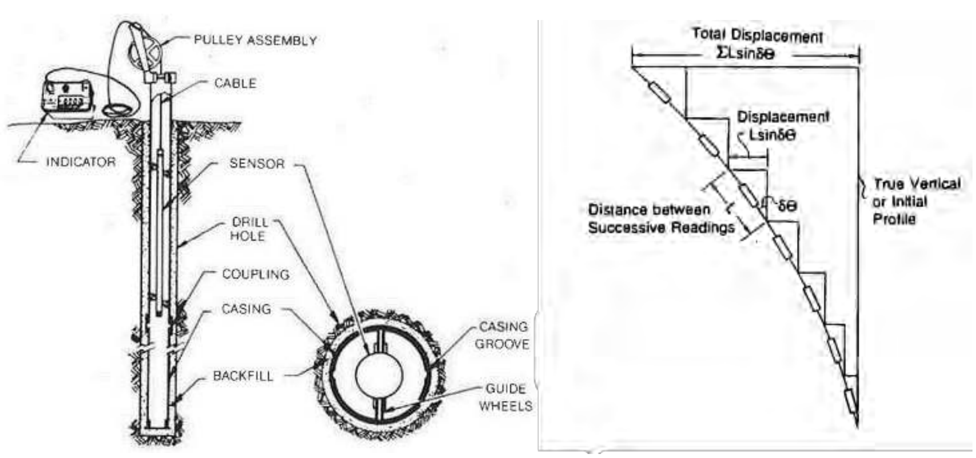


Figure 6-12 - Principle of Probe Inclinometer Operation [63]

6.4.2.1.3 Data Reduction

Extensive data reduction is needed for inclinometers, including rotational and translational transformations. There will be apparent noise in the results; however, the practitioner should be able to identify definitive trends. When in doubt, complete a supplemental set of readings to validate a data set. The A-axis is less typically more accurate than the B-axis readings, and hence we typically want to orient the A-axis in the direction of maximum movement (i.e., looking downslope).

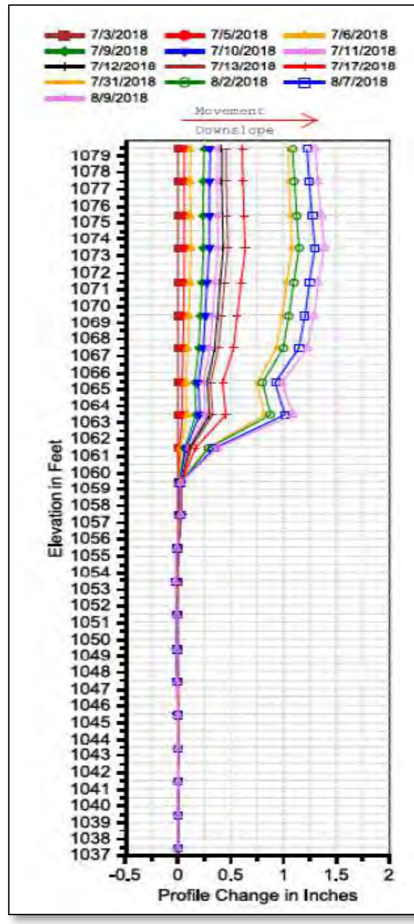


Figure 6-13 - Typical Elevation-Displacement Inclinometer Output Plot

Due to the high volume of data associated with inclinometers, specialized software is available to perform data reduction and graphing tasks. Although the practitioner should be sensitive to any limitations of the software and manually verify selected readings to confirm accuracy.

Whether using an in-place or probe style inclinometer, the manufacturer's literature will typically outline all the different types of error that may occur, what the error may look like on a plot of raw data, and how to correct for the error if it has occurred. The manufacturer's literature should be adhered to above all when identifying and correcting errors.

Using methods that are repeatable and reproducible by a third party is crucial to obtaining a defensible data set. Where outliers occur, obtain a supplemental reading set to reproduce the anomaly. Ideally, checksums should remain constant with depth within a given data set for both A and B sensors [63].

The most common data adjustment is a rotation transformation to account for possible jarring of the servo accelerometer in the sensor probe during transport. This is not an uncommon requirement, even when the operator takes great care to handle the sensor probe when transporting it to and from the site. To address this need, it is critical that either the bottom or top of the inclinometer (usually the bottom) is fixed in a zone that the practitioner is confident is not moving. Each set of readings is then transformed rotationally so that the fixed portion is aligned with the baseline reading set.

Another common data correction is when the inclinometer casing grooves are installed at a skew with respect to the maximum slope gradient (i.e., primary direction of slope movement). In this case, a geometric correction is needed to predict movement in the direction of interest.

In the past, spiraling of the inclinometer casing has been identified as a possible error that needs to be corrected for. However, given the current quality of inclinometer casing that is commercially available from reputable instrument manufacturers, this has not been an issue on routine inclinometer projects in over the past several decades.

Undoubtedly, the most common errors include:

- Inadequate care to completely backfill the borehole annulus when the inclinometer was initially installed.
- Insensitive handling of the sensor probe.
- Failure to let the sensor probe temperature stabilize at the bottom of the inclinometer casing prior to taking the next set of readings.
- Insufficient care to take successive readings at the same depth increment,
- Data recording errors.
- Failure to review the data in the field and compute "check sums" before moving on to the next inclinometer to obtain more readings.
- Failure to conduct routine maintenance and re-calibration of the sensor probe at the factory.

However, all of these errors are easily avoidable with attention to detail and quality control.

Successive movement profiles may be generated, preferably over similar time intervals, to aid in forecasting (Figure 6-13). Depending on the project circumstances, a time interval should be agreed upon by the interested parties and agencies and can be adjusted as more data is collected. It should be remembered that the initial reading needs to be taken to establish a baseline. Once the baseline is established and successive plots are generated, movement forecasting becomes possible.

6.4.2.2 In-Place Inclinometers

6.4.2.2.1 Uses and Limitations

Figure 6-14 - In Place Inclinometer [56]

An in-place inclinometer (Figure 6-14) involves the sensors being sealed and fixed within a casing of a near-vertical borehole. The sealed sensor packages are spaced with a standard grooved inclinometer casing by a series of rods. The rods and sensors are linked by universal joints so that they can deflect freely as the soil and casing move (Figure 6-15). The sensors are secured and aligned in the casing by spring-loaded guide wheels. Measurements are taken by determining the change in sensor tilt over the gauge length or spacing between sensors. The result is a relative displacement that can be summed to determine the total displacement at each inclinometer casing. Since the sensors are fixed in-place, monitor and alarm consoles and telemetry systems are available to use with in-place inclinometers.

In-place inclinometers are useful in automated monitoring at a regular frequency, which often produces more precise data on the rate of movement. Ultimately, readings taken at regular incremental depths allow for the determination of the change in slope at various points and the relative deflection between those points. This data provides the distribution of lateral movements to be determined as a function of time and depth below the ground surface. However, the failure plane should be accurately identified before deployment; proper positioning and installation increments (along the depth of casing) are important to obtaining valuable data.

Although these instruments provide value to long-term, high-value projects where an automated early warning alert system is important to warn of an acceleration in the rate of movement (e.g., when there is a fluctuation in rainfall), they are not typical for routine landslide projects in southwestern Pennsylvania.

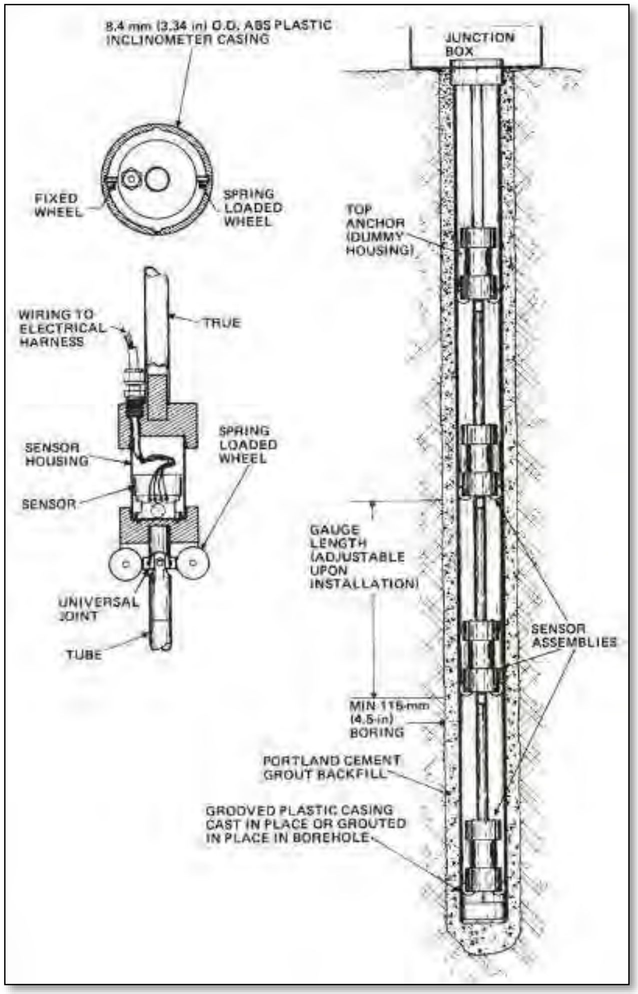


Figure 6-15 - Installation and Detail of Multi-Position In-Place Inclinometer [142]

6.4.3 Crack Gauge

Crack gauges (Figure 6-16) are utilized to demonstrate movement (or non-movement) of an existing crack, typically on a wall or face of a structure. The typical operating range for these devices is +/- 10 mm with an accuracy of 0.5 mm.

6.4.3.1.1 Uses and Limitations

Crack gauges are attached to the surface, traversing the existing crack, using epoxy and screws. Once installed, the instrument can discern differential movement through manual instrument readings.

Figure 6-16 - Crack Gauge

6.4.3.1.2 Deployment and Data Collection

Installation of multiple gauges is optimal to demonstrate repeatability and reproducibility by a third party. To validate results, it is advantageous to obtain measurements by another means (e.g., laser level, plumb lines, etc.) to complement the data obtained.

Based on experience, 1/8" crack width is a typical threshold, beyond which one would begin to consider structural distress. The practitioner should make allowance when interpreting the results for cracking due to thermal, humidity, and shrinkage change.

6.4.4 Piezometers

Groundwater and pore pressure fluctuation are major factors that contribute to the occurrence of landslides, which makes groundwater monitoring fundamental to an instrumentation program. A discussion follows below to present commonly used groundwater monitoring methods including observation wells and piezometers.

In addition to the discussion presented herein, the practitioner is further referred to the discussion in Freeze and Cherry [53], Driscoll [33], Fetter [40], and USACE [133] for detailed coverage of groundwater monitoring techniques and interpretation.

6.4.4.1 Observation Wells

6.4.4.1.1 Uses and Limitations

A simple groundwater monitoring tool is an observation well (Figure 6-17). An observation well consists of a small diameter casing, typically referred to as a riser pipe (typically plastic or steel), with a slotted end section installed within a borehole. The borehole annulus is backfilled with granular material that allows for groundwater entry into the slotted section and riser pipe, where depth measurement may be taken with a water-detecting probe. While economical and rapid to install, observation wells do not permit monitoring of pore water pressure from a discrete soil zone and create a vertical hydraulic connection between shallow and deeper soil units [35].

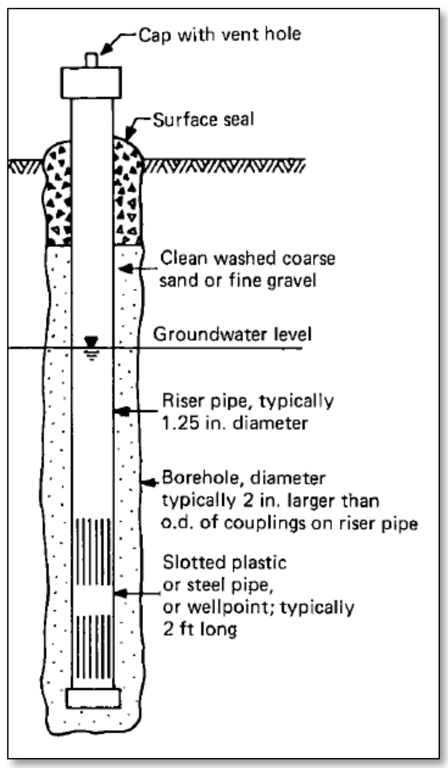


Figure 6-17 - Observation Well [35]

6.4.4.1.2 Deployment and Data Collection

Observation well data is collected through manual field measurements obtained by manually lowering a water-detecting probe down the casing.

6.4.4.2 Open Standpipe Piezometer

6.4.4.2.1 Uses and Limitations

A common type of groundwater monitoring instrument is the open standpipe piezometer (Figure 6-18). These are simplified instruments to obtain water level readings and pore water pressures for a selected stratum. Casagrande piezometers are a variation of open standpipe piezometers that employs the use of a specialized tip that consists of a porous stone tip embedded in sand in the sealed-off portion of a borehole.

Standpipe piezometers are cost-effective tools due to their simplicity; however, they can become labor-intensive due to the need for field measurements. However, continuous readings through the use of water level loggers are available which can provide high-density datasets and drastically reduce the number of labor hours associated with data collection; see Section 6.4.4.4 for further discussion.

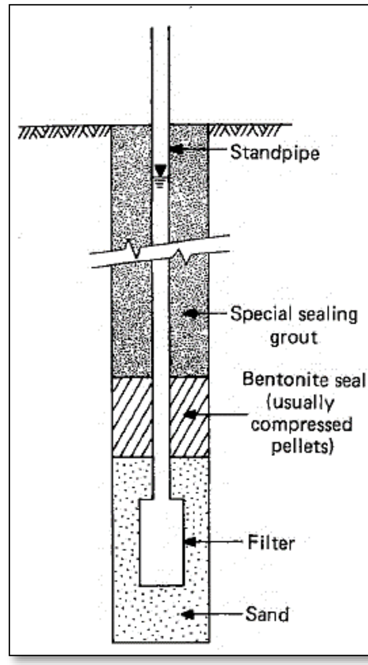


Figure 6-18 - Open Standpipe Piezometer [35]

6.4.4.2.2 Deployment and Data Collection

A simple open standpipe piezometer consists of a small diameter (commonly Sch. 40 PVC) casing inserted into a borehole, with a slotted end section (sensing section) positioned within the discrete soil depth interval (sensing section zone) where pore water pressure information is desired. The annulus of the sensing section zone is backfilled with granular material, above which impermeable material (typically bentonite) is placed to hydraulically isolate the sensing section zone from water infiltration outside of the sensing zone.

Similar to observation wells, the water level is measured by manually lowering a water-detecting probe down the casing. This measurement gives the average conditions over the entire zone that is being monitored. In impervious soils, the open standpipe may have a large time lag which can be lessened by reducing the riser pipe diameter and increasing the screen filter diameter (although slot size considerations to avoid piping/clogging from surrounding soil media often govern the selection of slot size).

For common practice in southwestern PA, PennDOT Publication 222 [101] provides recommended procedures and materials concerning open standpipe piezometers for transportation-related practice.

6.4.4.3 Vibrating-Wire Piezometers

6.4.4.3.1 Uses and Limitations

Vibrating wire (VW) sensors are more sophisticated piezometers compared to the open standpipe variety. The VW piezometer consists of a sealed tip containing a pressure-sensitive valve that opens or closes the connection between two tubes that lead to the surface [128]. The flow of air through the outlet tube is established as soon as the inlet-tube pressure equals the pore-water pressure. These sensors are simple to use and operate, have long-term stability, and virtually no time lag in all types of soils. VW piezometers are typically available with an operating range as low as 17 kPa (355 psf).

Although VW piezometers are ideal for remote monitoring, they should be used with a data logger to obtain readouts since manual measurements are not possible which will incur additional equipment costs. However, with regular use, the additional cost for remote monitoring will likely be offset by the reduced labor involved to obtain a reading.

6.4.4.3.2 Deployment and Data Collection

These piezometers can be installed with or without a sand intake zone and bentonite seal; refer to Figure 6-19 for the depiction of typical installation details.

Due to the electrical components of the system, lightning protection is advised; failure of electronics due to a local surge is not uncommon when lightning strikes extend from the ground to the sky within a 1/4 mile of the instrument.

Calibration factors should be applied to obtain pressure measurements. For unvented installations a correction to compensate for fluctuation in barometric air pressure should be made using Weather Underground or other nearby source; vented type installations do not require that correction. During installation, it is pertinent to saturate the porous sensor tip per the manufacturer's recommendations when deployed, or you may get misleading readings.

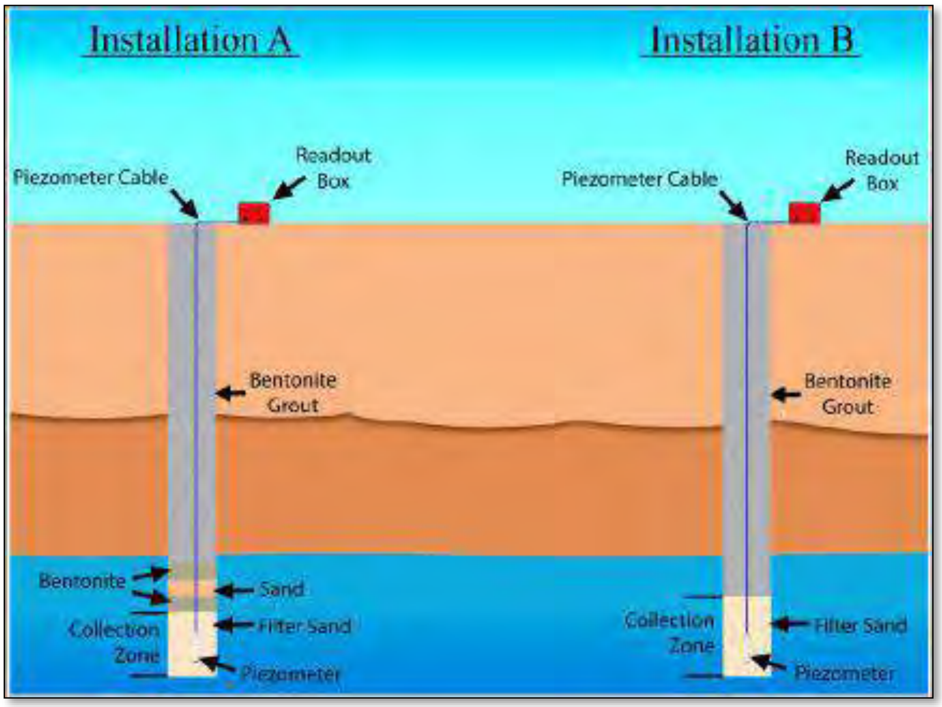


Figure 6-19 - Typical Installations of a Vibrating Wire Piezometer [59]

For more detail about VW piezometers and how to install them, refer to the following video link, https://www.youtube.com/watch?v=MEBgQJYko60.

6.4.4.4 Water Level Loggers

Water level logger instrumentation provides an additional means for groundwater monitoring, which involves the placement of a "plug and play" vented sensor in an open standpipe piezometer with self-contained data logging capability that can be retrieved during a follow-up site visits. This type of instrumentation may also be equipped with remote access communication to accommodate real-time monitoring if required. This equipment has to opportunity to provide continuous monitoring data and drastically reduce the amount of labor involved in data collection.

Results from these remote sensors may be validated by obtaining manual field measurements of the water levels during data collection events.

The high density of water level readings able to be obtained by the level loggers are useful to identify trends and correlate to precipitation data. See Figure 6-20 for an example of the output data able to be obtained with level logger systems. In this example water level readings (from the level logger) at a single piezometer location were plotted with daily precipitation levels and coal elevations (e.g. a potential water bearing unit). The manual water gauge readings (from a water level meter) that were collected during monthly level logger data collection visits were used to validate data and are also shown on the plot.

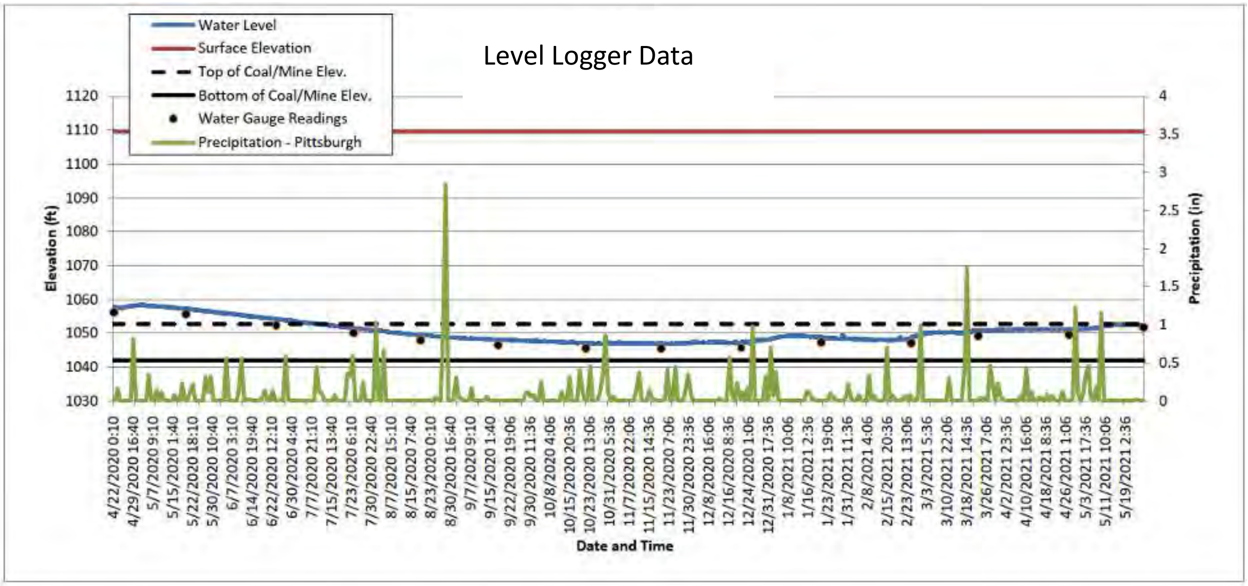


Figure 6-20 - Water Level Logger and Precipitation Data

6.5 DATA REDUCTION

The reduction of data obtained from installed instrumentation is either done manually by direct manipulation of the data by the user or by the assistance of computer software specialized for the particular type of data being reduced and used. In simple cases, such as water depth readings in open standpipe piezometers or observation wells, a tabular summary, and graphical plots may be generated with commonly available spreadsheet software so that the data may be evaluated and utilized. In many modern cases, data reduction software is supplied by the slope instrumentation manufacturer for direct use with the data collected by the instrumentation. Instrumentation data reduction is also important in many cases, whether by simple or computer-aided means, as a quality assurance measure to confirm that the data was collected in the manner intended by the instrumentation manufacturer and as a validation that the instrumentation components are performing satisfactorily.

Data reduction procedures are not covered in detail for the typical slope instrumentation methodologies presented herein, as the considerations are entirely dependent on the type of instrumentation being used, the specific components of the instrumentation, and the recommendations of the instrumentation manufacturer. Regardless of the age, type, and manufacturer of a particular slope instrumentation device, it is imperative for the user to have a complete understanding of the function and purpose of the individual components so that data reduction, evaluation, and subsequent use can be made without misinterpretation and any potential anomalies in the recorded data can be identified.

6.6 FORECASTING

Forecasting of future slope movement is often desired for designed soil embankment slopes, cut slopes (permanent or temporary), and for natural slopes in which a landslide is already occurring. Monitoring with instrumentation serves as a useful aid to assist with forecasting regardless of the type of slope. In order to effectively use data to forecast movement, special attention should be given to monitoring frequency. Both the quantity of data (data points) and the data acquisition frequency (e.g., to account for seasonal fluctuations, rate of predicted change, and the available precision of the instrument), are key considerations to make determinations about the desired frequency for forecasting purposes. These variables will vary for each project based on the purpose and type of instrumentation used, the magnitude/rate of slope movement involved, and the tolerable threshold that impacted parties can endure.

As an emphasis to the practitioner, slope instrumentation may serve as an important tool for slope movement forecasting but is often used in tandem with traditional field reconnaissance, site observation practices, site survey, and evaluation of remote sensing and aerial imagery (including that obtained from Unmanned Aerial Vehicles, i.e., drones). Instrumentation needs for forecasting purposes primarily involve monitoring groundwater conditions within a slope, fluctuations of which serve as a proxy for potential or pending slope movement, as well as direct monitoring of surficial and subsurface deformation rates, direction, and magnitude. As such, the use of many of the instrumentation types discussed herein is routinely utilized not only for monitoring for landslide characterization studies during design, but also for forecasting purposes. Forecasting slope movement allows for evaluation of the efficacy of mitigation measures, the protection of critical/sensitive infrastructure components (e.g., roadways, gas wells, utilities), and informed decision-making for necessary actions to pro-actively-prevent worsening slope conditions or the need to alert the public or specific owners, agencies, and other stakeholders.

Additionally, mitigation efforts for landslides often involve either temporary or permanent excavations and/or placement of fill material at planned slope locations to install stabilization and repair methods, as well as drainage measures (e.g., horizontal drains and aggregate-filled slot drains). As such, instrumentation further serves the purpose of forecasting and verifying reduction in slope movement as a result of completed mitigation actions, often in conjunction with slope stability modeling and analysis.

CHAPTER 7

Laboratory Testing

7.1 PURPOSE OF LABORATORY TESTING

Laboratory testing is conducted following the landslide investigation. Prior to laboratory testing, the data obtained during the landslide investigation (field reconnaissance, desktop study, topographic survey, subsurface investigation, etc.) and monitoring program should have provided a basis for the practitioner to define project constraint(s), landslide areal limits, depth to critical rupture plane, soil, rock and groundwater conditions, and other miscellaneous factors. Based on this information the laboratory investigation should be purpose driven to provide supporting data and enhance the characterization of subsurface conditions and classification of soils. A successful laboratory testing program is crucial to determination of representative soil and rock parameters for analysis and design.

The type and number of laboratory tests required are dependent on the project's needs, goals, budget, and urgency. Landslide investigations in southwestern Pennsylvania typically entail soil classification and shear strength testing on representative soil samples and unconfined compressive strength and/or point load index tests on intact rock core specimens in the laboratory.

Soil classification tests typically involve sieve analyses, hydrometer analyses and/or percent passing the #200 sieve, Atterberg limits, moisture content, and Shelby tube bulk density tests. Soils are typically classified by the Unified Soil Classification System (ASTM D2487) and/or the American Association of State Highway and Transportation Officials (AASHTO) Standard Specification for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes (AASHTO M145). Depending on the plausible mode of failure, shear strength tests are also completed in the laboratory on representative intact and remolded soil specimens to determine the following:

- Unconsolidated-undrained shear strength (UU, typically used for an end-of-construction condition).
- Consolidated-undrained shear strength with pore pressure measurement (CU, steady-state seepage, and rapid pore pressure change scenarios, typically used for total stress and effective stress conditions).
- Consolidated-drained shear strength (CD, free-draining, typically used for an effective stress condition) where complete consolidation has occurred under the existing overburden and failure is reached slowly so that excess pore pressures are dissipated.
- Residual shear strength (typically used for an effective stress condition with large strain – e.g., the portion of landslide mass that has slid), which is typically needed when dealing with material that has been subjected to prior shear displacement.

7.2 TESTING METHODS

Laboratory test methods typically follow standard specifications set forth by ASTM International (ASTM) and AASHTO. ASTM and AASHTO each provide laboratory testing specifications covering the most common types of laboratory testing procedures used for geotechnical engineering purposes. In many instances, ASTM and AASHTO provide generally equivalent testing specifications for the same types of soil tests. AASHTO Test Methods should be used for all other tests, but if an AASHTO Test Method does not exist, ASTM Test Methods may be used. For PennDOT projects, PennDOT Test Methods (PTMs) will take precedence over other test standards. Detailed testing standards and procedures are available on the AASHTO and ASTM websites. The practitioner should be aware that subscriptions often apply to the purchase of current standard testing methods, which are typically copyrighted and not reproducible without permission from the author/originator. PennDOT PTMs are available in PennDOT Publication 19.

The full array of testing methods commonly used in southwestern Pennsylvania are not enumerated in full herein but are listed and described in commonly used, publicly available publications including Chapter 4 of PennDOT Publication 293, Geotechnical Engineering Manual [102], and Section 3.9 of PennDOT Publication 222, Geotechnical Investigation Manual [101]. The practitioner is directed to these publications as an easy reference for geotechnical laboratory tests, testing method designations, sample types, a minimum quantity of samples needed for each test, and typical cost ranges for each test. A web link for retrieval of current PennDOT publications is provided under References and Additional Resources herein.

7.3 SOIL CLASSIFICATION AND INDEX PROPERTIES

As previously discussed, aside from the importance of characterizing the shear strength behavior, the soil classifications and index properties of the soils involved in landslide movement should be characterized as part of developing a complete understanding of the soil mechanics prevailing within the landslide. A brief discussion of the typical laboratory testing performed in conventional landslide site characterization practice follows below.

Soil classification is intended to characterize the grain size distribution, plasticity, and liquidity of the soil tested. Soil classifications can then be used to determine index properties and estimate engineering properties based on correlations with published information for similar soil types and origins. A variety of soil classification systems have been proposed and utilized in various industries over the years; however, this discussion does not attempt to provide a comprehensive survey of those, nor the merits and drawbacks of each concerning one another. For subsurface characterization of landslides in southwestern PA, the Unified Soil Classification System (USCS) is frequently utilized, often in conjunction with the AASHTO Soil Classification System. Both systems are predicated on quantifying the distribution of particle sizes for a soil specimen ranging in size from boulder-sized material to cobble, gravel, sand, and fine-grained (silt and clay) particles. The plasticity of the fine-grained portion is another key factor in classifying the soil specimen.

Soil classification (i.e., Index Testing) is typically used in conjunction with knowledge about former land use and geologic depositional characteristics to characterize subsurface conditions in a graphic format (e.g., subsurface section).

Moisture content determinations serve as a good source of data to quantify and illustrate a moisture profile on the subsurface section(s). This is particularly useful when perched groundwater is encountered, which is common in western Pennsylvania due to the cyclic nature of the structural geology and interbedded nature of the water-bearing members (e.g., limestone, fractured sandstone, and coal) and less permeable claystone and clay shale units. Due to the generally low cost of moisture testing, it is often advantageous to get a higher density of moisture data (versus classification tests) to confirm the moisture profile and confirm field observations.

Moisture content data can be used in combination with the Atterberg Limit to measure the “softness” of cohesive soil by computing the Liquidity Index (LI). The Liquidity Index is a measure of the “softness” of the soil, where a LI of 0 and 1.0 indicate the point at which the soil is at the plastic limit and liquid limit states, respectively.

7.4 SHEAR STRENGTH

Defining the shear strength properties of the soils constituting a slope experiencing movement is a critical aspect to support monitoring, evaluation, and mitigation of landslides. Conventional characterization efforts for landslides in southwestern Pennsylvania typically involve a subsurface investigation where representative samples are retrieved and provided to a qualified laboratory for further testing. Since the laboratory testing results are reliant on the quality of the samples obtained, carefully planned extraction and handling of soil samples from within or beneath the landslide mass is important.

Determination of shear strength parameters requires a comprehensive understanding of the potential failure mode and stress state. Placement of embankment fill, versus excavation of cut slopes, will differ concerning change in the stress state; such activity should be considered during the planning stage of the laboratory investigation so that the appropriate stress states are obtained. The practitioner should also consider the effect of proposed construction methods, for example:

- Various compaction conditions can lead to completely different shear-strain behavior in fine-grained soil.
- The moisture content of fine-grained soil can have a significant effect on the maximum deviator stress.
- Static versus kneaded compaction (e.g., smooth drum versus sheepsfoot roller) can have a significant effect on the shear strain behavior of fine-grained soil.

The practitioner should be sensitive of not only the final design condition, but also the data needed to accurately define stress states during the phases of construction (i.e., short term conditions). Thus, accurate assessment of the shear strengths for site soils will involve a general understanding of probable mitigation strategies prior to implementation of the laboratory testing program.

7.4.1 Unconfined Compressive Shear Strength

Relatively undisturbed Shelby tube samples can be extruded, trimmed, and tested in the laboratory to determine the unconfined compressive shear strength of cohesive soil specimens. The shear strength test results can be applied to in-situ soil conditions for rapid (e.g., undrained) loading of fine-grained soils at the existing moisture content (i.e., degree of saturation). The quality and reliability of the test results are dependent in part on the care taken in obtaining, handling, and preparing the soil specimens.

A more simplified test that can be completed to estimate the unconfined compressive shear strength of cohesive soil is the hand penetrometer test. The hand penetrometer is small tool with a spring-loaded calibrated piston (i.e., 1/4-inch diameter rod) that is pressed into a soil sample at a constant rate to measure the consistency and unconfined undrained shear strength of cohesive soil (Figure 7-1). An adapter foot attachment is available for hand penetrometers to test very soft to soft cohesive soil. It is important to avoid any isolated areas with gravel inclusions when performing this test so that the results are not skewed. These simplified tests can be performed in the field (on split spoon samples) or in a laboratory setting.

Figure 7-1 - Field Hand Penetrometer Reading

7.4.2 Direct Shear Strength

The direct shear test is a common laboratory test for soils involved in the characterization of landslides in southwestern Pennsylvania. Although undisturbed samples are preferable, the test has the benefit of being amenable to both relatively undisturbed samples (such as a 3-inch diameter Shelby Tube) and remolded soil specimens that are prepared and compacted to a user-specified density and degree of saturation.

Direct shear strength testing involves the placement of a soil specimen within a standard-size box that is split into upper and lower halves with porous stones on the top and bottom to allow for specimen drainage [128]. To maintain undrained conditions in the sample, loading is applied at a slow uniform displacement rate to prevent the build-up of pore water pressure. During the test, the sample is allowed to consolidate under the practitioner-defined normal stress (σ_z') and soaked to saturation, followed by shearing at a constant rate until the sample fails in shear along a pre-defined plane (a-a, Figure 7-2a).

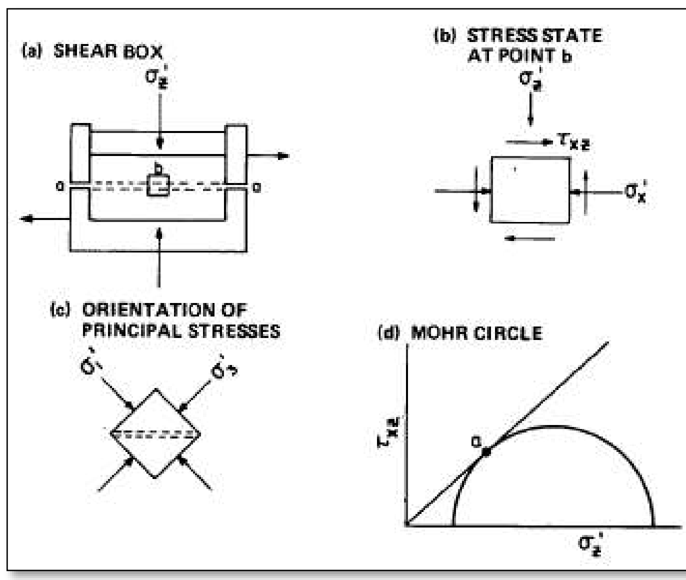


Figure 7-2 - Direct Shear Testing, Assumed Stress States, and Results Presentation [128]

During the direct shear test, both the shear stress (τ_{xz}) and horizontal displacement are measured and plotted. Failure shear stress is commonly taken as either a peak shear stress or at a specific % strain developed during the test. The rate of strain is a key consideration in performing direct shear tests to achieve a consolidated and drained condition during shearing. Multiple test specimens are sheared to represent failure at different overburden stress (see Figure 7-3). Typically, specimens are sheared at three overburden pressures (i.e., normal stress, σ_z'); the shear stress (τ_{xz}) and σ_z' for each failed test specimen is then plotted using a best-fit linear regression to approximate the Mohr-Coulomb failure envelope with corresponding friction angle and cohesion.

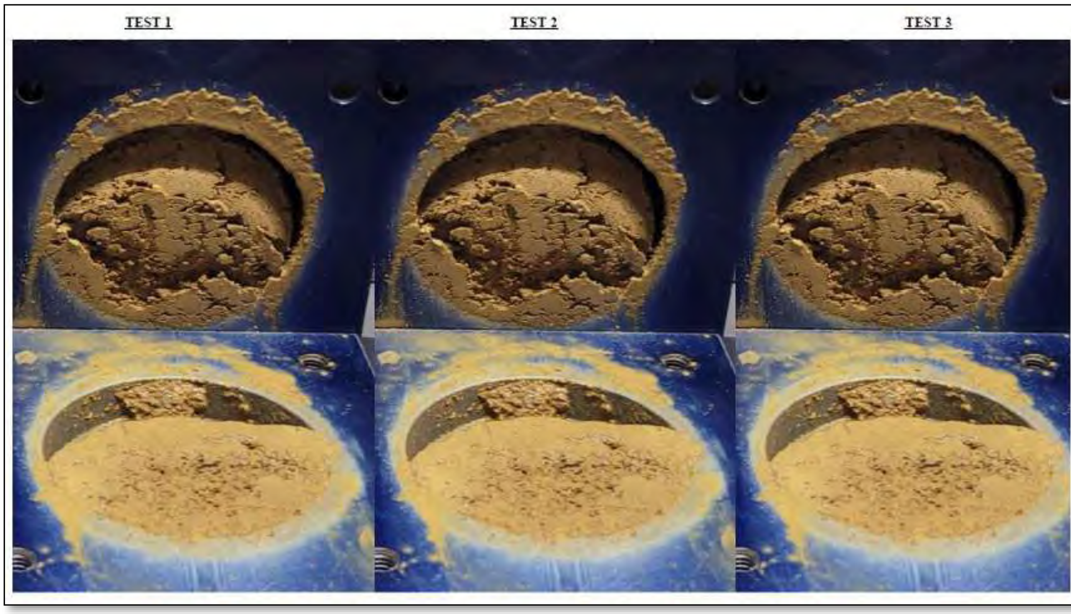


Figure 7-3 - Direct Shear Testing Specimens After Shearing to Failure

It is important to understand that although the direct shear test can be completed on both undisturbed and remolded samples, the test must be conducted under a consolidated and drained condition to provide valid results. "The test results apply to field situations where complete consolidation has occurred under the existing overburden and failure is reached slowly so that excess pore pressures are dissipated. The test is also useful in determining the shearing resistance along recognizable weak planes within the soil material. The slow rate of displacement provides for dissipation of excess pore pressures, but it also permits plastic flow of soft cohesive soils. Care should be taken that the testing conditions represent those being investigated."[3] Therefore, since the test is performed to model a fully drained condition, the cohesion should be small to none. If excessive cohesion is indicated by the test results, and the specimen is not over consolidated or cemented, then it may be an indicator that the test specimens may have been sheared too quickly (i.e., too high of a strain rate) to achieve a drained condition during testing.

Although not specifically stated in AASHTO T-236, it is important to specify AASHTO T-216 Method B so that time-deformation readings are obtained (i.e., obtain t_{50} and/or t_{90}) during the planning stage of the laboratory testing program, so that the direct shear strength test results can be properly interpreted. The swell potential is also a significant consideration that needs to be taken into account.

7.4.2.1 Practitioner Defined Testing Criteria

7.4.2.1.1 Normal Stress

The appropriate normal stress range to be applied during testing should be defined by the practitioner to reflect site specific conditions; the range of normal stress should include the approximate range of effective overburden stress acting along the rupture plane of the landslide mass. A typical range includes normal stresses equal to 0.5, 1.0, and 2.0 times the existing in-situ overburden stress to capture consolidation properties that exist both below and above the maximum past pressure of the test specimen; however, this only a general guideline and should be re-evaluated based on site specific conditions.

7.4.2.1.2 Rate and Magnitude of Displacement

Experience and knowledge about the soil type and stress history of the test specimen are needed to determine the required magnitude and rate of displacement.

The default shear deformation (10 percent of the original diameter of the shear box) may not be adequate for normally or lightly over consolidated soils. As a guide, use d_f = 12 mm (0.5 in) if the material is normally or lightly over consolidated fine-grained soil, otherwise use d_f= 5 mm (0.2 in).[4] It is also important to note that sufficient specimen displacement transpires when residual shear strength is required for slope stability analysis.

For routine testing, AASHTO T-236 offers the following guidelines to determine the rate of displacement.

$d_r = \dfrac{d_f}{t_f}$ where,

$d_r =$ rate of dispacement (inches per minute)

$d_f =$ estimated shear (lateral) deformation at maximum shear stress (inches)

$t_f =$ computed time to failure (minutes)

$t_f = 50t_{50}$ where,

$t_{50} =$ time required for the speciment to achieve 50% primary consolidation under normal force, using the log-of-time method

$t_{50} = \dfrac{t_{90}}{4.28}$ where,

$t_{90} =$ time required to achieve 90% primary consolidation under normal force, using the square root of time method

7.4.2.2 Quality of Data

Direct shear strength test results may be utilized to assign effective stress parameters given the slow rate of shear displacement applied and the ability to prevent pore pressure buildup during shearing; thereby yielding results comparable to those achieved by the more expensive, and time-consuming, consolidated-drained triaxial shear strength test. However, the results yielded by direct shear testing should be tempered with the understanding that failure is forced through a predetermined failure plane, which may not be representative of the weakest rupture plane within the soil based on the soil fabric, stress history, and critical rupture plane inclination of the landslide mass near the point of sampling [77]. Additionally, anomalous and/or high, unrepresentative results may be seen where the predetermined failure plane is forced through coarse rock fragments within the test specimen in the shear box.

The reliability and quality of the results for a direct shear test are highly dependent on understanding the nuances of the test method. For transportation projects, AASHTO Test Method T236 is typically specified to complete a direct shear test. Keys to success include, but are not limited to:

- Sample quality, transport, and preservation.
- Sample extrusion, preparation, and trimming with focus on maintaining the in-situ moisture content, structure, and density.
- Reasonable normal stresses were defined considering site specific conditions.
- Confirmation that calibration disks were used to complete the test.

- Confirmation that the test specimen was inundated in the shear box and permitted to completely swell before the commencement of shearing.
- Accurate measurement of the rate of consolidation to determine the time required to either reach 50 or 90 percent primary consolidation under normal force, to assure that a slow enough shearing rate is used to test the specimen.
- Confirmation that the two halves of the shear box are separated slightly during shearing to eliminate frictional forces from the equipment.
- Specimen shearing is continued until the shear stress becomes essentially constant; or until shear deformation is at least equal to 10 percent of the original diameter of the shear box (e.g., 0.25 inches for a 2.5-inch diameter shear box) or to the practitioner specified shear deformation.
- Verification of the degree of saturation for the sheared specimen at the completion of the test.
- Assessment and interpretation of the test results to discern the appropriate failure criterion are used.
- Equipment calibration and experienced personnel are needed to plan, execute and interpret the direct shear strength test results.

This means that quality results demand proper planning, execution, and interpretation of results to assure that representative test specimens are properly prepared, moisture conditioned, and inundated in a calibrated shearing device, specimens are allowed to swell completely before shearing, and the soil type and stress history of the soil is considered to determine a satisfactory displacement rate. Close communication with the laboratory staff is generally needed through the testing program.

7.4.2.3 Determination of Parameters

Direct shear tests should be complemented with index tests, Shelby tube bulk density, and natural moisture content determinations. Remnants of the sample trimming are commonly used to determine index properties and natural moisture content. These complimentary tests will aid in the determination of shear strength parameters for the assessment of slope stability.

Further discussion about direct shear strength testing is presented in ASTM Special Technical Publications (STP) 131, STP 361, and STP 740.

The practitioner should note that the direct shear test generally provides a representation of the peak shear strength of a soil specimen during shearing. However, at many landslides in southwestern PA, the shear strength of the soil is governed by prior movement and subsequent shear strength reduction from a peak value to a lower residual value. As discussed previously in this section, it is important that the shear displacement be specified by the practitioner if residual strength values are to be obtained through direct shear testing.

7.4.3 Triaxial Shear Strength Testing

Triaxial shear strength testing provides greater versatility than simple and direct shear strength testing methods by allowing for control of drainage conditions and stress states. This permits the practitioner to specify testing parameters that are representative of the in-situ conditions acting along the critical rupture plane within the landslide. In the triaxial shear test, either an undisturbed or remolded soil specimen is prepared and placed within a thin membrane inside a triaxial cell. The cell is subsequently filled with fluid to apply a specified cell pressure (σ_3). Back-pressure can be applied to fully saturate the soil specimen before shearing. Control of cell pressure (σ_3) and deviator stress ($\sigma_1 - \sigma_3$) and measurement of pore pressure can be used to replicate anticipated loading conditions; whether that be the addition of surcharge load during shearing (e.g., fill placement), removal of surcharge load during shearing (i.e., excavation) or maintenance of similar surcharge load during shearing (e.g., failure of existing slope). Refer to Figure 7-4a for a depiction of a typical test setup, a stress condition diagram for the prepared soil specimen, and typical representation of stress-strain results.

Multiple failure criteria should be considered to interpret the results. Mohr-coulomb failure criterion that is typically considered includes the maximum total stress obliquity ($\frac{\sigma_1}{\sigma_3}$), maximum deviator stress ($\sigma_1 - \sigma_3$), and maximum shear stress at 15 percent strain. When pore pressure measurements are obtained, it is common practice to also determine the maximum effective stress obliquity ($\frac{\sigma_{1-u}}{\sigma_3 - u}$).

Triaxial shear strength tests are typically performed on soil specimens that have been back-saturated before shearing. The three types of drainage conditions typically performed for triaxial testing are the consolidated-drained test, consolidated-undrained test, and unconsolidated-undrained test. Selection of appropriate loading and drainage conditions for the triaxial shear strength test should be made by the practitioner depending on the testing objectives, loading condition(s), and governing soil conditions.

Due to the cost, and time, associated with triaxial testing, these tests are less common for landslide investigations in the region compared to than more simplified tests such as direct shear; therefore, a brief description of the conditions prevailing during each of these tests and results yielded follows below.

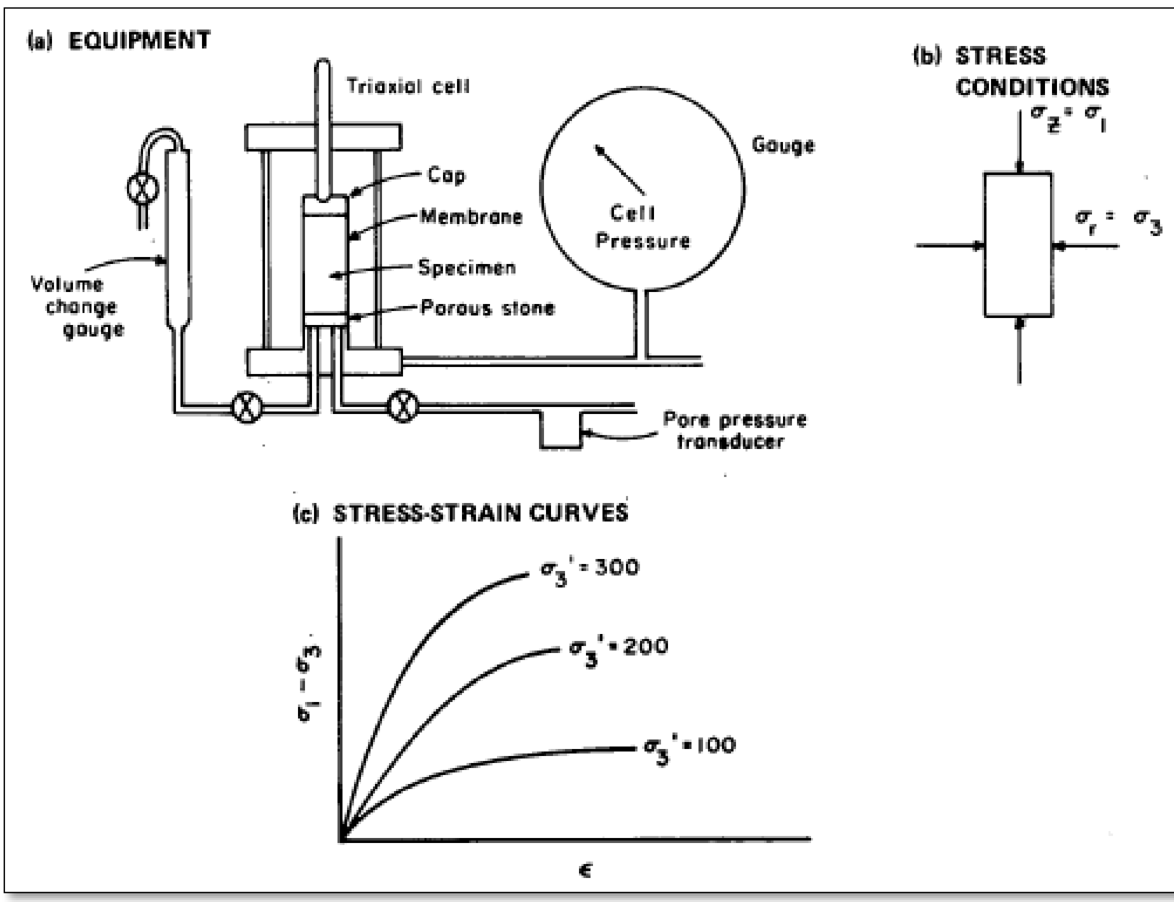


Figure 7-4 - Triaxial Shear Testing [128]

7.4.3.1 Consolidated-Drained Test

In the consolidated-drained (CD) test (e.g., ASTM D7181), test specimens are consolidated and sheared in compression at a slow strain rate to maintain a drained condition during shearing. This test is limited in practical application for landslides in southwestern Pennsylvania due to the fine-grained nature of the majority of soils involved in landslide activity. A consolidated-drained test of a single lean clay test specimen could require several weeks to perform; hence, CD triaxial shear strength tests are not usually performed to investigate landslides in southwestern PA.

Figure 7-5 - Test Specimen After Consolidated-Undrained Testing

7.4.3.2 Consolidated-Undrained Test

In the consolidated-undrained test (e.g., ASTM D4767), test specimens are isotropically consolidated and sheared in compression without drainage at a constant rate of strain. Back-pressure can be applied to fully saturate the test specimen before shearing. Pore pressure measurements can be obtained to determine effective shear strength parameters. The consolidated-undrained test has the advantage over the consolidated-drained test in that the test may be performed much quicker considering the sample can be saturated before shearing and pore water pressure can be measured during shearing. As a side note, the AASHTO T297 Standard Method of Test for Consolidated Undrained Triaxial Compression Test on Cohesive Soils was inactive as of May 17, 2022.

7.4.3.3 Unconsolidated-Undrained Test

In the unconsolidated-undrained test (e.g., AASHTO T296), a cohesive soil test specimen is subjected to a confining cell pressure (without drainage) and failed in axial compression at a constant strain rate. Interpretation of the results of this test is taken to yield an undrained shear strength (S_u) at the degree of saturation at which the sample was tested (since no drainage is permitted during the test).

7.4.4 Selection of the Number and Type of Shear Strength Tests

The practitioner is often faced with the dilemma of justifying the cost of performing site specific shear strength tests in the laboratory and using index properties to estimate shear strength parameters. This dilemma has challenged engineers in the past and will continue to challenge engineers in the future. It is imperative that laboratory testing as well as any other investigation activity be purpose-driven to balance risk and consequence with allowance for plausible variance.

An instance of plausible variance is the classic relationship between the effective peak friction angle and plasticity index for cohesive soil. Many practitioners will estimate shear strengths using average values provided by available correlations. For example, in Figure 7-6, the practitioner may use a vertical line that passes through the Plasticity Index, and then where that vertical line intersects the solid averaging line in the plot, extend a horizontal line to predict an estimated peak friction angle for analysis. However, the practitioner is cautioned to look deeper at Figure 7-6 and take into account the data scatter with a plausible range of peak shear strengths for a given plasticity index. Experience with local soils is a key factor in the final selection of shear strength parameters for analysis. Site-specific laboratory test data can be and is used to estimate shear strength parameters with a higher degree of confidence, which can then be used to assess the reliability and cost-benefit factors.

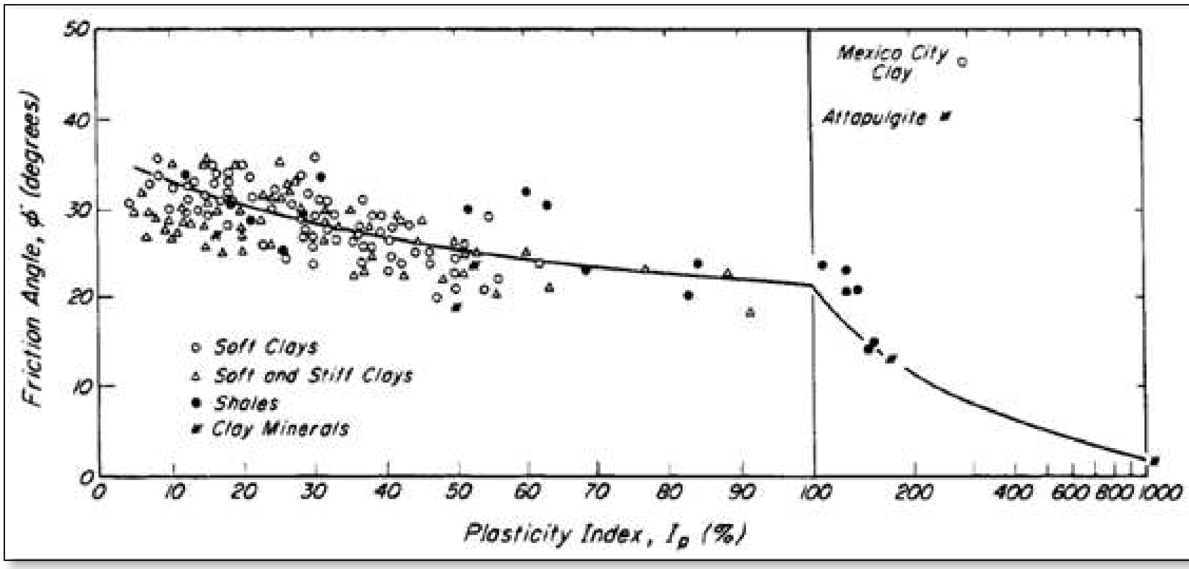


Figure 7-6 - Peak Effective Friction Angle as a Function of Plasticity Index [126]

7.5 DATA VERIFICATION

Data verification is an important step to complete a laboratory investigation. Data obtained should be integrated with results from the desktop study, field reconnaissance, and field investigation to formulate a holistic understanding of the subsurface setting.

The practitioner should conduct a critical review of the data obtained and perform a "reality check" to:

1. Conduct a quality assessment review to interpret the test results and identify possible errors,
2. Screen the data to identify possible data outliers,
3. Assess whether or not the results are reasonably correct, and
4. Determine if the laboratory test results make sense for the intended purpose.

After the laboratory test results are validated, it is typical practice to use the laboratory test results in combination with the field logs to prepare the final boring and test pit logs to document the results of the field and laboratory investigation. The final logs are then used as the basis for landslide analysis and design.

The first step in the data verification process is to interpret the test results and identify possible errors, which can then incite discussion with the laboratory for a deeper understanding of the test results. The following key factors should be considered for shear strength test results:

- Influence of sampling procedure.
- Condition of test specimens.
- Testing procedures.
- Side-by-side comparison of results for similar types and conditions of the material to identify trend lines and possible test outliers.

For strength testing, the following additional key factors should be considered:

- Specimen preparation, sealing, transport, and preservation.
- Specimen handling at the laboratory to maintain as-sampled moisture condition.
- Specimen preservation.
- Method of stress application.
- Re-saturation of test specimen(s) in the laboratory.
- Method of pore water pressure measurement.
- Strain rate.
- Failure criterion.

Further discussion about these key factors is presented in ASTM STP 361.[5]

Detailed shear strength test results are needed to interpret the results and identify possible errors. It is imperative that the practitioner have a fundamental understanding of the test methods employed to properly interpret the results. As a minimum, the practitioner should read the detailed testing procedure and insist that all of the available data be reported with the test results. Many of the test methods require the practitioner to define a specific procedure within a test method during the planning stage; consultation and coordination with the testing agency are advised so that the planning stage is complete to maximize the benefit of the laboratory investigation. It cannot be overemphasized that error identification requires an in-depth review of the laboratory test results; for instance, the A and B parameters are good indicators and need to be reviewed for each incremental reading that is recorded for a triaxial compression test.

The engineer should define the failure criterion and not rely on the testing laboratory since the criterion used needs to reflect the mode of failure of concern. Both stress and strain-based failure criterion exist; it is common practice to use a stress-based failure criterion for landslide investigations. Further discussion about the failure criterion is mentioned earlier in this chapter.

7.6 SELECTION OF SHEAR STRENGTH PARAMETERS

Shear strength parameters should be based on the ground conditions, evidence of prior ground movement, data from the laboratory test results, and other relevant data. Such data may include a back-analysis of an active landslide taking into account existing slope geometry, stratigraphy and groundwater conditions, correlation to index properties, field observations, and published data. Referencing local research such as Hamel [66][70][71] is a valuable resource to assess the reasonability of the data acquired and selection of parameters.

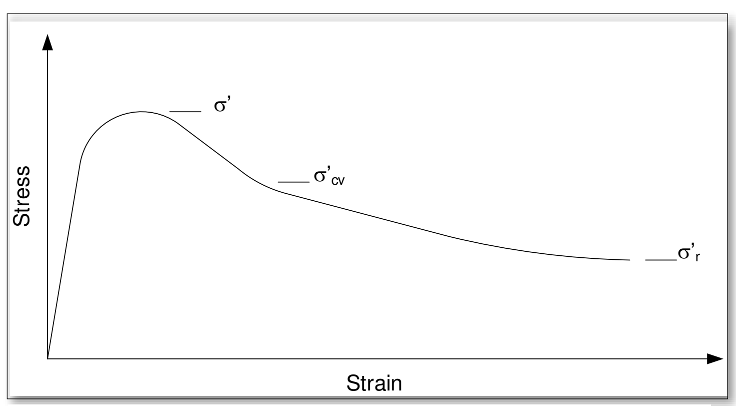


Figure 7-7 - Shear Strength Versus of Strain
(function of soil type, condition & mode of failure)

Parameter selection should consider the difference between the boundary conditions and test procedures used to measure shear strength in the laboratory and the actual conditions present in the field that govern the mode of failure of the landslide mass.

Allowance for possible soil dilation (e.g., volumetric expansion during shearing) should be considered. If soil dilation is a factor under plane strain conditions (e.g., no volumetric expansion normal to the direction of slope movement), then consideration should be given to using the critical state shear strength (i.e., σ'_{cv} and c'_{cv}) in place of the peak shear strength (i.e., σ' and c'). Strain softening and progressive failure of cohesive soils need to be considered to discern whether or not residual shear strength (i.e., σ'_r and c'_r) need to be considered to select appropriate shear strength parameters for analysis and design. Figure 7-7 provides a graphic illustration of typical plastic clay that might be involved.

Figure 7-8 provides a methodical thought process that the practitioner can use as a guide to determine shear strength parameters for landslide analysis and design.

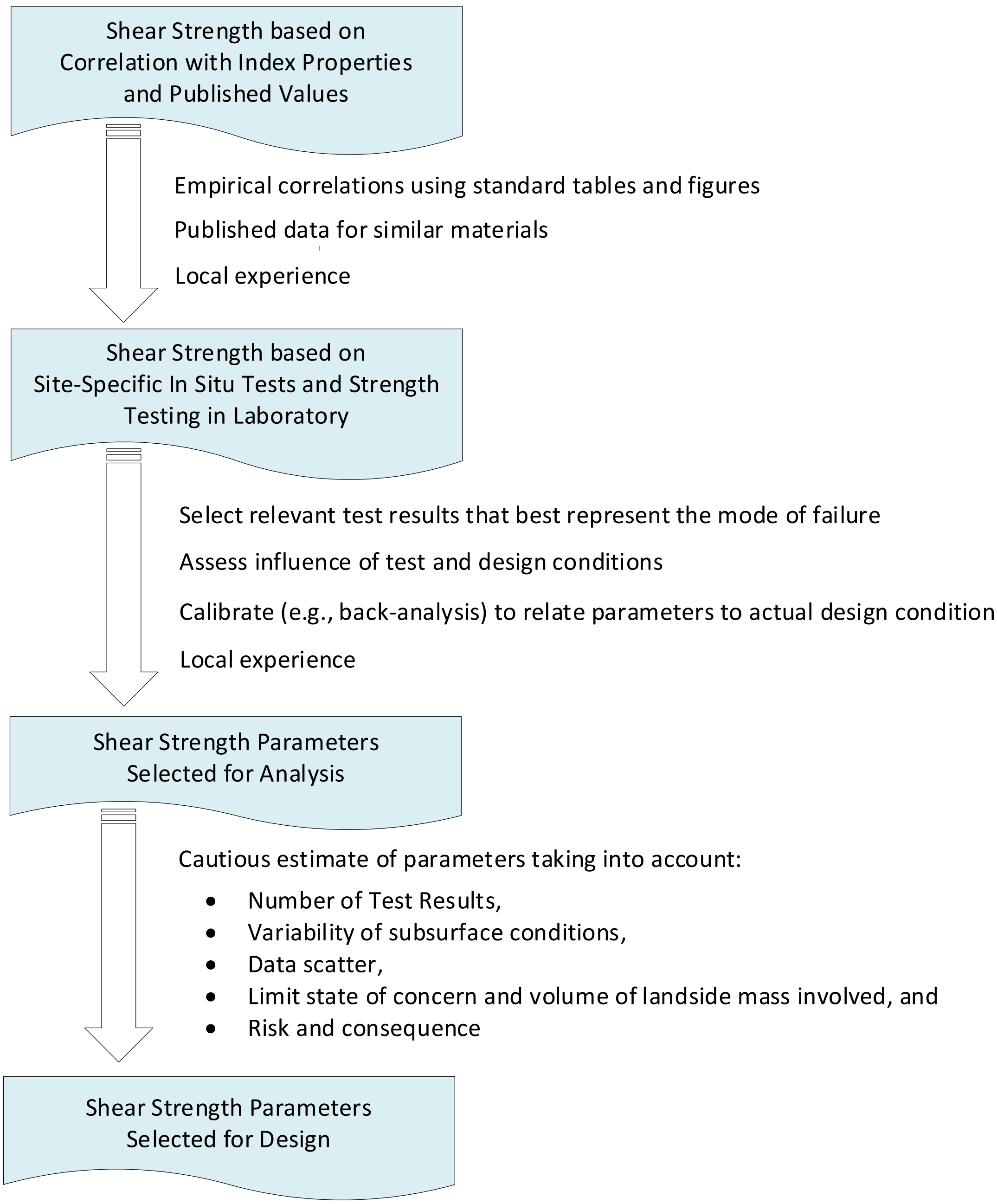


Figure 7-8 - Determination of Shear Strength for Analysis and Design [17]

CHAPTER 8

Slope Maintenance Best Practices

8.1 INSPECTION AND MAINTENANCE

Consistent and effective slope maintenance offers a proactive rather than reactive solution which can reduce risk and inconvenience to the public and minimize overall cost when compared to slope stabilization and repair methods. The lack of regularly conducted slope maintenance is a significant contributing factor to the high rate of landslide occurrences in southwestern PA.

The groundwork of an effective slope maintenance program consists of regular inspections by qualified individuals and a timely response to mitigate/repair unsatisfactory conditions that are identified. Additionally, adequate reporting and tracking of slope monitoring and maintenance activity is key to tracking and assessing potential risks to slope stability.

It may be prudent and beneficial for inspections to coincide with the wet seasons or anticipated significant precipitation events to allow for a more thorough inspection of surface drainage features, subsurface drainage facilities, and erosion prevention measures. Ideally, monitoring activities should be performed so that any necessary maintenance can be implemented before the next season when landslide activity is more prevalent. In southwestern PA, a seasonal increase in landslide activity is typically observed during late winter (early spring) snowmelt or elevated precipitation in the fall.

Factors affecting decision-making concerning appropriate regularity of slope monitoring and corresponding maintenance activities include:

- Previously established requirements set by an agency/owner/government entity for slopes within a particular jurisdiction.
- The uncertainty of underlying geologic and groundwater conditions.
- The importance of a slope concerning consequences resulting from slope displacement.
- Availability of budget to fund monitoring and maintenance activity.
- Age and condition of slope features, and drainage features.

8.2 MAINTENANCE SUMMARY

See below for a table of common deficiencies found during slope maintenance operations and the relative priority and cost of the remediation efforts. Further discussion on maintenance operations regarding the categories listed below is presented in the following sections.

Category	Deficiency	Potential Remediation Effort	Priority Level	Relative Cost
Surface Drainage	Clogged Drainage Ditches	Hand clear excessive debris or large obstructions; perform washout activities to remove excessive siltation buildup in the rock voids; install barrier devices to limit amount of siltation and/or debris able to enter the surface drainage systems	Medium	$-$$
	Excessive vegetation overgrowth	Clear excessive overgrowth within drainage channels; reline with fabric if necessary to discourage future growth	Low	$
	Drainage gullies have formed on the slope	Redirect surface water to existing surface drainage ditches or create new surface drainage to intercept drainage pathways and covey water away from slope then regrade and revegetate slope	Moderate	$$
Subsurface Drainage	Clogged Drain Pipes	Hand clear debris or washout line to restore positive drainage; install E&S controls as necessary to limit future blockages	High	$
	Damaged Drainage (i.e., cracked or separated)	Repair or replace the drainage system with new conduit and/or connections	High	$$
Subgrade Drainage	Seepage is observed	Report immediately; if seepage causes potential stability issues; mitigation (i.e., additional drainage) may be required	High	$$-$$$
Surface Maintenance/ Erosion Control	Material buildup of containment structures (i.e., slide fences, rockfall fences, catchment walls)	Clear buildup at containment structures	Low to Moderate	$$
	Bare soil or visible surface erosion	Regrade and replace or install vegetation	Low	$
	Depressions or areas of ponded water are observed	Regrade to establish positive surficial flow	Moderate to High	$$
	Cracked, rutted, or damaged slope surfaces are observed	Regrade to establish positive surficial flow and/or seal cracks	Low to moderate	$-$$

Category	Deficiency	Potential Remediation Effort	Priority Level	Relative Cost
	Tension cracks or surficial failures observed	Report immediately; obtain evaluation from a geotechnical engineer	High	--
Surcharge Loading	Unapproved material stockpiles, equipment, and/or structures observed near crest of slope	Report immediately; additional surcharge should be removed or evaluation from a geotechnical engineer may be obtained if surcharge loading is to remain in place	High	--
Toe Support Loss	Drainage channel erosion contributing to material loss at the toe of a slope	Relocate the drainage channel from the area of the slope toe or reconstruct the drainage channel with scour-resistant material (i.e. riprap)	High	$$-$$$
	Natural waterways are contributing to material loss at the toe of a slope	Relocate the waterway away from the slope toe or place slope armoring materials such as crushed stone riprap, segmental block walls, or equivalent remedial measures	High	$$-$$$

Table 8-1 - Summary of Common Maintenance Practices

8.3 DRAINAGE MAINTENANCE

The presence of water, either as uncontrolled surface runoff or in the form of groundwater and excess pore water pressure, is a principal contributing factor to slope movement and triggering mechanisms as discussed in Chapter 2. Providing and maintaining proper surface and subsurface drainage is a primary component of slope maintenance. Consideration for monitoring and maintenance of drainage systems and additional best practices in slope maintenance are presented below. See Chapter 10 for additional information regarding available drainage features.

8.3.1 Surface Drainage

Surface runoff channels, swales, interceptor ditches, rivulets, as well as perennial and intermittent seeps and springs should be kept unobstructed and operational year-round in diverting uncontrolled surface drainage away from the slope and mitigate against water ponding on any portion of the slope. All drainage channels and ditches should be clear of obstructions, excessive vegetation overgrowth, and debris; additionally, any aggregate used as part of the drainage system should be free draining and unclogged from accumulated siltation.

When performing cleanout activities for surface drainage features, maintenance personnel should avoid unnecessary over excavation of the underlying material, particularly along the toe and lower portions of the slope; over excavating and changing existing grade at the toe of slope can result in a reduction in slope stability. Regular cleanouts are effective in maintaining these features, however preventive measures can also be implemented to avoid repetitive maintenance efforts including check dams to intercept siltation and debris and providing filter fabric at the base of drainage features to discourage vegetation overgrowth. See Section 8.5 for further discussion on best practices.

Figure 8-1 - Example of Surface Drainage Maintenance Operations

8.3.2 Subsurface Drainage

Free drainage from weep holes and drainpipe outlets should be maintained at all times to prevent uncontrolled backup of flow and the potential for saturation of subsurface soils, which could lead to a buildup of excess pore pressure or excessive erosion.

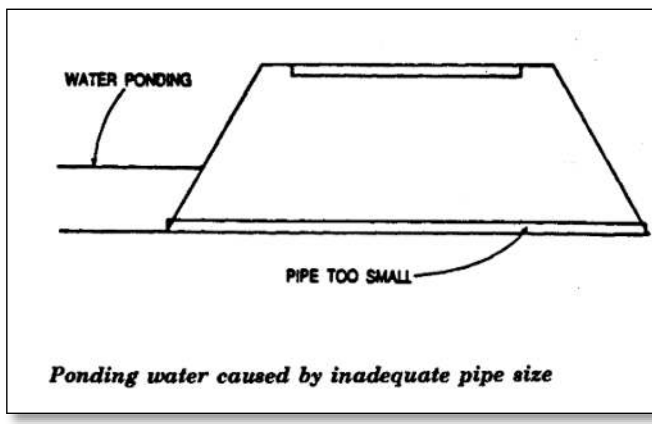


Figure 8-2 – Inadequate Drainage [49]

Inspect drainage pipes for cracks and separated joints which can cause eroded subgrade. Assess drainage inlets for water backup, which could be indicative of obstructed flow or inadequate pipe size; in the case that clogs or obstructions are observed, they should be documented and removed to promote unobstructed flow. If the drainage system is allowing water to back up on the slope or leak into the subgrade soil as a result of design or material deficiencies, immediate action to repair, replace, or upgrade the drainage system should be made. Regular cleanout of subsurface drainage systems is recommended to maximize performance.

Due to the concealed nature of subsurface drainage systems, it is important to mark the outlets of subsurface drainage systems with some form of permanent identification marker so that they can be easily identified in the field for monitoring and protection against inadvertent burial. Cleanouts for surface drainage systems should also be visibly marked. In addition, subsurface drainage systems should be clearly depicted regarding location, type, depth, and orientation on as-built records for future reference.

Figure 8-3 - Separated subsurface drainage pipe leading to erosion of the subgrade

Figure 8-4 - Separated underdrain causing excessive slope erosion leading to a landslide which comprised the shoulder of a roadway (PennDOT District 11)

Figure 8-5 - Stones obstructing drainpipe flow

8.3.3 Subgrade Drainage

In addition to inspection and maintenance of preexisting drainage systems, it is important to manage in situ drainage patterns along the slope as well. During inspection the technician should identify indications of saturated slope conditions and/or changing drainage patterns including:

- Areas of vegetation known to thrive in saturated conditions. Examples of these plants in southwestern Pennsylvania include, but are not limited to cattails, Japanese knotweed, skunk cabbage, and briars.
- Observations of slope seepage with special attention to recent seeps not previously observed.

If these features are identified, potential remedial measures can be taken by maintenance staff including installation of drains to intercept seepage and relieve any potential pore water pressure buildup. For shallow failures where surficial seepage is of concern, seepage interceptor drains and finger drains are options which require only minor earthwork and stone material (see Figure 8-6). These drains consist of a shallow excavated trenches lined with geotextile and backfilled with coarse aggregate. See Section 10.3.2.2 for additional detail.

Figure 8-6 - Seepage Interceptor Drains Installed

Figure 8-7 - Seepage observed on slope

8.4 SURFACE MAINTENANCE/EROSION CONTROL

A brief description of typical surface maintenance activity is provided below:

- Assess erosion control elements including ditches, slope paving, rip rap, vegetation, and culvert headwalls for signs of distress. Water bars, dikes, and berms should be maintained and assessed for debris buildup or washout so that they remain effective in redirecting surface water flow and reducing surface erosion. Remove buildup and resurface the erosion control features as needed for maintenance.
- Containment structures (i.e., slide fences, rockfall fences, catchment walls) should be clear of material buildup; remove the material as needed to confirm the containment structures can operate as designed. Minimum height of the structure needs to be maintained or slides/rockfalls may be able to overtop the structures.
- For areas exhibiting bare soil, vegetation should be installed or replaced to reduce the risk of further erosion (see Figure 8-10). The root systems of grass are effective for keeping soils in place.
- Visually confirm positive surface grades and identify isolated depressed areas or areas of stagnant, ponded water (see Figure 8-9). Where depressions or areas of ponded water are observed, maintenance activity should be immediately undertaken to drain ponded water and/or perform earthwork grading to reestablish positive surficial flow away from the slope.
- Evidence of cracked, rutted, or damaged slope surfaces should be sealed or regraded so that water cannot collect in the depressions and infiltrate into the soils.
- If shallow/surficial failures or visible surface erosion are observed, it is important to direct any drainage features or structures away from the slide area.
- For observed shallow/surficial failures or excessively hummocky ground causing concern for long term stability, additional maintenance measures could be implemented to stabilize the surficial soils such as installation of geoweb (see Figure 8-11).


Figure 8-8 - Rip Rap protection at drain outlet (left)
Figure 8-9 - Water Ponded at Toe, Bare spots observed on slope (right)

Figure 8-10 - The value of seeding/vegetation on a slope: the left (seeded) section shows almost no erosion; right side rills are quickly becoming gullies [82]

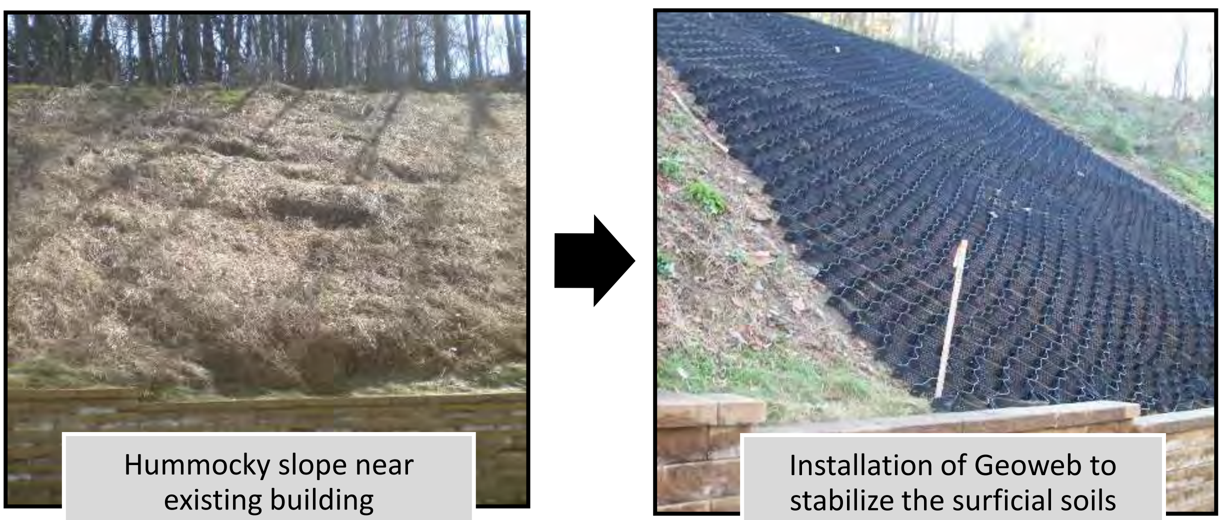


Figure 8-11 - Geoweb installation

8.5 CONTROL OF SURCHARGE LOADING

Good practice for slope maintenance further involves prohibiting activities whose effects will negatively impact the overall equilibrium of the slope. It is important that no excess loading or unapproved surcharges are placed near the crest of the slope as added surcharge loading increases driving forces and decreases overall slope stability. Specific aspects of this concept are summarized below:

- Maintenance activities performed on the slope should avoid stockpiling materials near the top of slope.
- Planned activities involving the application of surcharge loads at the top of slope should be evaluated by a qualified engineer in advance of the planned work to assess risk to the overall slope equilibrium. Any unauthorized surcharge loads should be removed immediately.
- Excessive patching of localized depressed surface areas due to slope failure is not advised; the additional surcharge weight will act as an additional surcharge load and worsen the slope condition(s); in these situations, the slope movement should be mitigated so that further patching is not required (Figure 8-12).

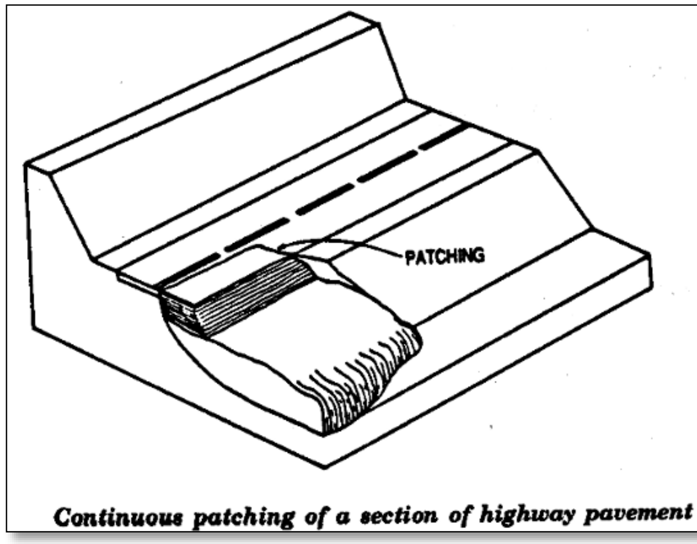


Figure 8-12 - Example of Excess Surcharge Loading [49]

- When structures are proposed at or near the existing slope, slope stability should be evaluated by a qualified engineer.

8.6 PREVENTION OF TOE SUPPORT LOSS

Corresponding to the control of surcharge loading described above, slope maintenance activities should further encompass the prohibition or careful control of activities resulting in a loss of support at the toe of a slope. While not entirely exclusive, these events commonly consist of mechanical excavation for construction-related purposes or natural erosion taking place near the toe of slope (Figure 8-13). Specific aspects of this concept are summarized below:

- Work performed near the toe of slope should avoid excessive removal of material from the area immediately beyond the toe of slope and/or the lower portion of the slope itself.
- Similar to planned placement of surcharge loads, a qualified engineer should be engaged to evaluate the magnitude of planned material removal and assess risk to the overall slope equilibrium. Unauthorized removal of slope toe materials should immediately be replaced and compacted in-kind.
- Where erosion from drainage channels or natural waterways are contributing to material loss at the toe of a slope, appropriate measures should be undertaken to either relocate the drainage channel or waterway away from the slope toe, reconstruction of the drainage channel with scour-resistant material, or placement of slope armoring materials such as crushed stone riprap, gabion baskets, segmental block walls, or equivalent remedial measures.

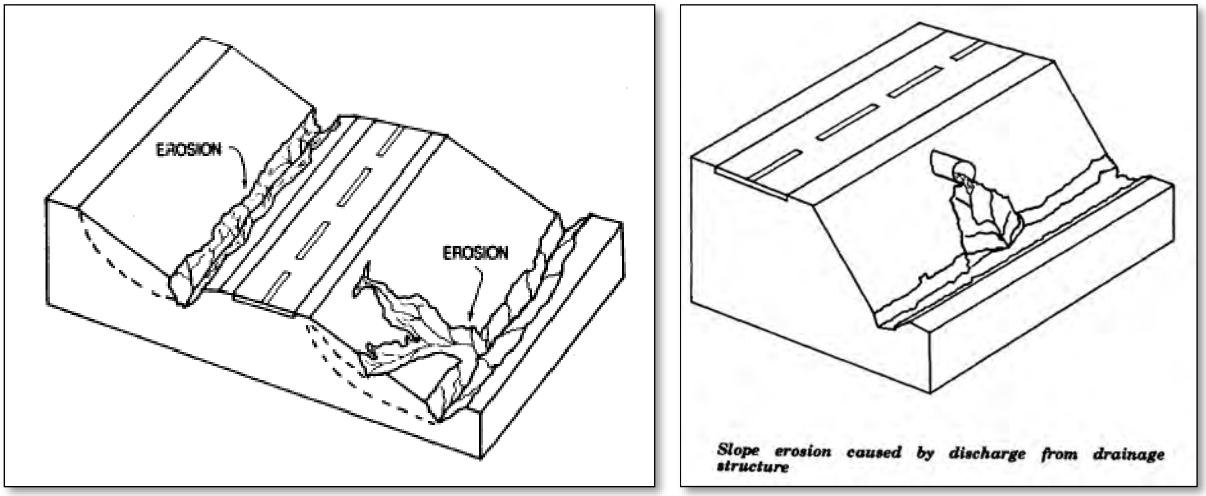


Figure 8-13 - Example of Toe Support Loss [49]

8.7 INFRASTRUCTURE

Assessment of site infrastructure can give insight to slope creep that may not otherwise be visible to maintenance staff. Identification of structural irregularities such as tension cracks in pavement, leaning or sagging guiderails, and/or wall cracks in nearby structures can indicate slope movement. In some cases, maintenance efforts may be able to address movement such as when it is surficial or drainage related; for example, sealing open tension cracks may be implemented to restrict infiltration of surface water to the subgrade (see Section 10.3.2.1 for further information). In other cases, evaluation of a qualified engineer may be warranted. Maintenance staff should report observations of infrastructure related deficiencies that are indicative of slope movement immediately.

Figure 8-14 - Tension cracks on pavement observed (left), leaning guiderail (right) (PennDOT District 11)

8.8 REPORTING AND DOCUMENTATION

Proactive response to slope maintenance requires considerable planning, especially when managing many landslide-prone slopes; therefore, prompt and adequate reporting of slope monitoring and maintenance activity is important to track and assess potential risk to slope stability. At a minimum, maintenance reporting should provide sketches, visual observations about surface and drainage features, photographs, and maintenance activities recommended and/or performed as a result of observed deficiencies. The benefit of archiving and tracking maintenance reports include:

- Documentation on the progression of slope movement.
- Documentation of vital information and historical data to establish priority and allocation of funds for slope repair(s).
- Documentation of historical data and observations which could help inform the engineer if a landslide were to occur.
- Documentation to aid in identifying the triggering cause(s) of potential slope movement.

Selection of appropriate reporting and documentation practices will vary case-by-case by agency, owner, and entity, depending on budgetary considerations and availability of personnel to perform the required tasks. However, a lack of documentation for slope maintenance activities, as described above, may lead to a lack of available information and records to help ascertain problematic versus low-risk slopes. The consequence is difficulty in making decisions regarding which slopes are at increased risk of slope failure and may require more frequent monitoring and maintenance activity.

See below for a tabular summary of the slope feature of interest during maintenance inspections, the available documentation methods, and the estimated documentation effort. See Chapter 4 for additional information on UAS and LiDAR. See Appendix A.3 for an example slope maintenance checklist which can serve as a template of the pertinent information to be collected during site inspections.

Category	Feature	Documentation Method	Documentation Effort
Surface Drainage	Drainage Channels	• field survey • UAS survey	• visual observations including measurements (size of cracks of surface depressions) • ground/air photos
Subsurface Drainage	Drainage Systems	• field survey	• visual observations • downhole camera to document discontinuities
Subgrade Drainage	Seeps	• field survey • UAS survey	• visual observations • measurements (size, flow rate) • sketches • ground/air photos
Surface Maintenance/ Erosion Control	Surface deformation and erosion	• field survey • UAS survey • LiDAR	• visual observations • measurements (size of cracks or surface depressions, changes in grade) • sketches • ground/air photos

Category	Feature	Documentation Method	Documentation Effort
Infrastructure	Structural integrity of roadways and homes	• field survey	• visual observations • measurements (displacement, tilt, cracking) • ground photos
Surcharge Loading	Excess Loading at Slope Crest	• field survey • UAS survey • LiDAR	• visual observations including approximate size and type of loading • sketches • ground/air photos
Toe Support Loss	Erosion at toe from natural waterways or surface drainage	• field survey • UAS survey • LiDAR	• visual observations • measurements (approximate volume of toe loss) • sketches • ground/air photos

Table 8-2 - Summary of Potential Documentation and Reporting Efforts

8.9 BEST PRACTICES FOR DRAINAGE AND EROSION CONTROL

The practitioner should refer the Pennsylvania Stormwater Best Management Practices Manual 363-0300-002 for best practices on stormwater management. Utilizing best practices such as level spreaders, erosion control matting, outlet stabilization and check dams can also be used to protect drainage features and minimize ongoing maintenance.

CHAPTER 9

Slope Management Systems

9.1 DEFINITION, PURPOSE, AND SCOPE

9.1.1 Definition

The breadth and magnitude of data associated with landslides and landslide-prone slopes, provides a unique challenge to the owner, agency, or governmental entity desiring to collect, store, manage, disseminate, and utilize the information for planning purposes. This data serves as the basis for assessing the probability and consequences associated with a landslide, preparing preliminary-level feasibility and conceptual alternative studies for landslide mitigation, and providing justification for procurement and allocation of funding for design and construction to either proactively address landslides and landslide-prone slopes or response measures once landslide activity has already commenced.

A Slope Management System may take on a variety of forms and the manner and frequency with which the information is collected and organized, depending on:

- The intended information usage and outcome.
- The nature of the infrastructure to which the system is applied (typically roadway transportation),
- Availability of funding and personnel to establish and maintain the system.
- The type of entity establishing the system (governmental, private, academia, etc.).
- The frequency and severity of landslides for the particular area or region to which the system is applied.
- The factors that most likely influence landslide occurrence in a given region (e.g., topography, lithology, vegetation, disturbance, climate, etc.).
- The type(s) and resolution (temporal and spatial) of data available to analyze and monitor a given area.

For this discussion, Slope Management Systems are broadly defined as a set of policies and procedures that are established for the maintenance and function of a civil infrastructure network to systematically organize a data set about landslides and landslide-prone slopes.

9.1.2 Purpose

Ultimately, the purpose of the Slope Management System needs to recognize the elements and the consequence of landslide risk(s) and develop a strategy to contend with such landslide risk(s). Elements at risk include people, property, transportation links (e.g., roads), public utilities, and natural resources (e.g., disruption of a watercourse). The consequence of landslide risk includes, but is not limited to, damage, injury and or loss of life, reduction and or loss of functional usage, direct or indirect cost, indirect consequential cost (e.g., litigation), and adverse social and environmental impact.

9.1.3 Scope

A fundamental working premise that forms the foundation for Slope Management System(s) is the hypothesis that similar landslides in similar material are caused by similar processes acting under similar conditions. This premise asserts that “natural slope failures in the future will most likely be in geologic, geomorphic and hydrologic situations that have led to past and present failures…we have the possibility to estimate the style, frequency of occurrence, extent, and consequences of failures that may occur in the future.”[28]

The scope of the Slope Management System needs to define functional constraints to be effective. Some examples of functional constraints include defining the areal limits of interest, clarification of what aspects the system will consider (e.g., property loss or damage, life safety, disruption of public services, etc.), extent and nature of the investigation phase, type of risk analysis and assessment, stakeholders involved, operational and financial constraints, legal responsibility and obligation, and basis for assessment of acceptable and tolerable risk. [11]

A targeted understanding of the mode of failure, plausible causal factor(s), and triggering cause(s) is important to the evaluation of landslide risk (Section 9.2.1.1), vulnerability (Section 9.2.1.2), and uncertainty (Section 9.2.1.3). Refer to Chapter 2 for a detailed discussion of slope movement mechanisms.

Inherently, a thoughtful Slope Management System is a key component in the overall landslide analysis, assessment, evaluation, management, and reduction process.

9.2 CAUSAL FACTOR(S), TRIGGERING CAUSE(S), AND REMEDIAL OPTIONS

9.2.1 Causal Factor(s) and Triggering Cause(s)

Landslide hazard is the probability that a first-time slope failure or an active landslide reaches a given rate of movement, and is linked to the point at which the triggering factor(s) reach(es) a threshold, beyond which slope failure occurs. For pre-failure and reactivation stages, the hazard that is associated with a rate of movement is related to the probability that the aggravating factor reaches a given value leading to this rate. For the post-failure stage, the hazard to a given rate of movement is very much governed by the materials involved and the predisposition factors and thus is more difficult to define [115]. Figure 9-1 presents an illustration of the interrelationship of causal factors and triggering cause. Figure 9-1 also illustrates how landslide movement that has been arrested can be reactivated by a triggering cause that was believed to cause only a small change in the existing factor of safety against global stability (e.g., seasonal fluctuation in groundwater during a period of elevated precipitation).

Determination of the “cause” of a landslide is not always essential to an accurate solution to a landslide problem and is secondary in importance to the understanding of the mechanics of the movement.[12]

Ground Causes
- Weak materials
- Sensitive materials
- Weathered materials
- Sheared materials
- Jointed or fissured materials
- Adversely-oriented, mass discontinuity (bedding, schistosity)
- Adversely-oriented, structural discontinuity (fault, unconformity, contact)
- Contrast in permeability
- Contrast in stiffness (stiff materials over plastic soils)

Geomorphological Causes
- Tectonic uplift
- Glacial rebound
- Fluvial erosion of the slope toe
- Wave erosion of the slope toe
- Glacial erosion of the slope toe
- Erosion of the lateral margins
- Subterranean erosion (solution, piping)
- Deposition loading the slope or its crest
- Vegetation removal (by forest fire, drought)

Physical Causes
- Intense rainfall
- Rapid snow melt
- Prolonged exceptional precipitation
- Rapid drawdown (of floods and tides)
- Earthquake
- Ice damming
- Thawing
- Freeze and thaw weathering
- Shrink and swell weathering

Artificial Causes
- Excavation of the slope or its toe
- Embankment failures though fill material
- Loading of the slope or its crest
- Drawdown (of reservoirs)
- Deforestation
- Irrigation
- Mining
- Artificial vibration
- Water leakage from utilities
- Defective surface drainage
- Dumping of loose materials

Table 9-1 - Checklist of Landslide Causes [28]

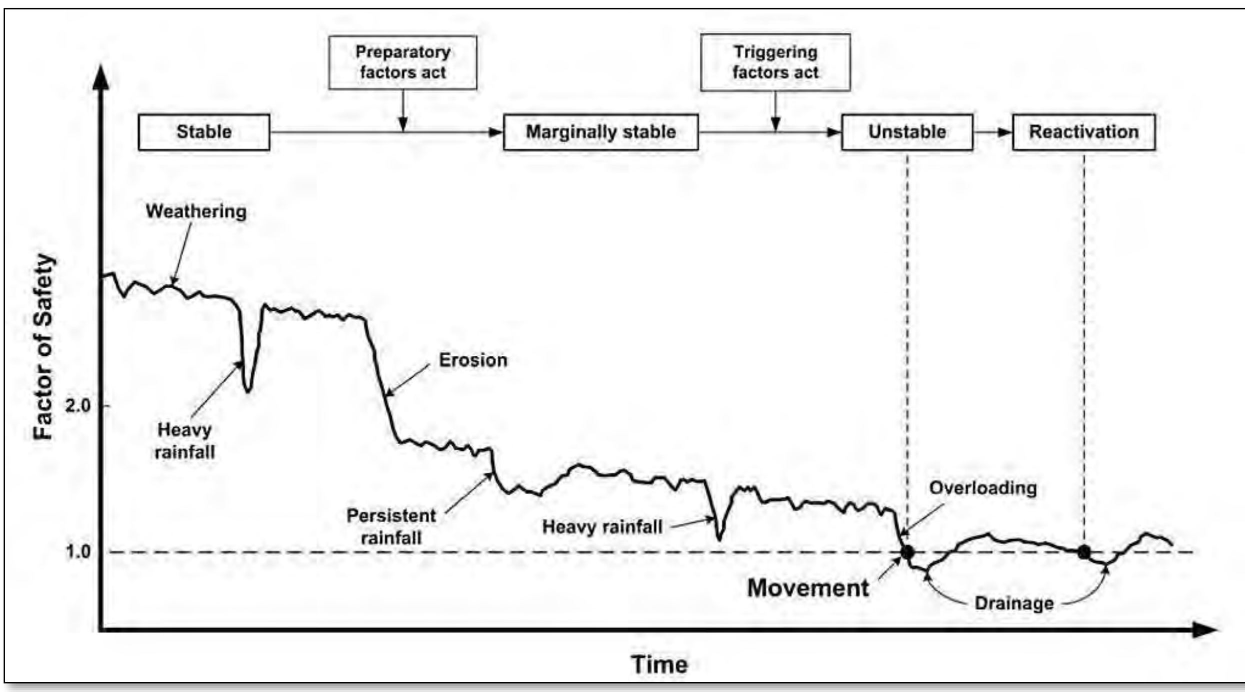


Figure 9-1 - Example of change in factor of safety with time, indicating some causal factors [27]

9.2.1.1 Landslide Risk

Consideration of risk is intrinsically related to landslide best practice. As Arthur Casagrande summarized in the Second Terzaghi Lecture [18] in 1964, engineering wisdom dictates that any attempt to mitigate adverse impact must take into account possible risk. Casagrande broke down design risks into two broad categories, namely human risk and engineering risk. Human risk entails unsatisfactory organization and division of responsibility, unsatisfactory use of available knowledge and judgment, and corruption. Engineering risks include both unknowns and calculated risk. Casagrande stated that “calculated risk” entails two distinct steps:

- "The use of imperfect knowledge, guided by judgment and experience, to estimate the probable ranges for all pertinent quantities that enter into the solution of a problem."
- "The decision on an appropriate margin of safety, or degree of risk, taking into consideration economic factors and the magnitude of losses that would result from failure."

This wisdom cannot be understated in the decision-making process to mitigate landslide hazard.

9.2.1.2 Landslide Vulnerability

For risk assessment, Landslide Vulnerability is defined as the degree of loss to a given feature (e.g., roadway, utilities, structures, environmental controls, etc.) or set of features within the area that is affected by the hazard. Landslide Vulnerability also includes a set of conditions and processes from physical, social, economic, and environmental factors, which increase the susceptibility of a community to the impact of the hazard(s).[38]

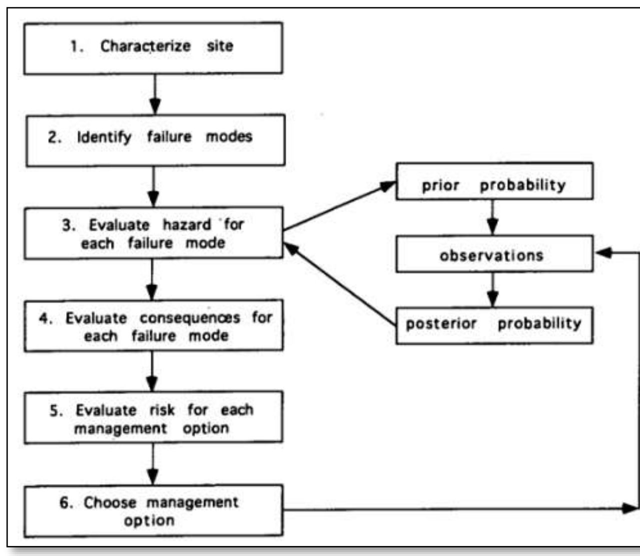


Figure 9-2 - Decision Making Process to Account for Landslide Mitigation Uncertainty [28]

9.2.1.3 Landslide Uncertainty

There is inherent uncertainty in the landslide risk analysis, assessment, and evaluation process. This requires recognition of and prioritization of characteristic features and potential failure modes to arrive at a preferred approach to managing landslide risk. It is important to appreciate the effect of changing assumptions and the resulting sensitivity. It is important to recognize and make known limitations and uncertainty when arriving at conclusions about the course of action moving forward.[11]

Examples of uncertainty to assess landslides include, but are not limited to, an accurate determination of the groundwater conditions, shear strength, evidence of prior slope movement, spatial variability, subsurface setting, and future loading.[128]

Figure 9-2 illustrates a typical progressive decision-making process to account for uncertainty.

9.3 LANDSLIDE RISK MANAGEMENT PROCESS

As mentioned at the beginning of this chapter, the Slope Management System is key to the overall landslide risk analysis, assessment, evaluation, management, and reduction process. Figure 9-4, Figure 9-3, and Figure 9-5 provide a visual representation of the overall risk management process.[38]

The design intent herein is to provide a brief overview of a logical process upon which a logical outcome can be accomplished. Based on the outcome, decisions can be made to arrive at a feasible, practical, and cost-effective solution to mitigate landslide hazard. For further details about this topic, refer to the following resources.

- Australian Geomechanics Society, Sub-committee on Landslide Risk Management, *Landslide Risk Management Concepts and Guidelines*
- Fell, R., State of the Art Paper 1, *A Framework for Landslide Risk Assessment and Management*
- Popescu, M. *Landslide Causal Factors and Landslide Remediation Options*
- Washington State Department of Transportation, *Landslide Mitigation Action Plan*

Figure 9-3 depicts the five (5) primary phases of the risk management process.[38]

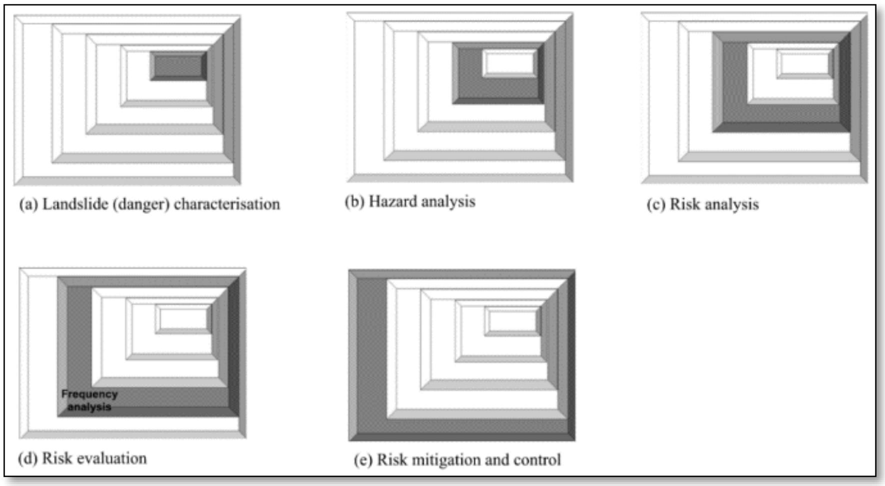


Figure 9-3 - The Five (5) Primary Phases of the Risk Management Process [38]

1) Initially, landslide characterization is typically performed which involves the slope locations and general mechanics of movement.
2) The hazard analysis builds upon the landslide characterization to predict the frequency of occurrence.
3) The Risk Analysis includes an assessment of the landslide hazard and its consequence.
4) Risk Assessment takes the output from risk analysis and assesses the results of the risk analysis against value judgments, and risk acceptance criteria. The outcomes of the risk assessment will be either: a) the risks are tolerable, or even acceptable, and no mitigation options need be considered, or b) the risks are intolerable, and risk mitigation options need to be considered.
5) Risk Management takes the output from the risk assessment, and considers risk mitigation, including accepting the risk, reducing the likelihood, reducing consequences (e.g., by developing monitoring, warning, and evacuation plans), transferring risk (e.g., insurance), or developing a risk mitigation plan and possibly implementing regulatory controls. This step also includes monitoring risk outcomes, feedback, and iteration when needed. It is an integral part of risk management that the estimated risks are compared to acceptance criteria (either quantitative or qualitative).

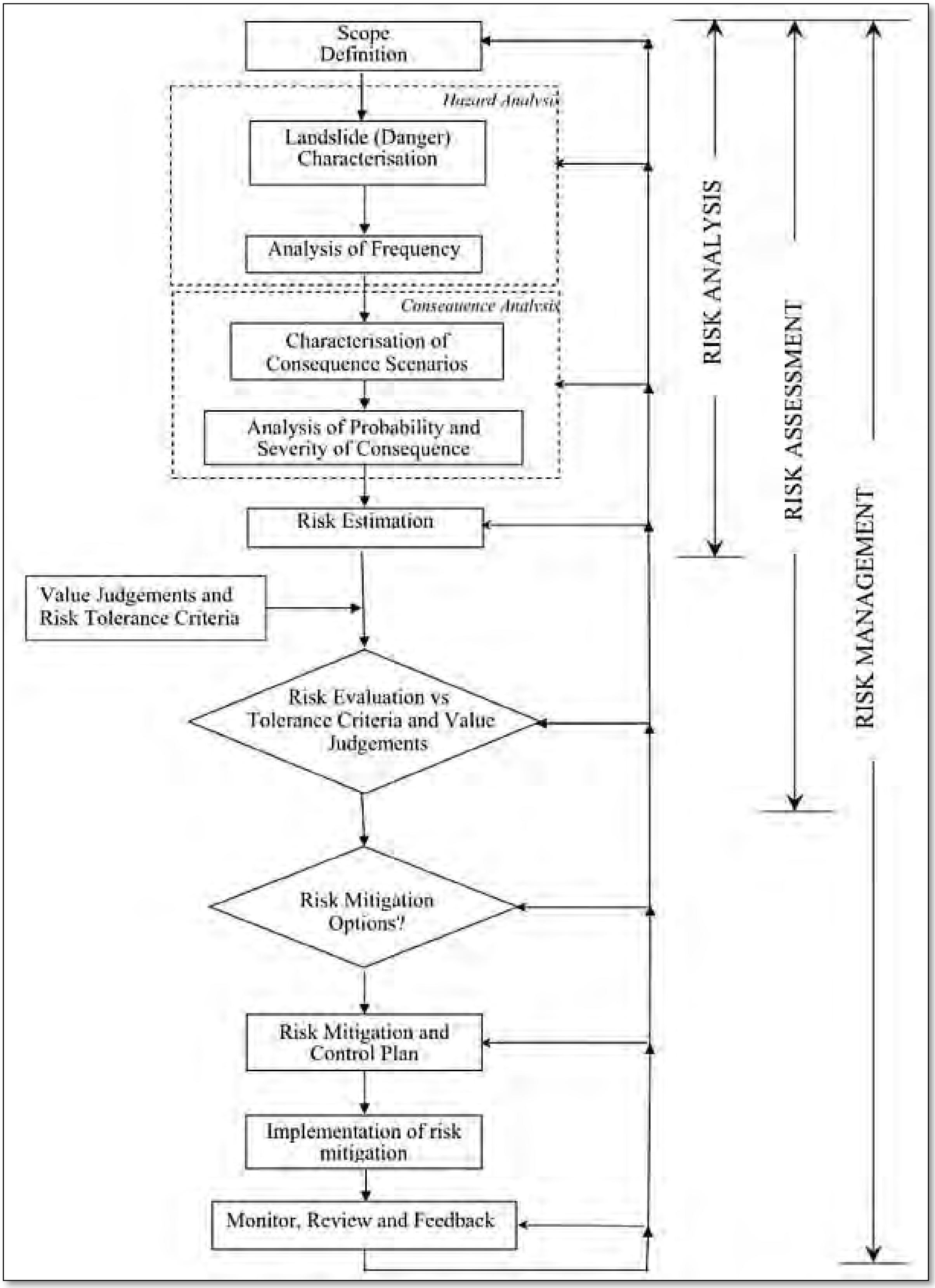


Figure 9-4 - Flowchart for Landslide Risk Management [38]

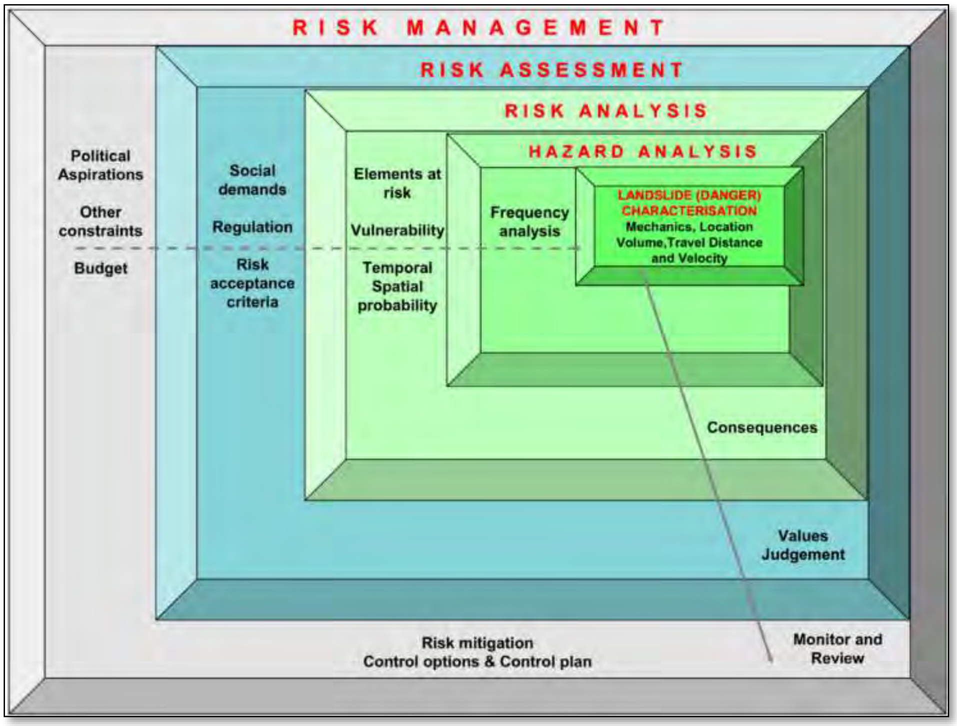


Figure 9-5 - Schematic Representation of the Integrated Risk Management Process [38]

9.3.1 Factor of Safety Criteria

A factor of safety of unity (1.0) represents a state of equilibrium, upon which the driving forces are counterbalanced by the available resisting forces. This applies to both shear and moment equilibrium. The most widely used and most generally useful simplified definition of a factor of safety (FS) for slope stability [34] is

$$FS = \frac{\text{shear strength of the soil}}{\text{shear stress required for equilibrium}} \qquad \textit{Equation 9- 1}$$

Uncertainty about shear strength needs to be accounted for to provide a minimum margin of safety. What this means, is that if all other factors are exactly as anticipated (although oftentimes they are not) and a FS of 1.5 is considered for slope stability, then the practitioner would essentially be attesting to the confidence that the actual effective friction angle (ϕ') is equal to or greater than $\tan^{-1}$ ($\tan \phi'$/FS). For example, considering a FS of 1.5 and soil with an assumed $\phi' = 30°$, the actual effective friction angle must be equal to or greater than 21.1° (= $\tan^{-1}$ (tan(30)/1.5). It is important to note that this would apply to the soils that are present over the portion of the rupture plane that is performing the majority of the work to provide shearing resistance. This approach assumes that there is no other change (e.g., groundwater fluctuation, site disturbance, the addition of a surcharge load, etc.); this is a simplistic example, however, it does reinforce the relationship between uncertainty and FS.

Drs. Duncan and Wright suggested the factor of safety should be proportionate to the uncertainty of analysis conditions and the significance of the consequence associated with slope failure. The recommended factors of safety by Duncan and Wright are presented in Table 9-2 as a guide for decisions about cost and consequence when it comes to the degree of uncertainty.

See Section 9.6.4 for further discussion on suggested “best practice” for assessing risk tolerance and defining factors of safety.

Cost and consequences of slope failure	Uncertainty of Analysis Conditions	
	Small *	Large **
The cost of repair is comparable to the incremental cost to construct a more conservatively designed slope	FS = 1.25	FS = 1.5
The cost of repair is much greater than the incremental cost to construct a more conservatively designed slope	FS = 1.5	FS = 2.0 or greater
* The uncertainty regarding analysis conditions is smallest when the geologic setting is well understood, the soil conditions are uniform, and thorough investigations provide a consistent, complete, and logical picture of conditions at the site. ** The uncertainty regarding analysis conditions is largest when the geologic setting is complex and poorly understood, soil conditions vary sharply from one location to another, and investigations do not provide a consistent and reliable picture of conditions at the site.		

Table 9-2 - Cost and Consequence Considerations Versus Uncertainty to Assess Required Minimum Factor of Safety [34]

9.4 EVALUATING POTENTIAL FOR SLOPE MOVEMENT AT A GIVEN SITE

During the early stages of the Landslide Risk Management Process, the practitioner needs to formulate a qualitative assessment to identify slopes that should receive a more in-depth study of landslide potential.

Dr. Adams developed an empirical model to evaluate the potential for slope movement based on an inventory of approximately 720 slope movements, with a detailed desktop study and site reconnaissance occurring at 220 locations. Input was solicited from 18 experts on slope movement in Allegheny County, which rated variables that may contribute to slope movement. Fourteen slope movement variables (SMV) were established and rated on a scale of 1 to 9 by the experts, where a rating of 5 or greater was the value at which the ratings began to indicate elevated importance in the slope movement process. The rating results (Figure 9-6) were used to define a Slope Movement Stability Rating (SMSR) which was considered to be representative of a potential slope failure. For each SMV in the SMSR equation, a value between 0 to 1 was chosen; where a value of 0 represents the absence of the variable's influence and 1 represents the full presence of the variable's influence. Equation 9.4-1 represents the recommended SMSR, where the bolded SMVs were found to be the most critical in the slope-movement process in Allegheny County. The SMSR was calculated for 25 slope movements that were field visited during the study; the results of the study are presented in Figure 9-7.[8] The conceptual model developed by Dr. Adams helps to identify the critical variables contributing to slope failures within Allegheny County and similar geologic settings in Southwestern PA.

The SMSR system can be used to assess the addition of a new causal factor (i.e., SMV) and develop a qualitative sense of how much of a reduction in relative stability rating that that addition might yield. A graphic example of how compounding causal factors can eventually trigger an active landslide is depicted in Figure 9-1.

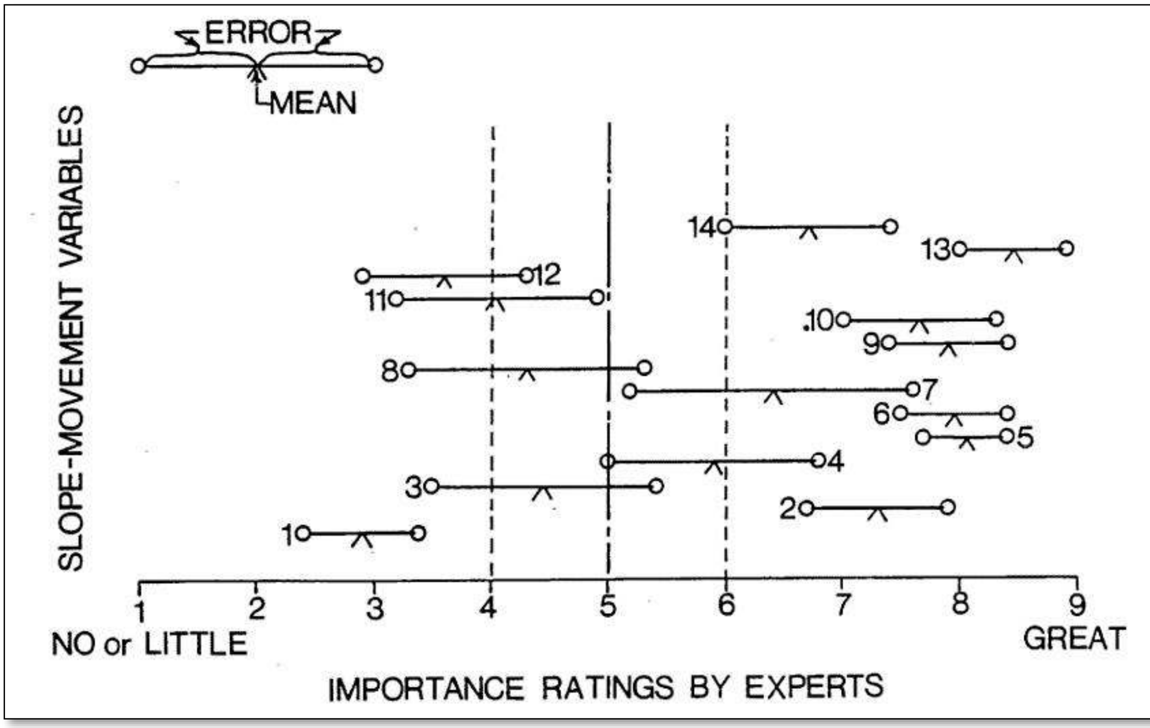


Figure 9-6 - Importance Ratings by Experts for Slope Movements at Landslide Prone Areas in Allegheny County, Pennsylvania [8]

SMSR = 7.3*SMV(2) + 4.5*SMV(3) + 5.9*SMV(4) + 8.1*SMV(5) + 7.9*SMV(6) + 6.4*SMV(7) + 7.9*SMV(9) + 7.7*SMV(10) + 8.5*SMV(13) + 6.7*SMV(14)

where,
SMV(1) – Orientation of slope face
SMV(2) – Gradient (e.g., steepness, inclination) of the ground surface, existing & final
SMV(3) – Position on the slope
SMV(4) – Slope configuration
SMV(5) – Surcharge on the slope
SMV(6) – Removal of lateral support (e.g., removal of an existing retaining wall downslope)
SMV(7) – Removal of underlying support (e.g., scour)
SMV(8) – Lateral pressure
SMV(9) – Previous movement
SMV(10) – Stratigraphic unit(s)
SMV(11) – Attitude (e.g., overdip) of bedding
SMV(12) – Bedrock discontinuities
SMV(13) – Increase in water content and/or change in groundwater level or pressure
SMV(14) – Inappropriate human actions

* **BOLD** indicates the parameter was included in the SMSR equation
** Each BPG variable is numerically represented as,
1 = Yes. The factor does exist and applies to the slope being considered.
0 = No. The factor does not exist or does not apply to the slope being considered.

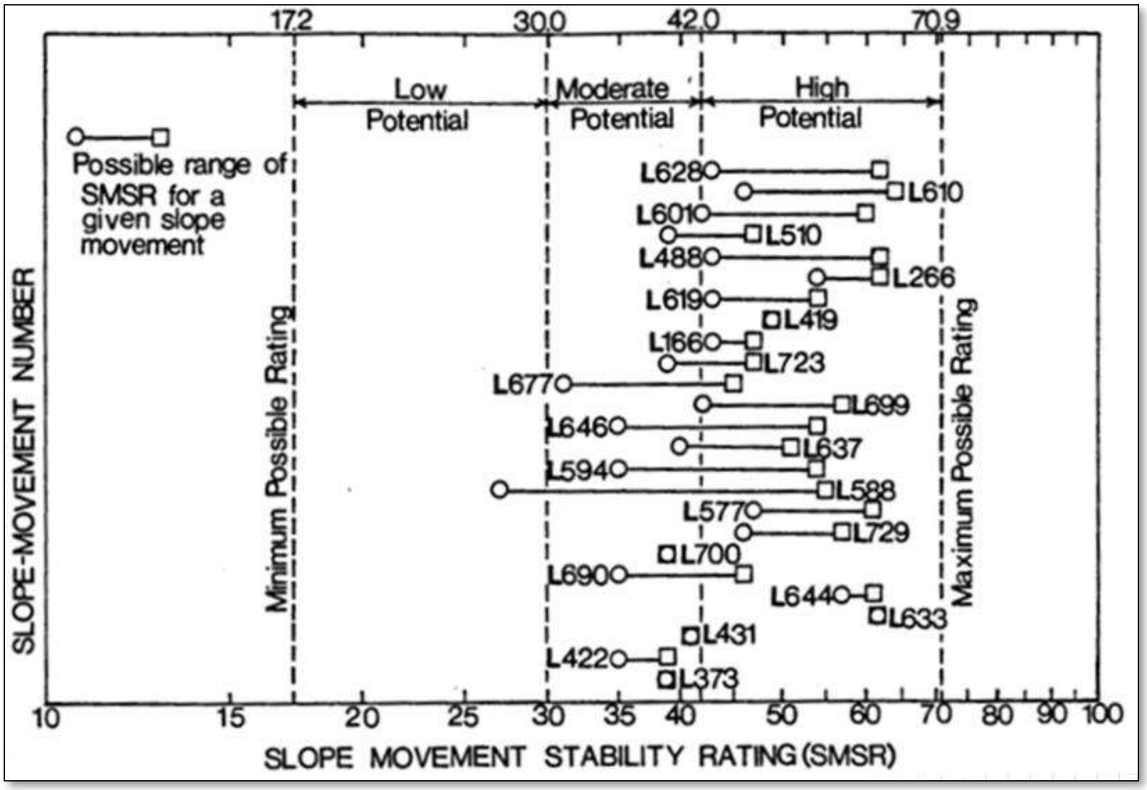


Figure 9-7 - Example Application of SMSR for 25 slope locations [8]

9.5 HAZARD RATING

Hazard rating is not unique to southwestern PA. Hazard ratings are utilized worldwide by international agencies, governmental entities, academic institutions, and private owners for managing data concerning landslides and landslide-prone slopes.

There are differences, particularly in the level of detail and data quality, in the hazard rating systems used in southwestern Pennsylvania and worldwide as presented in Table 9-3. This is understandable, given that agency or entity has been responsible to create unique Hazard Rating systems that would yield meaningful results.

To determine a “best practice” Hazard Rating (Section 9.6.2) and Threat Tolerance (Section 9.6.3) for use in southwestern Pennsylvania, several currently utilized Hazard Rating Systems were studied and compared to identify parameters of significance within the region.

No.	Parameter	PennDOT D-11	PennDOT D-12	Allegheny County	Oregon DOT	Ohio DOT	NYS DOT	FHWA FLM	NC DOT	Hong Kong	WS DOT
1	Slope Height						X			×	
	Height of Failure	X									
2	Slope gradient									×	
3	Slide mass/volume						X				
4	ADT	X	X		X	X	X	X			X

No.	Parameter	PennDOT D-11	PennDOT D-12	Allegheny County	Oregon DOT	Ohio DOT	NYS DOT	FHWA FLM	NC DOT	Hong Kong	WS DOT
	Traffic speed					X	X				X
	Avg. vehicle risk = f(ADT, failure length & speed limit)								X		
5	Highway classification		X		X				X		X
6	Detour delay (e.g., time, travel length, repair time)	X	X			X	X		X	X	X
7	Geohazard cause	X					X			X	
	Failure type and volume							X	X		
8	Depth to void / mined seam	X									
9	Evidence of H2O seepage						X		X	X	X
	Rainfall sensitivity								X		
10	Impact on road structure and adjacent features	X	X	X	X	X		X	X	X	X
	Length of roadway impact	X						X			
	Surrounding area impact	X	X								
	Landlock potential		X								
	Community impact			X							
	Usage impact							X			
	Potential future impact					X	X			X	X
11	Change in slide since the last visit			X							
	Scarp displacement					X					X
12	Expected damage					X	X				X
13	Annual maintenance cost					X			X		X
	Maintenance frequency				X				X		
	Previous mitigation								X		
14	Failure incidence					X	X			X	
	Rockfall history							X			
15	Benefit-cost factor				X	X					X

No.	Parameter	PennDOT D-11	PennDOT D-12	Allegheny County	Oregon DOT	Ohio DOT	NYS DOT	FHWA FLM	NC DOT	Hong Kong	WS DOT
16	Expected no. of landslide fatalities for a given facility					X	X				X
17	Decision sight distance					X	X				X
18	Risk to vehicle					X	X				X
	Accident history				X	X					X
19	Relative emergency					X					
20	Slide/erosion effects							X			
21	Rockfall ditch effectiveness							X			

Table 9-3 - Comparison of Several Hazard Rating Systems

9.6 SUGGESTED "BEST PRACTICE" FOR SOUTHWESTERN PA

In the development of the suggested "best practice" for slope management, lessons learned and experience played a key role in developing a practical, clear, useful and usable "best practice" metric. Guidance from others who are subject matter experts about landslide mitigation in southwestern Pennsylvania was also taken into consideration. An attempt was made to make a clear distinction between the potential for landsliding (i.e., risk) and consequence (e.g., impact on the public).

The following are some of the key lessons learned about the regional practice used to craft a suggested "best practice" for slope management.

1. A clear understanding of the plausible mode of failure is more important than the details.
2. Landslide risk, vulnerability, and uncertainty are fundamental components of the Hazard Rating that need to be acknowledged.
3. Causal factors need to be clearly defined and weighted to arrive at a correct understanding of potential slope movement at a specific site. As the number of primary causal factors increases at a specific site, the probability of possible slope movement will increase.
4. Simplified Hazard Rating systems focused on the primary causal factors may be the most effective.
5. Quantity of data is not equal to the quality of data.

The Hazard Rating, which is used to determine Threat Tolerance, should be flexible to make allowance for engineering judgment. The suggested "best practice" to determine the Landslide Hazard Rating and Threat Tolerance is presented in Sections 9.6.2 and 9.6.3, respectively.

After the Hazard Rating and Threat Tolerance is established, risk tolerance and an acceptable factor of safety should be determined to decide what type of remedial option is applicable. Further discussion about risk tolerance and factors of safety are presented in Section 9.6.4.

A decision will be needed to discern whether an emergency response or planned improvement is appropriate for the remedial option of choice. Discussion about the suggested "best practice" for the remedial option of choice is presented in Section 9.6.5. Discussion about "best practice" for emergency response and planned improvement is presented in Section 9.6.7.

9.6.1 “Best Practice” to Determine Stability Rating

A sample “best practice” stability rating to evaluate slope movement potential (e.g., risk) based upon the work of Dr. Adams (Figure 9-7) was derived based on subjective judgment that was rendered by a panel of experts on slope movement in Allegheny County. Only the variables adopted as part of the SMSR equation (Section 9.4) have been included in the “best practice” rating system.

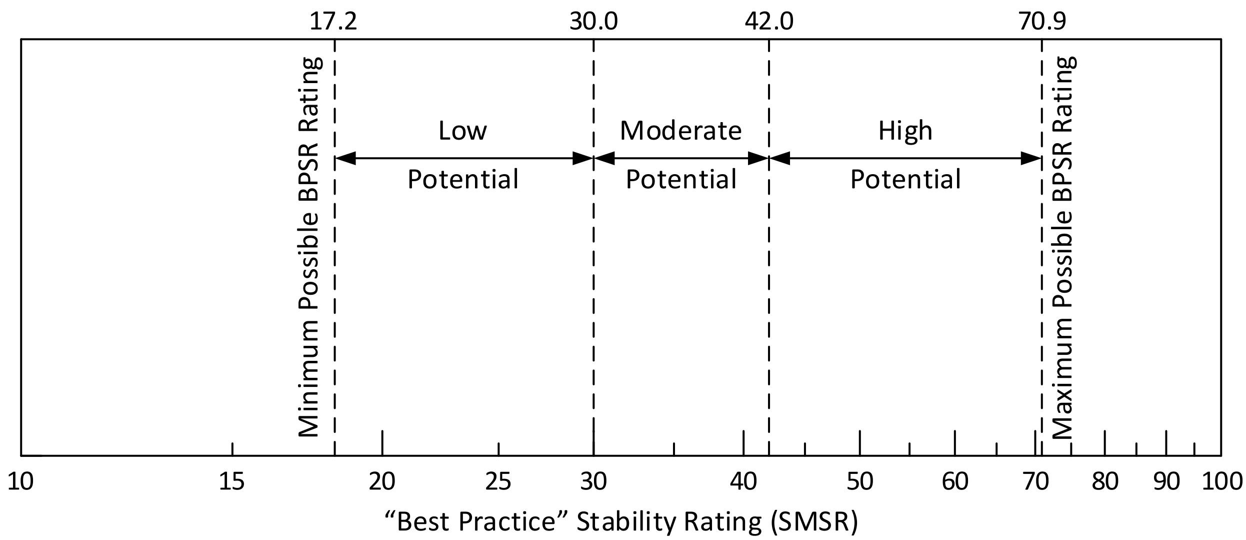


Figure 9-8 - Predicted Slope Movement Potential as a Function of the SMSR

Best Practice Slope Movement Stability Rating (SMSR) = 7.3*BPG1 + 4.5*BPG2 + 5.9*BPG3 + 8.1*BPG4 + 7.9*BPG5 + 6.4*BPG6 + 7.9*BPG7 + 7.7*BPG8 + 8.5*BPG9 + 6.7*BPG10

“Best Practice” Geohazards (BPG) include:
BPG1 = Slope Gradient (e.g., steepness, inclination) of the ground surface, existing & final
BPG2 = Position on the slope
BPG3 = Slope configuration
BPG4 = Surcharge load added on top of the existing slope
BPG5 = Removal of lateral support (e.g., removal of an existing retaining wall downslope)
BPG6 = Removal of underlying support (e.g., scour)
BPG7 = Evidence of prior slope movement
BPG8 = Proximity of landslide-prone stratigraphic unit
BPG9 = Increase in water content and/or change in groundwater level or pressure
BPG10 = Inappropriate human action
Where each BPG variables is numerically represented as,
1 = Yes. The factor does exist and applies to the slope that is being considered.
0 = No. The factor does not exist or does not apply to the slope that is being considered.

9.6.2 “Best Practice” to Determine Hazard Rating

A “best practice” hazard rating system was determined based on an evaluation of currently utilized Hazard Rating systems in the southwestern Pennsylvania geologic region, as well as other rating systems that reflect practice trends for landslide mitigation worldwide.

Key lessons learned were applied to identify the following key consequence and risk factors for the development of the "best practice" hazard rating.

Consequence Factors:

1. Road Classification
2. Roadway Impact
3. Average Daily Traffic (ADT)
4. Detour Length
5. Length of Roadway Impacted
6. Height of Failure Above Roadway
7. Surrounding Area Impact

Risk Factors:

a. Rate of slope movement and severity of impact.
b. Engineering judgment about possible wall failure.
c. Maintenance to maintain truck and bus traffic.
d. Detour potential for interstate highway traffic.
e. Property damage potential.
f. Landlock potential.

Derivation of a suggested "best practice" hazard rating system was based on the current rating systems for Allegheny County, District 11-0, and District 12-0. The unique consequence and risk factors for these models were normalized for a constant weighting factor of 1.0 and combined into a singular model. Where multiple agencies provided unique ratings for the same consequence and risk factor, an average value was derived. See Figure 9-9 and Figure 9-10 for a visual representation of how the rating points from multiple agencies for a single factor were averaged for the height of failure above the roadway and length of road impact factors, respectively.

A legend of consequence factors and associated rating points are presented in Table 9-4. A legend of risk factors and associated rating points are presented in Table 9-5. The Net Risk Factor (Net RF) is computed using an arithmetic average for the Risk Factors divided by 10.

The Hazard Rating matrix is then computed by multiplying the sum of the minimum and maximum consequence rating points times the minimum and maximum Net RF, as indicated in Table 9-5. This process resulted in a possible range of 6 to 640 for the Hazard Rating for low risk/low consequence and high risk/high consequence, respectively. These results fit well with current practice regarding the assessment of Risk-Consequence.

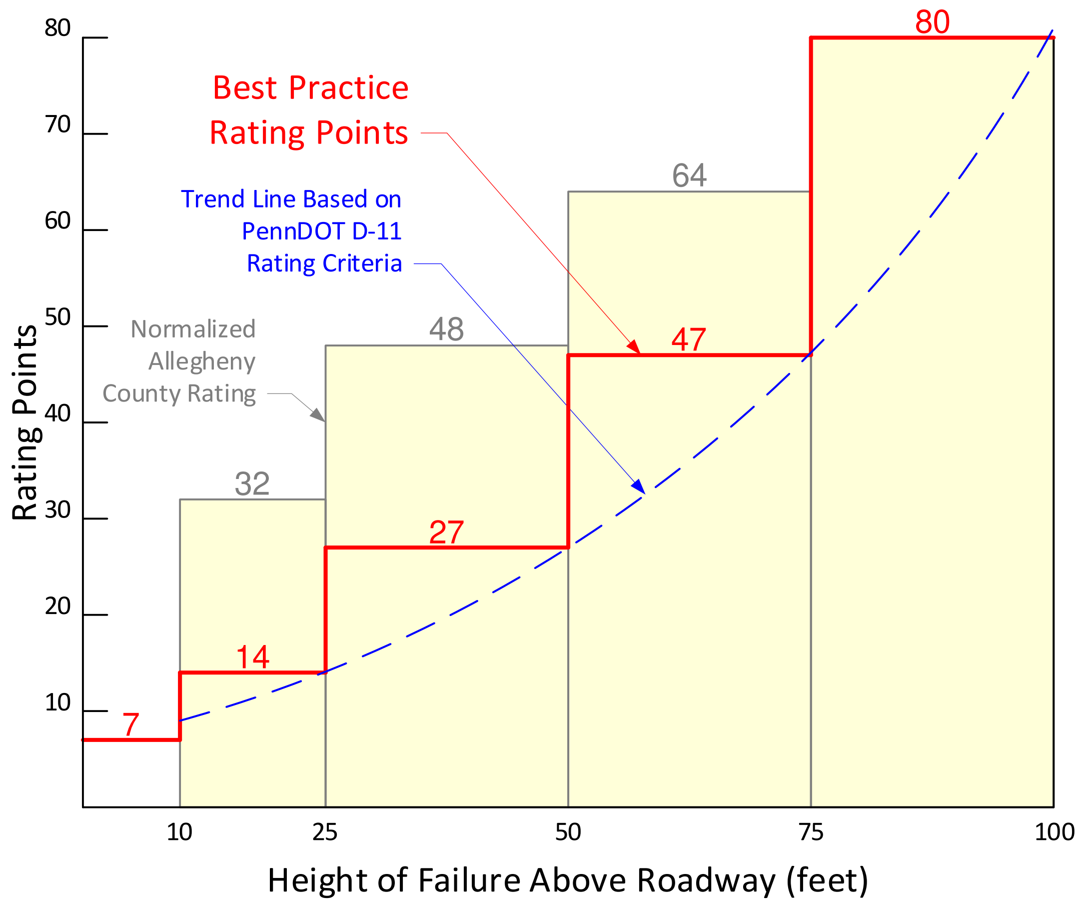


Figure 9-9 - Comparison of Rating Points Assigned to Assess Height of Failure Above Roadway

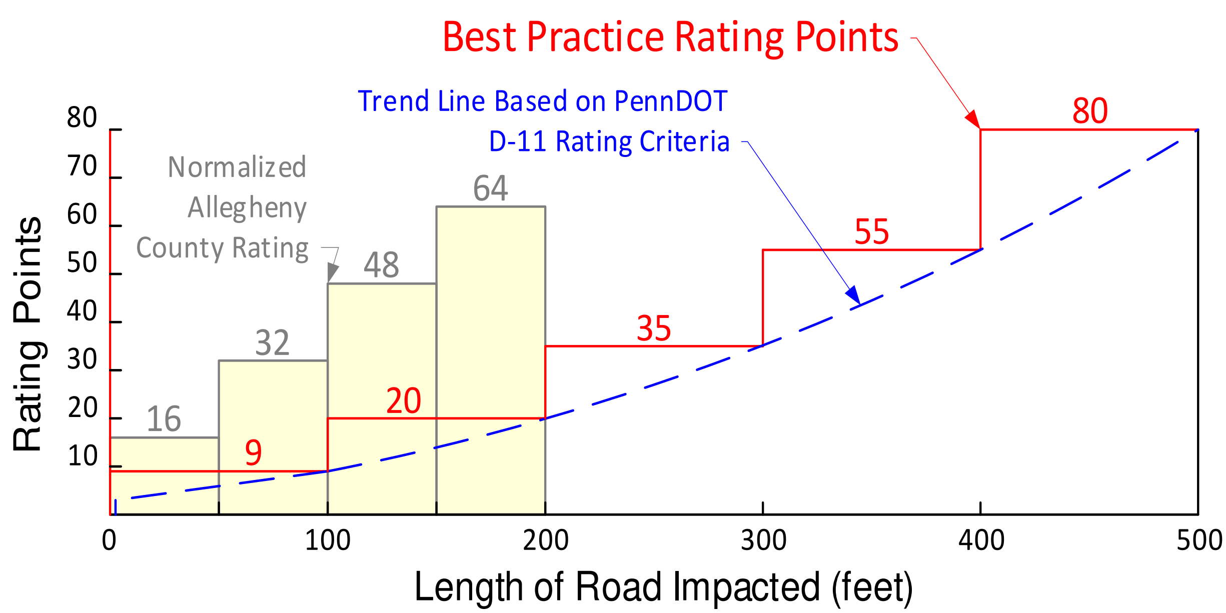


Figure 9-10 - Comparison of Rating Points Assigned to Assess Length of Road Impacted

Max. Rating Point	640													
Min. Rating Point	44													
Max. Points	80		160		80		80		80		80		80	
Min. Points	16		0		10		16		2		0		0	
Weighting Factor	1		1		1		1		1		1		1	
Max.	80		160		80		80		80		80		80	
Min.	16		0		10		16		2		0		0	
	Road Classification		Roadway Impact		ADT		Detour Length		Length of Roadway Impacted *		Height of Failure Above Roadway *		Surrounding Area Impact	
	Points	Threshold	Points	Threshold	Points	Threshold	Points	Threshold	Points	Threshold	Points	Threshold (feet)	Points	Threshold
	80	1-Interstate	160	Both Lanes Closed - Road Closed	80	>10000	80	None Available	80	401-500	80	76-100	80	Maintenance Costs of Closure
	64	2/3-Principal Arterial	128	One Lane Closed - Traffic Signals	65	7500 - 10000	64	>15 Miles	55	301-400	47	51-75	80	Political Implications
	48	4- Minor Arterial	96	One Lane Closed - Stop Signs	55	3000 - 7500	48	5-15 Miles	35	201-300	27	26-50	27	School, EMS, Police, and/or Fire Impacts
	48	5- Major Collector	64	Shoulder Closed - R-S Devices	40	1000 - 3000	32	<=5 Miles	20	101-200	14	11-25	9	Frequent Maintenance
	32	6- Minor Collector	32	Shoulder Closed - DPW Device	20	250 - 1000	32	Interstate Detour	9	26-100	7	1-10	9	Periodic Maintenance
	16	7- Local	27	Dip in Road	10	< 250	16	Not Needed	2	0-25	0	<1	3	Minor Impacts
			16	Edge of Pavement/Curb Impacted									3	Rural Road with Low ADT
			0	No impact to Road									0	Other

* Refer to Figures 9-9 and 9-10 for an illustration about the rating point(s) versus height of failure above the roadway (Figure 9-9) and rating point(s) versus length of roadway impacted (Figure 9-10).

Table 9-4 - "Best Practice" Hazard Rating - Rating Point Consequence Legend

	Geotechnical Risk Factor (RF)	Engineeering Judgement Risk Factor (RF)	Maintenance Risk Factors (RF)					Sum	Average Maintenance RF
Max. RF	10	10	10	10	10	10	10	50	10
Min. RF	1	2	2	2	0	0	0	4	0.8
	Change in Condition of Slide or Wall	Engineering Judgement	ADT-Truck	ADT-Bus	Interstate Detour	Potential Property Damage	Landlock		

Change in Condition of Slide or Wall

Points	Threshold
10	Wall/Slope Eminent on Collaspe Causing Loss of Entire Road Since Last Visit
10	Lane Sagging > 2' Drop @ CL and/or Both lanes in distress [>3"/day or >2"/mo], Imminent catastrophic failure with next storm
8	Landslide/Wall Deteriorated Since Last Visit Creating a Change in the Pavement - Creating Impact to Travel Lane **Added Road Closure Signals**
8	Lane Sagging and <2' Drop @ CL [6" to 24"/yr (2"/mo)]
6	Landslide/Wall Deteriorated Since Last Visit Creating a Change in the Pavement - Creating Impact to Travel Lane **Added Stop Signs/Traffic Signals**
6	Lane Sagging / Droppage [1" to 6"/yr (0.5"/mo)], Lane Closure Possible
4	Landslide/Wall Deteriorated Since Last Visit Creating a Change in the Pavement - No Impact to Travel Lane - Added Cones/Delinators/Barriers
4	Slide into Shoulder with <1' Drop @ White Line [0.5 to 1"/yr]
2	Landslide/Wall Deteriorated since last visit with noticable changes to the road surface without the addition of safety devices to block the shoulder or travel lane
2	Slide behind Guidrail [<0.5"/yr]
1	No Change to Wall/Landslide since last visit

Engineering Judgement

Points	Threshold
10	Wall/Slide has deteriorated and moving with changes in conditions to the road and/or has become unsafe
8	Wall/Slide is a safety concern to Public/Buildings but appears to be stable and not getting worse
2	No Safety Concern to the Public Buildings

Maintenance Risk Factors

ADT-Truck Points	ADT-Truck Threshold	ADT-Bus Points	ADT-Bus Threshold	Interstate Detour Points	Interstate Detour Threshold	Potential Property Damage Points	Potential Property Damage Threshold	Landlock Points	Landlock Threshold
10	>500	10	>100	10	Yes	10	Occupied Structure	10	Yes
8	250-500	8	60-100	0	No	5	Unoccupied Structure	0	No
6	100-250	6	30-60			0	None		
4	50-100	4	10-30						
2	0-50	2	0-10						

	Minimum	**Maximum**
Rating Point (Consequence) =	44	640
Net RF = (Geo RF + Engr RF + Maint RF)/3/10	0.13	1.00

Hazard Rating Matrix = Rating Point * Net RF

44	**640**
6	**83**

Table 9-5 - "Best Practice" Hazard Rating - Risk Factor Legend and Computation of Hazard Rating

9.6.3 "Best Practice" to Establish Landslide Threat Tolerance

Threat Tolerance is a function of the interrelationship of risk and consequence; that is evident based on the results of the development of the "best practice" hazard Rating system. The "best practice" threat tolerance takes into account landslide risk, vulnerability, and uncertainty. A schematic of the risk consequence for the "best practice" hazard rating is presented in Figure 9-11.

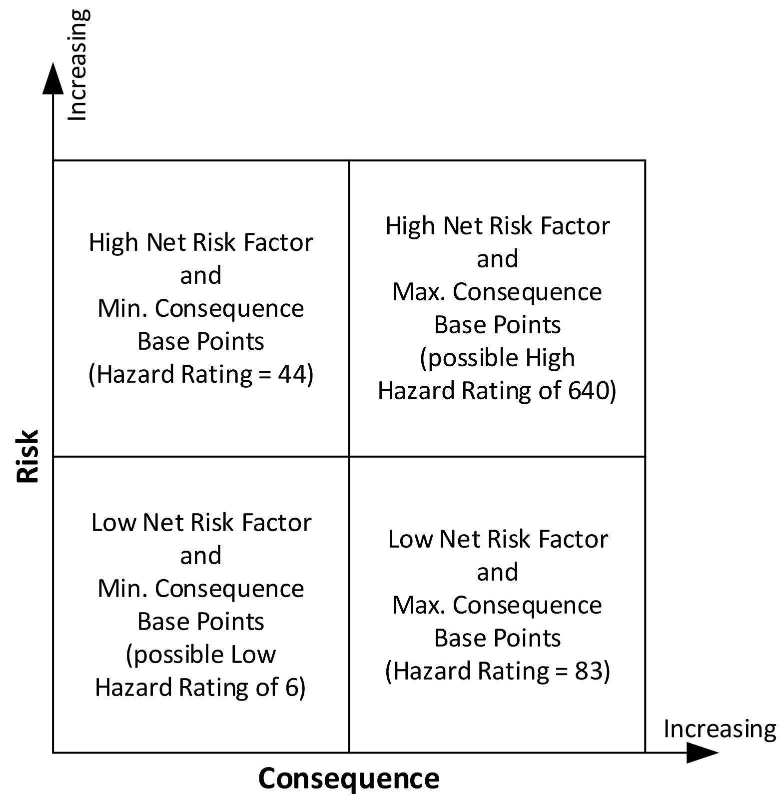


Figure 9-11 - Schematic of Risk versus Consequence for "Best Practice" Hazard Rating

Based on the results of the "best practice" hazard rating system, a threat tolerance of 100 is suggested, provided that there is no evidence of accelerated slope movement. However, threat tolerance values may vary by agency considering agency-specific tolerances for both risk and consequence.

As mentioned earlier, it is important to not only know the threat tolerance, but it is equally important to have a firm understanding of the mode of failure and the Best Practice Stability Rating (BPSR) for a given site (Figure 9-8).

9.6.4 "Best Practice" to Assess Risk Tolerance and Define Factors of Safety

Factors of safety against slope failure are used to establish a baseline by which to assess hazard and risk and execute risk management (Figure 9-4). It is important that the practitioner develop a comprehensive understanding of the causal factors and triggering cause (Figure 9-1) for a specific site. The practitioner should develop a site-specific understanding of landslide risk, vulnerability, and uncertainty (Sections 9.2.1.1, 9.2.1.2, and 9.2.1.3, respectively) to assess risk tolerance.

Economic factors and the magnitude of loss are key considerations. "best practice" suggests that the practitioner not use a "one size fits all" approach to assess risk tolerance, but rather select a required factor of safety unique to the site conditions to mitigate a landslide hazard.

In practice, local practitioners will often rely on a conventional deterministic analysis to assess and mitigate a landslide hazard; however, "best practice" dictates that measures be applied to account for the consequence and uncertainty of the factors that were mentioned earlier in this chapter. With that in mind, the suggested "best practice" follows a modified version of the approach that was derived by Dr. Duncan and Dr. Wright (Section 9.3.1) which is summarized in Table 9-6 below.

The following items may be used as a guide to discerning the amount of uncertainty and consequence of failure to determine the required minimum factor of safety against global stability:

- Subsurface setting is well-defined, uniform, and with minimal variability (reduction in uncertainty).
- Mode of failure is well-defined (reduction in uncertainty), and soil shear strength has been determined in the laboratory using test methods that are consistent with the mode of failure (e.g., consolidated drained, consolidated undrained, and reverse direct shear) (reduction in uncertainty).
- Seasonal fluctuation in groundwater conditions and pore pressure are well-defined (reduction in uncertainty).
- A detailed assessment of causal factors and triggering factor has been performed (reduction in uncertainty).
- Geohazards have been identified to assess the potential for slope movement (reduction in uncertainty).
- A risk hazard, hazard rating, and risk assessment have been completed (assessment of consequence).
- Consequence(s) has been identified and assessed.

Consequence of slope failure *	**Uncertainty of Analysis Conditions ***	
	Small	Large
The cost of repair is comparable to the incremental cost to construct a more conservatively designed slope	FS = 1.25	FS = 1.5
The cost of repair is much greater than the incremental cost to construct a more conservatively designed slope	FS = 1.5	FS = 2.0 or greater

* See above for items (1-8) to consider when discerning the amount of uncertainty and consequence of failure

Table 9-6 - "Best Practice" to Assess Risk Tolerance and Define Required Minimum Factors of Safety

9.6.4.1 Regulatory Requirements

For many projects, the minimum FS against global stability is prescribed by regulatory requirements and or design codes. Prescribed FS typically ranges from 1.3 to 1.8, depending on the circumstance and the anticipated lifespan of the solution (e.g., long-term mitigation versus an emergency repair). An emergency repair is interpreted herein to represent an expeditious response to reduce the hazard exposure and rate of slope movement, to permit sufficient time for a more long-term mitigation strategy to be executed.

To satisfy regulatory requirements, "best practice" dictates that the practitioner complete a thorough search of any regulatory documents applicable to the project including, but not limited to, design codes, design manuals, and local ordinances.

Since local ordinances are continually being updated, it is "best practice" to check with local agencies that have jurisdiction to identify any updates made to regulatory requirements. This is particularly true for landslides that involve private property. For instance, the Township of North Strabane (Washington County) updated its Grading and Excavation Ordinance in November 2018 where a new requirement was set forth. The ordinance now states that "a minimum factor of safety (against global stability) for all permanent slopes shall be 1.5 and for all temporary slopes shall be 1.3."[127]

For transportation-related projects in southwestern PA, a search of regulatory requirements to determine the minimum calculated FS will typically result in one or more of the prescribed minimum calculated FS against global stability (Table 9-7), depending on the project requirements, site constraints, and quality of shear strength data that is available.

Ref. Source	Min. Calc. FS	Description	Reference Section or Article
Pub. 293 [102]	1.5	Proposed cut slope in soil overburden	Sec. 8.6.1
	1.5	Compound slope stability of geosynthetic reinforced soil (GRS) structures for slopes steeper than 1H:1V (long-term static load and groundwater conditions)	Table 11.6.3-1
	1.3	Compound slope stability of GRS structures for 1H:1V slopes and flatter (long-term static load and groundwater conditions)	Table 11.6.3-1
Pub. 15M (DM-4) [99]	1.8	Abutment supported above a retaining wall, with shear strength parameters that are based on soil index properties and Standard Penetration Tests (SPTs), for Service I Load Combination (FS = 1/0.55).*	Table 10.6.2.5-1
	1.5	Abutment supported above a retaining wall, with shear strength parameters that are based on laboratory shear strength test results and Standard Penetration Tests (SPTs), for Service I Load Combination (FS = 1/0.65).*	Table 10.6.2.5-1
	1.5	Soil-nailed structures for long-term stability against global stability	App. O, Sec. 1.5.1
	1.35	Soil-nailed structures for short-term stability during construction against global stability	App. O, Sec. 1.5.1
AASHTO LRFD Bridge Design Specifications [2]	1.3	Overall (global) stability of retained fills **and earth slopes** where the geotechnical properties are well-defined, and the slope does not support or contain a structural element (FS = 1/0.75).*	Articles 3.4.1, 10.5.2.3, & 11.6.2.3
	1.5	Overall (global) stability of retained fills **and earth slopes** where the geotechnical properties are based on limited information (e.g., correlation to soil index properties), or the slope contains or supports a structural element (FS = 1/0.65).*	Articles 3.4.2, 10.5.2.3 & 11.6.2.3
	1.1	Stability of a non-gravity cantilevered wall with seismic earth pressure (FS = 1/0.9)	Article 11.8.6.1 Article 11.5.8

* Modified Bishop and Janbu slope stability analyses may be used for rotational and translational type failure surfaces, respectively.

Table 9-7 - Prescribed Minimum Calculated Factors of Safety (FS) against Global Stability

9.6.5 "Best Practice" Remedial Options

After a Landslide Risk Analysis and Assessment has been completed (Figure 9-4), a decision is required about which remedial option to select to move forward. The remedial option chosen is independent and parallel to any ongoing slope maintenance (Chapter 8).

The following are suggested "best practice" remedial options:

1) Develop and maintain a Slope Management System.
2) Reduce or eliminate the consequence (e.g., avoid the landslide-prone area of concern, by way of relocation, realignment, and or bridging).
3) Install a warning system to alert of the accelerated rate of movement.
4) Make capital improvements to reduce the risk of failure to an acceptable level (e.g., increase the factor of safety against slope failure).[115][141]

Option 1 requires funding and land ownership consideration. Landslide-prone area(s) need to be prioritized. The benefit of Item 1 is low since it does not prevent or reduce the frequency of an active landslide, but may be offset by a reduction in consequences.

Option 2 typically can be implemented faster than the other options, provide a lower cost than that required for capital improvement, and can adequately reduce public exposure to risk; however, this option will not prevent or reduce the frequency of an active landslide.

Option 3 will require allocated funding and resources to execute; this option does not prevent or reduce the frequency of an active landslide, but it can guide planning for capital improvements, and help justify further investment.

Option 4 requires funding and prioritization of objectives, but can reach the goal of long-term slope stabilization. This option typically will take more time to implement than Options 2 and 3.

9.6.6 "Best Practice" for Data Management

Data management has been and continues to be a challenge. Data is typically collected either in written form, or captured in cloud-based systems, and then processed using a variety of software depending on the data source. Technology has been well received, but due to the everchanging platforms and compatibility issues, relevant data may become obsolete and no longer retrievable. There is a current trend to move toward a regionally centralized system.

Some practitioners are striving to intensify the usage of technology advances to replace fieldwork to develop a firm understanding of the problem at hand – essentially trying to replace site reconnaissance with the off-site desktop study. However, this is fundamentally flawed; this practice conflicts with the primary objective for the practitioner to develop a detailed understanding of the problem and mode of failure, as well as a comprehensive understanding of risk, consequence, and potential at a given site for slope movement (i.e., BPSR, Section 9.6.1).

"Best practice" for data management encourages the use of the FOSE principle, that is

- <u>F</u>lexible.
- <u>O</u>rganized and accessible.
- <u>S</u>imple.
- <u>E</u>asy to understand and interpret.

The following are components of "best practice" data management:

- Quality (not quantity) and consistent data.
- Identification of the mode of failure.
- Identification of the areal extent and scale of landslide hazard.
- Data to support a fundamental understanding of causal and triggering factors.
- Data to support a fundamental understanding of landslide risk, vulnerability, and uncertainty.
- Data to support a fundamental understanding of site-specific constraints and consequence(s)
- Data to support a fundamental understanding of the slope movement potential.
- Records of the mitigation options executed, and costs incurred, with commentary for reference to advise future action including other sites with similar hazard scenarios.

9.6.7 "Best Practice" for Emergency Response versus Planned Improvement

A key consideration is the urgency of the need to mitigate a landslide hazard. A clear distinction between "needs" and "wants" is needed to determine whether an emergency response, a planned improvement, or a combination of both is warranted.

9.6.7.1 Emergency Response

"Best practice" suggests that when a situation implies urgency, then an Emergency Response is warranted. For instance, if the risk to life safety is a primary concern, immediate action may be needed to close a section of the roadway. In extreme cases, the local municipality may need to issue a Declaration of a State of Emergency, with possible removal of impacted residents from their homes.

Many of the typical steps that would be followed to characterize, evaluate, design, and execute a mitigation strategy are condensed to provide a timely response, based on collaboration between the practitioner and the vested stakeholders. Thus, action must be taken to implement Risk Management.

9.6.7.2 Planned Improvement

At the other end of the spectrum, the practitioner and vested stakeholders have rendered an opinion that a planned response is desirable. A planned response requires much more time to execute compared to an emergency response; in some cases, a planned response could require several years to complete. However, the benefit of a planned response is that the additional data and evaluations can be completed in the extended time frame allowing for rising confidence and diminishing uncertainty (Figure 9-2).

"Best practice" warrants a methodical approach with the intent to arrive at a feasible, practical, and cost-effective solution to mitigate a landslide hazard (Figure 9-4).

9.6.8 "Best Practice" to Tailor Response to Target Audience

"Best practice" demands that corrective actions be tailored to the needs of the target audience. Economic factors, consequences, degree of risk, and the magnitude of loss are of keen interest to all stakeholders when considering mitigation response.

"Best practice" suggests that the practitioner identify and discuss expectations to develop a mutual understanding with vested stakeholders at the beginning of the process to avoid unnecessary misunderstandings about the purpose, scope, and objectives to mitigate landslide hazard. The considerations needed to tailor the response to the target audience typically includes one or more of the following items.

1. Scale and serviceability of the mitigation response.

    a. What is the minimum required service life of the mitigation response?

    "Best practice" suggests that the response needs to be tailored to the situation at hand. For instance,

    - an emergency response may demand an action that can only provide serviceability for a few days or weeks;
    - a planned response may employ a temporary repair that can provide a useful life for less than 5 years; or
    - a planned response may be designed to provide a permanent repair with a service life that exceeds 50 years.

    An example of a situation that demanded mitigation measures with a service life of a few days involved a site where mobile crane access was blocked at both ends of a steep ravine by active landslide movement; the primary consequence was the loss of that equipment. After an emergency response was implemented and the crane was able to be recovered, subsequent measures were initiated to address longer-term solutions to mitigate landslide hazard.

    b. What is the target service life of a temporary repair?

    "Best practice" suggests that the service life could vary depending on need; at a minimum, a temporary repair should be sufficient to provide time to follow up with other options to provide a more resilient solution. Conventionally, the service life of temporary works is interpreted to be a maximum of 3 years.

    c. What is the service life of a permanent mitigation response?

    "Best practice" suggests that a permanent mitigation response should be designed to provide a service life of at least 50 years.

2. Available funds.
    a. How do available funds affect the implementation of landslide mitigation measures?

    "Best practice" suggests that economic factors need to be considered to establish an appropriate margin of safety (e.g., degree of risk) and tolerable magnitude of loss that would result from a slope failure.

3. Landslide vulnerability.
    a. What are the physical, social, economic, and environmental factors that are susceptible to the impact of the landslide hazard? What is the repair history and frequency? What (if any) constraints exist? Who is impacted by the landslide hazard and to what degree? What is impacted by the landslide hazard?

    "Best practice" dictates that these factors be identified and discussed with stakeholders early in the mitigation process, and then considered in any decision made about Risk Hazard, Assessment, and Management moving forward.

4. Elements at risk.
    a. What are the elements at risk (e.g., people, property, transportation links (e.g., roads), public utilities, and natural resources (e.g., disruption of a watercourse)?

"Best practice" dictates that elements at risk be identified and discussed with stakeholders early in the mitigation process, and then considered in any decision made about Risk Hazard, Assessment, and Management moving forward.

5. Consequence of landslide risk.
    a. What are the site-specific consequential factors (e.g., damage, injury, loss of life, reduction and or loss of functional use, direct or indirect cost, indirect consequential cost (e.g., litigation), and adverse social and environmental impact)?

    "Best practice" dictates that these factors be identified and discussed with stakeholders early in the mitigation process, and then considered in any decision made about Risk Hazard, Assessment, and Management moving forward.

    "Best practice" to determine "Hazard Rating" (Section 9.6.1) provides a basis that the practitioner can use to identify consequential factors and prioritize the pros and cons for any mitigation options considered.

CHAPTER 10

Stabilization and Repair Methods

10.1 GENERAL

At the beginning of this chapter, several key concepts are worth mentioning to frame, introduce, and describe “Best Practice” to stabilize and repair landslides in southwestern PA. The stabilization and repair methods presented in this chapter build upon a fundamental understanding of the project site which is based on the processes that are described in the preceding chapters, to provide actionable guidance with practical, clear, useful, and usable direction. A process flow chart is presented in Figure 10-1.

Implementation of an effective slope stabilization or repair method may build upon existing elements or “tools” that are available from participating agencies (namely, PennDOT and FHWA). It is understood that the practicing engineer that is using this Handbook has a working knowledge of soil mechanics and is reasonably familiar with the applicable FHWA and PennDOT publications including, but not limited to PennDOT Publication 222 - *Geotechnical Investigation Manual* [101]*,* and PennDOT Publication 293 - *Geotechnical Engineering Manual [102]*. The following publications provide additional guidance about items that are discussed herein. However, project specific requirements must be followed, which may take precedence over the publications presented herein; refer to Section 1.2 for additional information.

- Duncan, M, Wright, S., and Brandon, R. *Soil Strength and Slope Stability*, 2nd edition, 2014. <https://www.wiley.com/en-dk/exportProduct/pdf/9781118651650>
- FHWA Publication DP-90-068, Permanent Ground Anchors, Volume 1, Final Report. 1991. <https://www.fhwa.dot.gov/engineering/geotech/pubs/009989.pdf>.
- FHWA Publication DP-90-068, *Permanent Ground Anchors, Volume 2, Field Demonstration Project Summaries*. 1991. <https://www.fhwa.dot.gov/engineering/geotech/pubs/009988.pdf>.
- FHWA Publication NHI 08-101, Highway Slope Maintenance and Slide Restoration. October 2008. <https://rosap.ntl.bts.gov/view/dot/50373>.
- FHWA Publication NHI-14-007, *Geotechnical Engineering Circular No. 7, Soil Nail Walls, Reference Manual*. 2015. <https://www.fhwa.dot.gov/engineering/geotech/pubs/nhi14007.pdf>.
- FHWA Publication NHI-18-031, Geotechnical Engineering Circular No. 9, *Design and Analysis of Laterally Loaded Deep Foundations*. 2018. <https://www.fhwa.dot.gov/engineering/geotech/pubs/hif18031.pdf>.
- FHWA Publication NHI-18-024, Geotechnical Engineering Circular No. 10, *Drilled Shafts: Construction Procedures and LRFD Design Methods*. 2018. <https://www.fhwa.dot.gov/engineering/geotech/nhi18024.pdf>.
- FHWA Publication HRT-10-77, Composite Behavior of Geosynthetic Reinforced Soil Mass. 2013. <https://www.fhwa.dot.gov/publications/research/infrastructure/10077/index.cfm>.
- FHWA Publication SA-94-005, Advanced Technology for Soil Slope Stability. April 1994. <https://ntrl.ntis.gov/NTRL/dashboard/searchResults/titleDetail/PB95225819.xhtml>.
- FHWA Publication SA-94-005, Advanced Course on Soil Slope Stability: Volume 1. Slope Stability Manual. November 1993.

<https://ntrl.ntis.gov/NTRL/dashboard/searchResults/titleDetail/PB98104805.xhtml>.
- PennDOT Publication 293, Geotechnical Engineering Manual. November 1993. <https://ntrl.ntis.gov/NTRL/dashboard/searchResults/titleDetail/PB98104805.xhtml>

The publications cited above provide overall guidance about stabilization methods, slope stability concepts, methods of analysis, factors affecting slope stability, and other related topics. Stabilization methods discussed include landslide unloading, buttressing, drainage, reinforcement, retaining walls, vegetation, surface slope protection, hardening of soils, and several others.

This chapter highlights current practice and emerging technology to provide actionable guidance with practical, clear, useful, and usable direction for “Best Practice” to stabilize and repair landslides in southwestern Pennsylvania.

See Chapter 11 for discussion regarding the economics of various methods of landslide stabilization and repair.

See Chapter 12 for preferred details that align with the “Best Practice” that typically is employed in southwestern Pennsylvania to stabilize and repair landslides based on current practice. Chapter 12 will build upon information that is presented in Chapters 10 and 11 to transition from a discussion about available stabilization methods to the selection of a site-specific preferred solution. It is a given that practitioners should address site-specific, and client specific, needs to select a preferred solution that is cost-effective, practical, functional, and constructible.

To begin, the following twelve (12) key assertions should be considered to frame a systematic “how to” approach [12]:

1) Natural slopes generally stand at the steepest inclination possible, given the environmental conditions to which they are subjected. Natural slopes do not exhibit factors of safety of 1.5; closer to 1.15 [116].
2) Many instances exist where landslide control will not be the best solution. For instances that involve corrective action by the “elimination method” (e.g., avoidance), halting the landslide movement is not generally a factor in the solution.
3) Determining the "triggering cause” of a landslide is not always essential to execute a reliable solution, and is secondary in importance to understanding the mechanics of the landslide movement (i.e., mode of failure). More important than the “triggering cause”, is the realization that increased stability will result by eliminating or minimizing the effect of any contributing factor.
4) The works of man can measurably accelerate or decelerate the rate of landslide movement. The most permanent solutions to control mass movement will be those of a type that permanently (from a geologic viewpoint) assists nature's resistance.
5) Failure often occurs in the soil when the rupture plane is at the interface with the underlying stable media (e.g., bedrock). The mobilized shear strength at the soil-rock interface is a primary area of interest.
6) For a given landslide, there is usually more than one viable stabilization and repair method. The inference is that for any given landslide, one method is more desirable after consideration is given to economics, acceptable risk and consequence, urgency, project constraint(s), access, constructability, impact on the public and the environment, and aesthetics.
7) The decision as to the corrective action to be used for a given landslide is eventually reduced to a problem of economics.

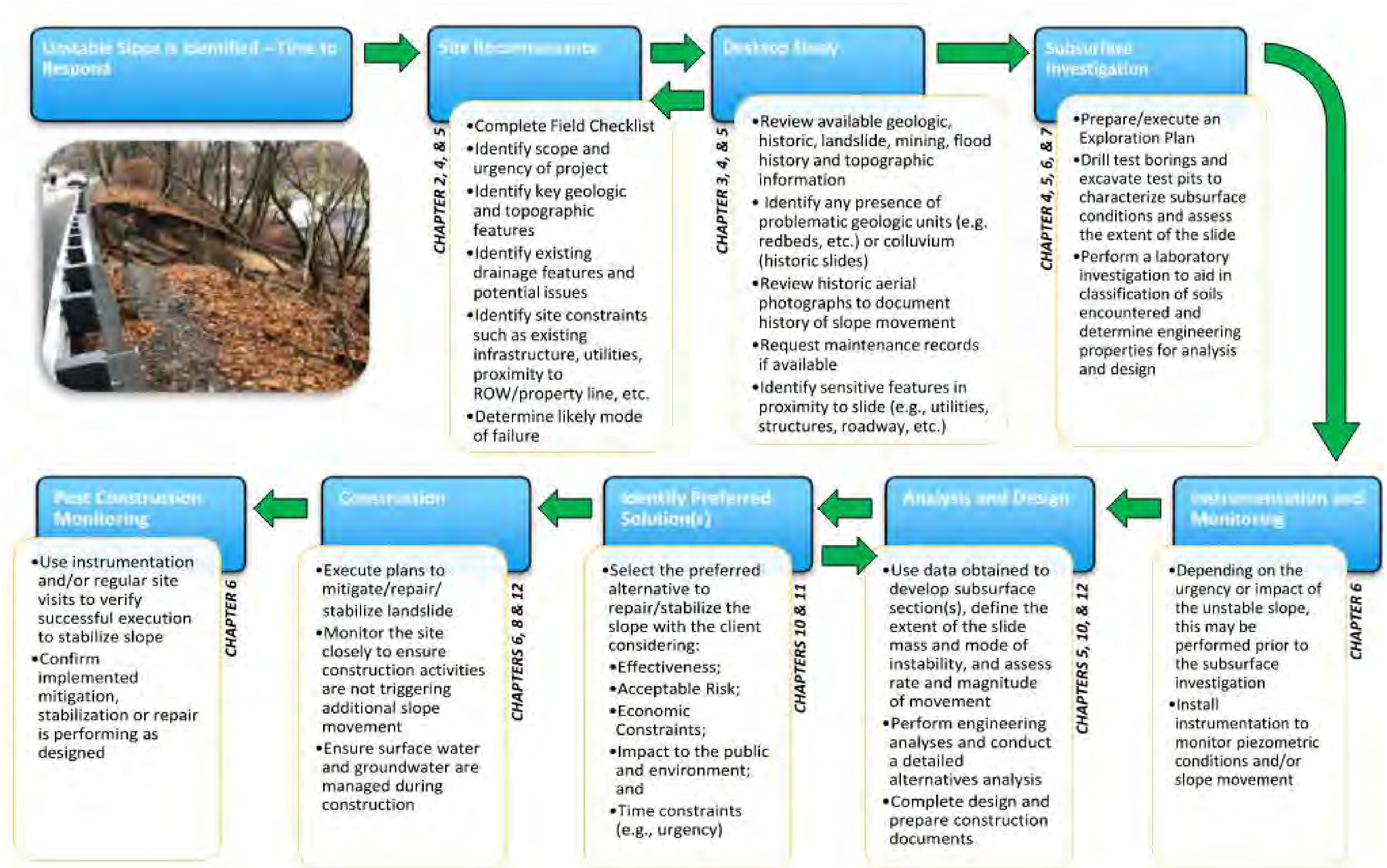


Figure 10-1 - Landslide Mitigation Flowchart
(Please note, the bidirectional arrows for several of the steps indicate that those steps may be done iteratively or in reverse order)

8) Water is a contributing factor in practically all landslides, particularly those involving unconsolidated material. Aside from the force of gravity, no factor is more generally present as a contributing factor.
9) The force of gravity is the sole contributing factor that is common to all landslides.
10) In all mass movements, and just before movement, the reactions tending to resist movement are for all practical purposes equal to the forces that are tending to cause movement.
11) The determination of the location of the rupture plane is the most critical factor in the use of a rational or semi-rational approach to assess and stabilize landslides. This requires critical thinking both during office study and field examination.
12) The analysis of a landslide should be governed by the basic principle of obtaining a more stable slope than that which existed before failure.

Stabilization and repair methods typically fall into one of two generalized categories: elimination and control. The following is a more detailed breakdown of these two generalized categories [12]:

- I. Elimination Methods
  - A. Relocation
  - B. Removal
  - C. Bridging (over Landslide Mass)
- II. Control Methods
  - A. Retaining Structures
    - 1. Buttress(es)
      - a. Rock-Fill Buttress
      - b. Stability Berm
      - c. Extended Fill (raise grade at toe)
      - d. Shear Key with Rock Buttress
    - 2. Geosynthetic Reinforced Soil (GRS)
    - 3. Slope Surface Enhancement(s)
      - a. Geocell Slope Protection
      - b. Turf Reinforcement Mat (TRM)
      - c. Rock Slope Armoring
      - d. Gabion slope (Reno mattress)
    - 4. Shoulder Back-Up and Moment Slabs
    - 5. Single-Face Barrier
    - 6. Gravity Walls
      - a. Conventional Concrete Gravity Wall
      - b. Gabion Wall
      - c. Modular Block Wall
      - d. Crib Wall
    - 7. Cantilevered Pile Walls
      - a. Soldier Pile and Lagging Wall
      - b. Buried Soldier Pile Wall
      - c. Tangent Pile Wall
    - 8. Tieback (Ground) Anchors
      - a. Soldier Pile Wall with Tiebacks
      - b. Buried Panel Wall with Tiebacks
      - c. Sheet Pile Wall with Tiebacks

II. Control Methods (continued)
    - 9. Soil Nails
    - 10. Soil Nail Launcher
    - 11. Articulated Micropile
    - 12. Pipe Dowels
  - B. Rebalance Ratio Between Mobilized Resistance and Driving Force(s)
    - 1. Surface Drainage Improvement
      - a. Upslope Interception & Diversion
      - b. Ditch Lining
      - c. Surface Reshaping/Regrading
      - d. Sealing Open Tension Cracks
    - 2. Subsurface Drainage Improvement
      - a. Interceptor (French) Drain
      - b. Spring Drain
      - c. Finger Drain
      - d. Horizontal Drains
    - 3. Lightweight Fill
    - 4. Partial Unloading (Removal of Material at Top)
    - 5. Slope Flattening
    - 6. Remove and Replace
- III. Emerging Technology
  - A. Control Methods
    - 1. Soil Nails and Grillage
    - 2. Cruciform Structure with Anchor Slab
    - 3. Debris-Flow Fence
    - 4. Deep Polymer Injection
    - 5. Bio-Remediation

Consideration of feasible stabilization and repair methods needs to begin at the earliest opportunity in the landslide mitigation process (see Figure 10-1). This process requires a fundamental understanding of the problem, site history, topographic relief, surface drainage, overall understanding of the soil, rock, and groundwater conditions, and the plausible mode of failure.

There is no simple rule as to which stabilization and repair method used is more prudent. Rather, constraints will lead to elimination of several options, from which affordability and cost-benefit should be considered. Regardless, the tabular list that is presented on the following page can be used as an initial screening tool to help identify preferred stabilization and repair method(s) as a function of the mode of failure (Chapter 2, Figure 2-1). Please note, only rebalancing methods are presented in the table below as these are generally able to be more quickly implemented than retaining structure options.

Stabilization and Repair Method	**Shallow**		**Deep**	
	Rotational	**Sheet Flow**	**Rotational**	**Translational**
B. Rebalance Ratio Between Mobilized Resistance and Driving Force(s)				
1. Surface Drainage Improvement				
a. Upslope Interception & Diversion	x	x	x	x
b. Ditch Lining	x	x	x	x
c. Surface Reshaping/Regrading	x	x	x	x
d. Sealing Open Tension Cracks	x	x	x	x
2. Subsurface Drainage Improvement				
a. Interceptor (French) Drain	x	x		
b. Spring Drain	x	x		
c. Finger Drain	x	x	x	x
d. Horizontal Drains				
3. Lightweight Fill				
4. Partial Unloading (Mat'l. Removal @ Top)			x	x
5. Slope Flattening				
6. Remove and Replace				

Table 10-1 - Suggested Preventative Measures when stability issue(s) is(are) first noticed

10.2 ELIMINATION METHODS

The first step of landslide stabilization or repair is to conduct a cursory review to rationalize whether or not an Elimination Method is practical or desirable. An early-action decision to implement an Elimination Method can yield significant savings both in time and economics to execute a repair solution.

10.2.1 Relocation

The relocation method is reliant on shifting a structure to a firm foundation. This method applies to all landslides, but may not be viable due to cost, right-of-way restrictions, or undesirable impact on function.

Analysis Consideration(s).

1) Typically, relocation does not require a formalized slope stability analysis but does require a fundamental understanding of where to relocate or change the function of a facility.

2) There are times when there is no practical solution other than to change the function of a facility. An example of a relocation, where the roadway width was reduced to one lane with bi-directional traffic control, is illustrated in Figure 10-2 If available right-of-way had existed, an excavation cut could have been made into the hillside to add a 2nd travel lane.

Figure 10-2 - Relocation Example
Extended Term Lane Closure

10.2.2 Removal

The removal method involves excavation and wasting part, or all, of the landslide mass. Removal is typically utilized in situations where the movement has come down onto a structure. This method is best suited for shallow slope failures. An example of a landslide removal to restore vehicular access to a community is depicted in Figure 10-3.

Figure 10-3 - Removal Example
Massive Landslide

Analysis Consideration(s):

1) A key component of the analysis for landslide removal is an accurate determination of the aerial extent and depth of excavation required, as well as the determination of a final slope geometry after the landslide mass is removed. Additionally, calculations may be warranted to analyze the

slope stability of the remaining excavation cut slope or of the site where the excavation spoil is wasted.

2) Total removal is preferred over partial removal at the toe of a landslide mass. Partial removal typically entails the removal of landslide debris to relieve pressure on a structure or to remove an obstacle but should rarely be used except in emergencies. Slope movement is likely to continue and may accelerate after a portion of the landslide toe is removed.

10.2.3 Bridging

There are times when "bridging" over a landslide is the only viable solution to mitigate the adverse impact. One such example is the solution that was employed to carry McArdle Roadway over a prehistoric landslide. This prehistoric landslide is located approximately 900 feet west of the intersection of McArdle Roadway and the approach to the Liberty Bridge. The location of this prehistoric landslide, and a view of the bridge that spans over the prehistoric landslide, are shown in Figure 10-5 and Figure 10-4, respectively.

Figure 10-4 - "Bridging" Example
Aerial View of McArdle Roadway Spanning over Landslide, Pittsburgh, PA [60]

Analysis Consideration(s):

The most important part of the bridging analysis is the determination of the span length required and suitable locations to place bridge substructures (abutments and possible piers). Single-span is usually preferred over multiple-span bridges to eliminate the need to construct substructures within the landslide mass. Future slope movement should be considered to assure that the bridging structure is bearing on a stable subgrade. Potential external loading or loss of soil that may occur as a result of additional slope movement should also be considered.

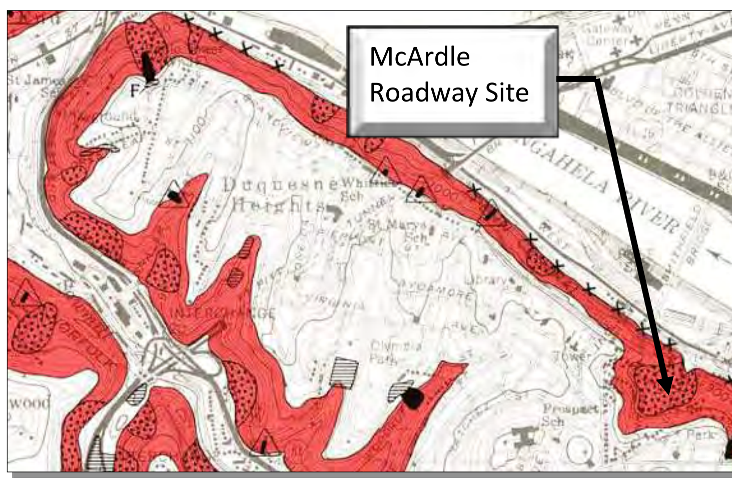


Figure 10-5 - "Bridging" Example
Location of Prehistoric Landslide along McArdle Roadway, Pittsburgh, PA [112]

10.3 CONTROL METHODS

Control Methods can frequently afford the option to isolate landslide stabilization and repair activity to a discreet area (as compared to that typically required for Elimination Methods). Utilization of control methods offers a significant advantage for projects where the right-of-way is limited and may otherwise necessitate an extended period to acquire additional right-of-way/easement and/or authorization from adjacent property owners. Control methods may also offer the benefit of mitigating multiple issues such as arresting slope movement and enhancing scour protection along a stream channel.

In typical applications, a combination of multiple control methods that are presented herein may be used in combination to stabilize and repair a landslide. For example, horizontal drains may be utilized with a stability berm, or a rock fill buttress may be used in combination with a cantilevered soldier pile and lagging wall.

Analysis Consideration(s). Key concepts regarding the analysis and design of Control Methods that are used to stabilize and repair landslides are presented herein. For detailed information about analysis methods and considerations, refer to Section 10.1 for a list of references. Additional analysis considerations specific to the Control Methods presented below are available in the individual subsections.

1) A comprehensive understanding of the mode of failure is a critical factor, and the first step, to selecting an effective Control Method.
2) Since water is often a contributing factor in most landslides in southwestern PA, consideration should be given to implementing measures to reduce pore water pressure that is acting on the rupture plane in conjunction with the preferred Control Method.
3) The practitioner needs to acknowledge that often there may be multiple modes of incipient failure present and that stabilization of the active mode of failure may not stabilize alternate (future) modes of failure that are also threatening the stability of the slope. Thus, all the potential failure modes should be considered. A comprehensive holistic understanding of the problem is important to stabilize and repair landslides.
4) The proposed solution should provide acceptable slope stability analysis results, not only for the long-term stress state (i.e., final condition), but also during and immediately after construction. Both total stress (e.g., short-term, undrained) and effective stress (e.g., long-term, drained) conditions need to be considered where applicable.
5) Residual shear strengths within the slide mass may be considered due to the loss of peak shear strength resulting from the large strain displacement of the landslide mass along the rupture plane.
6) Landslide movement that has been activated by a rise in pore pressure along the rupture plane will frequently dissipate pore pressure when landslide movement occurs. The dissipation in pore pressure will oftentimes permit the landslide movement to temporarily slow down or stop until the pore pressure has had time to build up again. Particular attention needs to be given to a temporary rise in the driving force when surface water seeps into and fills tension cracks near the ground surface.
7) For slope stability analysis, a complete search of a range of plausible rupture planes is needed to identify the critical failure surface. This analysis is typically completed with one of several commercially available software packages; and the search should consider both circular and translational (i.e., wedge) type failure surfaces.
8) The impact of varying piezometric surfaces, including seasonal water level changes and potential clogging of proposed drainage structures need to be considered.
9) A back-analysis may be performed to "reverse engineer" the existing landslide to calibrate the analytical model and soil parameters. It is important that this model is based on the data collected up to this point and that the results are consistent with the practitioner's understanding regarding the site conditions and mode of failure; this understanding will allow the practitioner to calibrate the appropriate parameters within the model. A factor of safety of unity (e.g., 1.0) against global stability is usually assumed for an active landslide that has slid and has temporarily arrested itself.

10) There are times when a hand calculation should be made to verify and complement the slope stability analysis results, to properly account for boundary conditions and structure performance. For instance, it is not uncommon to establish a boundary condition whereby rupture planes are restricted from penetrating below the surface of a competent stratum (e.g., the top of competent rock). Or for embedded structural elements (i.e. cantilevered walls) which provide shearing resistance at the rupture plane to stabilize a landslide, additional consideration should be given to confirm that both force and moment equilibrium is satisfied.

10.3.1 Retaining Structure(s)

Retaining structures are used to increase the resisting forces through external stabilizing elements. The type and location of the subgrade, or bearing stratum, is a key to successful stabilization efforts. Oftentimes, the retaining structure must penetrate below the critical rupture plane to drive the critical failure plane deeper and enhance mobilized resistance.

Typical retaining structures used for landslide stabilization and repair in southwestern Pennsylvania include:

- Buttresses.
- Slope surface enhancements.
- Shoulder Back-Up and Moment Slabs.
- Single-Face Barriers.
- Gravity Walls.
- Cantilevered Pile Walls.
- Tieback (Ground) Anchors.
- Soil Nails.
- Soil Nail Launcher.
- Articulated Micropile.
- Pipe Dowels.

10.3.1.1 Buttress(es)

Buttresses are used to provide additional surcharge weight to the toe of a slope to achieve additional shear resistance. Conceptually, the additional mass will increase the effective normal stress acting on the rupture plane which, in turn, increases the available frictional resistance.

Multiple applications and iterations of buttresses are available to the practitioner including:

- Rock-Fill Buttresses.
- Stability Berms (i.e. Toe Berms).
- Extended Fills (i.e. Valley Fills).
- Shear Keys with Rock Buttress.

Analysis Consideration(s):

1) Adequate preparation of the embankment subgrade before fill placement is key to the performance of the buttress. The existing turf, slide debris, and vegetation should be stripped off at the toe before the placement of new fill. The author of this chapter is aware of at least one instance where the earthwork contractor failed to strip topsoil on about a 5H:1V existing ground surface before placement of about a 30-foot high 2H:1V side-hill embankment; and that had

subsequently led to active slope movement of side-hill embankment with active rupture plane forming near the former topsoil interface.

2) To initiate design efforts, a volume of rock approximately 1/4 to 1/2 the volume of the slide mass may be considered [45].
3) The buttress must be constructed deep enough to penetrate the existing rupture plane and provide meaningful stability improvement; where the rupture plane extends below the ground surface, a shear key (i.e., toe key) may be utilized to extend below the rupture plane and provide adequate shear resistance.
4) For fine-grained soil (e.g., silt and clay), consideration should be given to short-term slope stability during excavation and the end of construction condition immediately after fill placement (if applicable) using undrained shear strength parameters in addition to long-term slope stability using effective shear strength parameters.
5) Consideration of the types of materials readily available (i.e., on-site materials or borrow sources), available right-of-way, urgency (i.e., available time to implement a repair), and mode of failure are key to the selection of potential buttress configurations.
6) Excavation and replacement of material may be done in discreet widths (slots) to shed load to either side of the slot excavation and maintain overall slope stability during construction

10.3.1.1.1 Rock-Fill Buttress

A rock-fill buttress (Figure 10-6) is comprised of a rock-fill placed at the toe of slope and built up in front of the existing landslide mass.

The rock-fill for the buttress may consist of coarse open-graded and durable rock, typically Class R-3 riprap or larger (refer to PennDOT Publication 408 [103] Section 850). This material is free-draining and provides a high level of frictional resistance with a typical mass angle of repose on the order of 38 to 42 degrees. Refer to Figure 10-7 for guidance to select an appropriate mass angle of repose for the rock fill.

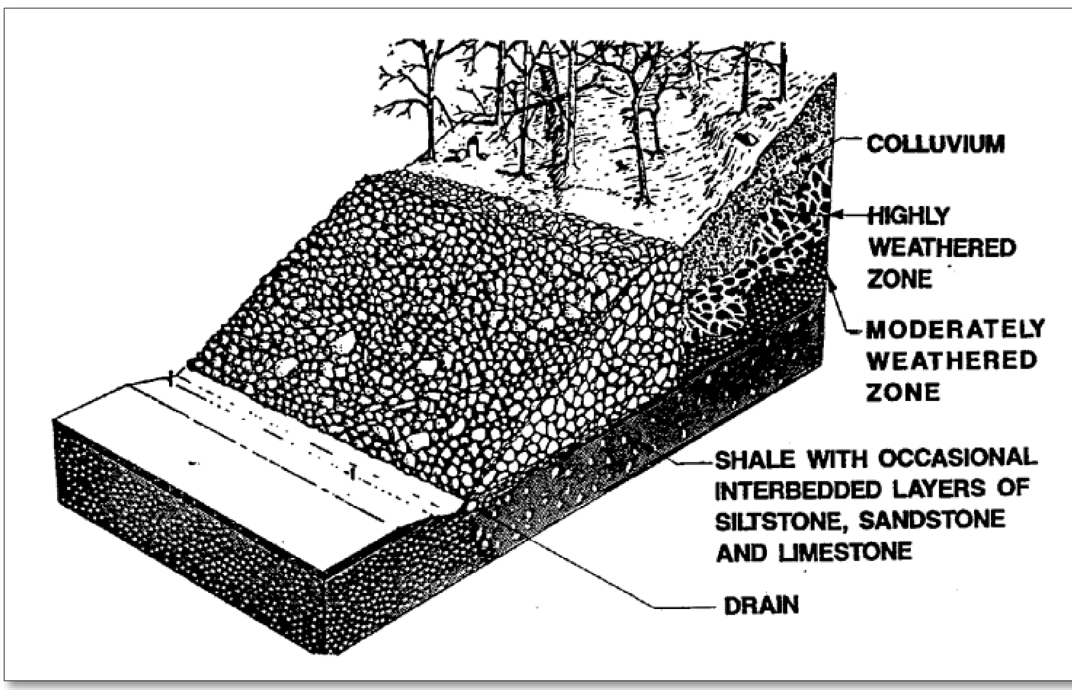


Figure 10-6 - Rock-Fill Buttress Conceptual Isometric [50]

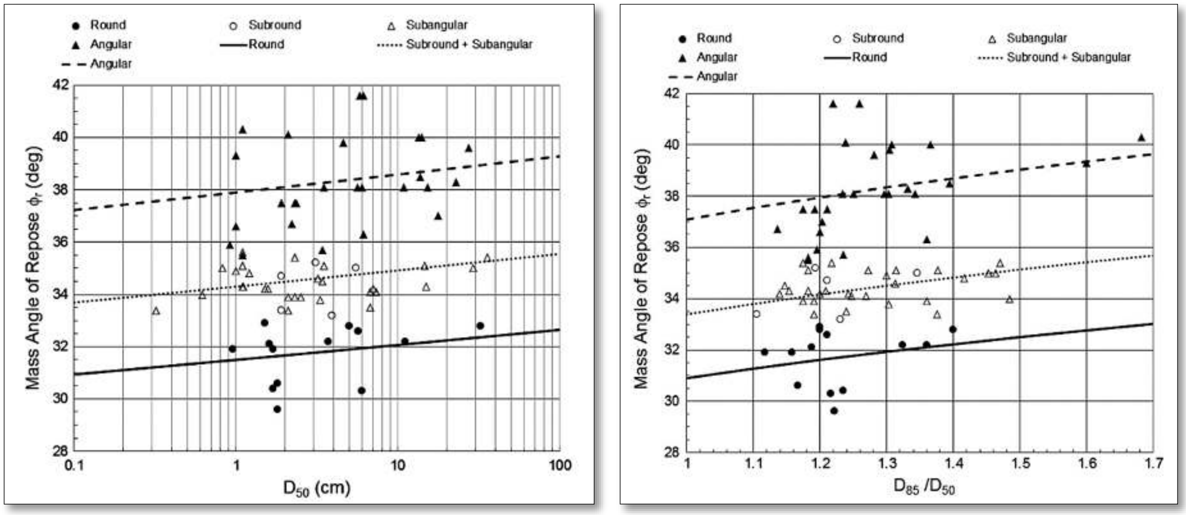


Figure 10-7 - Mass Angle of Repose as a Function of Particle Shape and D_{50} (left) and D_{85}/D_{15} (right) [54]

To provide perspective, the author is aware of quarried Loyalhanna sandstone that was loosely dumped from the end of a conveyor that is standing at a mass angle of repose of about 38 to 40 degrees, e.g., 1.2 to 1.3H to 1V (Figure 10-8).

To place this material steeper than the mass angle of repose (Figure 10-7 and Figure 10-8), consideration should be given to the material quality and source, as well as the method of placement and compaction. Select rock fill (e.g., angular rip rap), quarried from specific geologic members in the region, has been reported to have a peak internal friction angle of 45 to 50 degrees. This material typically consists of quarried rip rap that originates from the Loyalhanna limestone member of the Allegheny geologic Group (siliceous limestone/calcareous sandstone) and the Vanport limestone member of the Clarion formation, Allegheny Group (ferriferous limestone) in western Pennsylvania and northern WV.

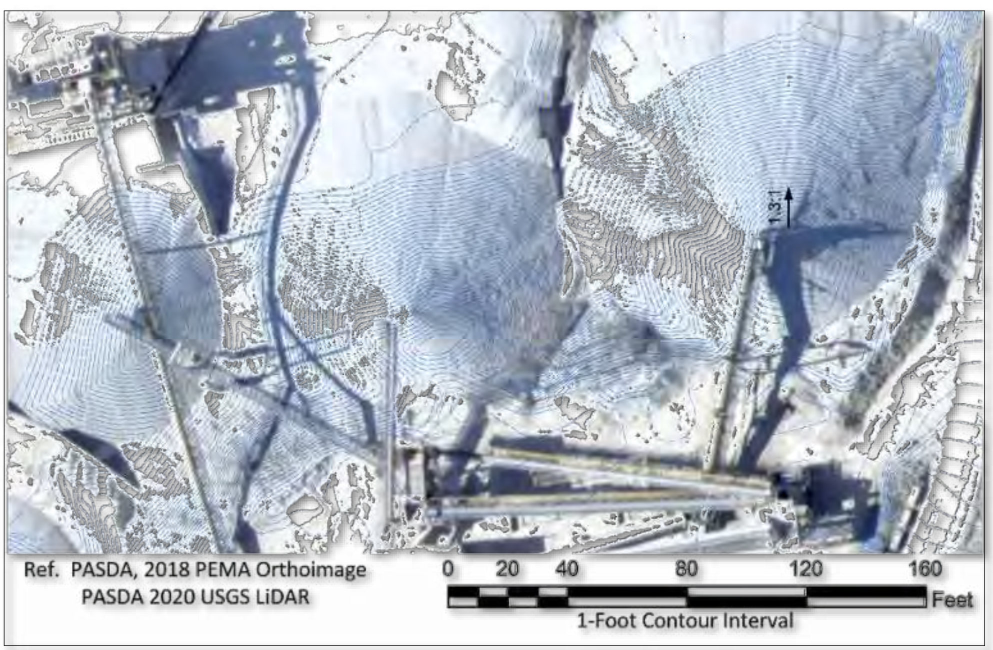


Figure 10-8 - Typical Quarried Stockpile of Loyalhanna Sandstone
(note, the portion near top of stockpile that was placed with conveyor is at 1.3H:1V)[96]

The effective friction angle of the rock fill may exceed the angle of repose depending on the method of placement and compaction. Where durable rip rap is carefully placed and compacted, rather than end dumped, an effective friction angle over 42 degrees is achievable. The author is aware of at least one location where rock fill was installed, and is still performing satisfactorily, at a 1.3H:1V slope (see Figure 10-9).

Figure 10-9 - Construction of Rock-Fill Buttress to Repair Landslide Along Stream Channel in Armstrong County, PA [96]

An example where an active landslide mass was removed and replaced with rock fill buttress on the same day, to restore service at a critical transportation link and protect an existing gas main, is illustrated in Figure 10-10 and Figure 10-11 (similar to the isometric that is depicted in Figure 10-6). Speed of execution in combination with reliance on standup time were critical to achieve satisfactory results.

Another example where rock fill buttress needed to be constructed with material that was dumped and spread due to logistical challenges is illustrated in Figure 10-12 and Figure 10-13.

Figure 10-10 - Rock-Fill Buttress Example - Active Slope Failure Looking Down Roadway (left) and Upslope from Toe (right)

Figure 10-11 - Rock-Fill Buttress Example – Completed Repair Looking Upslope from Toe

Figure 10-12 - Slope Failure
(prior to mobilization of construction equipment)

Figure 10-13 - Construction of Rock-Fill Buttress to Repair Landslide

10.3.1.1.2 Stability Berm

Stability berms (i.e. toe berms) entail excavating and replacing weaker soils at the toe of a landslide (if applicable) and adding a berm of higher-strength material in front of the toe of slope. This buttressing technique is oftentimes combined with the addition of some form of underdrain. Improving drainage at the toe can lead to an increase in effective vertical stress at the rupture plane which will provide improvement to the overall stability of the slope.

Where the failure surface is below the ground surface, a toe key may also be utilized in combination with a stability berm to stabilize multiple failure planes. See Figure 10-14 and Figure 10-15 for examples of stability berms used in combination with a toe key.

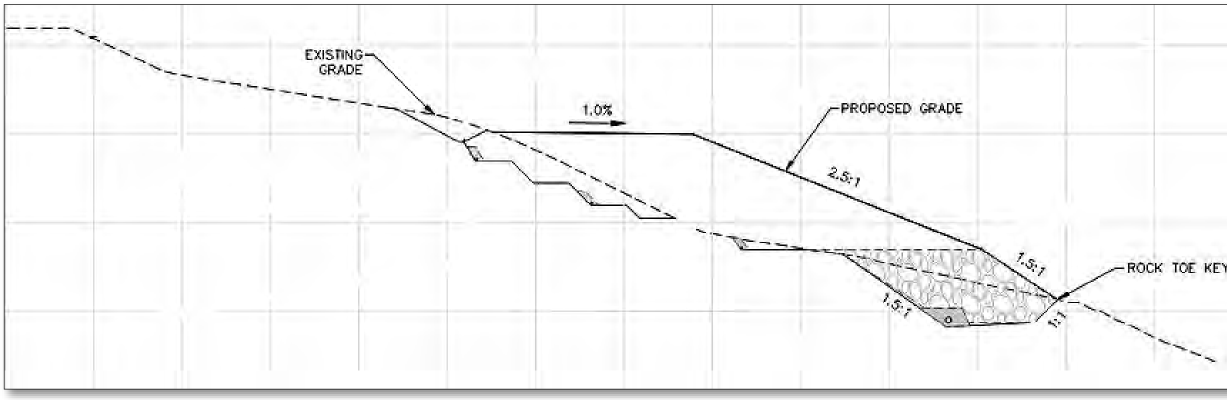


Figure 10-14 - Cross Section - Stability Berm with Toe Key [34]

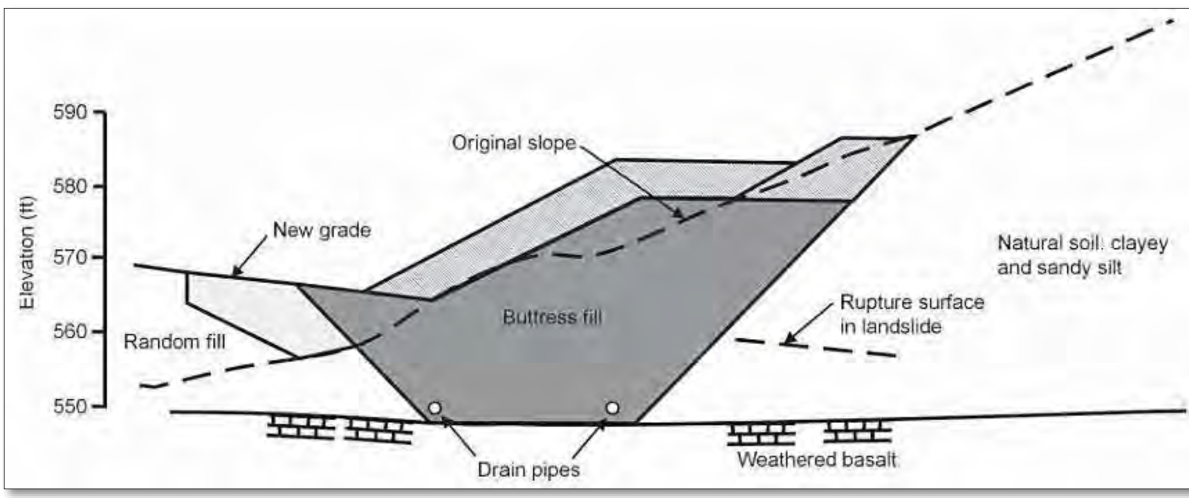


Figure 10-15 - Schematic of Stability Berm with Toe Key [34]

10.3.1.1.3 Extended Fills (Raise Grade at Toe)

Extended fills (Figure 10-16) involve placing fill at the toe of a landslide for an extended length, typically by filling in a valley or grade depression, to reduce the effective slope height. Decreasing the effective height of the slope typically will improve the overall factor of safety against global stability.

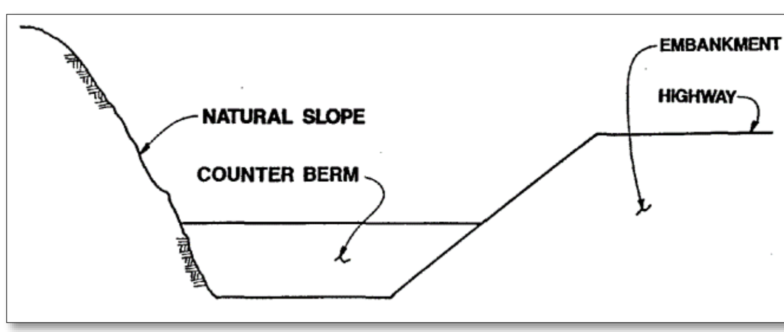


Figure 10-16 - Extended Fills (Raise Grade at Toe) [50]

Some form of subsurface drainage is warranted for this approach, where possible. This approach may be combined with culvert installation where stream crossings are involved; however, the practitioner may encounter an extensive permitting process where waterways are involved.

10.3.1.1.4 Shear Keys with Rock Buttress

Shear keys consist of a series of discontinuous trenches that are excavated in the direction of slope movement and backfilled with stronger material (typically lean concrete) to improve sliding resistance (Figure 10-17). Shear keys are similar to toe keys in concept. A typically shear key configuration may consist of 3- to 5-foot-wide trenches, spaced on the order of 12 to 15 feet center to center, that are filled neat with lean concrete. However, determination of the length and depth of shear key is dependent on the required factor of safety, which will require a slope stability analysis.

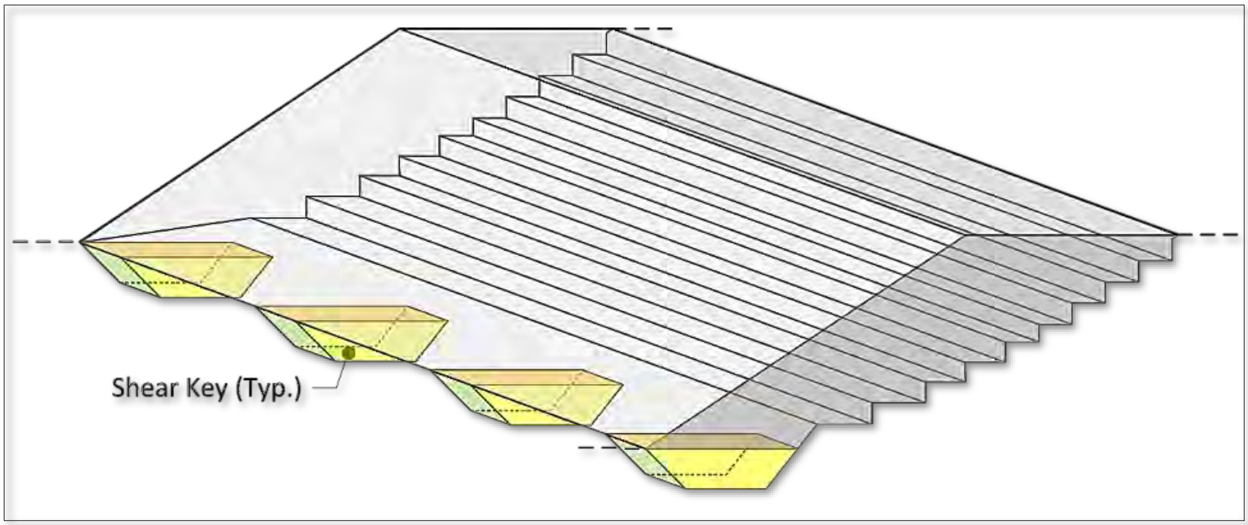


Figure 10-17 - Shear Key with Rock Buttress Sketch

10.3.1.2 Geosynthetic Reinforced Soil (GRS)

Where space is limited, Geosynthetic Reinforced Soil (GRS) has been used as a viable alternative to regrading. GRS generally consists of soil embankment that is strengthened with a series of geosynthetic fabric layers to provide tensile reinforcement to improve global stability.

Similar to regrading, the slide mass is completely removed and replaced with GRS. The side slope for GRS can typically be steeper than an earthen embankment due to the benefit that the geosynthetic contributes to enhancing the internal and external stability of the reinforced soil mass. Reinforcement should be placed across the old failure plane of slope failures when possible, or, as shown in Figure 10-18. See Figure 10-19 and Figure 10-20 for a photograph of a constructed GRS slope.

The advantages of reinforced walls and slopes are as follows [78]:

- On-site and failed (unstable) materials usually may be used.
- Space may be saved when right-of-way or other conditions are restricted.
- Fill requirements may be reduced when compared to unreinforced slopes.
- Provides an economical alternative and may be less expensive than other conventional methods.
- Provides a means of building over weak foundations.
- Tolerates large horizontal and vertical movement.

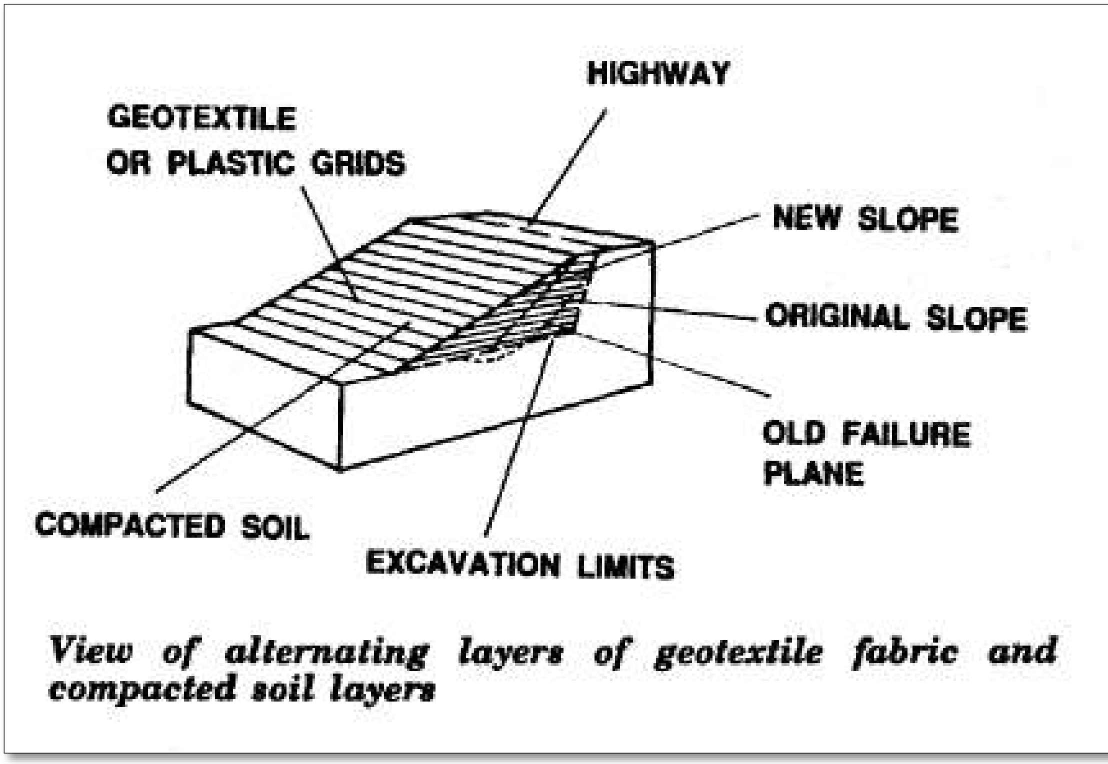


Figure 10-18 - Schematic of a GRS Slope to Repair a Landslide [78]

Figure 10-19 - Constructed GRS Slope Along a Roadway
(PennDOT District 11)

Figure 10-20 - GRS Slope
(PennDOT District 11)

10.3.1.3 Slope Surface Enhancement(s)

Surface runoff at slopes with loamy soils can lead to the formation of deep erosion gullies which diminish the stability of the near-surface soils by infiltration of surface water through the deep gullies. Where instability is due to surface erosion or shallow "carpet slides", slope surface enhancements are a viable repair option. Slope surface enhancements typically entail some form of armoring to protect the slope surface and mitigate the adverse impact from surface infiltration and erosion. Typical slope surface enhancement utilized in the region includes:

- Geocell Slope Protection.
- Turf Reinforcement Mat (TRM).
- Rock Slope Armoring.
- Gabion Slope Wall (Reno Mattress).

10.3.1.3.1 Geocell Slope Protection

Geocell consists of a honeycomb grid of cells that is attached to the surface of a slope, infilled with soil, and seeded to enhance erosion protection and armor slopes to stabilize shallow "carpet" slides.

An example of this method application is depicted in Figure 10-21 through Figure 10-23. For this site, hand labor was used to smooth the ground surface, then the geocell material was laid and anchored to the smoothed ground surface with a series of steel pins. The geocells were then filled with topsoil and the slope was reseeded and mulched.

Figure 10-21 - Shallow slope failure

Figure 10-22 - Ground Preparation for the Geocell System

Figure 10-23 - Geocell Installation

10.3.1.3.2 Turf Reinforcement Mat (TRM)

Turf mat reinforcement is comprised of synthetic material that does not biodegrade and provides support for vegetation on slopes. The main benefit of this enhancement is to decrease soil erosion by providing protection from shear stress that is caused by flowing water.

An example of the formation of rivulets that led to the formation of deep erosion gullies and subsequent slope failure is presented in Figure 10-24 and Figure 10-25. For that instance, cement-bentonite grout was tremie placed in the tension cracks at the ground surface to prevent additional surface water infiltration; the surface was then protected with a Green Armor TRM and hydroseeded with Flexterra Flexible Growth Medium to stabilize the surficial soils and protect against the formation of the erosion gullies (Figure 10-26).

Figure 10-24 - Erosion Gullies in Loamy Soil

Figure 10-25 - Close-Up View of 4+ Foot Deep Erosion Rivulets Near the Ground Surface

Figure 10-26 - Green Armor TRM System with Flexterra Flexible Growth Medium

Another example where surface drainage enhancement, TRM, and hydroseeding were used to stabilize a "carpet slide" is depicted in Figure 10-27 through Figure 10-29.

Figure 10-27 - Head Scarp of Carpet Slide

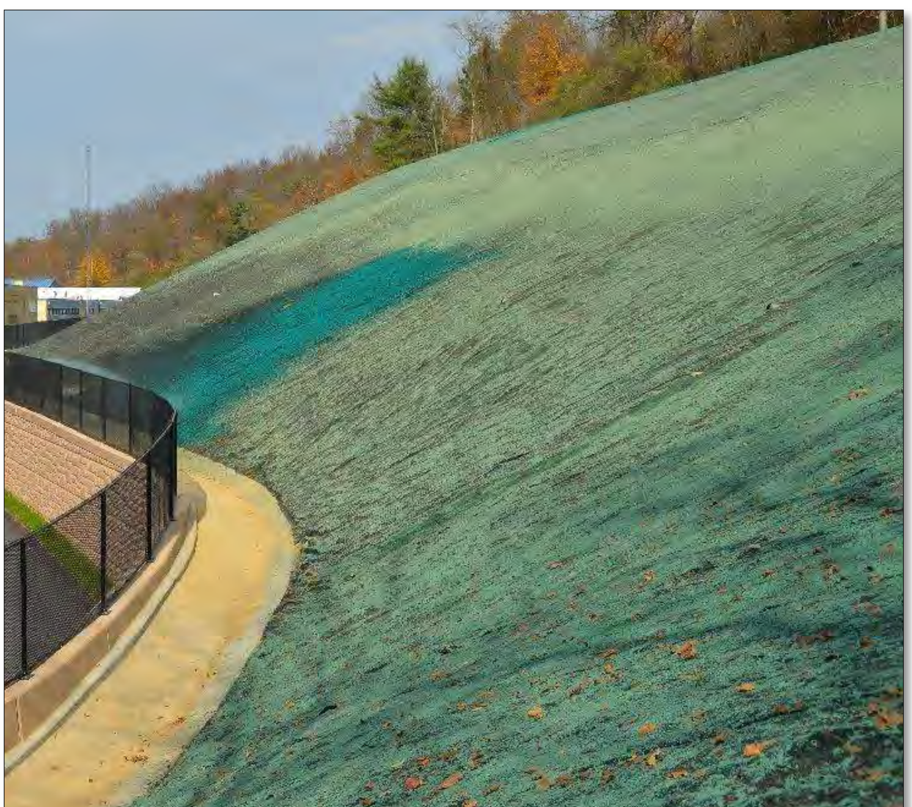


Figure 10-28 - Turf Reinforcement and Hydroseed to Installed Repair and Stabilize Slope

Figure 10-29 - Turf Reinforcement and Hydroseed to Installed Repair and Stabilize Slope

10.3.1.3.3 Rock Slope Armoring

Rock slope armoring (i.e. riprap revetment) is a viable option for slope erosion repair for stability issues related to scour at the toe or surficial landslides. The rock slope armor increases slope stability by providing higher frictional resistance at the surface and adding additional normal stress to the subgrade soils; riprap can also help resist erosion due to surface runoff or stream scour at the toe.

Example rock slope armoring project performed in the region is presented in Figure 10-30 and Figure 10-31.

Figure 10-30 - Rock Slope Armoring

Figure 10-31 - Rock Slope
(PennDOT District 11)

The minimum thickness of the riprap should be 1.5 times d_{50} or d_{100}, whichever is greater, where d_{100} and d_{50} equal the nominal gradation size that 100 and 50 percent of the riprap can pass through that sieve size, respectively. Guidelines for placement of riprap revetment on slopes are presented in Figure 10-32 and Figure 10-33. The ambient bed elevation (shown in Figure 10-33) is the initial (unscoured) bed elevation in a stream, where applicable. The rock slope armoring should at least extend to the base of the stream bed considering long-term degradation and scour; where a buried toe is not feasible, a mounded toe may be employed as part of the rock slope armoring design [44].

The size of the rock used for rock slope armoring (R-3 or larger) may cause issues with subsequent guide rail installation; where this is a concern the upper portion of the riprap (near roadway elevation) may be replaced with AASHTO #57 or AASHTO #1 aggregate. See Section 10.3.1.4 for further discussion.

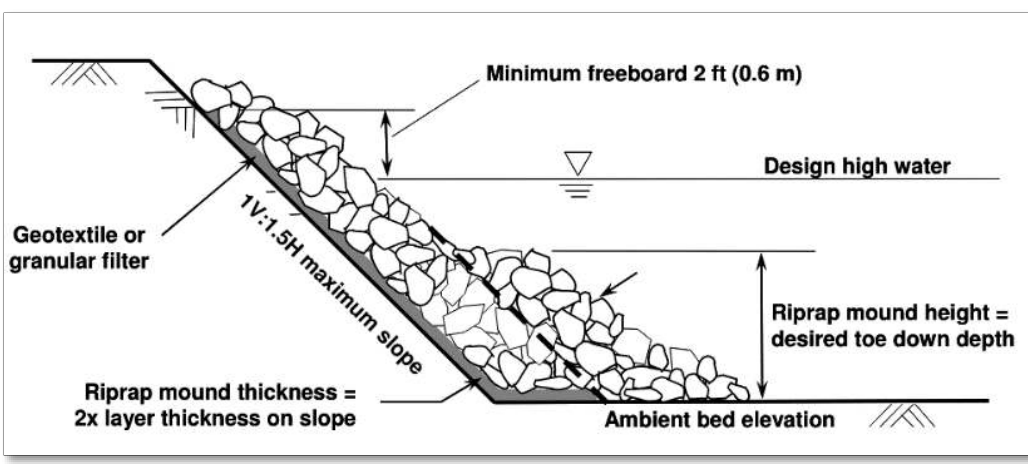


Figure 10-32 - Riprap Revetment with Mounded Toe [44]

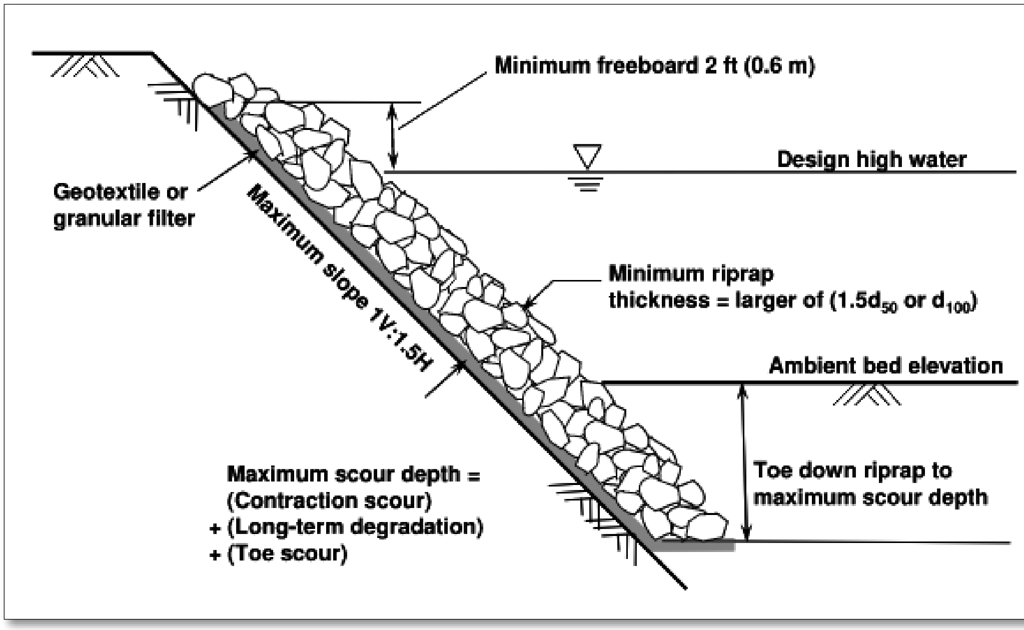


Figure 10-33 - Riprap Revetment with Buried Toe [44]

Analysis Consideration(s).

1) Where scour is a concern and additional H&H analysis may be warranted to discern the minimum required size of the riprap.
2) Two potential issues that need to be addressed during design are the potential for translational and rotational failure of riprap revetments (Figure 10-34 through Figure 10-36).
3) It is important to carefully design, select, and install the filter material that is placed under the riprap. The primary function of the filter is to retain and filter the base soil, be resilient to blinding (clogging from fines), and provide adequate flow capacity to mitigate against the buildup of excess pore pressure that could lead to instability. Nonwoven (NOT woven) geotextile should be used to satisfy filter requirements. See Chapter 12 for additional information on detailing and geosynthetic requirements.

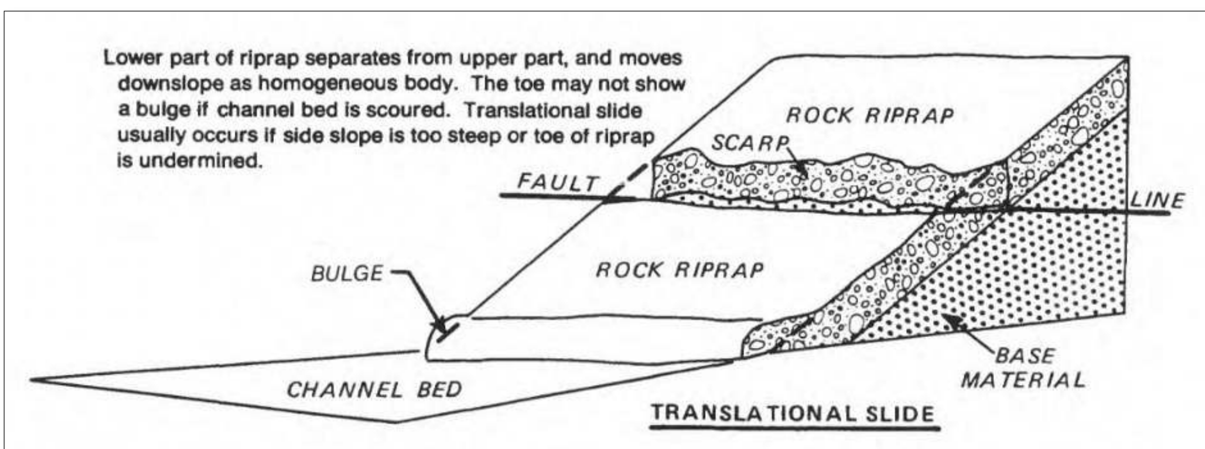


Figure 10-34 - Riprap Failure – Translational [14]

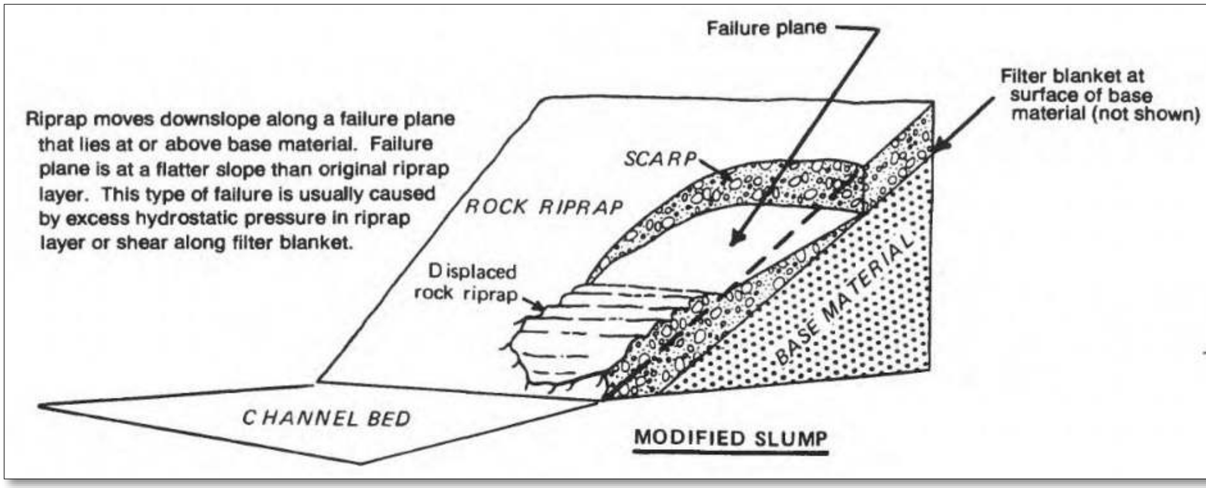


Figure 10-35 - Riprap Failure- Modified Rotational Slide [14]

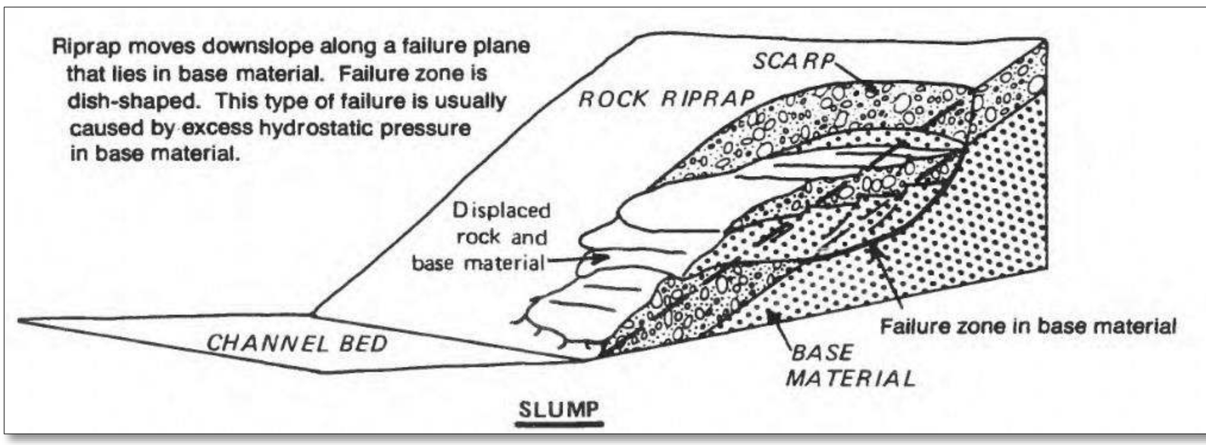


Figure 10-36 - Riprap Failure – Rotational Slide [14]

10.3.1.3.4 Gabion Slope Wall (Reno Mattress)

A gabion slope wall is a variant of a riprap revetment and follows the same general design principles. Gabions can reduce the potential to form a localized failure; however, the basket wires are susceptible to corrosion or breakage so the design life may be reduced with this method. Refer to Figure 10-37 and Figure 10-38.

Figure 10-37 - Reno Mattress Installation [55]

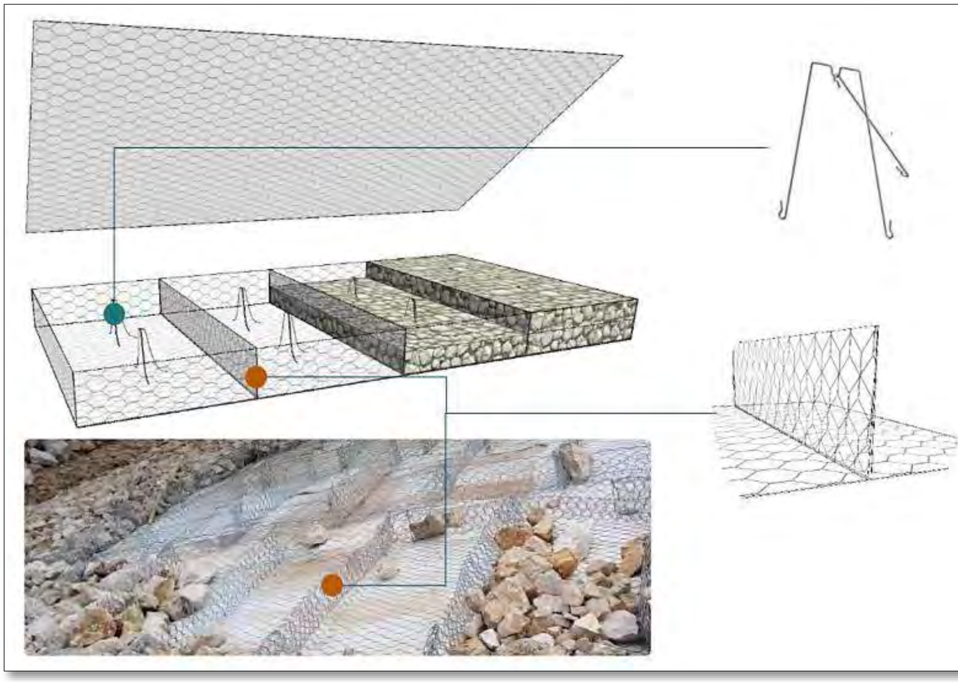


Figure 10-38 - Reno Mattress Schematic [88]

10.3.1.4 Shoulder Back-Up and Moment Slabs

There are circumstances where additional slope treatment is needed at road level to back-up existing shoulder and guide rail. This treatment, referred to as a shoulder back-up, is at times used to “top off” rock slope armoring (Figure 10-39).

Figure 10-39 - Shoulder Back-Up

There are situations when there is insufficient room along a roadway to install the shoulder back-up rock slope that is illustrated above. This is particularly true at locations where a steep slope near the surface may be required or in areas adjacent to an existing stream. In such situations, a possible solution might be to consider using a moment slab with a concrete barrier to replace the guide rail (Figure 10-40).

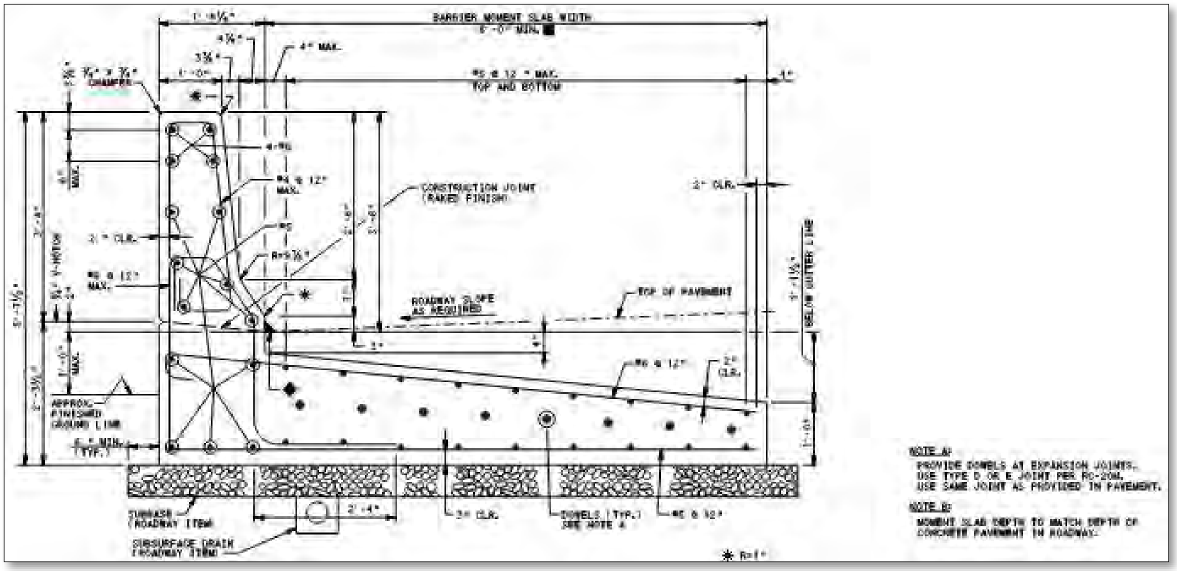


Figure 10-40 - Moment Slab [104]

10.3.1.5 Single-Face Barrier

Where slide debris has extended onto a roadway, excavation at the toe of a cut slope may be required to remove and control the accumulation of slope debris. This is a frequent issue on secondary roads along the "high side" roadway shoulder. In such situations, consideration should be given to assess the viability of placing a single-face barrier at the edge of a shoulder (Figure 10-41). In such situations, consideration should be given to adding a drainage treatment to mitigate ponding and reduce the buildup of excess pore pressure.

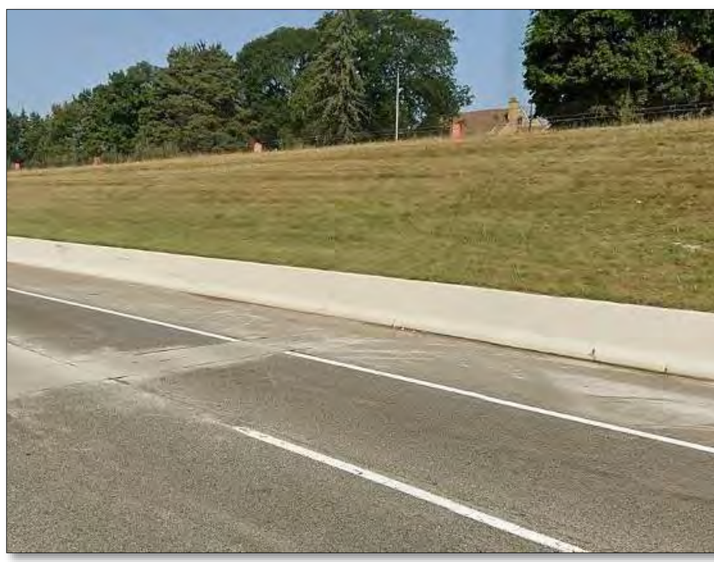


Figure 10-41 - Single-Face Barrier at Toe of Slope [60]

10.3.1.6 Gravity Walls

Gravity walls increase slope stability by providing a stabilizing mass near the toe of the landslide and increasing the resisting force. Gravity walls rely on their mass and the overburden pressure of the backfill to provide adequate shear resistance. Gravity walls are a viable repair method when there is sufficient space to excavate for the footing and construct from the "bottom-up" (versus "top-down") without destabilizing the landslide mass. Gravity walls are a feasible alternative to rock buttresses where the right-of-way or site boundaries are limited. Typical gravity walls constructed for landslide stabilization and repair in southwestern Pennsylvania include:

- Conventional Concrete Gravity Wall.
- Gabion Wall.

- Modular Block Wall.
- Crib Wall.

Analysis Consideration(s).

1) In addition to satisfying global stability (i.e. overall slope stability), the gravity walls themselves must also be stable; thus, the design must also satisfy external stability checks for sliding, overturning, and bearing capacity.
2) For active landslides, the thrust of the slide mass along the failure plane should be quantified and included as a load case for the external stability analysis.

10.3.1.6.1 Conventional Concrete Gravity Wall

Conventional concrete gravity walls may consist of a cantilevered structure consisting of a reinforced concrete stem and footing or a semi-gravity structure with a tapered backwall. Concrete gravity walls are typically used at the toe of a landslide to retain the slide mass (see Figure 10-42). An example of a conventional concrete gravity wall that was used to stabilize a landslide is shown in Figure 10-43.

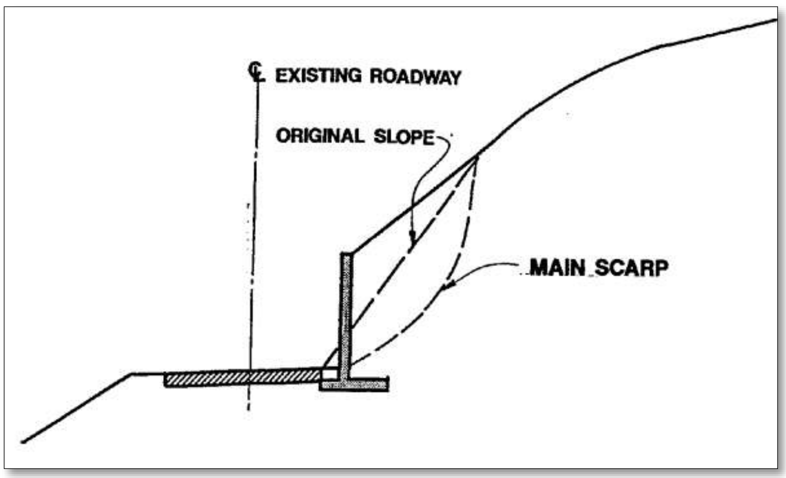


Figure 10-42 - Schematic of a Conventional Concrete Cantilever Wall to Repair a Landslide [50]

Figure 10-43 - Concrete Gravity Wall Construction, Landslide Repair

10.3.1.6.2 Gabion Wall

Gabion walls are comprised of stacked gabion baskets and are typically used to support roadways along local tributaries. Gabion walls are not as stiff as conventual concrete gravity walls and can tolerate more deformation without failure. However, consideration should be given to potential undermining due to stream scour. A local example where stream scour has undermined a gabion wall is presented in Figure 10-44 and Figure 10-45. Due to the high potential for scour, and the susceptibility of the wire mesh to abrasion, these walls may have a reduced service life compared to other landslide stabilization and repair methods.

Figure 10-44 - Gabion Wall

Figure 10-45 - Gabion Wall

10.3.1.6.3 Modular Block Wall

Modular block walls consist of stacked interlocking concrete units. Modular block walls offer a longer lifespan than gabions but are still prone to instability due to stream scour.

Consideration should be given to how the masonry units are interlocked to engage shear resistance and to required setback distance to install guide rail, where applicable.

An example of a modular block wall installation along a stream is presented in Figure 10-47 through Figure 10-48.

Figure 10-47 - Slope Failure along Stream Bed in Washington County, PA (PennDOT District 11)

Figure 10-46 - Modular Block Wall Installed for Landslide Repair in Washington County, PA (PennDOT District 11)

Figure 10-48 - Jumbo Block Slope Repair in Washington County, PA (PennDOT District 11)

Figure 10-49 - Modular ("Jumbo") Block Wall

10.3.1.6.4 Crib Wall

Crib walls consist of an interlocking grid system of stretchers and headers made of timber or concrete; the stretchers and headers are typically placed on a concrete leveling pad and backfilled in lifts with free-draining granular backfill from the "bottom up" to form a gravity structure.

An example crib wall that was used to reconstruct a section of roadway is shown in Figure 10-50 and Figure 10-51.

Figure 10-50 - Distressed Roadway Before Slope Repair

Figure 10-51 - Crib Wall Construction to Repair Slope and Support Road

An example crib wall that was constructed in PennDOT District 11 is presented in Figure 10-52.

Figure 10-52 - Durahold Wall
(PennDOT District 11)

10.3.1.7 Cantilevered Pile Walls

Cantilevered pile walls provide an opportunity to work within a constrained space to stabilize and repair a landslide. This type of retaining wall permits the use of the “top-down” construction method where the walls can be installed before any major excavation; this is especially useful in active landslides where minimal site disturbance is required.

Typical cantilevered walls constructed for landslide stabilization and repair in southwestern Pennsylvania include:

- Soldier Pile and Lagging Wall.
- Buried Solider Pile and Lagging Wall with Bridging Plug.
- Tangent Pile Wall.

Analysis Consideration(s).

1) For active landslides. the thrust of the slide mass along the failure plane should be quantified to size the piles and determine pile spacing and minimum embedment.
2) Where there is potential for future slope movement in front of the wall, consideration should be given to a final slope condition where predicted future soil loss in front of the wall is accounted for.
3) Proper drainage behind these walls is necessary to limit the buildup of unbalanced hydrostatic pressure against the wall elements.
4) Arching should be considered for the analysis of discreet piles.

5) Where piles are placed at close spacing, a reduction in lateral resistance for discreet piles may be considered where applicable.

10.3.1.7.1 Soldier Pile and Lagging Wall

Soldier pile and lagging walls consist of soldier piles, typically installed at 8- to 12-foot center-to-center spacing, with structural elements (i.e., lagging) placed between the flanges to support the soil between the piles. Typical structural elements may include timber lagging, precast concrete lagging, steel plates, or cast-in-place concrete wall panels. Timber lagging or reinforced shotcrete is typically used to provide temporary excavation support until such time a permanent lagging or wall facing is installed. Prefabricated strip drains are typically installed with these walls to limit the buildup of unbalanced hydrostatic pressure against the wall elements. An example of a landslide repair that used a cantilevered soldier pile and precast concrete lagging is presented in Figure 10-53 and Figure 10-54.

Figure 10-53 - Landslide Area, Before Stabilization with a Soldier Pile and Lagging Wall

Figure 10-54 - Landslide Area, After Stabilization with a Soldier Pile and Lagging Wall

10.3.1.7.2 Buried Soldier Pile Wall with Bridging Plug

Where there are significant site constraints, a cantilevered soldier pile wall can be buried below a moment slab. Since only a minor portion of the wall is left exposed, the concrete lagging can be replaced by a bridging plug.

A bridging plug is constructed by performing a vacuum excavation between the soldier piles and casting the concrete in place between the flanges; this allows for minimal excavation disturbance and execution without the need for additional right-of-way. This method relies on stand-up time of the surrounding soils so that the excavation remains open for rebar and concrete placement neat against the existing soils.

Figure 10-55 - Landslide at edge of shoulder

For example, a landslide occurred along a roadway where the right-of-way was less than 7 feet from the edge of the shoulder, and acquisition of additional right-of-way was not an option (Figure 10-55). A buried cantilevered soldier pile wall with a bridging plug was the preferred solution for this site due to the space constraints; an illustration of a moment slab that is underlain by a buried soldier pile wall and bridging plug is shown in Figure 10-56.

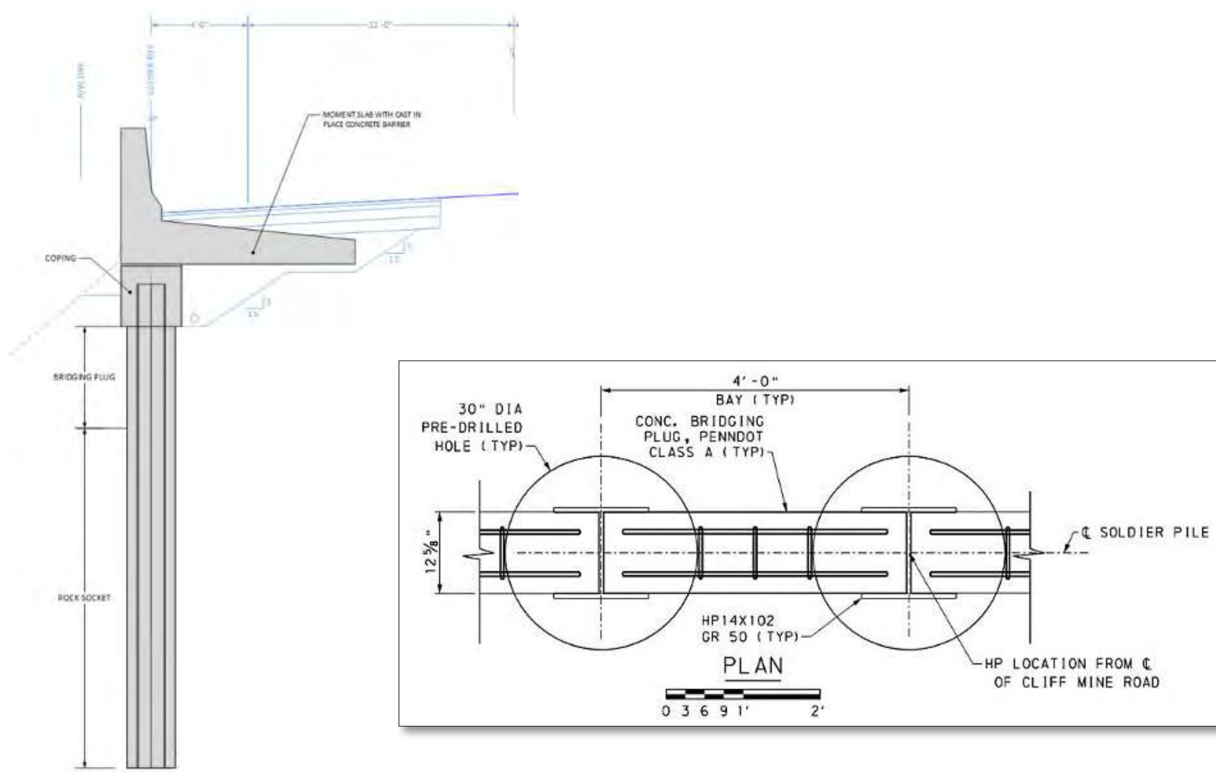


Figure 10-56 - Typical Section (left) and Plan View (Right) – Buried Soldier Pile Wall with Bridging Plug

10.3.1.7.3 Tangent Pile Wall

Tangent pile walls consist of a continuous row of drilled shafts that are installed at center-to-center spacing approximately equal to the shaft diameter. A small gap (e.g., 6 to 12 inches) is typically left between the drilled shafts to mitigate against constricting groundwater flow through the wall and permit clear space to install horizontal drains, if required. Steel dowels may be drilled and embedded into the drilled shafts to attach a wall facing that consists of reinforced shotcrete or conventional reinforced concrete wall facing with prefabricated wall drains. See Figure 10-58 through Figure 10-60 for photographs of tangent pile wall construction.

Due to the small gap that is left between the tangent piles, this method will typically rely on soil arching to retain soil. Significant research on the effect of arching on this type of wall system has been done to restrain landslides by the Ohio Department of Transportation. For more detail, refer to FHWA General Engineering Circular (GEC) No. 9 [47].

The shear loading on the tangent piles within an active slide may be significantly higher than typical active earth loading; where additional structural resistance is needed drilled shafts may be stiffened with a steel rolled shape that is embedded within the drilled shaft.

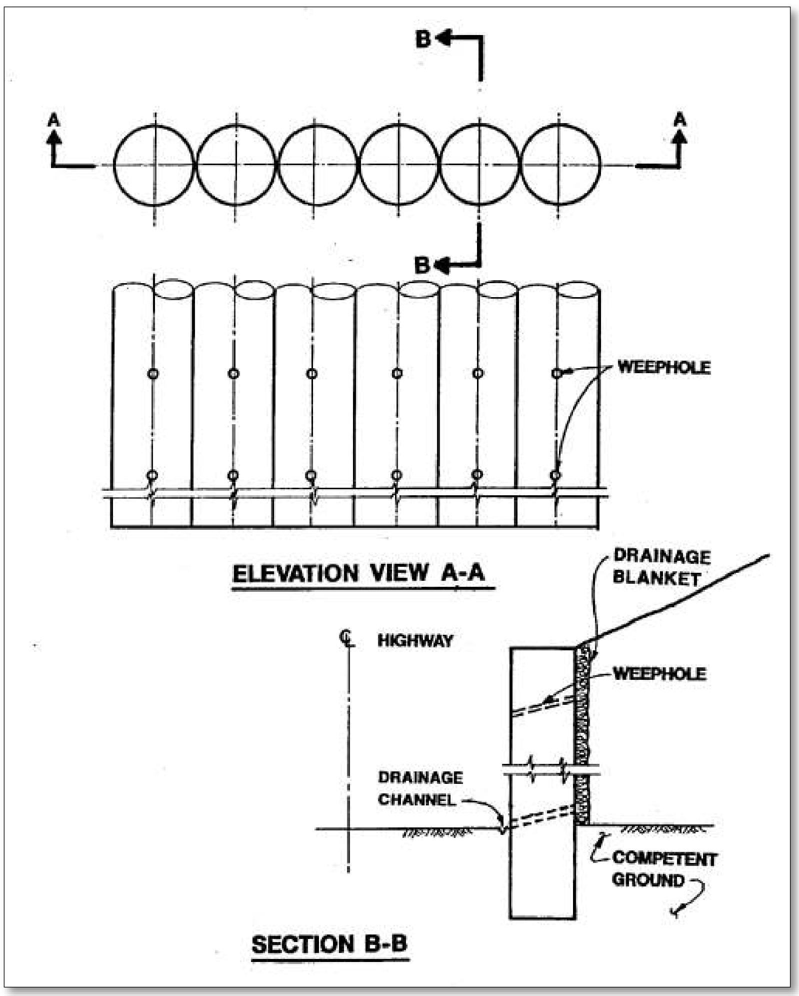


Figure 10-57 - Tangent Pile Wall Plan, Section, And Elevation [50]

Figure 10-58 - Tangent Pile Wall [48]

Figure 10-59 - Tangent Pile Wall Supporting Roadway Shoulder [23]

Figure 10-60 - Tangent Piles (PennDOT District 11)

10.3.1.8 Tieback (Ground) Anchors

Where sufficient resistance is not able to be reasonably or economically developed - tiebacks may be utilized to provide additional shear resistance directly through the slide mass. Due to the load resistance of the tieback anchors, they may also allow for a less robust wall design compared to a cantilevered wall. Tieback anchors may also be preferred where wall deflection is a concern.

Tieback anchors consist of a bond zone that is developed in stable material and a steel tension member to engage the bond zone to resist lateral load that is applied to the wall facing. The tension member may consist of low relaxation prestressed wire strand(s), all thread bar (Grades 75 and 150), reinforcing steel, or other steel shape that is capable of providing tensile strength.

Where space is limited, equipment is available to install tiebacks at locations with restricted space. An example of where space was limited, and alternate equipment was utilized is depicted in Figure 10-61 and Figure 10-62. For this project, tiebacks were installed on top of an existing mechanically stabilized earth (MSE) wall that was moving with a landslide downslope.

Typical applications for tieback anchors for landslide stabilization and repair in southwestern Pennsylvania include:

- Soldier Pile Wall with Tiebacks.
- Buried Panel Wall with Tiebacks.
- Sheet Pile Wall with Tiebacks.

Figure 10-61 - Example Tieback Installation with a Small Duplex Drill on Top of an Existing MSE Wall, Allegheny County

Figure 10-62 - Example Tremie Grout Placement for Tieback with Grout Tube, Allegheny County, PA

Analysis Consideration(s).

1) Tieback (ground) anchors must develop resistance in the stable mass beyond the rupture plane. The potential to develop a deeper-seated failure should be taken into consideration (see Figure 10-63). For instance, the author is aware of at least one instance where internal erosion led to the development of a deeper-seated slope failure that started to engulf part of the anchor bond zone, continued progressive slope movement, and the eventual need to install more soil anchors with an anchor bond zone that was developed at greater depth.
2) Earth pressure distribution for anchored walls should be considered for the final condition.

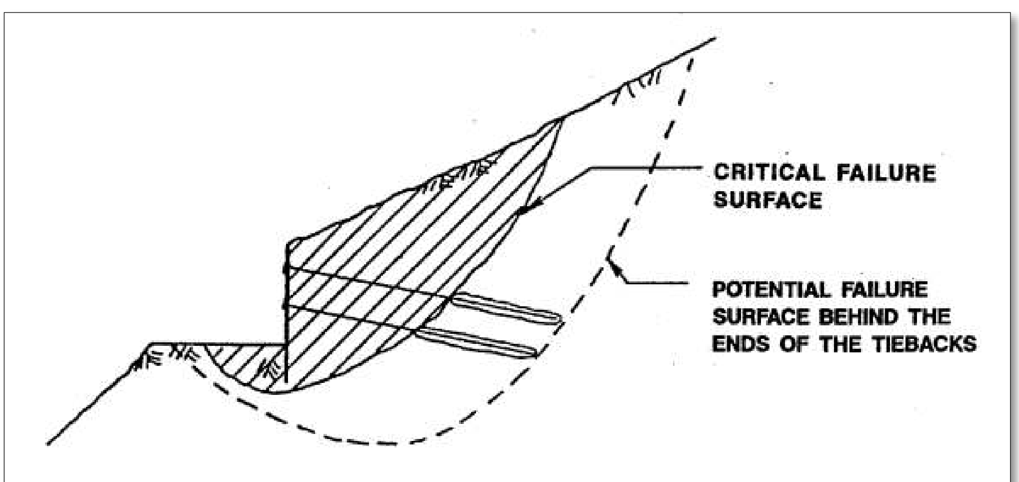


Figure 10-63 - Tieback Anchors to Stabilize a Slide Mass [50]

10.3.1.8.1 Soldier Pile Wall with Tiebacks

Soldier pile walls with tiebacks follow the same general concept as cantilevered soldier pile and lagging walls, except that tieback anchors are used to provide supplemental lateral support. Load at the anchor head may be transferred directly to the soldier pile through the concrete facing, through an external waler, or an internal waler. Tiebacks are often inclined between 10 and 45 degrees from horizontal.

These types of walls may be utilized in areas of very high loads (e.g., large active landslide) or where the top of rock is deep. See Figure 10-64 and Figure 10-65 for construction photographs of tieback soldier pile walls that were used to stabilize landslides in the region.

Figure 10-64 - Example Soldier Pile Wall with Tiebacks, Allegheny County, PA

Figure 10-65 - Example Tieback Wall with Timber Lagging and Multistrand Anchors, Allegheny County, PA

10.3.1.8.2 Buried Panel Wall with Tiebacks

A buried panel wall with tiebacks offers an opportunity to stabilize active shallow earth flow type slides (e.g., "carpet slides"), which frequently occur in soils that are geologically associated with the Dunkard geologic Group.

An example buried panel wall application that was used to stabilize a "carpet" slide is illustrated in Figure 10-66 through Figure 10-71. To construct the buried panel wall, a trench excavation was made, a rebar cage was placed with a steel sleeve and styrofoam blockout, and concrete was placed neat into the trench (without formwork). After several weeks of concrete curing, the concrete panel was partially exposed on the downslope side, so that rock anchors could be installed through the styrofoam blockout. Following installation, the rock anchors were tested and locked off, then fill was placed to bury the exposed face of the concrete panel. The burial of the concrete panel eliminated the need for a formed finish.

Figure 10-66 - Example Buried Panel Wall with Tiebacks, Trench Excavation Through Slide Mass

Figure 10-67 - Example Buried Panel Wall with Tiebacks, Rebar Cage with Anchor Sleeve, and Styrofoam Blockout

Figure 10-68 - Example Buried Panel Wall with Tiebacks, Trench After Rebar Cage is Set and Concrete is Placed Neat

Figure 10-69 - Example Buried Panel Wall with Tiebacks, Front Face of Panel Partially Exposed

Figure 10-70 - Example Buried Panel Wall with Tiebacks, Drilling to Install Tiebacks

Figure 10-71 - Example Buried Panel Wall with Tiebacks, Anchors Tested, Stressed, Locked Off, and Capped

10.3.1.8.3 Sheet Pile Wall with Tiebacks

There are instances where sheet pile is required instead of temporary timber lagging, to provide temporary excavation support. For landslide applications, tiebacks are often used with sheet piles to provide supplemental support. External walers may also be used to increase the anchor spacing and thus reduce the number of tiebacks required. An example of an anchored sheet pile wall is shown in Figure 10-72.

Figure 10-72 - Installation of Tiebacks During Sheet Pile Wall Construction

10.3.1.9 Soil Nails

Soil nails are closely spaced grouted steel bars (i.e., nails) that are used to provide passive resistance to a slide mass and stabilize landslides. Soil nails are grouted over their full length and are not tensioned during installation; soil nails require some amount of slope movement to develop resistance.

Soil nails are typically installed in staggered patterns with an average tributary area of 20 to 25 square feet per soil nail. The primary components of a soil nail wall (e.g., hollow bar, a bit that is left in place after completion of drilling, external bar coupling, steel bearing plate, beveled washer when needed, and heavy-duty hex head nut) are shown in Figure 10-73 and Figure 10-74. Soil nails are typically inclined at 10 to 20 degrees. Beveled washers are available to make inclination adjustments in 5- to 10-degree increments; a typical beveled washer is illustrated in Figure 10-73 The exposed cut face is supported with a variety of materials ranging from wire mesh to fiber-reinforced shotcrete (Figure 10-76 and Figure 10-77). After the soil nails and wire mesh are installed, the stabilized slope is hydroseeded or lined with riprap to provide erosion control (Figure 10-78 through Figure 10-80).

Since soil-nailed slopes are typically constructed top-down, they are an option for installation in confined areas. Drilling attachments are available to be fitted to the end of the boom for a hydraulic excavator, similar to that shown in Figure 10-75.

A modular block wall can be added in front of a soil-nailed slope where aesthetics are a concern, similar to that shown in Figure 10-79 and Figure 10-80.

Some conditions that may preclude the use of soil nails include sites with poor soils, high plasticity soils, and/or a high groundwater table.

Figure 10-73 - Soil Nail

Figure 10-74 - Soil Nails

Figure 10-75 - Soil Nail Drilling

Figure 10-76 - Soil Nails and Wire Mesh

Figure 10-77 - Soil Nails and Wire Mesh

Figure 10-78 - Hydroseeded Soil Nail Slope

Figure 10-79 - Shotcrete and Modular Block Installation in Front of Soil Nail Slope

Figure 10-80 - Shotcrete and Modular Block Installation in Front of Soil Nail Slope

10.3.1.10 Soil Nail Launcher

The Soil Nail Launcher (Figure 10-81) is a variant of the soil nails that are described above. In lieu of drilling and grouting to install soil nail (bars), steel or fiberglass tubes are launched into the ground using a compressed air cannon. The primary use of the Soil Nail Launcher is to repair shallow slope failures at a quicker rate then typically needed to execute other retaining structures. According to the manufacturer, the equipment is capable of accelerating a 1.5-inch diameter, 20-foot long, solid steel bar to a velocity in excess of 200 miles per hour. The Soil Nail Launcher is also able to perform within a small footprint for minimal impact to the project site (Figure 10-82).

Figure 10-81 - Soil Nail Launcher [13]

Figure 10-82 - Soil Nail Launcher [57]

10.3.1.11 Articulated Micropile

Micropile has been used successfully for decades to stabilize landslides. Articulated micropile rely upon frame action with a cap beam and to engage a portion of the micropile in tension and another portion in compression to stabilize lateral movement. The terms used to describe this method have varied but the principle is fundamentally the same; over time this method has been referred to as “root piles”, “pin piles”, “dowels”, “articulated piles”, and “A-frame anchor piles”, among others. See Figure 10-83 and Figure 10-84 for examples of this method.

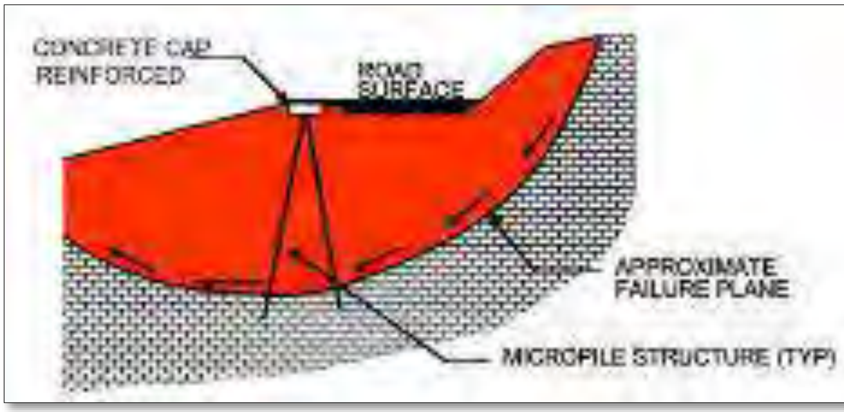


Figure 10-83 - Articulating Micropile Schematic [50]

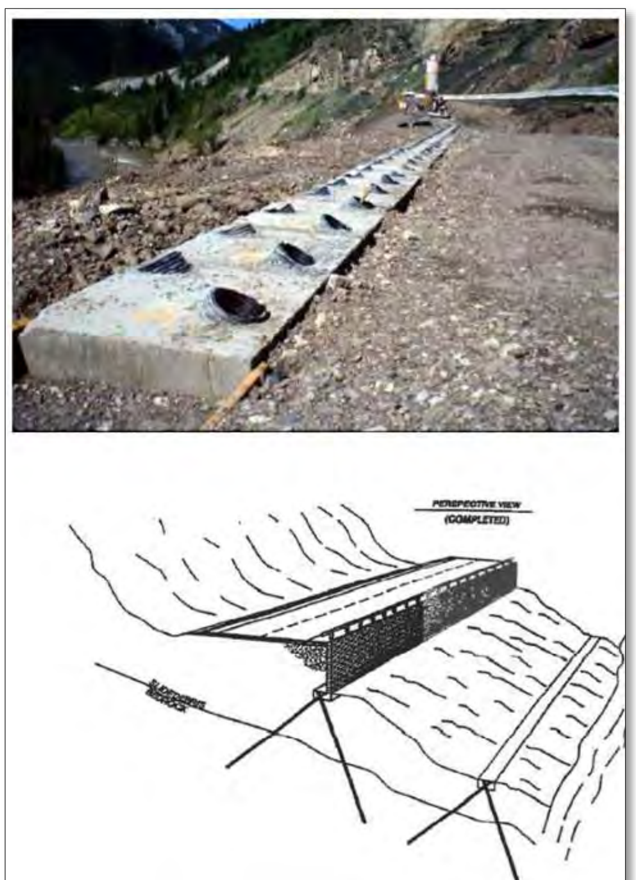


Figure 10-84 - Example “Articulated” Micropile at Base of GRS Wall [130]

10.3.1.12 Pipe Dowels

Pipe dowels consisted of an array of buried pile elements consisting of pipes, dowels, solider piles, or drilled shafts which rely on soil arching to retain the overburden soils (see Figure 10-85). Pipe dowels provide the opportunity to work in limited space with minimal site disturbance. These are of particular interest at landslides that are at or near equilibrium, where additional ground disturbance could induce significant slope movement.

Analysis Consideration(s).

1) For active landslides, the thrust of the slide mass along the failure plane should be quantified to size the piles and determine pile spacing and minimum embedment.
2) Where there is potential for future slope movement in front of the wall, consideration should be given to a final slope condition where predicted future soil loss in front of the wall is accounted for.
3) Soil arching should be considered for the analysis of discreet piles. For more detail, refer to FHWA GEC 9 [47].
4) Where piles are placed at close spacing, a reduction in lateral resistance for discreet piles may be considered where applicable.

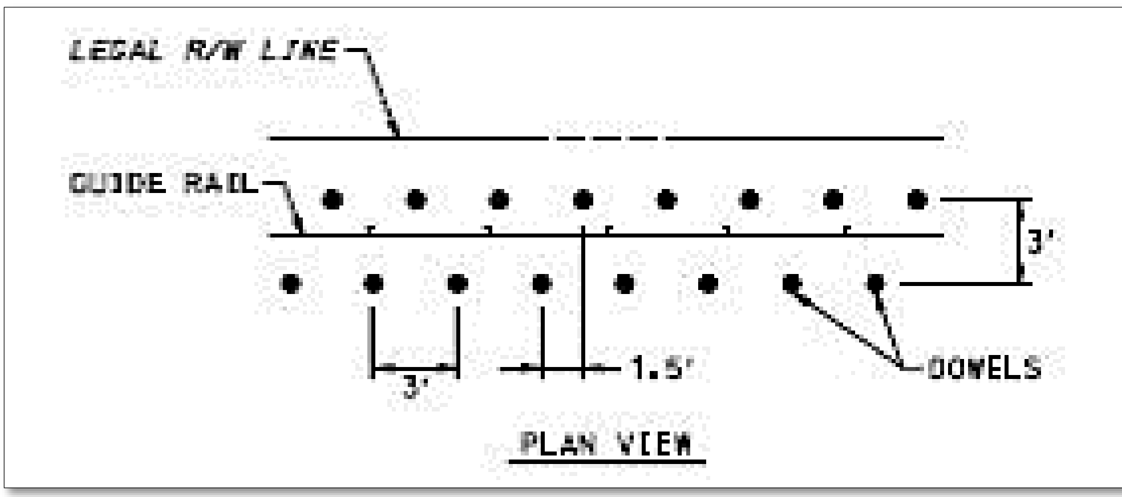


Figure 10-85 - Plan View of Pipe Dowels for Landslide Stabilization (PennDOT District 12)

10.3.2 Rebalance Ratio Between Mobilized Resistance and Driving Force(s)

Landslide movement can be stabilized by rebalancing mobilized resistance and driving force(s). Such a rebalancing method will typically involve the combined action of reducing the driving force and increasing the available resistance. Common methods that are used to rebalance landslide loads include:

- Surface Drainage Improvement.
- Subsurface Drainage Improvement.
- Lightweight Fill (to replace part of the landslide mass).
- Partial Unloading (at top of slide mass).
- Slope Flattening.
- Removal and Replacement (of slide mass).

Analysis Consideration(s).

1) Rebalancing the mobilized resistance and driving forces of a slope in not only determined by the size or shape of the improvement, but also by the position on the slope. Hutchinson [79] provides details of the "neutral line" method to assist in finding the best location to place a stabilizing fill or cut.

10.3.2.1 Surface Drainage Improvement

Control of surface drainage is a key part of landslide stabilization and repair. Concentrated runoff can lead to the formation of erosion gullies, which allow surface flow to enter the subgrade through tension cracks and may eventually cause a substantial increase in the driving force to the point that stability equilibrium is compromised. Therefore, surface drainage systems should be designed and maintained to remain functional and effective. An illustration of how surface drainage can exacerbate slope movement is presented in Figure 10-86.

Figure 10-86 - Concentrated Overland Flow, Leading to an Eventual Landslide, Fayette County, PA

Existing surface drainage systems, including ditches and drains, should be repaired to remove any obstructions and convey surface water away from the slope and slide mass. Where site drainage conditions have changed, additional drainage systems may be necessary; this is especially important near the crown of the slide mass. Ditch drainage systems should keep a minimum 2 percent gradient along the length of the ditch near the slide mass to minimize the risk of forming dips and subsequent ponding on the slope.

Overland sheet flow is typically preferred over concentrated overland flow (Figure 10-86) to minimize the risk of creating erosion gullies that can activate slope movement.

Typical surface drainage options for landslide stabilization and repair in southwestern Pennsylvania include:

- Upslope Interception and Diversion.
- Ditch Lining.
- Surface Reshaping/Regrading.
- Sealing Open Tension Cracks.

In addition to surface drainage infrastructure, effective erosion control is important to manage surface drainage. There are a variety of methods of erosion control available including Turf Reinforcement Mats (TRMs) and hydroseeding applications. See Section 10.3.1.2 for further detail.

10.3.2.1.1 Upslope Interception and Diversion

If possible, opportunities should be considered to intercept surface runoff and convey flow in a controlled manner. Figure 10-87 illustrates one such example, where a series of sandbags and HDPE pipe was successfully used to divert surface runoff away from an active landslide.

Figure 10-87 - Example of How Sandbags and HDPE Pipe Are Used to Divert Flow from an Active Landslide

10.3.2.1.2 Ditch Lining

A lined ditch can be an effective means to promote the conveyance of surface runoff in a controlled manner. Examples of lined conveyance ditches include:

- Rock-lined ditches.
- Grouted rock-lined ditches.
- Fabriform lined ditches.
- Smart Ditch.

A "Smart Ditch" can help to reduce reliance on material transport (e.g., trucking to deliver rock or redi mix concrete) where ditch construction is required. The benefit of a "Smart Ditch" over a reinforced concrete lined ditch is apparent for a ditch that is difficult to access by anything over than a track hoe.

See Figure 10-88 through Figure 10-91 for example photographs of lined conveyance ditches.

Figure 10-88 - Smart Ditch

Figure 10-89 - Rock Lined Ditch

Figure 10-90 - Grouted Rock-Lined Ditch

Figure 10-91 - Fabriform Lined Ditch

10.3.2.1.3 Surface Reshaping/Regrading

The surface of the slope may be scarified and re-graded to eliminate localized depressions and reduce potential ponding of surface water on the slope.

10.3.2.1.4 Sealing Open Tension Cracks

Particular attention needs to be given to a temporary rise in the driving force when surface water seeps into and fills tension cracks near the ground surface. When landslide movement has been activated by a rise in pore pressure along the rupture plane, dissipation of pore pressure will often occur as the landslide movement occurs. This dissipation in pore pressure will oftentimes permit the landslide movement to temporarily slow down or stop until the pore pressure has had time to build up again. Therefore, identifying and sealing tension cracks within the slide mass area is critical to prevent surface water from infiltrating and reaching the failure plane. Typical materials used to seal open tension cracks include grout, compacted soils, and bentonite. See Figure 10-92 for a photograph of an observed tension crack prior to crack sealing.

Figure 10-92 - Tension Crack

10.3.2.2 Subsurface Drainage Improvement

Subsurface drainage can improve slope stability by lowering the piezometric surface(s) acting on the sliding mass. Subsurface drainage aids to increase resisting forces through the reduction of pore pressure and increase in the effective shear strength that is acting along the rupture plane. Subsurface drains may also help to eliminate spikes in the seasonal buildup of pore pressure during periods of higher-than-normal precipitation.

It is worth pointing out that the benefit of enhanced subsurface drainage, as well as the selection of installation locations, can at times be difficult to predict. For example, there are periods when the presence of intermittent (e.g., seasonal) seeps is difficult to identify in the field, particularly during dry seasons. Additionally, the interception of one seepage exit point on the ground surface may not necessarily prevent another seepage pathway from manifesting. A significant lag time may be required to effectively drain cohesive soils, especially the slide-prone red bed soils of southwestern PA.

Typical subsurface drainage options for landslide stabilization and repair in southwestern Pennsylvania include:

- Interceptor (French) Drains.
- Spring Drains.
- Finger Drains.
- Horizontal Drains.

10.3.2.2.1 Interceptor (French) Drain

French drains are particularly useful to intercept, manage, and convey a line of groundwater springs and seeps near their exit point at the ground surface. A buildup of excess pore pressure at the exit point can lead to internal erosion and piping of fine-grained soil, particularly loamy soil, and lead to shallow slope failures (e.g., “carpet” slides). French drains are also useful to manage surface water infiltration that seeps into and flows through surficial turf, sod, and topsoil.

French drains are typically comprised of a lined trench with a perforated pipe that is backfilled with free-draining material. An example of an interceptor drain is presented in Figure 10-93.

Figure 10-93 - Interceptor (French) Drain

10.3.2.2.2 Spring Drain

Spring drains are similar to french drains; however, spring drains are designed to target a specific area or point where a groundwater spring or seep is identified rather than collecting from a large area. An example of a spring drain is presented in Figure 10-94.

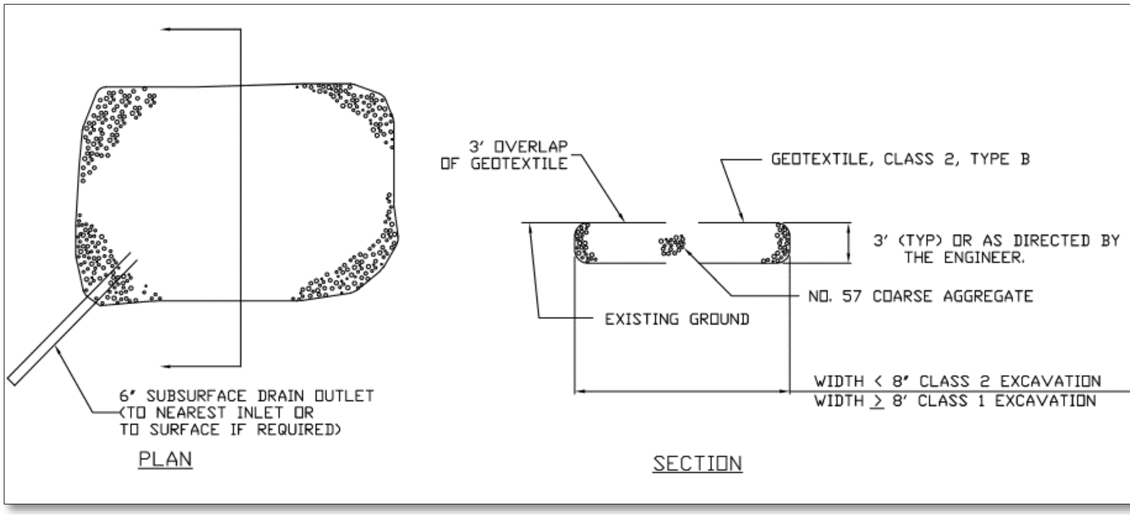


Figure 10-94 - Spring Drain
(PennDOT District 12)

10.3.2.2.3 Finger Drains

Finger (e.g., slot) drains are designed to target, intercept, collect, and convey water that is emanating from groundwater springs and seeps away from the slide-prone mass. Finger drains consist of a shallow excavation (e.g., trench) that is typically lined with a geotextile filter fabric and backfilled with free-draining coarse aggregate. The purpose of the geotextile filter and coarse aggregate is to retain and filter the surrounding base soil (geotextile) while providing sufficient flow capacity (free-draining coarse aggregate). These trenches are typically shallow, produce minimal surface disturbance, and can be constructed in small sections which are preferable to minimize the risk of destabilizing the slope during drain installation.

An example schematic of a finger drain is presented in Figure 10-95. An example construction sequence where finger drains were used to stabilize a slide mass is presented in Figure 10-96 through Figure 10-100.

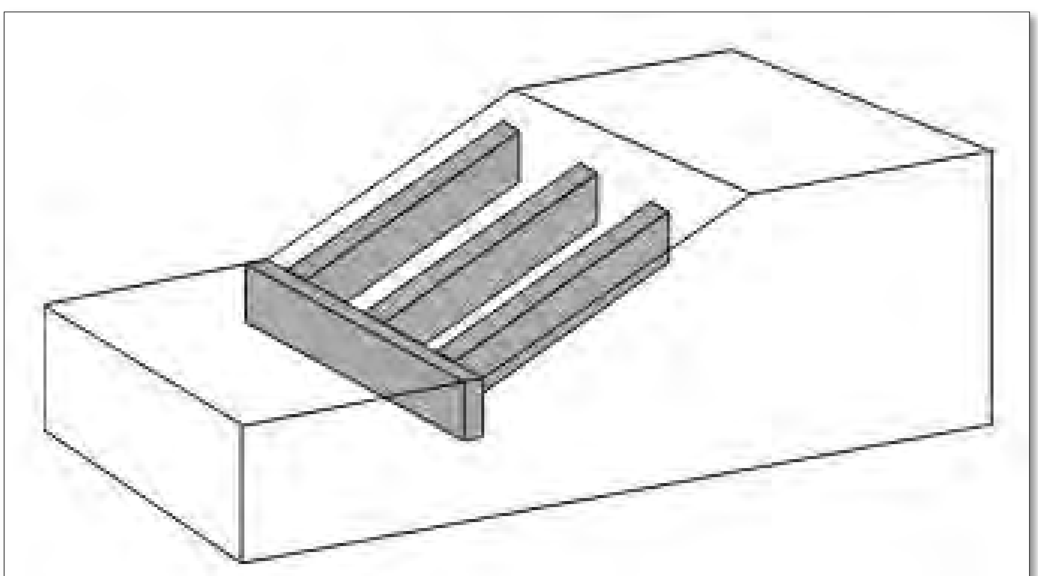


Figure 10-95 - Schematic Finger Drain Concept [34]

Figure 10-96 - Head scarp forming due to loss of shear strength from seepage

Figure 10-97 - Seepage and erosion gullies formed on the slope below the head scarp

Figure 10-98 - Slope cleared to allow for identification of seepage points and installation of finger drains

Figure 10-99 - Finger Drains on Slope

Figure 10-100 - Finger Drains on Slope

10.3.2.2.4 Horizontal Drains

Horizontal drains increase slope stability by lowering the piezometric surface and reducing the porewater pressure acting on the rupture plane. Due to the frequent relationship between elevated piezometric head and slope failure in southwestern PA, horizontal drains are typically an effective method to stabilize and repair landslides. However, to be effective, the drains must penetrate the failure surface and successfully intercept and drain the source of water. Due to the unique hydrologic conditions in the region, including variance in seasonal groundwater table and the presence of perched water, the design of these drainage systems should be tailored to site-specific subsurface conditions.

Horizontal drains are constructed by drilling to the desired depth and installing surface casing. The casing is then cleared of soil, and sections of slotted PVC drainpipe encapsulated with geotextile filter fabric are pushed into the surface casing and coupled together. Following the installation of the PVC drainpipes, a screen is installed over the exposed end of the horizontal drain. It is important to note that the drain holes should be thoroughly cleaned of drill cuttings and mud; uncleaned holes are significantly less effective, possibly only 25 percent effective [75].

Due to the generally free-draining nature of sandy soils, the groundwater table will typically lower within a few months but will likely fluctuate with rainfall. In clayey soils, the full change in the groundwater table may take up to five years, with 50 percent of the improvement taking place within the first year. However, once water tables are lowered in clayey soils, the change is fairly permanent. Seasonal fluctuations may still occur, but rainfall will generally not alter the ground-water level in clayey soils provided the horizontal drains do not clog [75].

An example installation of a horizontal drain is depicted in Figure 10-101 through Figure 10-104.

Figure 10-101 - Drilling to Install Horizontal Drains, Allegheny County, PA

Figure 10-102 - Drilling to Install Horizontal Drains, Allegheny County, PA

Figure 10-103 - Slotted Well Screen and PVC Conveyance Pipe Through Surface Casing to Install Horizontal Drains

Figure 10-104 - Completed Horizontal Drains, Note Seepage Emanating Beside Surface Casing for Drain

10.3.2.3 Lightweight fill

Figure 10-105 - Geofoam for Landslide Repair [124]

Lightweight fill is used to stabilize and repair landslides by removing and replacing a portion of the landslide mass with a lighter material, thus reducing the driving force(s) to improve the factor of safety against slope stability. Lightweight fills may offer similar strength to compacted soils with as low as one-quarter of the density.

Where a structure such as a roadway is at/near the top of the slope, soil or rock fill may be placed with discreet thickness at the top to improve subgrade stability to support the pavement.

The topic of the beneficial use of recycled material frequently is entertained when the option of lightweight fill is considered. The author is aware of at least one instance in southwestern Pennsylvania where shredded tires were used as lightweight fill. Other lightweight materials that have been considered in southwestern Pennsylvania are expanded shale, cementitious Elastizell, expanded polystyrene geofoam, and low-density cellular concrete (i.e., foam concrete).

An example of a geofoam application to stabilize a landslide is illustrated in Figure 10-105. This site has a long history of slope failures, and conventional remedies had failed to correct the problem. Geofoam enabled engineers to reduce the landslide driving force without lowering the grade at the head of the slide.

An example of where Elastizell was used to stabilize a landslide is illustrated in Figure 10-106.

Figure 10-106 - Elastizell for Landslide Repair [36]

10.3.2.4 Partial Unloading (Removal of Material at Top)

Partial unloading is a method that is typically used to slow and/or arrest slope movement for deep-seated landslides. Removal of a sufficient quantity of the sliding mass at the top of the landslide will typically yield a significant reduction in driving force with a small reduction in sliding resistance, creating a net increase in the factor of safety against slope stability.

Unloading should consider the stability of the temporary excavation slope that is created during the unloading process. The material that is removed during unloading can be used to form a stability berm and/or contribute to net slope flattening, which can also aid in improving the factor of safety against slope stability. An example of where partial unloading was used to slow the rate of movement at an active landslide is depicted in Figure 10-107; at this site, the removed slide mass was utilized as a temporary stability berm at the toe of the landslide mass until the final design could be completed.

Figure 10-107 - Partial Unloading of Slide Mass

10.3.2.5 Slope Flattening

Regrading and flattening the slope is a common and reliable method of correcting a landslide in the region. For highway applications, it is typically the preferred and most economical method to repair small failures since it is often easier to implement and less expensive than other remedial methods. In the case of a highway fill failure, this method usually causes minimal disturbance to the existing pavement and fill. In the case of a cut, generally sufficient material may be removed to permit the passage of traffic. However, if additional right-of-way is required, this method may be cost and time prohibitive. The time delay to acquire the required right-of-way and/or slope easement may contribute to an aggravation of the instability.

Typically, the failed slope is regraded, so the new slope is flatter than the slope that failed. It is not advisable to regrade the slope to the same inclination as the failed slope unless supplemental support is provided such as a toe wall which is typically not a permanent solution. Due to the large strain associated with the slope failure, the soil along the rupture plane is often weaker than the soil that exists above and below the rupture plane.

10.3.2.6 Remove and Replace

Removal and replacement of the slide mass with a higher-strength material is an alternative to flattening the slope. This method involves excavation, drainage, and backfilling to reconstruct the entire slope. Due to the extensive amount of earthwork involved, removal and replacement are typically considered for smaller slides where there is sufficient space to safely complete the temporary excavation and removal of the rupture plane.

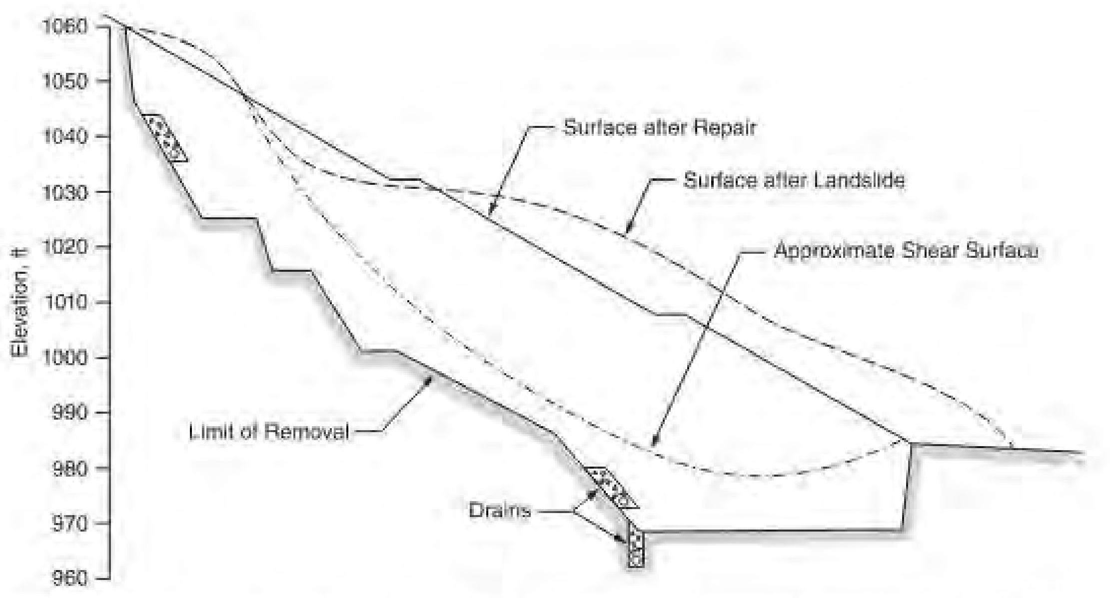


Figure 16.20 Repair of landslide at Lawrence Berkeley Laboratory by removal and replacement of the sliding mass (after Harding, Miller, Lawson, and Associates, 1970).

Figure 10-108 - Schematic of Removal and Replacement of a Slide Mass [34]

It is important to locate a competent subgrade for the toe key to support the reconstructed embankment when this method is employed. Where acceptable soils are available onsite, they may be excavated, and reused as compacted embankment fill to reconstruct the slope; however, where site soils are not able to be properly compacted (typically due to moisture issues) then borrow material or soil improvements (e.g., blending with hydrated lime or cement) may be needed. It is preferable to remove the entire slide mass when using this method, however in some cases that may not be possible; in those cases, a thorough analysis considering the reduced strength of the rupture plane should be conducted.

A schematic of this method is presented in Figure 10-108. An example construction sequence using this method to repair a landslide is illustrated in Figure 10-109 through Figure 10-112.

Figure 10-109 - Existing Rotational Slide

Figure 10-110 - Toe Key Excavation

Figure 10-111 - Drain Installation in Toe Key

Figure 10-112 - Regraded Slope

10.4 EMERGING TECHNOLOGY

10.4.1 Control Method(s)

10.4.1.1 Soil Nails and Grillage

This method involves soil nails that are interconnected with reinforced concrete ribs and was developed in the early 2000s by a working group of the Hong Kong Institution of Engineers [20]. An example of this method is illustrated in Figure 10-113.

(a) Construction of soil nails and grillage.

(b) Completion of soil nails and grillage construction.

Figure 10-113 - Example Soil Nailing with Reinforced Concrete Grillage [20]

10.4.1.2 Cruciform Structure with Anchor Slab

A variant of a Maccaferri cruciform structure fence has been used to stabilize landslides. The cruciform structure consists of a buried anchor slab that is linked to a pyramid shape steel cross-frame retaining structure. This structure is depicted in Figure 10-114.

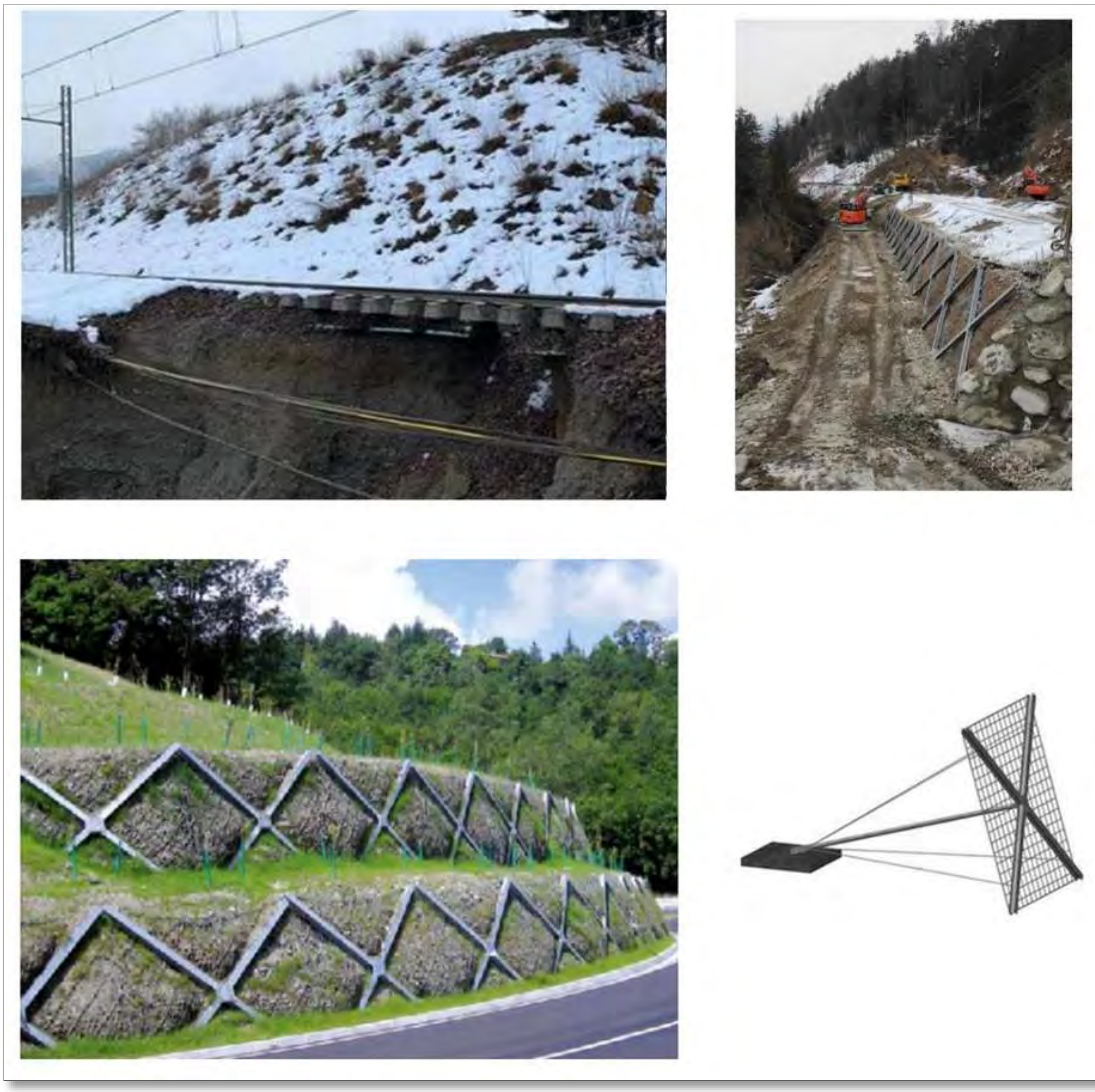


Figure 10-114 - Pyramid-Shape Steel Frame Retaining Elements [95]

10.4.1.3 Debris-Flow Fence

A debris flow fence is used to protect the motoring public below an active landslide. This is particularly useful when shallow debris flow is involved.

An example of a debris flow fence application is illustrated in Figure 10-115 and Figure 10-116. At this location, it was a priority to restore the serviceability of the roadway. Holes were drilled down the center of the roadway and used to set steel posts and erect a debris flow fence. Landslide debris was removed, salvaged debris flow fence was delivered/erected, and a temporary traffic signal was put in service to restore one lane of bidirectional traffic.

Figure 10-115 - Debris Flow Fence Installation

Figure 10-116 - Debris Flow Fence

10.4.1.4 Deep Polymer Injection

Deep polymer (e.g. foam) injection is used to stabilize a slide mass through filling voids and densifying the soils with a lightweight, high strength, expansive foam material. This solution allows for minimal site disturbance and the process is relatively quick compared to other construction methods. These products are typically proprietary so design may need to be coordinated with the manufacturer.

This method has been successfully implemented in Fayette County to remediate severe cracking in a roadway section due to slope movement (see Figure 10-117). Deep polymer (e.g. foam) injection was used to stabilize the soils and lift the pavement where it had settlement due to the lateral soil movement. This solution has kept the roadway operational for two years with no signs of additional distress; however, based on inclinometer data the slope at this location still is exhibiting minor relative movement.

Figure 10-117 - Deep Polymer Injection, Fayette County, PA (PennDOT District 12)

10.4.1.5 Bio-Remediation

Environmental considerations have increasingly become an important factor in the choice of suitable remedial measures, particularly issues such as visual intrusion in scenic areas or the impact on nature or geological conservation interests. An example of a “soft engineering” solution, more compatible with the environment, is the stabilization of slopes by the combined use of vegetation and man-made structural elements working together in an integrated manner known as biotechnical slope stabilization.

The basic concept of vegetative stabilization is not new. Vegetation has a beneficial effect on slope stability by the processes of interception of rainfall, and transpiration of groundwater, thus maintaining drier soils and enabling some reduction in potential peak groundwater pressures. In addition to hydrological effects, vegetation roots reinforce the soil and increase soil shear strength. Where trees may be utilized, tree roots can anchor into deeper firm strata, providing support to the upslope soil mantle through buttressing and arching.

Even the small increase in soil cohesion induced by the roots has a major effect on shallow landslides. While the mechanical effect of vegetation planting is not significant for deeper seated landslides, the hydrological effect is beneficial for both shallow and deep landslides.

The “Geotechnical Manual for Slopes” [19] includes useful information on the hydrological and mechanical effects of vegetation. The concept of biotechnical slope stabilization is generally cost-effective as compared to the use of structural elements alone; it increases environmental compatibility and allows the use of local natural materials. Interstices of the retaining structure are planted with vegetation whose roots bind together the soil within and behind the structure. The stability of all types of retaining structures with open grid work or tiered facings benefits from such vegetation [115]. Stabilization of slopes using a combination of vegetation and retaining structures as an integrated solution is known as biotechnical slope stabilization[128]. See Table 10-2 for examples of biotechnical stabilization.

CATEGORY		EXAMPLES
LIVE CONSTRUCTION	Conventional Plantings	• Grass seeding • Sodding • Transplants
MIXED CONSTRUCTION	Woody plants used as reinforcements and barriers to soil movement	• Live staking • Contour wattling • Brush layering • Soft gabions • Brush mattress
	Plant/ structure associations	• Breast walls with slope face plantings • Revetments with slope face plantings • Tiered structures with bench plantings
	Woody plants grown in the frontal openings or interstices of retaining structures	• Live cribwalls • Vegetated rock gabions • Vegetated geogrid walls • Vegetated breast walls
	Woody plants grown in the frontal openings or interstices of porous revetments	• Joint plantings • Staked gabion matresses • Vegetated concrete block revetments • Vegetated cellular grids • "Reinforced" grass
INERT CONSTRUCTION	Conventional Structures	• Concrete gravity walls • Cylinder pile walls • Tie back walls

Table 10-2 - Classification of different biotechnical slope protection and erosion control measures [128]

Vegetation may be used to control the erosion of unstable masses. This method provides a protective surface on the slope and is used as a means of lining ditches. Roots of plants and grasses absorb moisture and aid in preventing the infiltration of water into the unstable mass. Seeding and the establishment of vegetation should be used in the reconstruction of slope failures. In some cut slopes, seeding may not be practical because of the steepness and nature of the materials on the slope. On fill slopes, vegetation also serves another purpose -- the prevention of small shallow slides. Roots form a matrix or a mat that holds soil particles in place. This matting helps resist failure and aids in holding the top few feet of soil in place. Roots of vegetation generally increase the strength of the soil. Consult a geotechnical engineer or plant pathologist in the local area for a list of native plants and grasses that form a thick mat of roots, absorb water, and are adaptable to the local climate.

Examples of plantings used for bio-stabilization in southwestern Pennsylvania include tall fescues and bristly locust.

CHAPTER 11

Economics of Repair Methods

11.1 GENERAL

This chapter highlights the economics of various methods of landslide stabilization and repair, which can be referenced for programming use.

One of the first questions that are asked when a landslide has occurred or when slope distress is evident is "how much will it cost to stabilize or repair the landslide?" A preliminary cost estimate is needed to identify available resources and guide decision-making. For example, estimates are needed to determine which contracting vehicle is best suited to act, whether it be the engagement of "in-house maintenance forces", execution under an open-end maintenance contract to engage an independent contractor (if one exists, which can address the need), or advertisement to solicit bids from independent contractors. An opinion of probable construction cost (OPCC) is needed to evaluate the reasonableness of cost proposals that are solicited to complete the landslide stabilization/repair. Cost estimates are also needed to determine the most appropriate response, whether it be a temporary repair or more-permanent stabilization.

When preparing a complete OPCC, the practitioner should be aware and prepared to detail the total project costs which will include costs outside of the components of the landslide repair. For example, mobilization, equipment access, erosion and sedimentation controls, utility relocations, traffic control, etc. These costs are not insignificant and can be critical to estimating the complete cost to repair the landslide.

An opinion about the probable landslide mitigation/repair cost estimate(s) should be site-specific. "The most cost-effective repairs are combination techniques, dictated by construction access and the haul distances of any imported materials."[116] Additionally, where excavation is required, consideration should be given to the characterization the material and suitability to be re-used as fill or handled and disposed as "clean fill". Additional costs will be incurred where contaminated soils or acid producing soils and rock are encountered; tipping fees at a permitted landfill may exceed $100 per cubic yard depending on the type and condition of actual waste material involved. Thus, a holistic understanding of the disposition of key cost drivers is critical to develop a reliable OPCC.

Direct, indirect, and hidden cost needs to be considered. User cost is subject to interpretation and may not be measurable. Indirect cost needs to be acknowledged, such as protection against adverse impacts on infrastructure, sensitive ecosystems (i.e., wetlands and streams), public utilities, and an adjacent property owner (e.g., legal liability) and possible property/easement acquisition(s). Consideration needs to be given to hidden costs (e.g., repeated pavement patching and overlay) and allocation of funds to complete routine slope and drainage maintenance.

Please note, maintenance costs may not be accurately reflected in maintenance records. "Tracking costs for maintenance and restoration of slopes as well as user costs associated with such failures is extremely difficult."[49]

Consideration should be given as to what, when (e.g., urgency), where, and how disrupted utility service is restored; and who is ultimately responsible to restore that disrupted utility service. Disruptions to public transportation may also be a consideration when roadway closures are necessary to perform work,

Allowance needs to be given to the complexity, scale, market fluctuation, availability/reliability of available resources, risk tolerance, supervision experience, and performance expectation which should be considered as the foundation upon which an OPCC is rendered.

11.2 SUGGESTED GENERAL GUIDELINES TO ASSESS REPAIR ALTERNATIVES

Mode of failure, the extent of slope movement, rate of slope movement, response time, risk, consequence, redundancy, and resiliency are key factors in the decision-making process. See Chapter 9 for further discussion about risk and consequence.

There is no simple rule as to which stabilization and repair method used is more prudent. Rather, constraints will lead to the elimination of several options, from which affordability and cost-benefit should be considered. It is a given that practitioners should address site-specific needs and constraints to select a preferred solution that is cost-effective, practical, functional, and constructible.

The practitioner is advised to not follow strict rules to formulate an opinion about the probable construction cost to assess viable stabilization/repair alternatives. However, some general guidelines are suggested and discussed below, which are broken down into three (3) generalized categories: eliminate, control, and rebalance.

11.2.1 Eliminate

The following is a list of elimination methods in order of increasing cost, and some of the principal items to consider in developing an OPCC. [12]

1. Removal of landslide - partial
    - Excavation
    - Relocation of excavation spoil, particularly if disposal is required off-site
2. Relocation of supported structure - complete
    - Excavation
    - Structure (e.g., dwelling, pavement, etc.)
    - Right-of-Way Damage
3. Removal of landslide - entire
    - Excavation
    - Relocation of excavation spoil, particularly if disposal is required off-site
    - Right-of-Way Damage
4. Bridging
    - Bridging structure

11.2.2 Control

The following is a list of control methods in order of increasing cost, and some of the principal items to consider in developing an OPCC.[12] The actual order of increasing cost can vary and is a function of project size and constraints.

1. Single-Face Barrier
    - Excavation/regrading

  - Repaving, if required
  - Structure (e.g., concrete barrier and structure backfill) and drainage
  - Right-of-Way Damage
2. Armoring (e.g., slope surface enhancement)
  - Excavation/regrading
  - Relocation of excavation spoil, particularly if disposal is required off-site
  - Backfill (e.g., geocell material, riprap, reno mattress, select borrow, etc.)
  - Right-of-Way Damage
3. Buttress
  - Excavation
  - Backfill (rock or soil) and drainage
  - Right-of-Way Damage
4. Geosynthetic Reinforced Soil (GRS)
  - Excavation
  - Relocation of excavation spoil, particularly if the disposal is required off-site
  - GRS (e.g., reinforced soil, wire mesh units/forms/gabions)
  - Select borrow, if required
  - Repaving, if required
  - Right-of-Way Damage
5. Shoulder Back-Up and Moment Slabs
  - Structure excavation
  - Repaving, if required
  - Structure (e.g., moment slab and barrier/guide rail)
  - Right-of-Way Damage
6. Gabion Wall, Modular Block Wall, and Crib Wall
  - Structure excavation
  - Repaving, if required
  - Structure (e.g., gabions, modular block, headers, and stretchers) and drainage
  - Right-of-Way Damage
7. Pipe Dowels, and Articulated Micropile
  - Repaving, if required
  - Structure (e.g., pipe dowels, micropile, load transfer mat if required)
  - Right-of-Way Damage
8. Soil Nails
  - Structure excavation
  - Repaving, if required
  - Structure (e.g., soil nails, high strength steel mesh facing and/or shotcrete with reinforcement, wall facing) and drainage
  - Right-of-Way Damage
9. Cantilevered Pile Walls
  - Structure excavation
  - Repaving, if required
  - Structure (e.g., soldier pile, timber lagging or reinforced shotcrete for temporary excavation support if required, structure backfill, wall facing) and drainage
  - Right-of-Way Damage
10. Anchored Walls
  - Structure excavation
  - Repaving, if required

- Structure (e.g., post-tensioned soil/rock anchors, walers if required, soldier pile, timber lagging or reinforced shotcrete for temporary excavation support if required, structure backfill, and wall facing) and drainage
- Right-of-Way Damage

11. Conventional Cantilevered Concrete Wall
    - Structure excavation
    - Temporary excavation support, if required
    - Repaving, if required
    - Structure (e.g., concrete, steel reinforcement, structure backfill) and drainage
    - Right-of-Way Damage

11.2.3 Rebalance (ratio between mobilized resistance and driving force)

The following is a list of rebalancing methods in order of increasing cost, and some of the principal items to consider in developing an OPCC.[12] The actual order of increasing cost can vary and is a function of project constraints.

1. Surface Drainage Improvement
    - Excavation/regrading
    - Structure (e.g., ditch lining)
2. Seepage Interceptor Drain
    - Excavation/regrading
    - Backfill (e.g., riprap, drainage collection system)
    - Right-of-Way Damage
3. Horizontal Drain
    - Drilling
    - Materials (e.g., drainage collection system)
    - Right-of-Way Damage
4. Partial Unloading (Removal of Material at Top)
    - Excavation
    - Relocation of excavation spoil, particularly if the disposal is required off-site
    - Regrading
    - Repaving, if required
    - Right-of-Way Damage
5. Lightweight Fill
    - Excavation
    - Relocation of excavation spoil, particularly if the disposal is required off-site
    - Lightweight fill
    - Repaving, if required
    - Right-of-Way Damage
6. Slope Flattening
    - Excavation
    - Relocation of excavation spoil, particularly if the disposal is required off-site
    - Repaving, if required
    - Right-of-Way Damage
7. Remove and Replace
    - Excavation
    - Relocation of excavation spoil, particularly if the disposal is required off-site
    - Select borrow material

  - Repaving, if required
  - Right-of-Way Damage

11.2.4 Emerging Technology

The following is a list of emerging technology methods in order of increasing cost, and some of the principal items to consider in developing an OPCC. The actual order of increasing cost can vary and is a function of project constraints.

1. Deep Polymer Injection
    - Drilling injection holes
    - Repaving, if required
    - Right-of-Way Damage

11.3 HISTORIC UNIT COST DATA FOR PROGRAMMING PURPOSES

Historic unit cost data is presented herein to support programming decisions. This data is not to be construed to be comprehensive or complete. The practitioner is advised to account for constraints, needs, and level of detail required to guide site-specific decision-making.

It is understood that the practitioner is knowledgeable of standard practice to develop an informed opinion about the probable construction cost, such as how and to what extent contingency should be applied as a function of known unknowns and stage of design development (e.g., conceptual, preliminary, final). As such, the discussion about how to develop an OPCC is beyond the scope of this document. For further guidance, the reader is advised to refer to the Association for the Advancement of Cost Engineering (AACE) and engage the services of a professional construction cost estimator.

11.3.1 Construction Cost Index to Normalize Historic Cost Data

Oftentimes, the practitioner has access to a limited amount of historic cost data upon which to develop a site-specific OPCC. This is particularly true when using multiple sources of data that represent construction that was advertised and bid over several years, sometimes over more than a decade. Hence, the practitioner is advised to normalize the historic data to a current base year, and then use that normalized data to make informed decisions to arrive at a site-specific OPCC. With that said, the practitioner is advised to not “overthink” the normalization process, but rather focus on a logical thought process to arrive at a reasonable and rational OPCC. The practitioner should apply judgment, in addition to experience and reliance on others to discern if the site-specific OPCC is reasonable.

One such approach to normalizing historic cost data is to defer to paid subscriptions of cost data that is broken down into categories, such as the Engineering News-Record (ENR) or RSMeans data from Gordian. If the practitioner does not have access to a paid subscription to access indexed historic cost data, then the practitioner is advised to consider the FHWA’s National Highway Construction Cost Index (NHCCI). NHCCI provides an interactive dashboard that presents quarterly data that is indexed to a 2003 base year and includes a filter that is broken down into 25 construction components (e.g., categories). To view the growth trend of the overall construction cost index, see Figure 11-1.

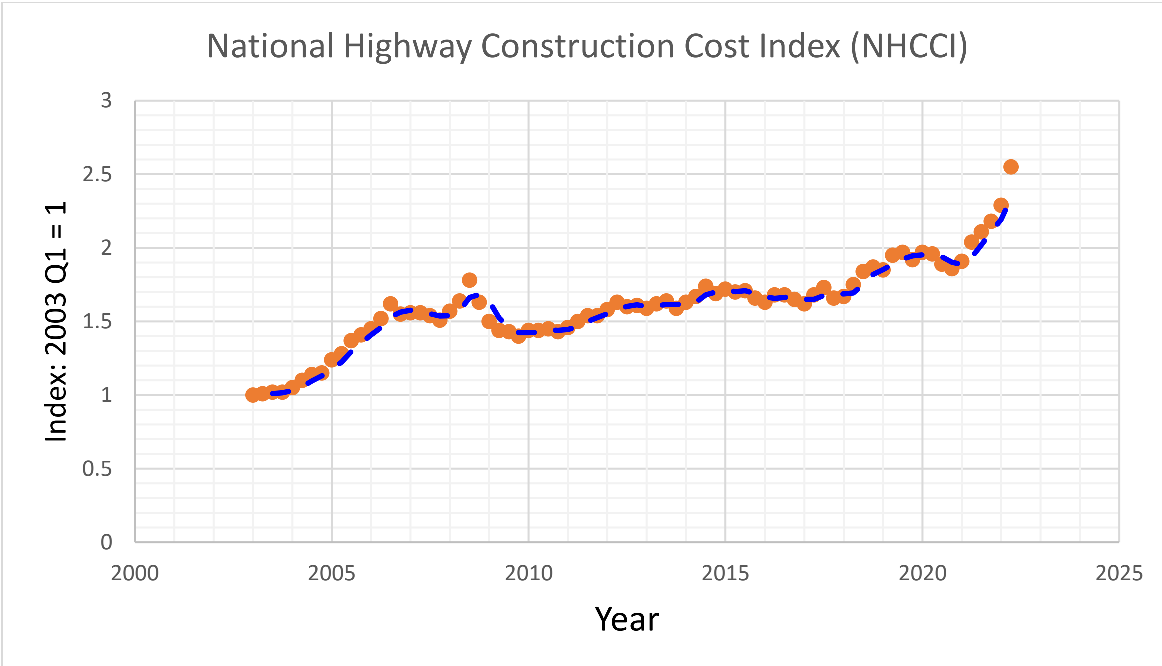


Figure 11-1 - FHWA National Construction Cost Index Unit Cost Data [93]

Sometimes it is better to estimate cost based on an average cost per unit of structure, such as the average cost per square foot of exposed wall face. Another approach is to estimate based on an average unit price for key components such as the unit cost per cubic yard of excavation. Typical unit price ranges (including average unit pricing), sourced primarily from the PennDOT Engineering and Construction Management System (ECMS), are presented below for several landslide mitigation items to provide perspective for programming purposes. The unit prices shown below were compiled from projects bid between 2018 and 2022 and have been normalized to reflect the 1st quarter 2022 trend for unit costs that have been reported by FHWA for the transportation market across the United States (see Section 10.3.1).

- $10 to $70 per cubic yard ($25/CY avg.) for Class I Excavation (for a minimum 1,000 CY) (Pub. 408 Sec. 203).
- $45 to $130 per cubic yard ($70/CY avg.) for durable rock fill (for a minimum 1,000 CY) (Pub. 408 Sec. 205 & 850).
- $220 to $420 per cubic yard ($290/CY avg.) for Class C concrete (for a minimum 200 CY) (Pub. 408 Sec. 1001).
- $550 to $3,100 per cubic yard ($1,200/CY avg.) for Class AA concrete (for a minimum 200 CY) (Pub. 408 Sec. 1001).
- $25 to $175 per cubic yard ($85/CY avg.) for Select Structure Backfill (for a minimum 100 CY) (Pub. 408 Sec. 1001).
- $0.85 to $11.85 per pound ($2.95/LB avg.) for epoxy-coated rebar (for a minimum 500 LB) (Pub. 408 Sec. 1002).
- $0.80 to $6.75 per pound ($1.80/LB avg.) for uncoated steel H-Piles (Pub. 408 Sec. 1005).
- $1.00 to $6.30 per pound ($2.60/LB avg.) for galvanized steel H-Piles (Pub. 408 Sec. 1005).
- $150 to $525 per linear foot ($325/LF avg.) for 36-inch dia. drilled shafts in soil (Pub. 408 Sec. 1006).
- $270 to $625 per linear foot ($425/LF avg.) for 42-inch dia. drilled shafts in soil (Pub. 408 Sec. 1006).

- $335 to $895 per linear foot ($510/LF avg.) for 30-inch dia. drilled shaft rock sockets (Pub. 408 Sec. 1006).
- $395 to $600 per linear foot ($475/LF avg.) for 36-inch dia. drilled shaft rock sockets (Pub. 408 Sec. 1006).
- $105 to $225 per linear foot ($165/LF avg.) for 7-inch OD micropile (Pub. 408 Sec. 1007).
- $135 to $370 per linear foot ($240/LF avg.) for 9.625-inch OD micropile (Pub. 408 Sec. 1007).
- $15 to $250 per cubic foot ($100/CF avg.) for grout for micropile (Pub. 408 Sec. 1007).
- $50 to $120 per square foot ($70/SF avg.) for modular architectural block wall (ECMS 57921).
- $95 to $215 per linear foot ($145/ LF avg.) for pipe dowels.
- $2,000 to $4,000 per soil nail (avg. $3,100 per each) for soil nails.
- $30 to $60 per SF (avg. $45 per SF) for shotcrete facing (including steel reinforcement).
- $10,000 to $30,000 per rock anchor (avg. $15,000 per each) for rock anchors.
- $600 per square foot for a 10’ high reinforced concrete cantilevered wall (including excavation & backfill) (RSMeans).
- $1,700 per square foot for 20’ high reinforced concrete cantilevered wall (including excavation & backfill) (RSMeans).
- $30 to $230 per square foot ($100/SF avg.) for GRS (Pub. 408 Sec, 223).
- $5 to $65 per square yard ($35/SY avg.) for 6-inch and 8-inch geocell (minimum 100 SY) (Pub. 408 Sec. 222).

CHAPTER 12

Typical Details

12.1 GENERAL

The preferred details that are presented in this chapter align with "Best Practice", which is typically consistent with current practice in southwestern Pennsylvania to stabilize and repair landslides.

The details that are presented in this Chapter build upon a fundamental understanding of the project site which is based on the processes that are described in the preceding chapters, to provide actionable guidance with practical, clear, useful, and usable direction. A process flow chart is presented in Figure 12-1.

This Chapter is intended to build upon information that was presented in Chapters 10 and 11 to transition from a discussion about available stabilization methods to detailing a site-specific preferred solution.

The typical details include notes that are meant to highlight key considerations for each option. Typical details presented in this chapter should not be substituted for the judgement of an experienced and licensed engineer. Additionally, project specific requirements must be followed, which may take precedence over the typical details presented herein; refer to Section 1.2 for additional information

Several of the mitigation details (e.g., Soil Launcher, Deep Polymer Injection, etc.) mentioned in this chapter are proprietary and are listed for general informational purposes only and to depict possible commercially available solutions. If a proprietary solution is preferred, the practitioner is advised to contact the manufacturer for guidance to develop site specific details. However, it is important to note that standardized design methodology and specifications may not be available for these emerging technologies and guidance from the manufacturer does not replace engineering judgment; any proprietary solutions shall be reviewed, assessed, and approved by the engineer.

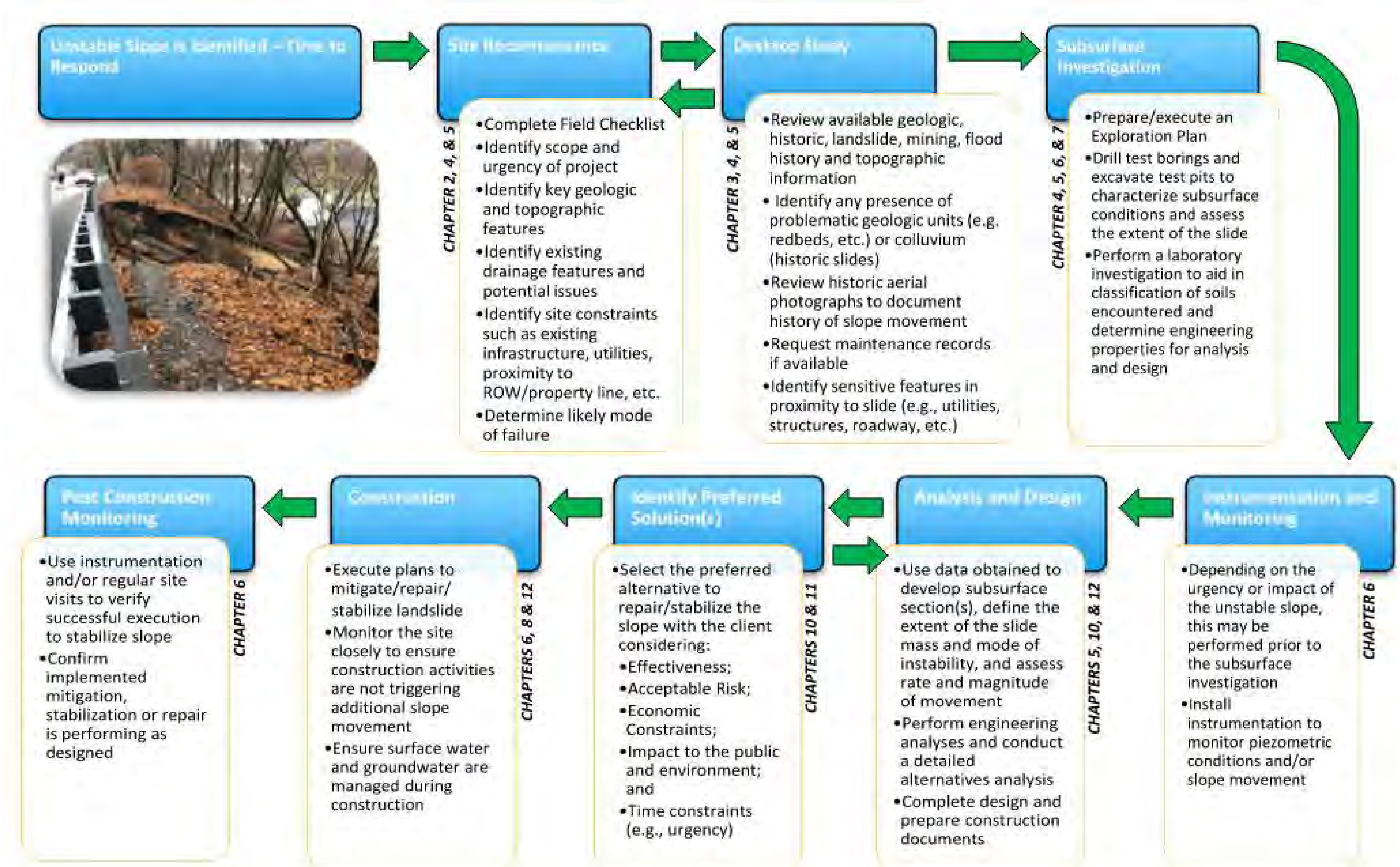


Figure 12-1 - Landslide Mitigation Flowchart
(Please note, the bidirectional arrows for several of the steps indicate that those steps may be done iteratively or in reverse order)

See Chapter 10 for a detailed discussion of the available stabilization and repair methods for which details are presented herein. See below for a summary of the stabilization and repair methods included in this chapter. It is a given that practitioners should address site-specific needs to select a preferred solution that is cost-effective, practical, functional, and constructible.

- I. Elimination Methods
  - A. Relocation
  - B. Removal
  - C. Bridging (over Landslide Mass)
- II. Control Methods
  - A. Retaining Structures
    - 1. Buttress(es)
      - e. Rock-Fill Buttress
      - f. Stability Berm
      - g. Extended Fill (raise grade at toe)
      - h. Shear Key with Rock Buttress
    - 2. Geosynthetic Reinforced Soil (GRS)
    - 3. Slope Surface Enhancement(s)
      - e. Geocell Slope Protection
      - f. Turf Reinforcement Mat (TRM)
      - g. Rock Slope Armoring
      - h. Gabion slope wall (Reno mattress)
    - 4. Shoulder Back-Up and Moment Slabs
    - 5. Single-Face Barrier
    - 6. Gravity Walls
      - e. Conventional Concrete Gravity Wall
      - f. Gabion Wall
      - g. Modular Block Wall
      - h. Crib Wall
    - 7. Cantilevered Pile Walls
      - d. Soldier Pile and Lagging Wall
      - e. Buried Soldier Pile Wall
      - f. Tangent Pile Wall
    - 8. Tieback (Ground) Anchors
      - d. Soldier Pile Wall with Tiebacks
      - e. Buried Panel Wall with Tiebacks
      - f. Sheet Pile Wall with Tiebacks

- II. Control Methods (continued)
    - 9. Soil Nails
    - 10. Soil Nail Launcher
    - 11. Articulated Micropile
    - 12. Pipe Dowels
  - C. Rebalance Ratio Between Mobilized Resistance and Driving Force(s)
    - 1. Surface Drainage Improvement
      - e. Upslope Interception & Diversion
      - f. Ditch Lining
      - g. Surface Reshaping/Regrading
      - h. Sealing Open Tension Cracks
    - 2. Subsurface Drainage Improvement
      - e. Interceptor (French) Drain
      - f. Spring Drain
      - g. Finger Drain
      - h. Horizontal Drains
    - 3. Lightweight Fill
    - 4. Partial Unloading (Removal of Material at Top)
    - 5. Slope Flattening
    - 6. Remove and Replace
- III. Emerging Technology
  - B. Control Methods
    - 1. Soil Nails and Grillage
    - 2. Cruciform Structure with Anchor Slab
    - 3. Debris-Flow Fence
    - 4. Deep Polymer Injection
    - 5. Bio-Remediation

12.2 RETAINING STRUCTURE(S)

12.2.1 Buttress(es)

Typical details for buttresses, and several variants thereof, are presented below. For detailed descriptions and photographs, refer to Chapter 10:

- Rock-Fill Buttress/Rock Slope Armoring (Figure 12-2).
- Stability Berm (Figure 12-3).
- Widened Embankment (Figure 12-4).
- Placement of Claystone & Marginal Material in Embankments (Figure 12-5).
- Shear Key with Rock Buttress – for general guidance refer to Figure 12-2; considerations unique to this option include discontinuous shear keys filled neat with lean cement concrete in lieu of a rock toe key. Typical shear key configuration may consist of three- to five-foot-wide trenches that are spaced on the order of 12 to 15 feet center to center; however, final determination of the length, depth, and spacing of the shear keys will require a stability analysis. See Section 10.3.1.1 for additional information.

12.2.2 Geosynthetic Reinforced Soil (GRS)

For detailed descriptions and photographs of Geosynthetic Reinforced Soil (GRS), refer to Chapter 10. See Figure 12-6 for a typical detail.

12.2.3 Slope Surface Enhancement(s)

Typical slope surface enhancement utilized in the region, and included in this section, are listed below. For detailed descriptions and photographs, refer to Chapter 10.

- Geocell Slope Protection (Figure 12-7 through Figure 12-10).
- Turf Reinforcement Mat (Figure 12-11).
- Rip Rap Revetment (Figure 12-12).
- Gabion Slope, e.g., Mattress (Figure 12-13).

12.2.4 Shoulder Back-Up and Moment Slabs

Typical details for shoulder back-up and moment slabs, and variants thereof, are presented below. For detailed descriptions and photographs, refer to Chapter 10.

- Shoulder Back-Up (Figure 12-14).
- Moment Slab (Figure 12-15).
- Moment Slab with Toe Wall (Figure 12-16).

12.2.5 Single Face Barrier

For detailed descriptions and photographs of single-face barriers, refer to Chapter 10. See Figure 12-17 for a typical detail.

12.2.6 Gravity Walls

Typical gravity walls constructed for landslide stabilization and repair in southwestern PA, and included in this section, are listed below. For detailed descriptions and photographs, refer to Chapter 10.

- Conventional Concrete Gravity Walls (see Figure 12-18).
- Gabion Wall (see Figure 12-19).
- Modular Block Wall (see Figure 12-20).
- Crib Wall (see Figure 12-21).

There are multiple proprietary wall products and designs within this section; the practitioner may reference the list of PennDOT Approved Bridge and Structure Products for an example of available options.

12.2.7 Cantilevered Pile Walls

Typical cantilevered pile walls constructed for landslide stabilization and repair in southwestern PA, and included in this Section, are listed below. For detailed descriptions and photographs, refer to Chapter 10.

- Soldier Pile and Lagging Wall.
- Buried Solider Pile Wall with Bridging Plug (Figure 12-31).
- Tangent Pile Wall (Figure 12-32).

For general detailing guidance, refer to Figure 12-22 through Figure 12-30. The practitioner may use elements of these details as applicable to cantilevered soldier pile walls.

12.2.8 Tieback (Ground) Anchors

Typical applications for tieback anchors for landslide stabilization and repair in southwestern PA, included in this Section, are listed below. For detailed descriptions and photographs, refer to Chapter 10.

- Soldier Pile Wall with Tiebacks.
- Buried Panel Wall with Tiebacks.
- Sheet Pile Wall with Tiebacks.

See Figure 12-22 through Figure 12-30 for typical details for anchored walls.

12.2.9 Soil Nails

For detailed descriptions and photographs of Soil Nail systems, refer to Chapter 10. See Figure 12-34 for a typical detail.

12.2.10 Soil Launcher

For detailed descriptions and photographs of the Soil Launcher system, refer to Chapter 10. Soil Launcher is proprietary; therefore, site specific detailing of Soil Launcher may be done through the manufacturer. Please see GeoStabilization International for guidance.

12.2.11 Articulated Micropile

For detailed descriptions and photographs of articulated micropile, refer to Chapter 10. Derivatives of the Pipe Dowel detailing (Figure 12-33) may be used for articulated micropile; considerations unique to articulated micropile is that the piles will be battered, and a structural cap is needed to engage frame action. The number of piles, spacing, and batter layout will be site specific based on the geotechnical analysis.

12.2.12 Pipe Dowels

For detailed descriptions and photographs of pipe dowels, refer to Chapter 10. See Figure 12-33 for a typical detail.

12.3 REBALANCE RATIO BETWEEN MOBILIZED RESISTANCE AND DRIVING FORCE(S)

12.3.1 Surface Drainage Improvement

Typical surface drainage options for landslide stabilization and repair in southwestern PA, and included in this Section, are listed below. For detailed descriptions and photographs, refer to Chapter 10.

- Upslope Interception and Diversion (Figure 12-35).
- Smart Ditch (Figure 12-36).
- Surface Reshaping/Regrading.
- Tension Crack Sealing (Figure 12-37).

12.3.2 Subsurface Drainage Improvement

Typical subsurface drainage options for landslide stabilization and repair in southwestern PA, and included in this Section, are listed below. For detailed descriptions and photographs, please refer to Chapter 10. Spring drains and finger drains are variants of the interceptor drain detail that is presented in Figure 12-38.

- Interceptor (French) Drains (Figure 12-38).
- Spring Drains.
- Finger Drains.
- Horizontal Drains (Figure 12-39).

12.3.3 Lightweight Fill

For detailed descriptions and photographs of lightweight fill, refer to Chapter 10. Typical earthwork detailing is applicable to lightweight fill applications; however, material specific considerations should be made including unique strength properties, shrink/swell, compaction requirements, and propensity to degradation. These materials are typically proprietary; therefore, site specific detailing may be coordinated through the manufacturer.

- Expanded Polystyrene Geofoam, see individual supplier for guidance.
- Aerolite, see Aeroaggregates for guidance.
- Cementitious Elastizell, see Elastizell for guidance.
- Expanded Shale, see individual supplier for guidance.

12.3.4 Partial Unloading (Removal of Material at Top)

For detailed descriptions and photographs of partial unloading, refer to Chapter 10. The practitioner should be sensitive to practical access and the effective reach (e.g., up to 25 to 30 feet maximum) for mobilized equipment (depending on the specific equipment manufacturer, make, and model). Placement of the excavated material stockpiles should be considered; these should be placed so that no adverse surcharge loading is added to another area of the slope.

12.3.5 Slope Flattening

For detailed descriptions and photographs of slope flattening, refer to Chapter 10. The degree of slope flattening will be dependent on the available right of way and proximity to other structures. Geotechnical analysis should be performed to assess long-term stability improvement for the flattened slope. Placement of the excavated material stockpiles should be considered; these should be placed so that no adverse surcharge loading is added to another area of the slope.

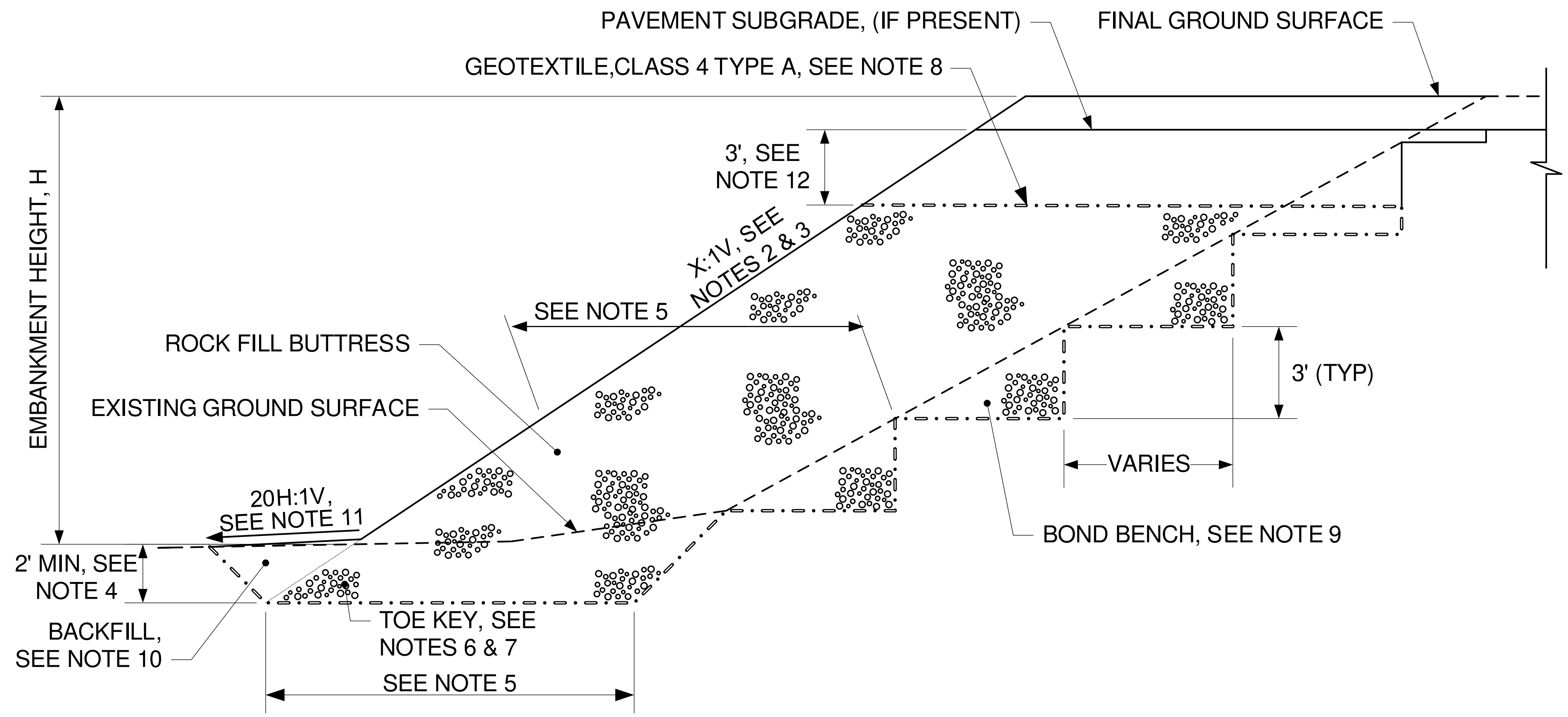
12.3.6 Remove and Replace

For detailed descriptions and photographs of the “remove and replace” alternative, refer to Chapter 10. Derivatives of the typical detail presented in Figure 12-4 may be used for general guidance. Considerations unique to removal and replacement is that onsite soils may be used for embankment; the practitioner should assess these materials for suitability to be placed as embankment. Where unsuitable materials are identified, borrow material may be used to reconstruct the embankment. Where slaking or marginal material is encountered, the detail presented in Figure 12-5 may be considered, which is subject to a site-specific stability analysis.

12.4 EMERGING TECHNOLOGY

Emerging technology for landslide stabilization and repair in southwestern PA, and included in this Section, are listed below. For detailed descriptions and photographs of these options, refer to Chapter 10.

- Soil Nails and Grillage.
- Cruciform Structure with Anchor Slab (proprietary, see Maccaferri for guidance).
- Debris-Flow Fence.
  Deep Polymer Injection (proprietary, see Uretek for guidance).
- Bio-Remediation.



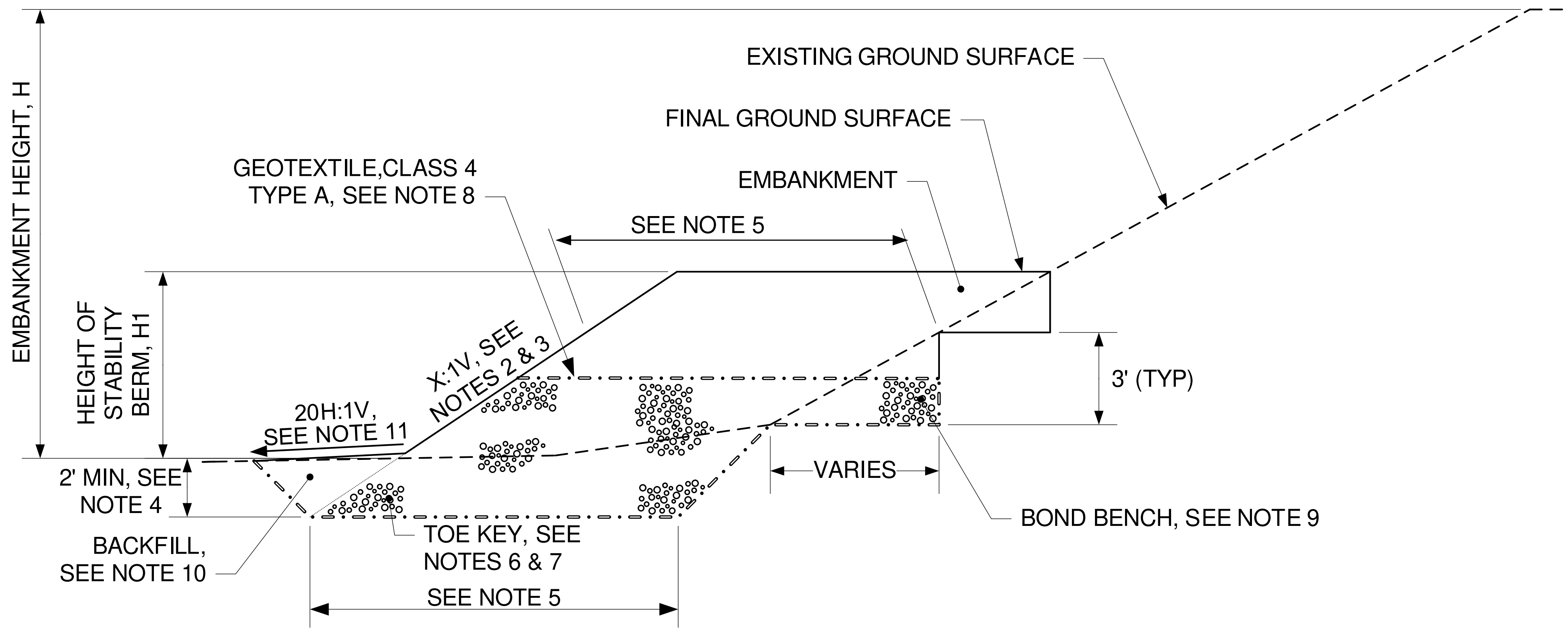
TYPICAL DETAIL – ROCK-FILL BUTTRESS

(NOT TO SCALE)

NOTES.

1. Provide materials and construct as specified in PennDOT Pub. 408, Sec. 203 for Class 1 & 1A excavation, & Sec. 206 for embankment.
2. Analyze slope stability to determine rock-fill buttress geometry. Consider both shallow and deep-seated modes of failure. Check temporary cut slope stability, where applicable.
3. As a general guide, consider 1.5H:1V, or flatter for rock-fill buttress. Consider the quality of rock-fill source material and method of placement, to determine if a steeper slope is viable. See Section 10.3.1.1 for additional information.
4. Undercut as required to satisfy slope stability and develop toe key on competent material (e.g., top of rock or residuum, where present within a reasonable depth).
5. Consider constructability to set minimum width required. Consider 8' and 12' minimum width for smaller and typical size earthmoving equipment, respectively.
6. Consider sloping the bottom of the toe key at 20H:1V towards the back of the toe key. Provide a bench drain surrounded with No. 8 coarse aggregate and wrapped in geotextile (Class 4, Type A) at the back of the toe key, whenever possible.
7. Provide subsurface drainage to maintain positive collection and conveyance downslope away from the toe key, whenever possible.
8. Construct the toe key with free-draining, competent, & durable material, such as rock fill. Use slope stability analysis results to determine the minimum width of toe key required.
9. Use geotextile (Class 4, Type A) to separate rock fill from granular fill embankment and in situ soils.
10. Construct bond (i.e., compaction) benches from bottom up (after topsoil is stripped) during fill placement.
11. Use excavation spoil to backfill space that exists outside the theoretical slope for the rock-fill buttress. Use similar soil type as the adjacent undisturbed soils when constructing adjacent to wetland.
12. Slope surface to maintain positive surface runoff away from the toe of rock-fill buttress to prevent ponding near the toe.
13. At locations where guide rail is proposed at top of slope, consider using No. 57 coarse aggregate or AASHTO No. 1 at the top of embankment to allow for guiderail installation.

Figure 12-2 - Typical Detail, Rock-Fill Buttress



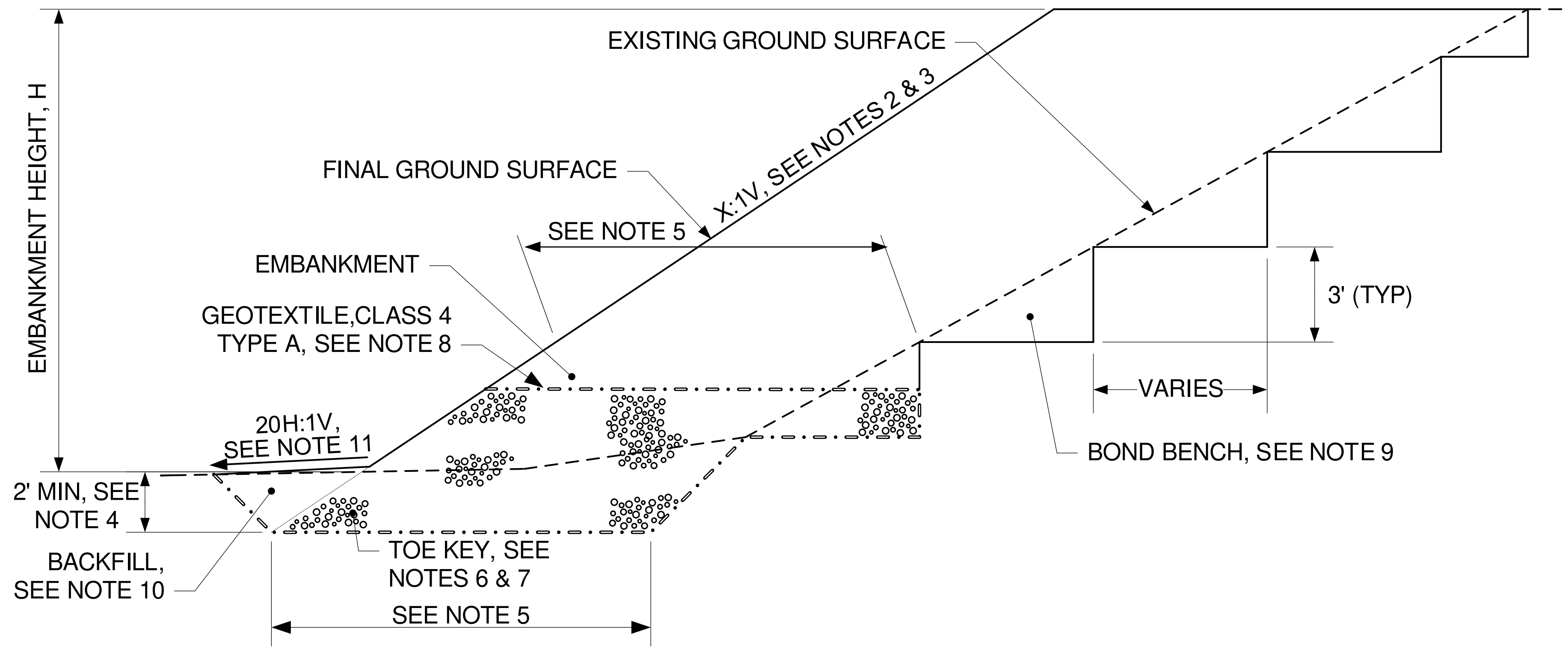
TYPICAL DETAIL – STABILITY BERM

(NOT TO SCALE)

Notes.

1. Provide materials and construct as specified in PennDOT Pub. 408, Sec. 203 for Class 1 & 1A excavation, & Sec. 206 for embankment.
2. Analyze slope stability to determine stability berm geometry that is required (e.g., width and height, H1). Consider both shallow and deep-seated modes of failure. Check temporary cut slope stability, where applicable.
3. As a general guide, consider 1.5H:1V and 2H:1V, or flatter for rock-fill and soil embankments, respectively. For rock fill, consider the quality of the source material and method of placement, to determine if a steeper slope is viable. See Section 10.3.1.1 for additional information.
4. Undercut as required to satisfy slope stability and develop toe key on competent material (e.g., top of rock or residuum, where present within a reasonable depth).
5. Consider constructability to set minimum width required. Consider 8' and 12' minimum width for smaller and typical size earthmoving equipment, respectively.
6. Consider sloping the bottom of the toe key at 20H:1V towards the back of the toe key. Provide a bench drain surrounded with No. 8 coarse aggregate and wrapped in geotextile (Class 4, Type A) at the back of the toe key, whenever possible.
7. Provide subsurface drainage to maintain positive collection and conveyance downslope away from the toe key, whenever possible.
8. Construct the toe key with free-draining, competent, & durable material, such as rock fill. Use slope stability analysis results to determine the minimum width of toe key required.
9. Use geotextile (Class 4, Type A) to separate rock fill from granular fill embankment and in situ soils.
10. Construct bond (i.e., compaction) benches from bottom up (after topsoil is stripped) during fill placement.
11. Use excavation spoil to backfill space that exists outside the theoretical slope for the stability berm. Use similar soil type as the adjacent undisturbed soils when constructing adjacent to wetland.
12. Slope surface to maintain positive surface runoff away from the toe of stability berm to prevent ponding near the toe.

Figure 12-3 - Typical Detail, Stability Berm



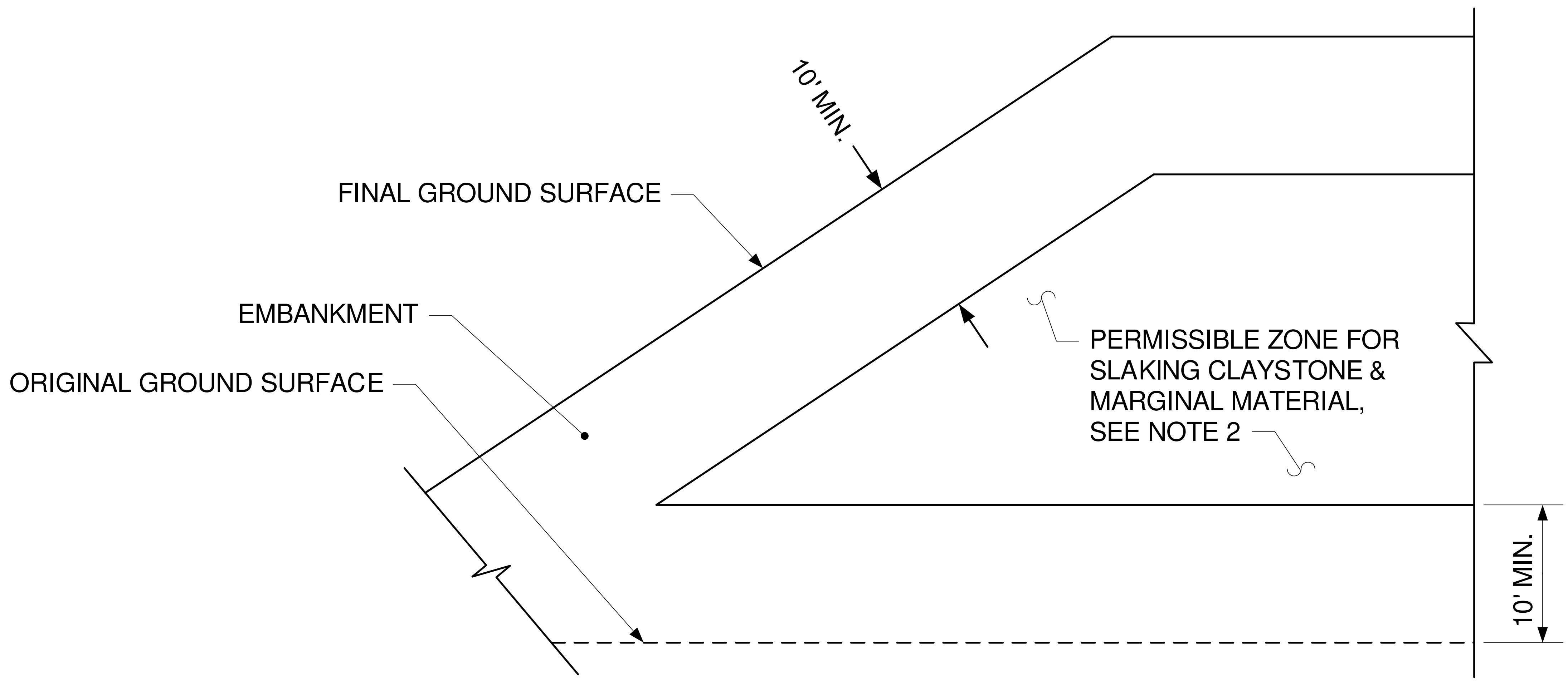
TYPICAL DETAIL – WIDENED EMBANKMENT

(NOT TO SCALE)

NOTES.

1. Provide materials and construct as specified in PennDOT Pub. 408, Sec. 203 for Class 1 & 1A excavation, & Sec. 206 for embankment.
2. Analyze slope stability to determine widened embankment geometry. Consider both shallow and deep-seated modes of failure. Check temporary cut slope stability, where applicable.
3. As a general guide, consider 1.5H:1V and 2H:1V, or flatter for rock-fill and soil embankments, respectively. For rock fill, consider the quality of the source material and method of placement, to determine if a steeper slope is viable. See Section 10.3.1.1 for additional information.
4. Undercut as required to satisfy slope stability and develop toe key on competent material (e.g., top of rock or residuum, where present within a reasonable depth).
5. Consider constructability to set minimum width required. Consider 8' and 12' minimum width for smaller and typical size earthmoving equipment, respectively.
6. Consider sloping the bottom of the toe key at 20H:1V towards the back of the toe key. Provide a bench drain surrounded with No. 8 coarse aggregate and wrapped in geotextile (Class 4, Type A) at the back of the toe key, whenever possible.
7. Provide subsurface drainage to maintain positive collection and conveyance downslope away from the toe key, whenever possible.
8. Construct the toe key with free-draining, competent, & durable material, such as rock fill. Consider soil to construct the toe key when the embankment height is less than 10 feet. Use slope stability analysis results to determine the minimum width of toe key required.
9. Use geotextile (Class 4, Type A) to separate rock fill from granular fill embankment and in situ soils.
10. Construct bond (i.e., compaction) benches from bottom up (after topsoil is stripped).
11. Use excavation spoil to backfill space that exists outside the theoretical slope for the widened embankment. Use similar soil type as the adjacent undisturbed soils when constructing adjacent to wetland.
12. Slope surface to maintain positive surface runoff away from the toe of widened embankment to prevent ponding near the toe.

Figure 12-4 - Typical Detail, Widened Embankment



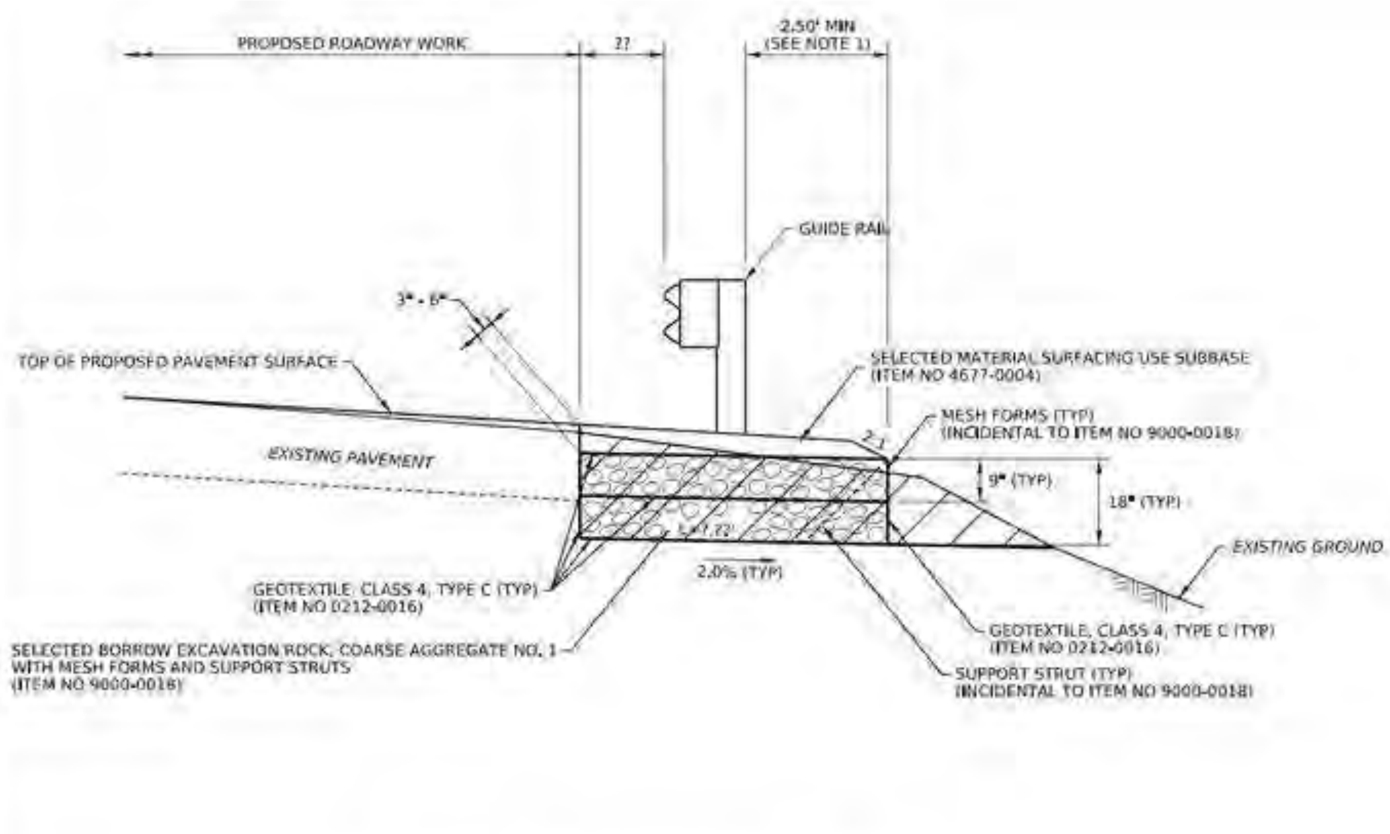
TYPICAL DETAIL – PLACEMENT OF CLAYSTONE & MARGINAL MATERIAL IN EMBANKMENTS

(NOT TO SCALE)

NOTES.

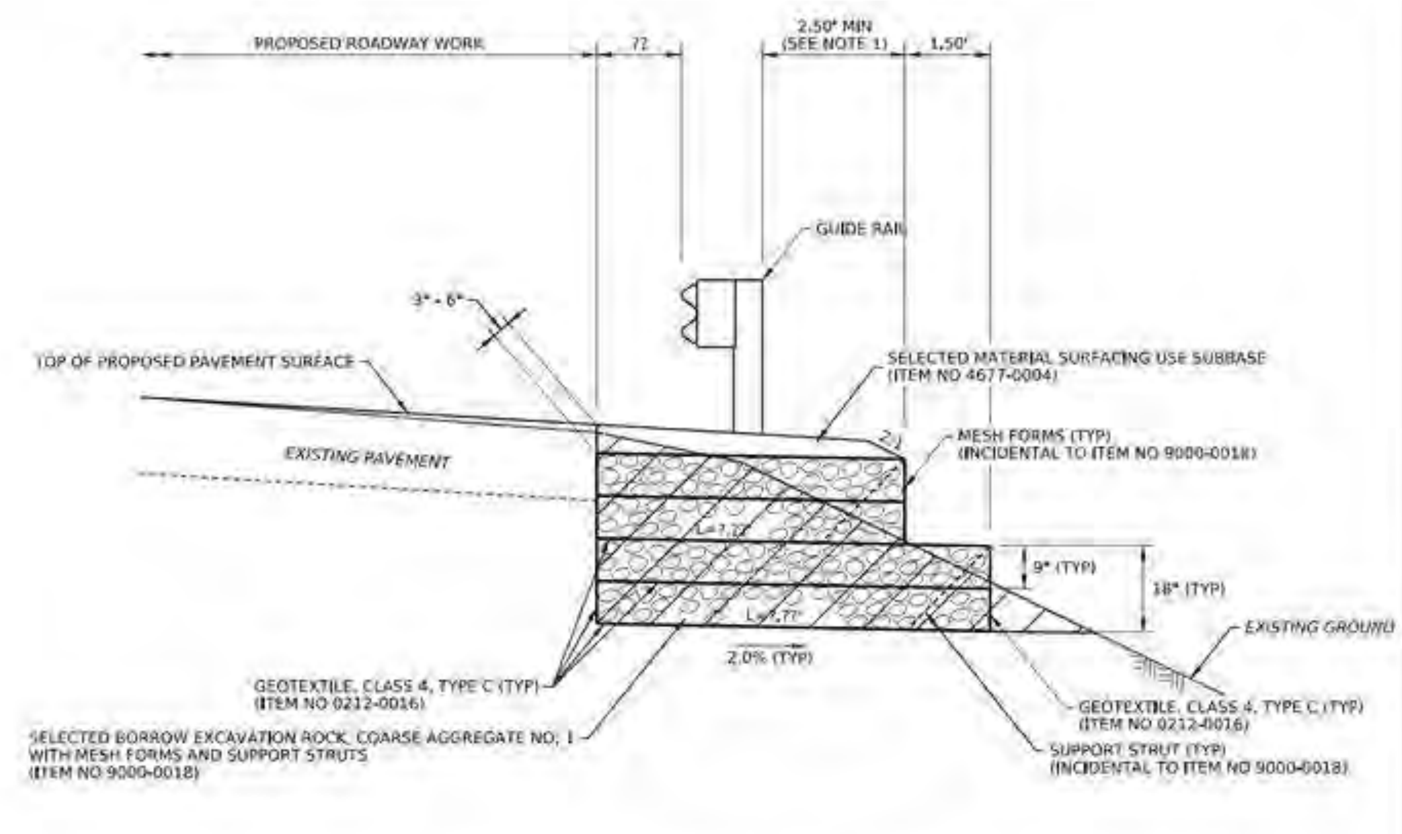
1. Provide materials and construct as specified in PennDOT Pub. 408, Sec. 203 for Class 1 & 1A excavation, & Sec. 206 for embankment.
2. Analyze slope stability to determine widened embankment geometry. Consider both shallow and deep-seated modes of failure. Check temporary cut slope stability, where applicable. Consideration should be given to the shear strength parameters used for this application as slaking claystone and marginal materials may degrade over time. See Section 7.6 for additional detail regarding parameter development.
3. The practitioner must be sensitive to the risks and potential consequences of using of poor-quality embankment material. Based on the intended application, detailed notes must be provided to declare the limits of application for this detail.

Figure 12-5 – Typical Detail, Placement of Claystone & Marginal Material in Embankments



SLOPE STABILIZATION (1 MESH FORM)

NOT TO SCALE



SLOPE STABILIZATION (2 MESH FORMS)

NOT TO SCALE

TYPICAL DETAIL – GEOSYNTHETIC REINFORCED SOIL (GRS)

NOTES.

1. Provide materials and construct in accordance with PennDOT Pub. 408 Sec. 223.
2. The layout, orientation, and extent of excavation are based on the number and offset of mesh forms necessary to provide a minimum 2' backup behind the proposed guide rail post. The back of the excavation should extend to provide the minimum geotextile reinforcement lengths.
3. Provide adequate subsurface drainage when GRS slopes are constructed in a floodplain or when high soil moisture is anticipated behind the GRS.
4. Provide GRS with an open-graded drainage gallery (typically consisting of No. 57 coarse aggregate, collection pipe, and Class 4 Type A geotextile encapsulation) when GRS is constructed against seeps or springs.
5. Do not dump fill directly on exposed geotextile. Place on previously spread material and blade out.
6. Do not leave the geosynthetic face exposed for more than 7 days. Place a UV protective cover over any geosynthetic that is exposed for more than 7 days until backfill is in place.
7. The top of the GRS slope may be either a paved or unpaved surface (e.g., the roadway may be either over the top of the GRS slope or below at the toe of the GRS slope).
8. Depending on the client, available materials, and site constraints, other iterations of this detail may be derived. For an alternative typical detail please refer to PennDOT Publication 72M Standard Detail RC-14M

Figure 12-6 - Typical Detail, GRS

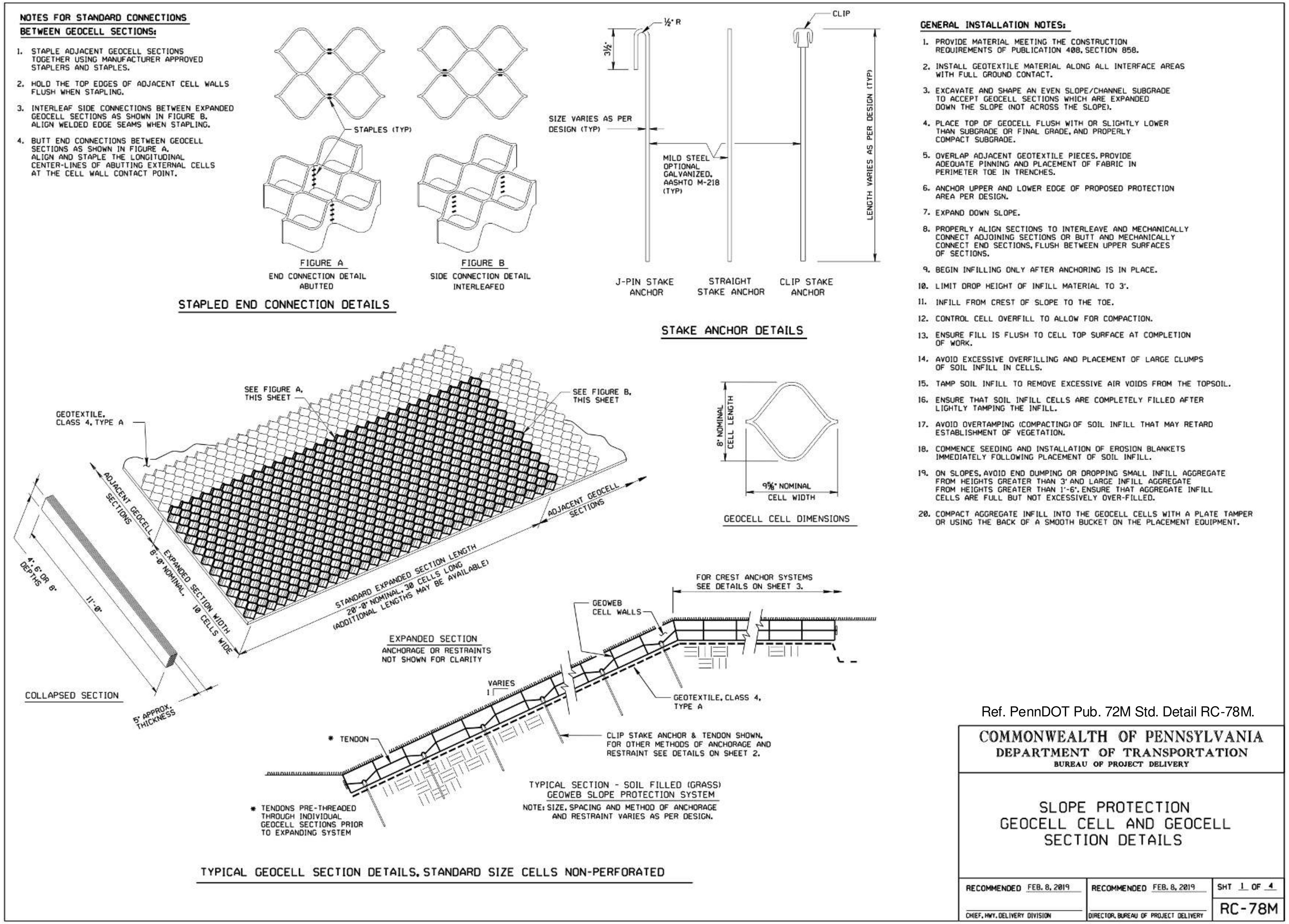


Figure 12-7 - Typical Detail, Geocell (Sheet 1 of 4)

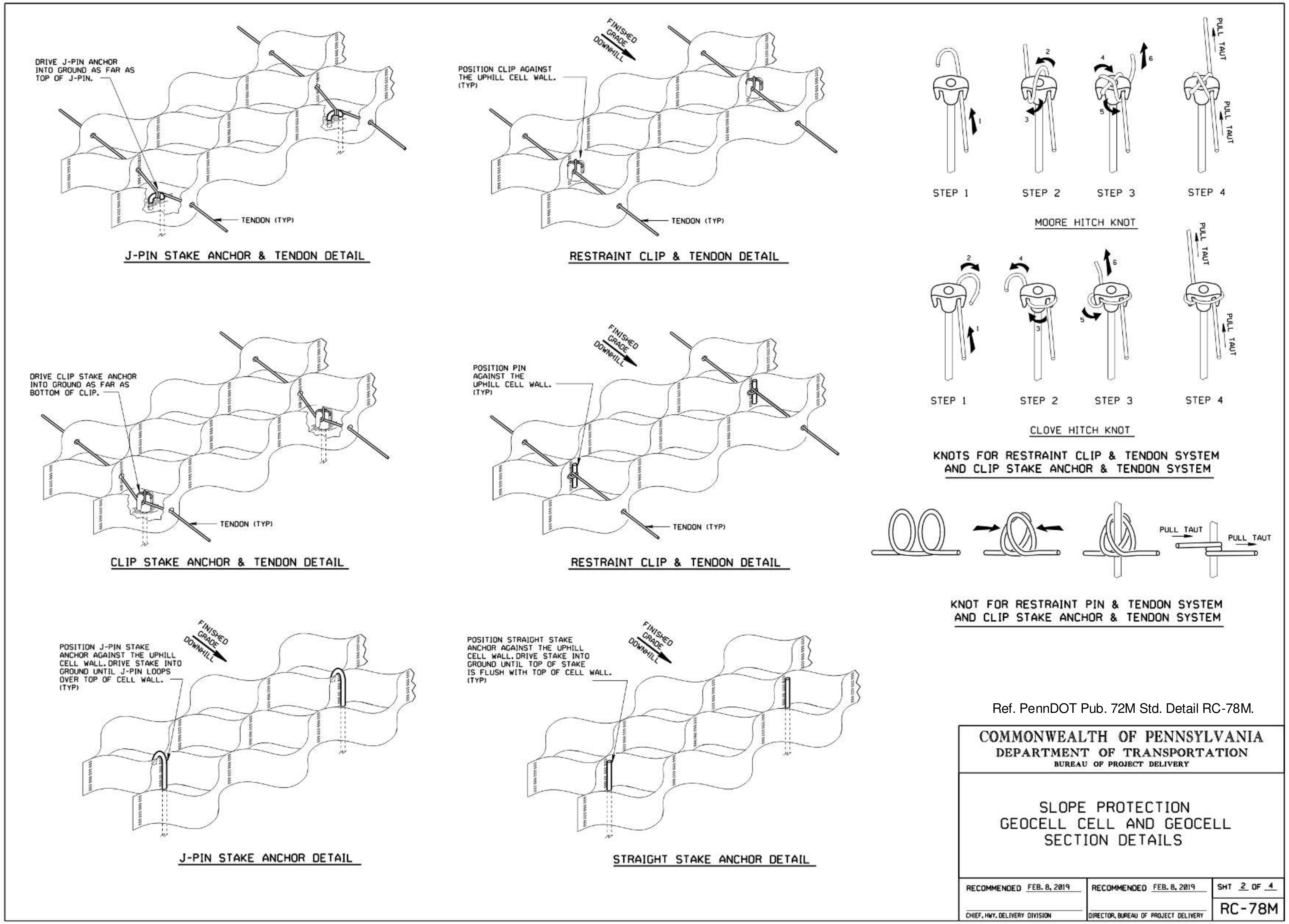


Figure 12-8 - Typical Detail, Geocell (Sheet 2 of 4)

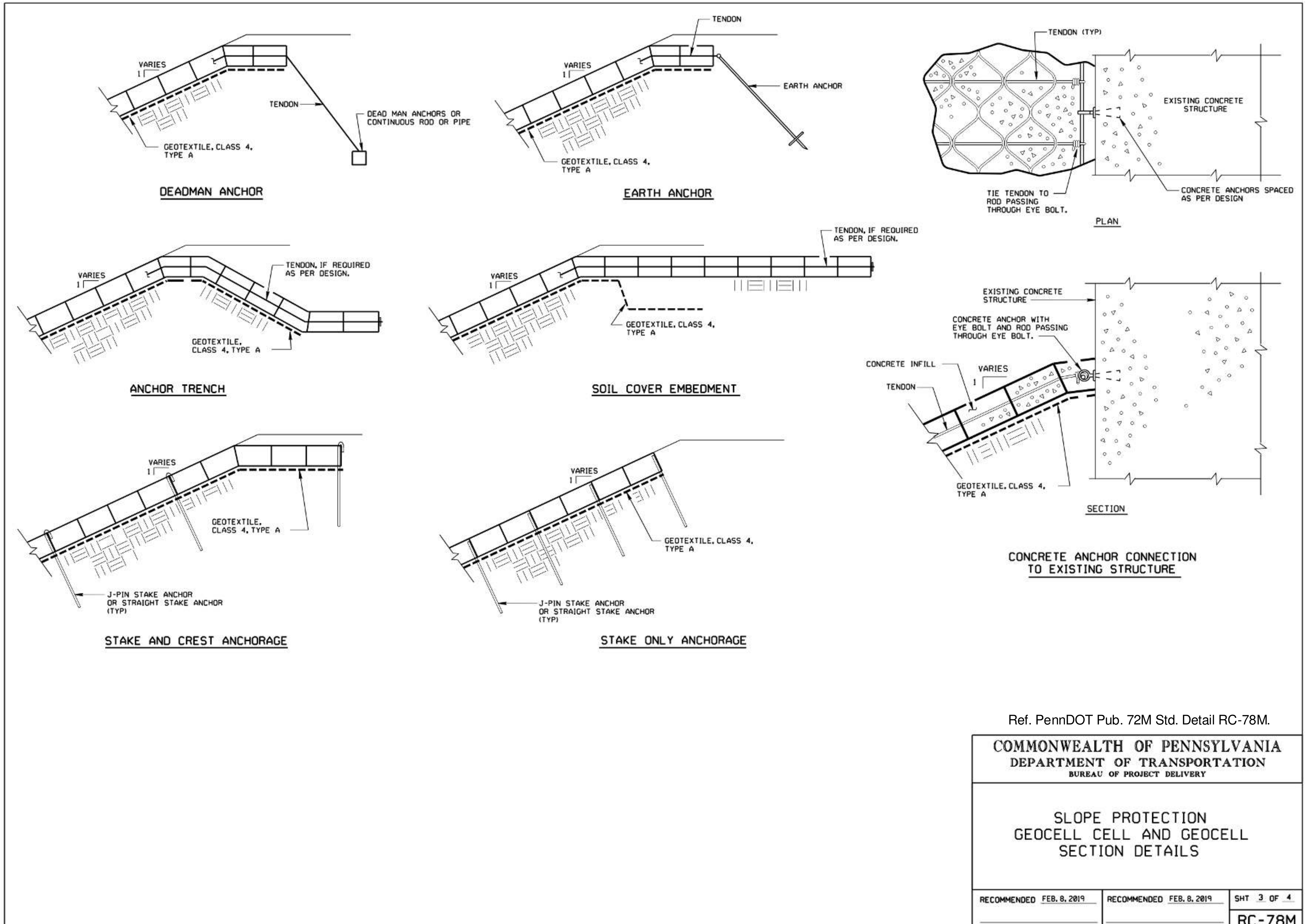


Figure 12-9 - Typical Detail, Geocell (Sheet 3 of 4)

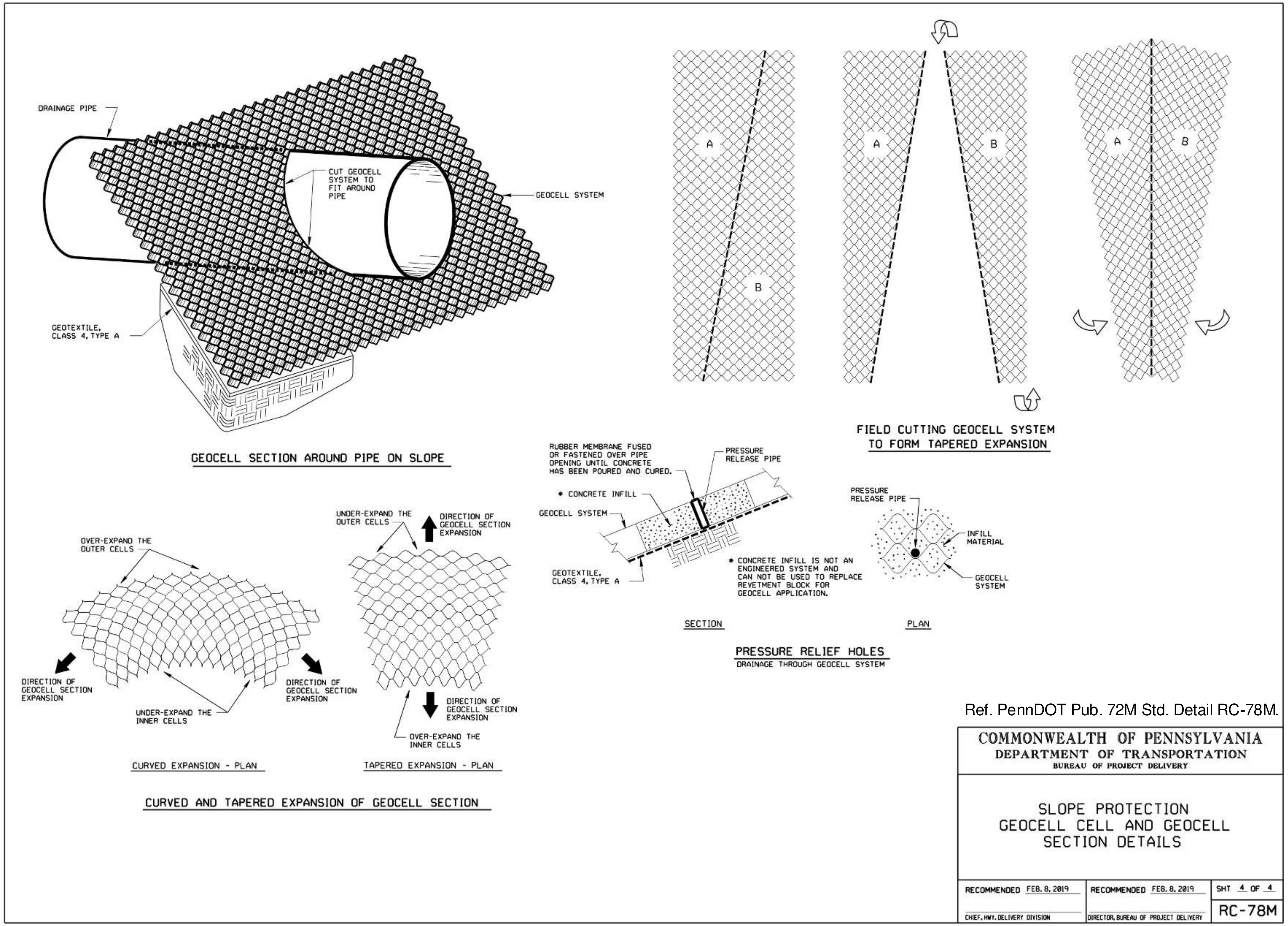
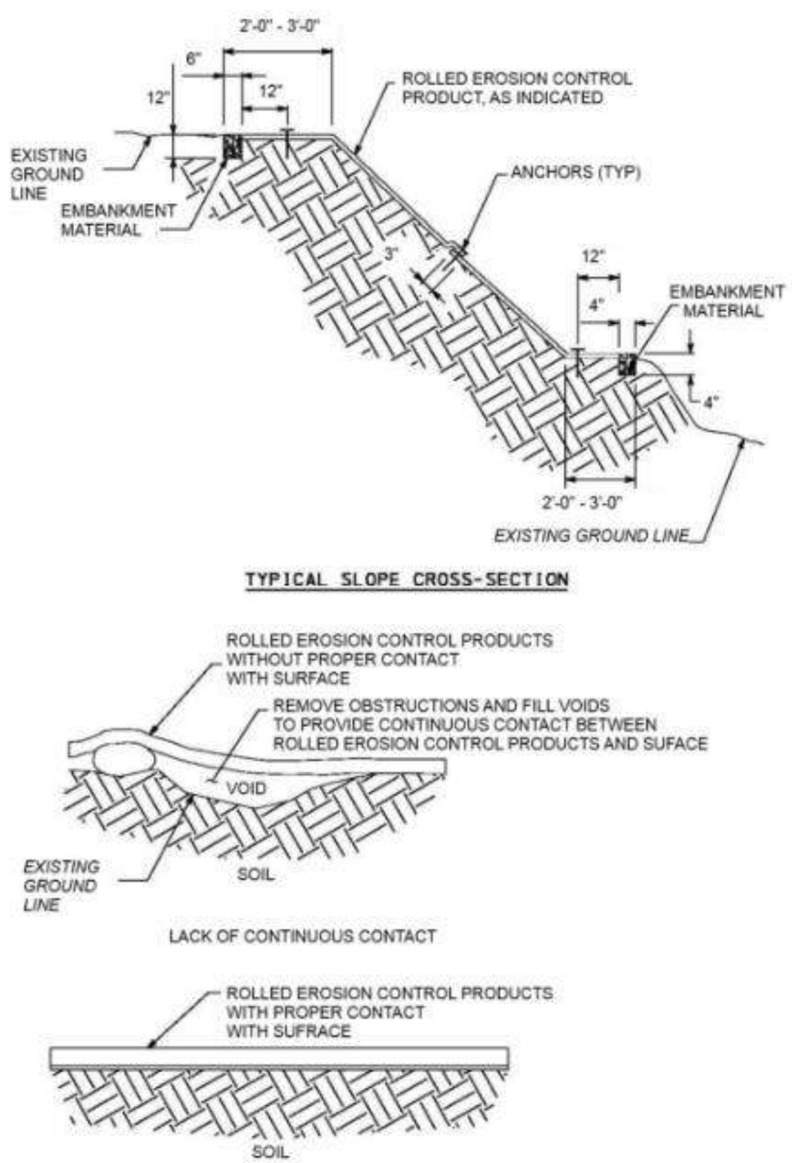
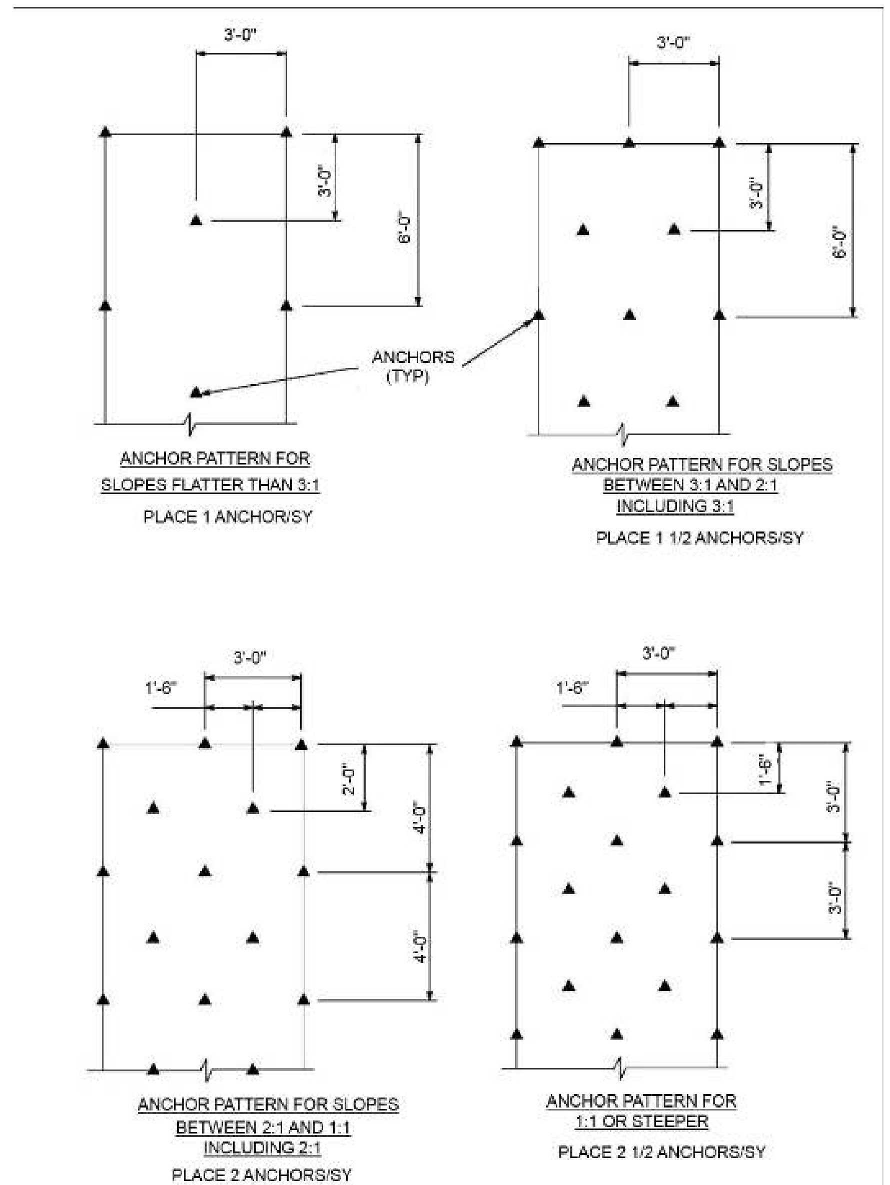


Figure 12-10 - Typical Detail, Geocell (Sheet 4 of 4)



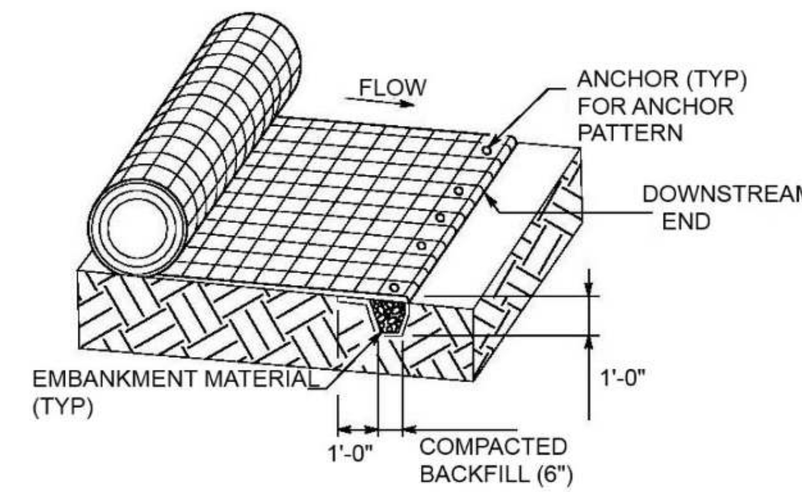
Blanket / Mat Orientation

(NOT TO SCALE)



Anchor Patterns

(NOT TO SCALE)



Anchor Trench

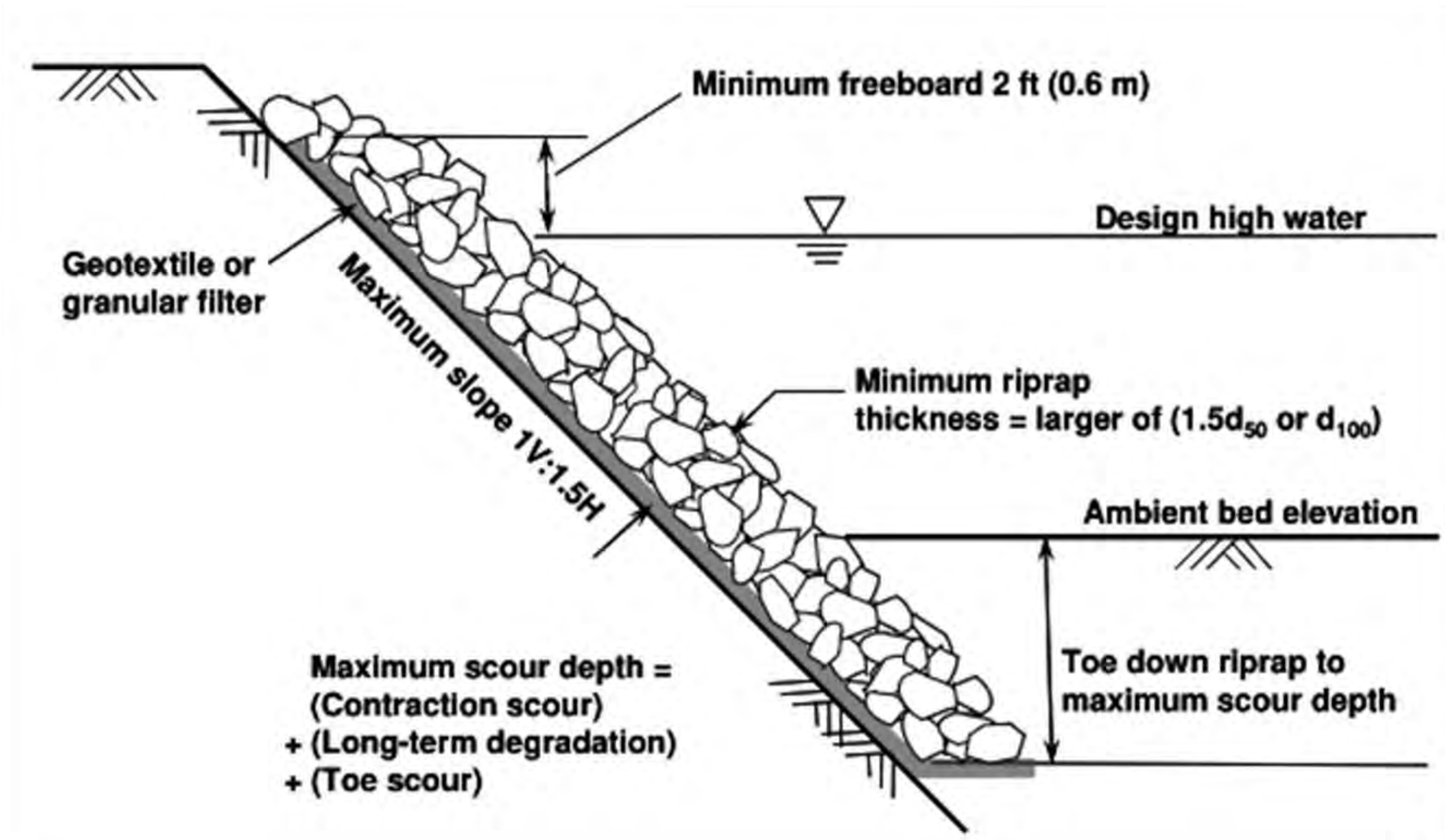
(NOT TO SCALE)

REF. PennDOT PUB. 464 (2013)

NOTES.

1. Provide materials and construct as specified in PennDOT Pub. 408, Sec. 806 for rolled erosion control products (e.g., turf reinforcement mat).
2. Use (Turf Reinforcement Mats) TRM on 1H:1V slopes, or flatter.
3. Dress the ground surface to provide a smooth and uniform surface, and promote continuous contact between the TRM and the ground surface.
4. Use an anchor trench to secure the upslope edge of the TRM. Provide shiplap splices at TRM seams. Use anchor trenches at the remaining three (3) terminal edges.
5. After the TRM is installed, apply seed and soil supplements such that the TRM and seed mixture is in intimate contact with the soil surface. Consider hydroseeding.
6. Install TRMs in accordance with the manufacturer' recommendations.
7. Inspect blanketed areas weekly and after each runoff event until perennial vegetation is established to a minimum uniform 70% coverage throughout the blanketed area.

Figure 12-11 - Typical Detail, Turf Reinforcement Mat



d_{50} and d_{100} is equal to the nominal gradation size that 50 and 100 percent of the riprap can pass through that sieve size, respectively.

The ambient bed elevation is the initial (unscoured) bed elevation in a stream, where applicable.

REF. NCHRP 568 (2006) and HEC 23 Vol 2 Design Guideline 4 (2009)

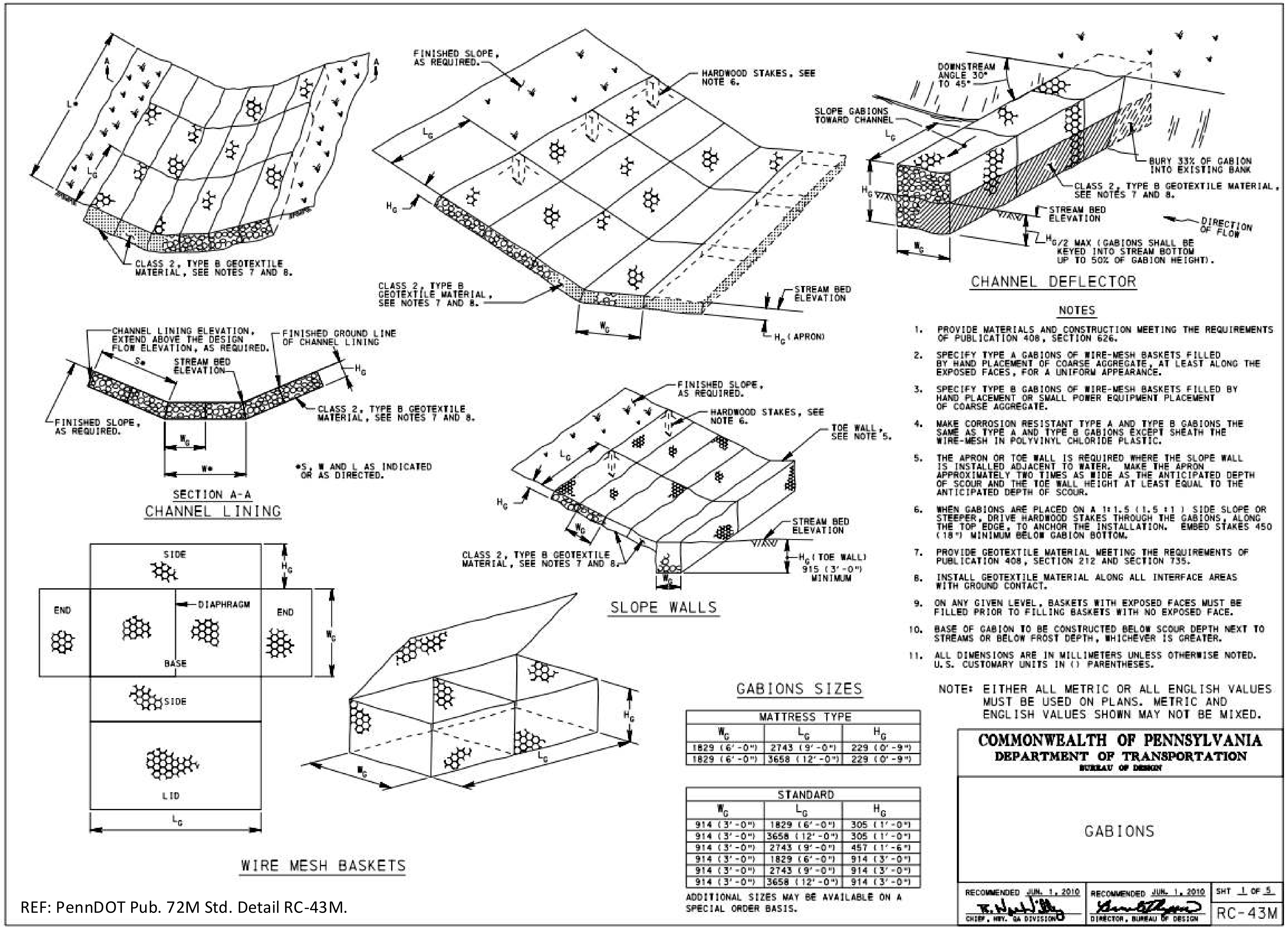
TYPICAL DETAIL – RIPRAP REVETMENT

(NOT TO SCALE)

NOTES.

1. Provide materials and construct as specified in PennDOT Pub. 408, Sec. 850 for rock lining.
2. Refer to HEC 23 Volume 2 Design Guideline 4 [44] to determine riprap filter requirements, edge treatment and termination details, and design guidelines.
3. For PennDOT projects, refer to PennDOT Pub. 72M Std. Detail RC-40M for additional guidance.

Figure 12-12 - Typical Detail, Rip Rap Revetment



GABIONS SIZES

MATTRESS TYPE		
W_G	L_G	H_G
1829 (6'-0")	2743 (9'-0")	229 (0'-9")
1829 (6'-0")	3658 (12'-0")	229 (0'-9")

STANDARD		
W_G	L_G	H_G
914 (3'-0")	1829 (6'-0")	305 (1'-0")
914 (3'-0")	3658 (12'-0")	305 (1'-0")
914 (3'-0")	2743 (9'-0")	457 (1'-6")
914 (3'-0")	1829 (6'-0")	914 (3'-0")
914 (3'-0")	2743 (9'-0")	914 (3'-0")
914 (3'-0")	3658 (12'-0")	914 (3'-0")

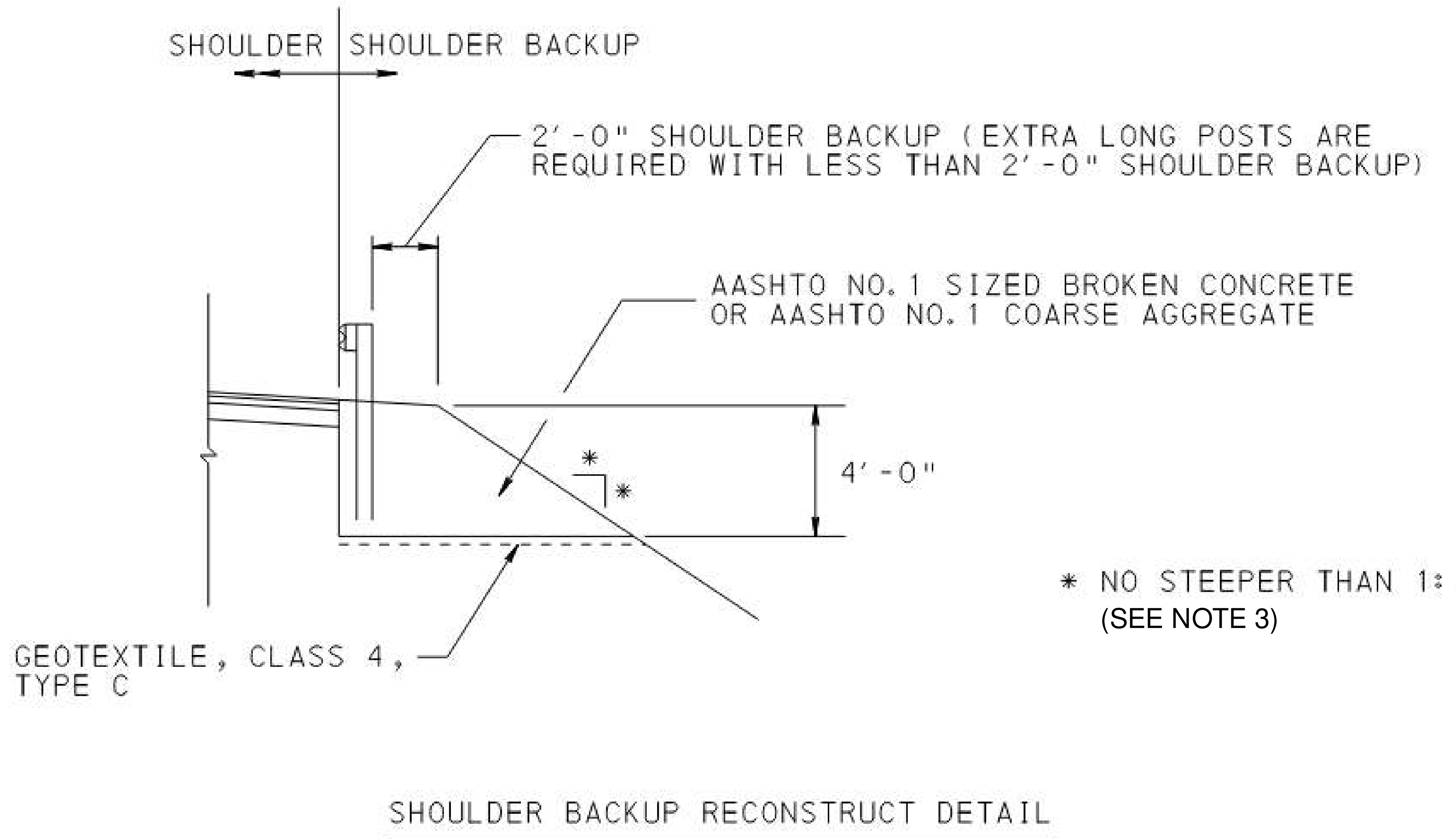
ADDITIONAL SIZES MAY BE AVAILABLE ON A SPECIAL ORDER BASIS.

TYPICAL DETAIL – GABION SLOPE (E.G., MATTRESS)

Figure 12-13 - Typical Detail, Gabion Slope (e.g., Mattress)

Notes.

12. Refer to HEC 23 Volume 2 Design Guideline 6 [44] for additional design guidelines.
13. Install gabions in accordance with the manufacturer' recommendations.
14. Fill gabions with free-draining material (e.g., no grout).

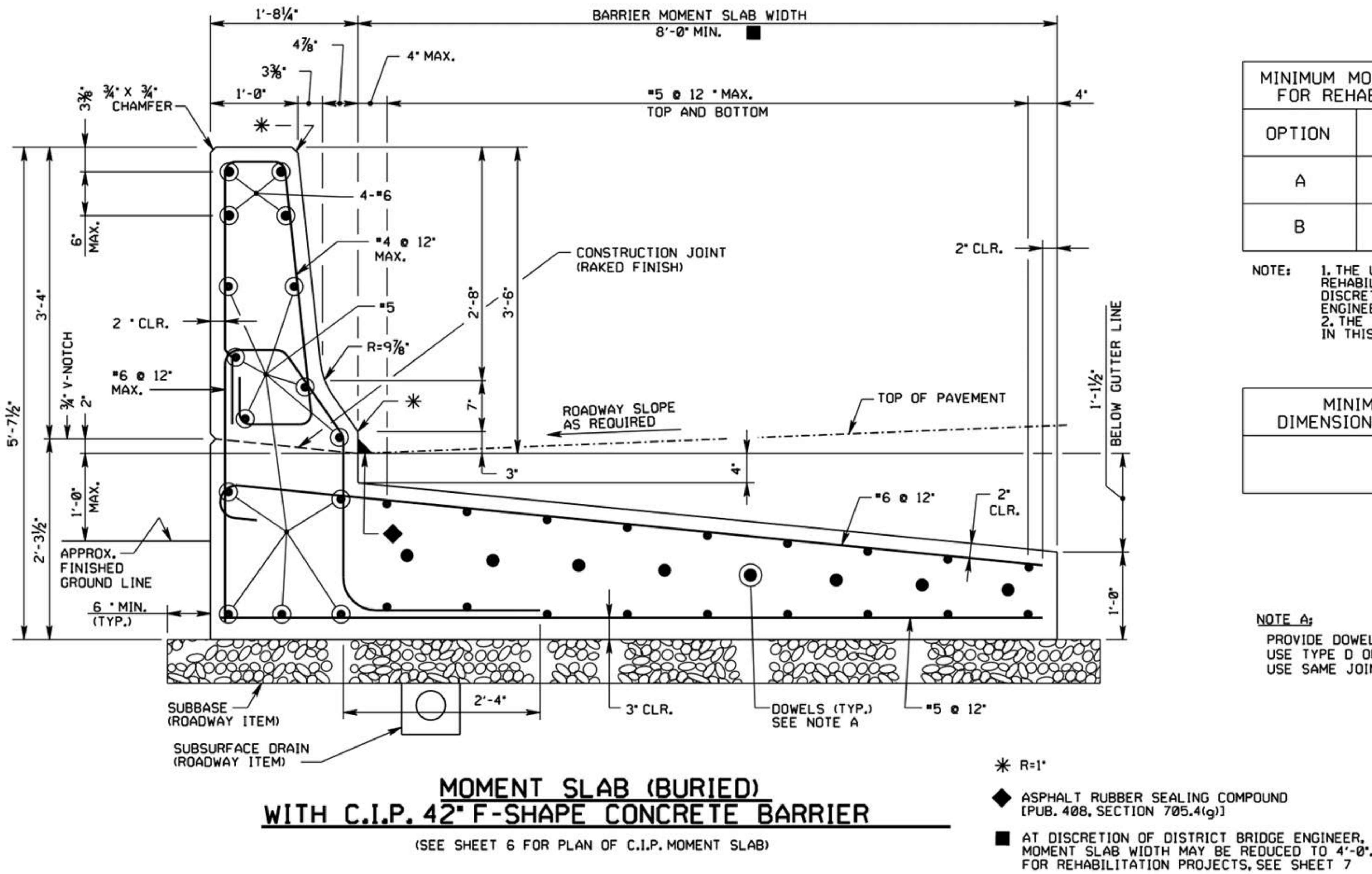


REF. PennDOT District 11

NOTES.

1. Provide materials and construct as specified in PennDOT Pub. 408, Sec. 206 for placement and compaction of embankment.
2. When a strong guide rail system is installed, maintain a minimum 2-foot clearance between the rear face of guide rail post and the fill slope break point. If a 2-foot clearance cannot be maintained because of topography or other constraint, then install longer guide rail posts in accordance with PennDOT Pub. 72M Std. Detail RC-51M Sht. 1 of 16, Note 10.
3. See Section 10.3.1.1 for additional information regarding the maximum slope angle to be considered.

Figure 12-14 - Typical Detail, Shoulder Back-Up



MINIMUM MOMENT SLAB DIMENSIONS FOR REHABILITATION PROJECTS		
OPTION	L	W
A	20'-0"	4'-0"
B	16'-0"	5'-0"

NOTE: 1. THE USE OF THIS TABLE IS FOR REHABILITATION ONLY AND AT THE DISCRETION OF THE DISTRICT BRIDGE ENGINEER.
2. THE DESIGNED CRASH TEST LEVEL IN THIS TABLE IS TL-3.

MINIMUM MOMENT SLAB DIMENSIONS FOR NEW PROJECTS
L=16'-0" W=8'-0"

NOTE A:
PROVIDE DOWELS AT EXPANSION JOINTS.
USE TYPE D OR E JOINT PER RC-20M.
USE SAME JOINT AS PROVIDED IN PAVEMENT.

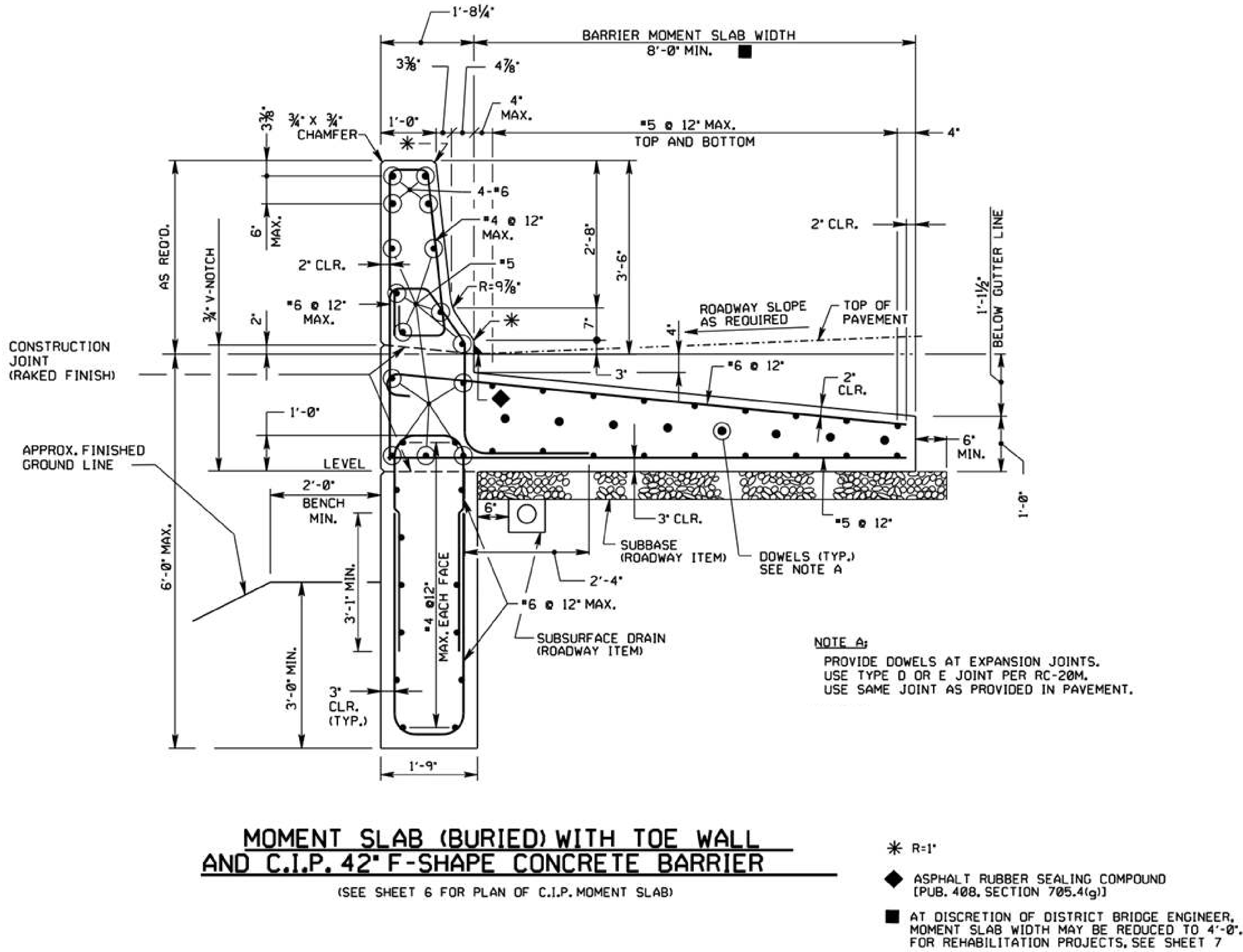
TYPICAL DETAIL – MOMENT SLAB

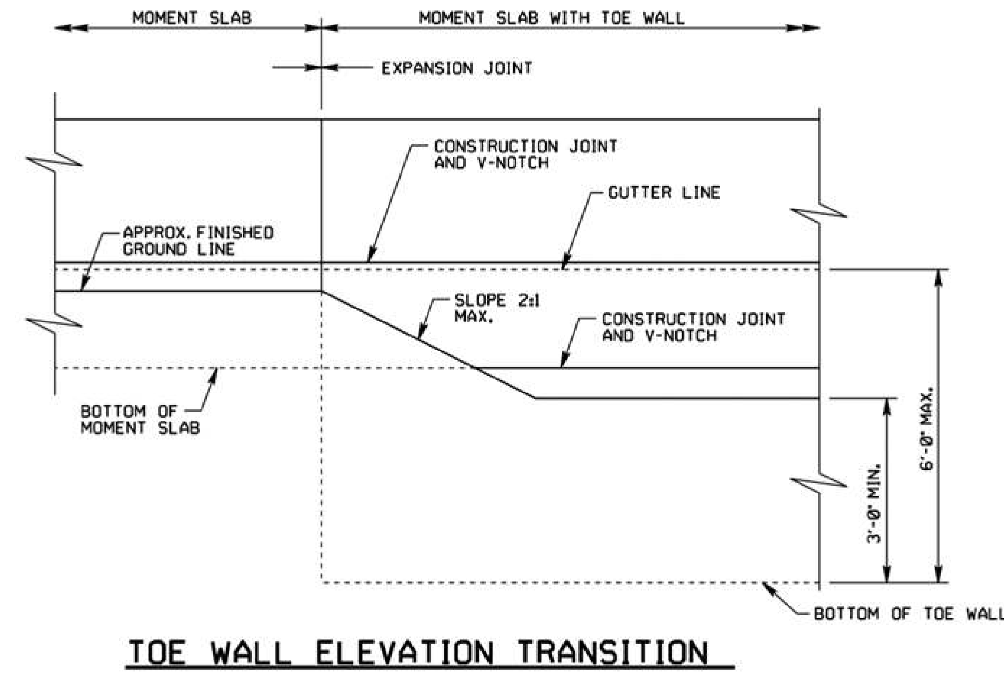
REF. PennDOT Pub. 218M Std. Detail BD-627M.

(NOT TO SCALE)

NOTES.
1. Provide materials and construct in accordance with applicable sections of PennDOT Pub. 408 and PennDOT Pub. 218M Std. Dwg. BD-627M.
2. Check to see if the moment slab is required to be compliant with MASH TL-3 or TL-4 rating requirements. The 32-inch and 42-inch F-shape concrete barrier, which is attached to the moment slab, are designated as MASH TL-3 and TL-4 compliant, respectively.

Figure 12-15 - Typical Detail, Moment Slab





REF. PennDOT Pub. 218M Std. Detail BD-627M.

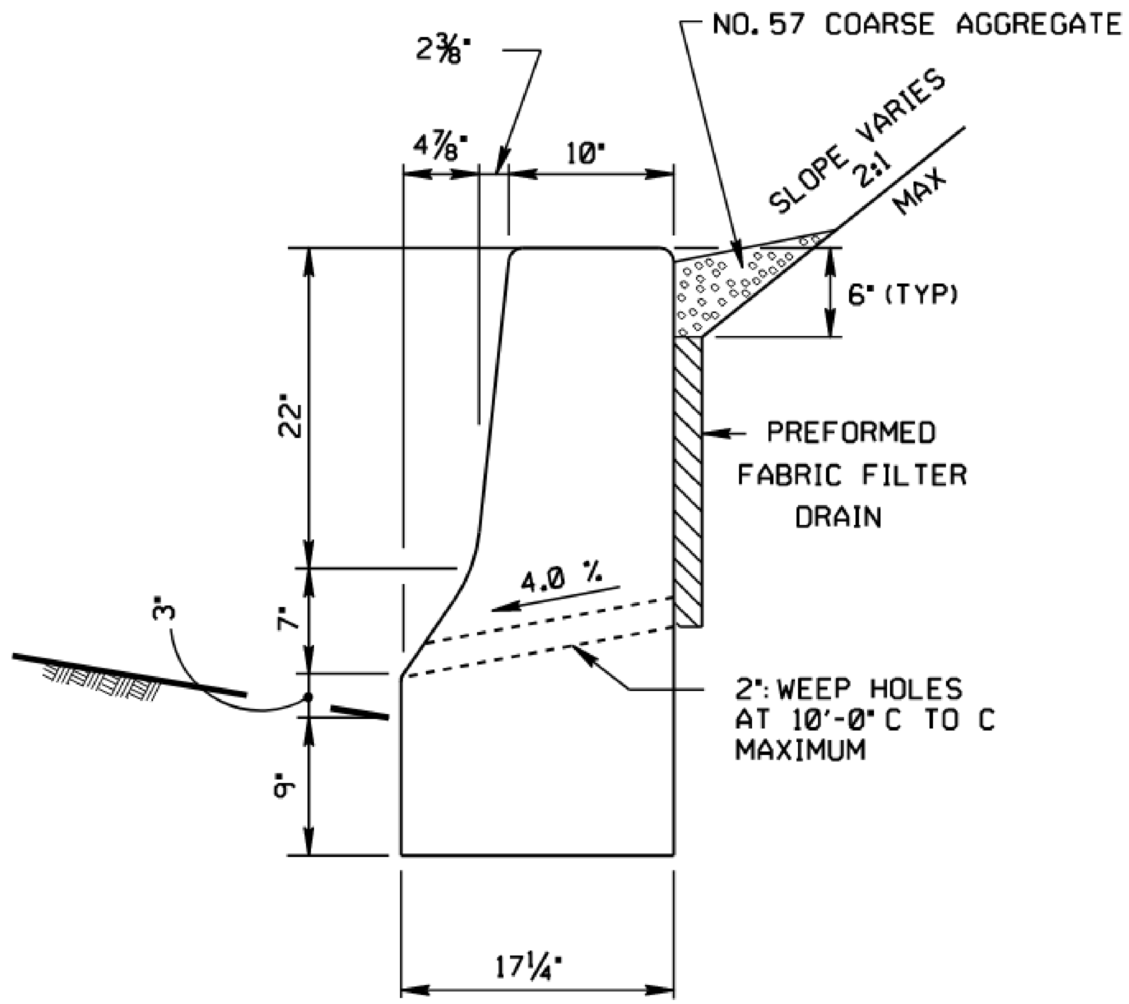
TYPICAL DETAIL – MOMENT SLAB WITH TOE WALL

(NOT TO SCALE)

NOTES.

1. Provide materials and construct in accordance with applicable sections of PennDOT Pub. 408 and PennDOT Pub. 218M Std. Dwg. BD-627M.
2. Check to see if the moment slab is required to be compliant with MASH TL-3 or TL-4 rating requirements. The 32-inch and 42-inch F-shape concrete barrier, which is attached to the moment slab, are designated as MASH TL-3 and TL-4 compliant, respectively.

Figure 12-16 - Typical Detail, Moment Slab with Toe Wall



TYPICAL DRAINAGE TREATMENT

SEE NOTE 2.

NOTES

1. PROVIDE STRUCTURAL STEEL PLATES MEETING THE REQUIREMENTS OF PUBLICATION 408, SECTION 1105. FOR PERMANENT BARRIER, GALVANIZE PLATES AS SPECIFIED IN PUBLICATION 408, SECTION 1105.02(s). ALTERNATE CONNECTIONS MAY BE USED AS APPROVED BY THE BUREAU OF DESIGN. FOR TEMPORARY BARRIER, DO NOT GALVANIZE THE STRUCTURAL STEEL PLATES.
2. WHERE SINGLE FACE CONCRETE BARRIER IS SPECIFIED FOR USE AS A RETAINING WALL AND DRAINAGE TREATMENT IS NECESSARY, CONSTRUCT A PREFORMED FABRIC FILTER DRAIN AS INDICATED AND IN ACCORDANCE WITH PUBLICATION 408, SECTION 610. CHECK STABILITY OF BARRIER USED AS A RETAINING WALL AND PROVIDE COMPUTATION WITH THE CONSTRUCTION PLANS.
3. ROUND OR CHAMFER ALL EDGES WITH A RADIUS OF 1" EXCEPT AS SHOWN.

Ref. PennDOT Pub. 72M Std. Detail RC-58M.

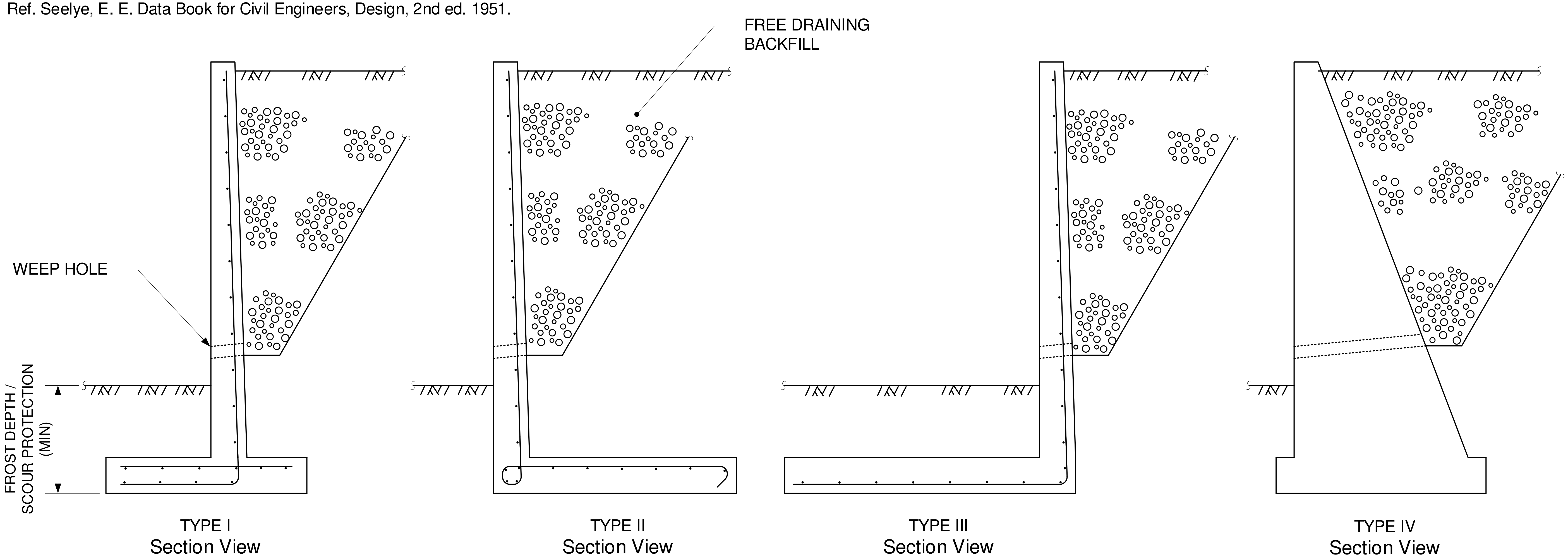
TYPICAL DETAIL – SINGLE FACE BARRIER-TYPE RETAINING STRUCTURE

NOTES.

4. Provide materials and construct as specified in PennDOT Pub. 408, Sec. 623 for concrete barrier.
5. It is typical to include an additional hydraulic design component to this detail at the top of the wall to collect and convey surface water into a drainage system.
6. Where single face concrete barriers is designed to function as a retaining wall, check external stability (e.g., sliding, overturning, and bearing resistance) of the barrier.

Figure 12-17 - Typical Detail, Single Face Barrier

Ref. Seelye, E. E. Data Book for Civil Engineers, Design, 2nd ed. 1951.



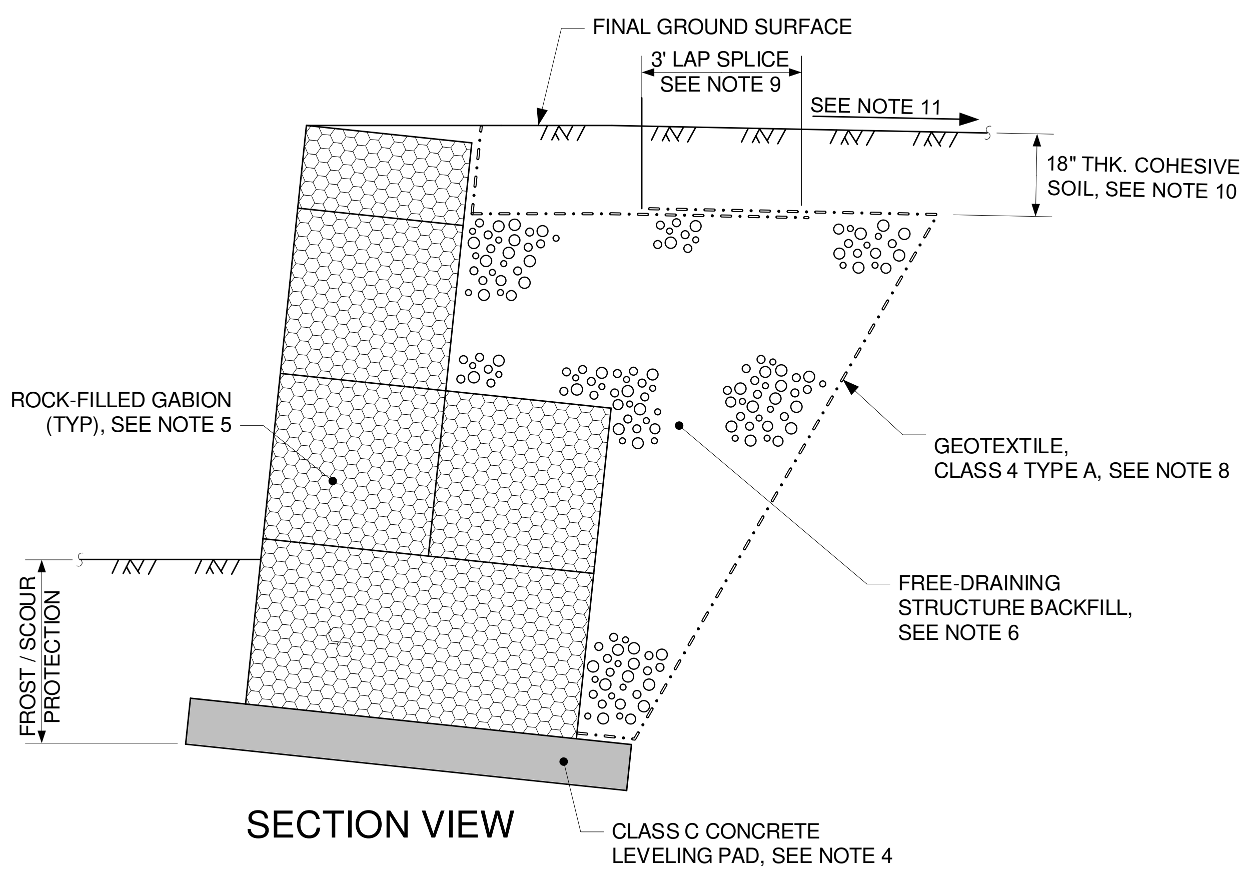
TYPICAL DETAIL – CONCRETE GRAVITY WALLS

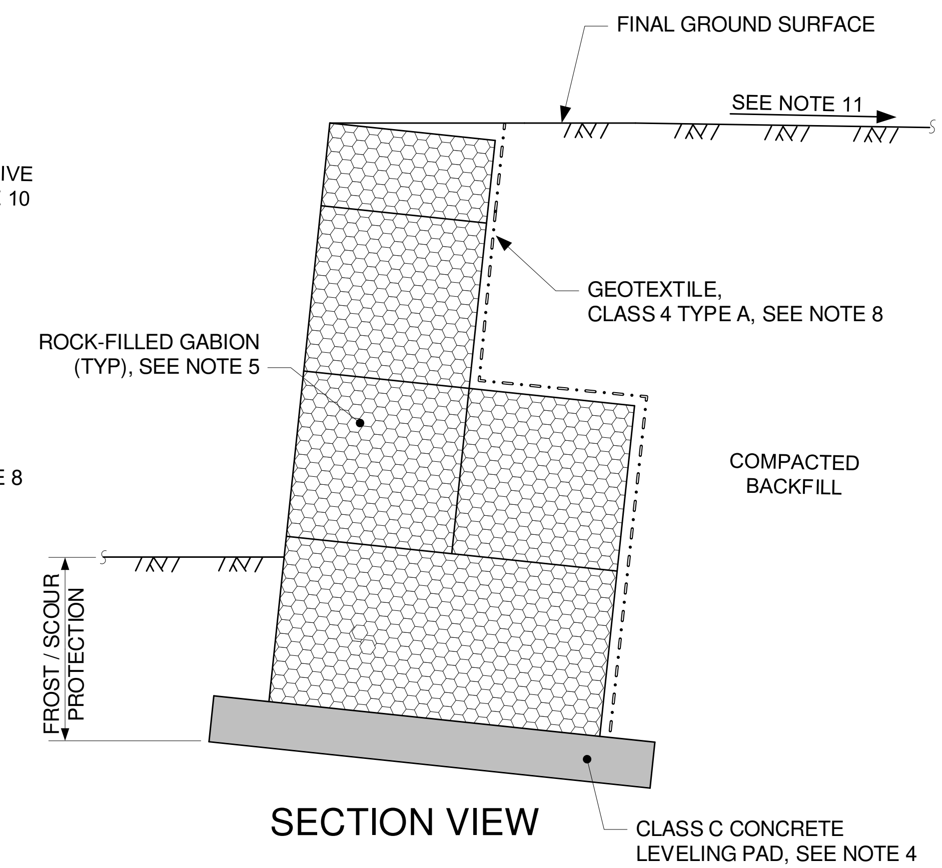
(NOT TO SCALE)

NOTES.

1. Provide materials and construct in accordance with PennDOT Pub. 408.
2. Complete external stability and overall (global) stability analyses and use those results to design the cantilevered retaining wall in accordance with the applicable design standards and/or codes.
3. Four (4) traditional types of concrete gravity walls (Types I, II, III, and IV) are depicted on this sheet. Each of these wall types are drawn proportionately with respect to each other to provide reader perspective about the geometric difference between these wall types. Wall type selection and wall geometry will depend on the site constraints, structural analysis, and stability analysis.
4. Provide free-draining structure backfill and a positive means (e.g., pipe, weep holes, etc.) to dissipate the buildup of hydrostatic pressure behind the walls and at the critical rupture.

Figure 12-18 - Typical Detail, Conventional Concrete Gravity Walls





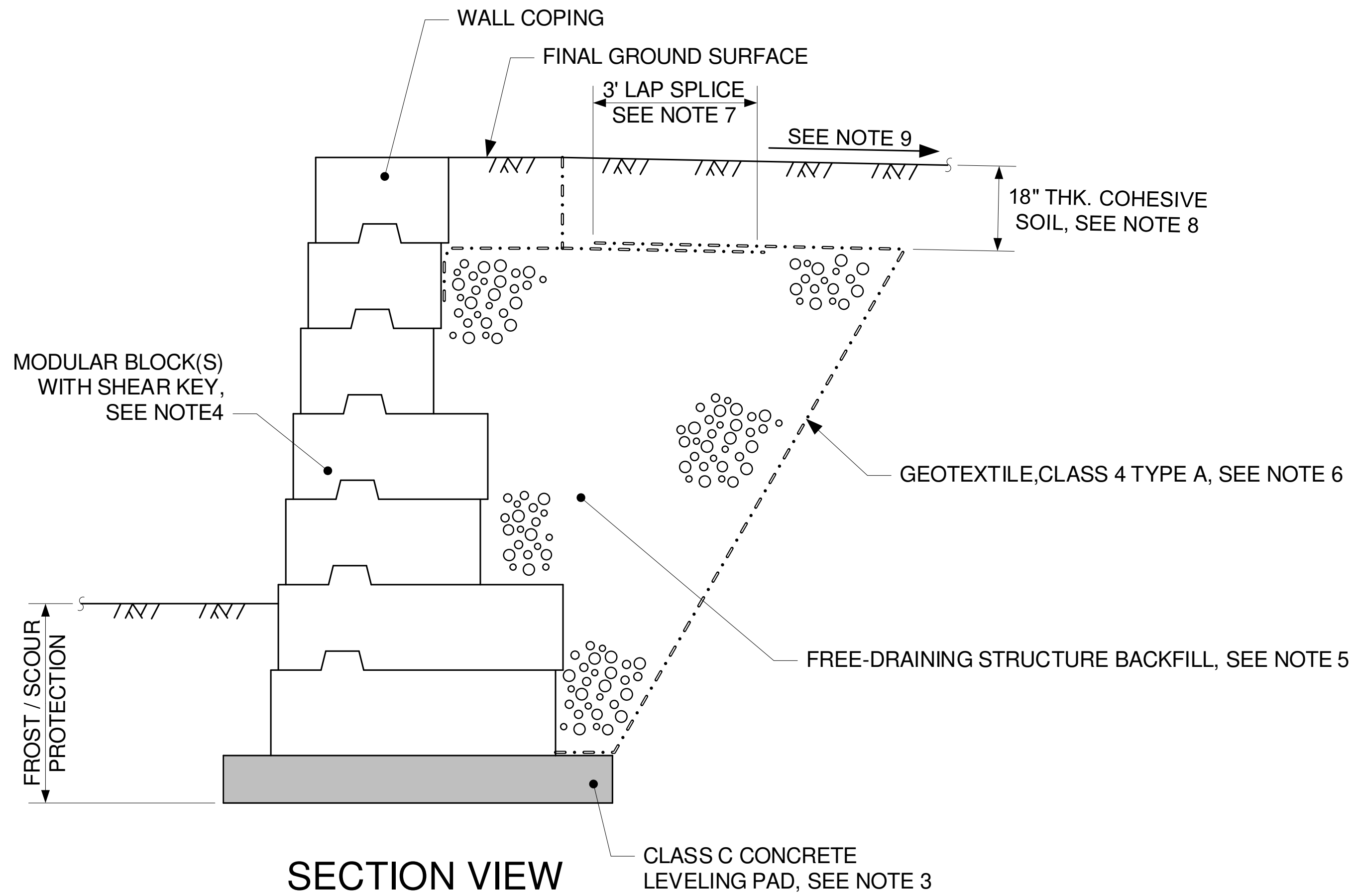
TYPICAL DETAIL – GABION WALL

(NOT TO SCALE)

NOTES.

1. Provide materials and construct in accordance with PennDOT Pub. 408, Sec. 626.
2. See PennDOT DM-4 [99] Section 11 for PennDOT design requirements for use of gabion retaining walls
3. Complete external (e.g., sliding, overturning, and bearing resistance) and overall (global) stability (shallow and deep-seated) to determine the number and size of gabions required to maintain wall stability. Consider adding geogrid reinforcement (not shown in section view), if required, to enhance stability.
4. Install the Leveling Pad to develop a uniform base upon which to set the gabion baskets. Step the leveling pad to compensate for the sloping ground along the face of the wall.
5. Place gabion baskets from the bottom up. Compensate for wall batter (e.g., 5 to 6 degrees from vertical is common).
6. Where compaction behind the gabions is not possible or where additional dissipation of pore pressure along the rupture plane is required for global stability, backfill with free-draining structure backfill; control gap width at gabion joints to mitigate against piping erosion of structure backfill.
7. Where possible provide a positive means of drainage (e.g., drainage pipe, etc.) at the base of the gabions at the interface with the backfill material.
8. Install non-woven geotextile to prevent migration of in-situ soils.
9. Lap splice the geotextile to mitigate against contamination of the structure backfill with soil fines.
10. Install a compacted cohesive soil cover to minimize surface water infiltration.
11. Slope the ground surface to provide positive drainage away from the gabion wall and prevent ponding.
12. Width of the top block should allow adequate room for guiderail installation.
13. Consult manufactures specifications for various slope heights and gabion configurations

Figure 12-19 - Typical Detail, Gabion Wall



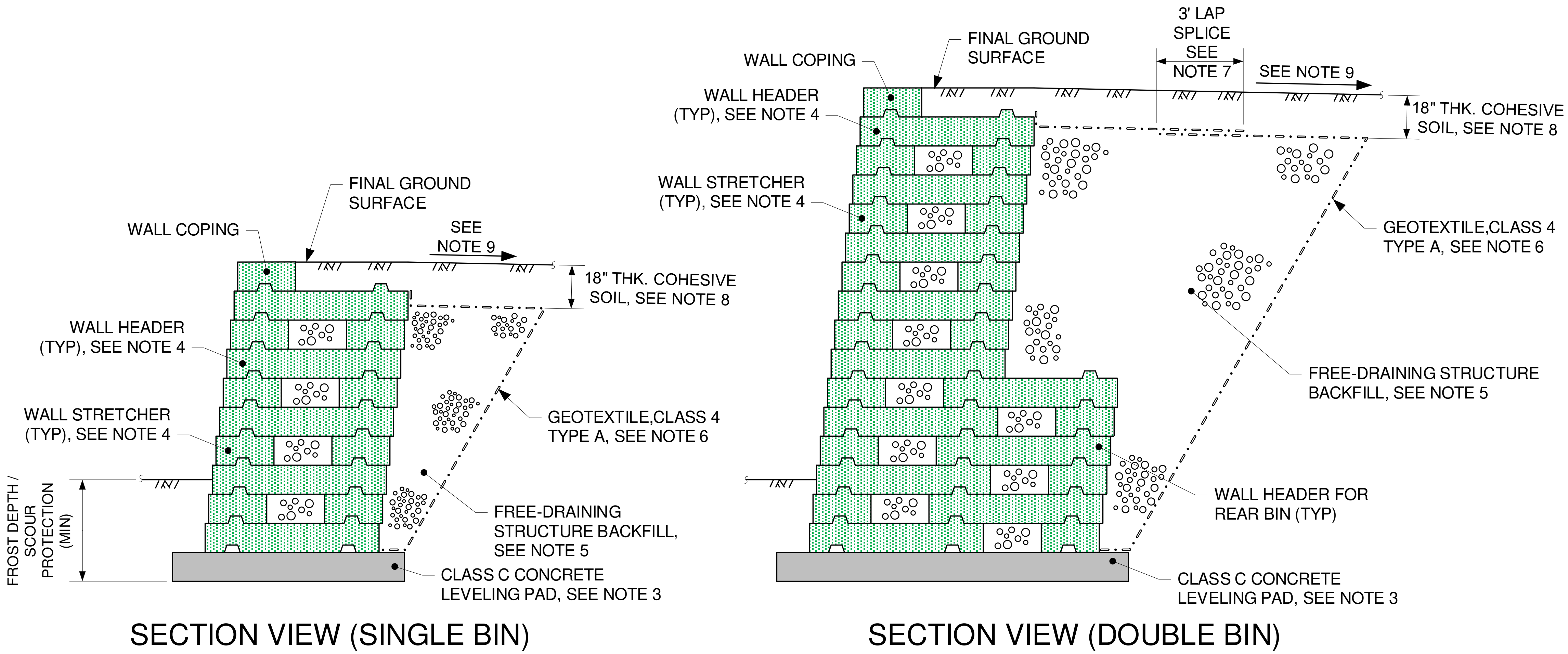
TYPICAL DETAIL – MODULAR BLOCK WALL

(NOT TO SCALE)

NOTES.

1. Provide materials and construct in accordance with PennDOT Pub. 408.
2. Complete external (e.g., sliding, overturning, and bearing resistance) and overall (global) stability (shallow and deep-seated) to determine the number and size of modular blocks required that are required to maintain wall stability. Consider adding geogrid reinforcement (not shown in section view), if required, to enhance external and/or internal stability.
3. Install the Leveling Pad to develop a uniform base upon which to set the modular blocks. Step the leveling pad to compensate for the sloping ground along the face of the wall.
4. Place modular blocks from the bottom up. Compensate for wall batter (e.g., 5 to 6 degrees from vertical is common). Slide each modular block forward (when it is set) to engage shear key(s).
5. Backfill the modular block wall with a free-draining structure backfill. Provide a positive means (e.g., pipe, small gaps at block joints, etc.) to dissipate the buildup of hydrostatic pressure behind the modular block wall. Control gap width at block joints to mitigate against piping erosion of structure backfill.
6. Install non-woven geotextile to encapsulate the structure backfill.
7. Lap splice the geotextile to mitigate against contamination of the structure backfill with soil fines.
8. Install an impermeable soil cover to minimize surface water infiltration.
9. Slope the ground surface to provide positive drainage away from the modular block wall and prevent ponding.
10. Size top block to allow adequate room for guiderail installation.
11. See the list [PennDOT Approved Bridge and Structure Products](#) to reference approved typical sections for select modular block wall systems.

Figure 12-20 - Typical Detail, Modular Block Wall



TYPICAL DETAIL – CRIB WALL

(NOT TO SCALE)

NOTES.

1. Provide materials and construct in accordance with PennDOT Pub. 408.
2. Complete external (e.g., sliding, overturning, and bearing resistance) and overall (global) stability (shallow and deep-seated) to determine the number and size of crib wall units that are required to maintain wall stability
3. Install the Leveling Pad to develop a uniform base upon which to set the bottom row of the wall header. Step the leveling pad to compensate for the sloping ground along the face of the wall.
4. Place wall headers and stretchers from the bottom up. Compensate for wall batter (e.g., verify acceptable batter with the manufacturer; however, 5 to 6 degrees from vertical is common). Openings in the wall will require a special stretcher and may require additional headers. For non-tangent wall alignment, special-length stretchers may be required.
5. Backfill the crib wall with free-draining structure backfill including, but not limited to, the space between the headers and stretchers. Provide a positive means (e.g., pipe) to dissipate the buildup of hydrostatic pressure behind the crib wall. Control the gap width between the stretchers (e.g., spacers) to mitigate against piping erosion of structure backfill.
6. Install non-woven geotextile to encapsulate the structure backfill.
7. Lap splice the geotextile to mitigate against contamination of the structure backfill with soil fines.
8. Install an impermeable soil cover to minimize surface water infiltration.
9. Slope the ground surface to provide positive drainage away from the modular block wall and prevent ponding.
10. Install in accordance with manufacturer specifications.
11. See the list PennDOT Approved Bridge and Structure Products to reference approved typical sections for select crib wall systems.

Figure 12-21 - Typical Detail, Crib Wall

GENERAL NOTES

(FOR CONTRACT DRAWINGS)

1. PROVIDE MATERIALS AND PERFORM WORK IN ACCORDANCE WITH SPECIFICATIONS PUBLICATION 408, AASHTO/AWS D1.5 BRIDGE WELDING CODE AND THE SPECIAL PROVISIONS.
2. DESIGN SPECIFICATIONS: AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS AS SUPPLEMENTED BY DESIGN MANUAL PART 4, STRUCTURES.
3. USE CLASS A CEMENT CONCRETE FOR CONCRETE EMBEDMENT, CAST-IN-PLACE CONCRETE, LAGGING, WALLS, AND CORBELS.
4. FOR PRECAST CONCRETE LAGGING FOLLOW SECTION 714 OF PUB 408 AND USE 4000 PSI CONCRETE. CHAMFER EXPOSED CONCRETE EDGES ¾" × ¾" WHERE NOTED.
5. PROVIDE GRADE 60 REINFORCING BARS THAT MEET THE REQUIREMENTS OF ASTM A 615, A 996, OR A 706. DO NOT WELD GRADE 60 REINFORCING BARS UNLESS SPECIFIED. GRADE 40 REINFORCING STEEL BARS MAY BE SUBSTITUTED WITH A PROPORTIONAL INCREASE IN CROSS-SECTIONAL AREA IF APPROVED BY THE CHIEF BRIDGE ENGINEER. DO NOT USE RAIL STEEL ASTM A 996 WHERE BENDING OR WELDING OF THE REINFORCEMENT BARS IS INDICATED.
6. PROVIDE STRUCTURAL STEEL CONFORMING TO AASHTO M270 (ASTM A709) GRADE 36 OR 50. PIPE CONFORMING TO API GRADE N-80 OR ASTM A53 MAY BE USED AS PERMANENT CASING.
7. PROVIDE WELDED STUD SHEAR CONNECTORS MANUFACTURED FROM STEEL CONFORMING TO ASTM A108.
8. IF REQUIRED, PROVIDE PERMANENT CASING CONFORMING TO ASTM A 53 AT THE ANCHOR HEAD. API GRADE N-80 CASING WITH FLUSH JOINT THREADED CONNECTION TO ASTM A 53 PIPE MAY BE USED FOR THE REMAINDER OF THE CASING . [USE FOR LATENT ANCHOR SYSTEMS WHERE STRUCTURAL WELDS ARE REQUIRED.]
9. PROVIDE PERMANENT CASING IN THE BACKFILL WHEN CONSTRUCTING A WALL FROM THE BOTTOM UP. SPLICING OF PILES IS DISCOURAGED. IF CONDITIONS DICTATE, PROVIDE SPLICE A MINIMUM OF 5' BELOW FINISHED GROUND LINE IN FRONT OF THE WALL. USE SPLICE DETAIL IN ACCORDANCE WITH BC-757M.
10. CONTROL PILE DRIVING BY THE WAVE EQUATION ANALYSIS. DRIVE TEST PILES TO ABSOLUTE REFUSAL. THE ENGINEER SHALL VERIFY FROM THE TEST PILE DRIVING RESULTS THE CAPABILITY OF THE PILE HAMMER SELECTED BY THE CONTRACTOR. DRIVE BEARING PILES TO ABSOLUTE REFUSAL INTO THE STRATUM DEFINED BY A TIP ELEVATION WHICH IS PREDETERMINED BY THE ENGINEER FROM TEST PILES. THE STRUCTURE CONTROL ENGINEER SHALL DETERMINE THE ACCEPTABILITY OF THE BEARING PILES WHICH ATTAIN ABSOLUTE REFUSAL ABOVE THE PREDETERMINED TIP ELEVATIONS.
11. PROVIDE PILE TIP REINFORCEMENT FOR DRIVEN PILES.
12. FOR CAST-IN-PLACE WALLS DRIVE OR PLACE PILES TO WITHIN 2" IN 10' OF VERTICAL. DRIVE OR PLACE EACH PILE TO WITHIN 3" OF THE INDICATED LOCATION AT FINISHED GROUND LINE IN FRONT OF THE WALL.
13. FOR PRECAST LAGGING DRIVE OR PLACE PILES TO WITHIN 1" IN 10' OF VERTICAL, BUT DO NOT ALLOW AN OUT-OF-PLANE OFFSET OF MORE THAN 1" IN 10" WITH RESPECT TO ADJACENT PILES. DRIVE OR PLACE EACH PILE WITHIN 2" HORIZONTALLY OF THE INDICATED LOCATION AT FINISHED GROUND LINE. PROVIDE MINIMUM BEARING DISTANCE FOR PRECAST LAGGING AT EDGE OF PILE FLANGE AS INDICATED IN PRECAST CONCRETE LAGGING DETAILS. IF NECESSARY, FABRICATE PRECAST LAGGING AFTER DRIVING OR PLACING PILES TO ENSURE PROPER FIT AND BEARING DISTANCE.
14. PROVIDE ANCHOR TENDONS CONSISTING OF 7-WIRE UNCOATED STRANDS CONFORMING TO AASHTO M 203 (ASTM A 416) WITH LOW-RELAXATION WIRE CONFORMING TO ASTM A 421, GRADE 270. [FOR ANCHORS CONSISTING OF STRANDS]
15. PROVIDE ANCHOR TENDONS CONSISTING OF STEEL BARS CONFORMING TO AASHTO-M275, TYPE II.
16. FOR GROUTING ANCHORS USE NEAT CEMENT OR SAND CEMENT GROUT WITH TYPE I, II, OR III PORTLAND CEMENT CONFORMING TO AASHTO M85. [INDICATE TYPE II FOR CORROSIVE ENVIRONMENTS.] BULLETIN 15 APPROVED NONSHRINK OR EXPANSIVE ADDITIVES MAYBE USED.
17. PROVIDE NO. 57 COARSE AGGREGATE FOR STRUCTURAL BACKFILL. PLACE BACKFILL IN ACCORDANCE WITH SECTION 1001.3(q)2b OF PUB. 408.
18. FOR AS-DESIGNED PERMANENT ANCHOR WALL - THE CONTRACTOR IS RESPONSIBLE FOR THE FINAL DESIGN AND DETAILED DESIGN OF THE FOLLOWING:
    - ANCHOR STRAND DESIGN
    - TRUMPET DESIGN
    - UNBONDED STRESSING LENGTH
    - BOND LENGTH DESIGN
    - STEEL CASING EMBEDMENT LENGTH [LATENT ANCHORS ONLY]
    - ANCHOR HEADS
    - CENTRALIZERS
    - ANCHOR CORROSION PROTECTION SYSTEM
    - GROUTING PROCEDURE
    - TIMBER LAGGING DESIGN
    - JACKING ASSEMBLY - HYDRAULIC JACK AND PUMP, STRESSING ANCHORAGE, PRESSURE GAGES/LOAD CELLS, DIALS TO MEASURE MOVEMENT AND JACK CHAIR
19. FOR CONTRACTOR DESIGNED PERMANENT ANCHOR WALL - THE CONTRACTOR IS RESPONSIBLE FOR THE FULL DESIGN, DETAILING, FABRICATION AND CONSTRUCTION OF THE PERMANENTLY ANCHORED WALL IN ACCORDANCE WITH THE SPECIAL PROVISION PERMANENT ANCHORED WALL.
20. PROVIDE PERFORMANCE, PROOF, AND CREEP TESTING OF ANCHORS AND INDICATE TESTING REQUIREMENTS AND RESULTS IN ACCORDANCE WITH THE SPECIAL PROVISIONS. FOR ANCHORS THAT FAIL TEST REQUIREMENTS, REPLACE ANCHORS OR MODIFY THE STRUCTURE TO MEET ALL DESIGN CODES AND REQUIREMENTS IN ACCORDANCE WITH THE SPECIAL PROVISION.
21. SUBMIT FINAL DESIGN CALCULATIONS AND DESIGN DETAILS IN ACCORDANCE WITH THE SPECIAL PROVISIONS.
22. CHAMFER EXPOSED CONCRETE EDGES 1" X 1" EXCEPT AS NOTED.
23. GALVANIZE MATERIAL IN ACCORDANCE WITH SECTION 1105.02(s)1 OF PUB. 408. REPAIR GALVANIZED SURFACES DAMAGED DURING CONSTRUCTION IN ACCORDANCE WITH SECTION 1105.02(s)2 OF PUB. 408.
24. REPAIR EPOXY COATED SURFACES DAMAGED DURING CONSTRUCTION IN ACCORDANCE WITH SECTION 1092.3(e) OF PUB. 408.
25. ENSURE INTIMATE CONTACT BETWEEN EXCAVATION FACE AND THE BACK FACE OF TIMBER LAGGING PRIOR TO STRESSING ANCHOR. [FOR TOP DOWN INSTALLATION]
26. GRIND AND FINISH ANCHOR OPENINGS IN DOUBLE PILES TO A SMOOTH CONDITION.
27. APPLY SHEAR STUDS TO WEBS OF DRIVEN PILES AFTER DRIVING PILES TO REFUSAL. WELD SHEAR STUDS IN ACCORDANCE WITH AASHTO/AWS D1.5 SECTIONS 7.5.5 AND 7.6.
28. WELDING SPECIFICATIONS: ANSI/AASHTO/AWS/D1.5 BRIDGE WELDING CODE AND IN ACCORDANCE WITH SECTION 1105.03(m) OF PUB. 408 AND THE SPECIAL PROVISIONS. USE QUALIFIED WELDERS IN ACCORDANCE WITH AWS D1.5 SECTION 5 PART B. FOLLOW D1.1 FOR TUBULAR (API OR ASTM A53) MATERIAL.
29. FIELD WELDING OF STEEL: USE THE SHIELDED METAL ARC PROCESS AND LOW HYDROGEN ELECTRODES WHICH ARE COMPATIBLE WITH THE BASE METAL AS SPECIFIED, AND IN ACCORDANCE WITH AN APPROVED WELD PROCEDURE SPECIFICATION.
30. DO NOT WELD WHEN SURFACES TO BE WELDED ARE MOIST OR EXPOSED TO RAIN, SNOW OR WIND, OR WHEN WELDERS ARE EXPOSED TO INCLEMENT CONDITIONS THAT WILL ADVERSELY AFFECT THE QUALITY OF THE WORK.
31. DO NOT WELD OR BURN WHEN THE TEMPERATURE IS BELOW 0-DEGREES F. PREHEAT AND MAINTAIN THE TEMPERATURE OF THE METAL TO AT LEAST 70-DEGREES F WHEN THE TEMPERATURE OF THE METAL IS BETWEEN 0-DEGREES AND 30-DEGREES F DURING WELDING OR BURNING. EXTEND THE AREA TO BE HEATED 3 INCHES BEYOND THE WELD IN ALL DIRECTIONS.
32. REMOVE ANY MOISTURE PRESENT AT POINT OF WELD BY APPLICATION OF HEAT. PROVIDE WINDBREAKS FOR PROTECTION FROM DIRECT WIND.
33. THOROUGHLY CLEAN ALL PORTIONS OF NEW SURFACES TO RECEIVE WELDS OF ALL FOREIGN MATTER, INCLUDING PAINT FILM, FOR A DISTANCE OF 2" FROM EACH SIDE OF THE OUTSIDE LINES OF WELD PRIOR TO PLACING THE WELD.
34. TEST INDICATED WELDS USING NON-DESTRUCTIVE METHODS IN ACCORDANCE WITH AASHTO AWS D1.5 2002 BRIDGE WELDING CODE, SECTION 6.7.
35. LAGGING MAY BE PLACED INSIDE THE REAR FLANGE, IF BLOCKED, OR INSIDE THE FRONT FLANGE.

NOTES TO DESIGNER

1. APPLICABILITY OF THIS STANDARD DRAWING:
    - THIS STANDARD APPLIES TO PERMANENT ANCHORED WALLS WITH DISCRETE VERTICAL ELEMENTS.
    - THIS STANDARD APPLIES TO ANCHORS BONDED IN ROCK. ANCHORS BONDED IN SOIL ARE PERMITTED WITH APPROVAL OF THE CHIEF BRIDGE ENGINEER.
    - THIS STANDARD APPLIES TO DISCRETE VERTICAL ELEMENTS WITH FOUNDATIONS ON OR INTO ROCK. DESCRETE VERTICAL ELEMENTS TERMINATING IN SOIL ARE PERMITTED WITH APPROVAL OF THE CHIEF BRIDGE ENGINEER.
2. SOLDIER PILES MAY BE DESIGNED USING H-PILES, WIDE FLANGE BEAMS OR CONCRETE DRILLED SHAFTS. ANCHORED WALLS MAY BE DESIGNED USING STEEL SHEET PILES.
3. PROVIDE REINFORCEMENT BAR DEVELOPMENT LENGTHS AND SPLICE LENGTHS IN ACCORDANCE WITH AASHTO LFRD BRIDGE DESIGN SPECIFICATIONS, AS SUPPLEMENTED BY DESIGN MANUAL PART 4.
4. PROVIDE NOT LESS THAN TWO LAYERS OF CORROSIVE PROTECTION FOR THE TENDONS BY ENCAPSULATION IN A GROUT-FILLED CORRUGATED PLASTIC OR DEFORMED STEEL SHEATH.
5. APPLY PROTECTIVE COATINGS FOR REINFORCED CONCRETE SURFACES IN ACCORDANCE WITH DESIGN MANUAL PART 4 WHERE WALL IS EXPOSED TO SALT SPRAY. SEE SECTION 1019 OF PUBLICATION 408.
6. PROVIDE EPOXY COATED REINFORCING BARS IN ACCORDANCE WITH DESIGN MANUAL 4 CONSIDERING POTENTIALLY CORROSIVE ENVIRONMENTS.
7. PROVIDE CONCRETE CAP OVER ANCHOR PLACED BELOW FINAL GRADE.
8. STEEL PILES MAY BE GRADE 50 STEEL; HOWEVER, BASE POINT BEARING CAPACITY ON NOT MORE THAN 36 ksi. DESIGNER MAY USE 50 KSI WHEN EVALUATING COMBINED BENDING AND AXIAL LOADS FOR GRADE 50 PILES.
9. SPECIFY PROTECTIVE COATINGS FOR STEEL COMPONENTS IN ACCORDANCE WITH DESIGN MANUAL PART 4 CONSIDERING POTENTIALLY CORROSIVE ENVIRONMENTS. THE FOLLOWING PROVIDES GENERAL GUIDANCE:
    - STEEL ANCHORAGE FULLY ENCASED IN CONCRETE - NO ADDITIONAL PROTECTION REQ'D.
    - STEEL EMBEDDED IN ROCK SOCKET / SHAFT SECTION AND ENCASED IN CONCRETE, INCLUDING LEAN CONCRETE FILL - ZINC PRIMER, GALVANIZING, OR EPOXY COATING
    - EXPOSED STEEL OR STEEL ENCASED IN LEAN CONCRETE - THREE-COAT PAINT SYSTEM, GALVANIZING, OR EPOXY COATING
    - DRIVEN PILES - DEDUCT 1/16" SACRIFICIAL STEEL AROUND FULL PERIMETER
    - PILES IN CONTACT WITH BACKFILL - THREE-COAT PAINT SYSTEM, GALVANIZING, OR EPOXY COATING
    - WHERE PROTECTIVE COATING IS REQUIRED, GALVANIZING IS PREFERRED.

    PROVIDE THREE-COAT PAINT SYSTEM IN ACCORDANCE WITH SECTION 1060 OF PUB.408. PROVIDE GALVANIZING IN ACCORDANCE WITH SECTION 1105.02(s) OF PUB.408. PROVIDE EPOXY COATING IN ACCORDANCE WITH SECTION 1092 OF PUB.408.
10. DO NOT USE STEEL BARS FOR ANCHORS WITH BOTTOM UP INSTALLATION DUE TO POTENTIAL DAMAGE FROM SETTLEMENT, UNLESS APPROVED BY THE CHIEF BRIDGE ENGINEER.
11. INDICATE ON THE DESIGN DRAWINGS "ANCHOR DESIGN LOAD". USE SER-I LIMIT STATE TO DETERMINE THE ANCHOR DESIGN LOAD. THE PERFORMANCE TEST LOAD IS DEFINED AS A MULTIPLIER (1.33) TIMES THE ANCHOR DESIGN LOAD. ANCHOR DESIGN LOAD WILL NOT EXCEED 0.6 GUARANTEED ULTIMATE TENSILE STRENGTH (GUTS) AND ANCHOR PROOF TEST LOAD WILL NOT EXCEED 0.8 GUTS.
12. LOCK-OFF LOAD SHALL NOT BE LESS THAN 50% OF THE ULTIMATE TENSILE STRENGTH OF THE STRANDS. IF ADDITIONAL STRANDS ARE REQUIRED FOR REDUNDANCY, A MINIMUM OF 40% OF THE ULTIMATE TENSILE STRENGTH OF THE STRANDS MAY BE USED. IN ANY CASE, THE STRANDS AND ANCHOR ASSEMBLY (INCLUDING WEDGES) ARE CLEANED OF ANY CONTAMINANTS IMMEDIATELY BEFORE LOCK-OFF, AND THE STRANDS AND WEDGES ARE CLEANED, GREASED, AND CAPPED WITHIN 48 HOURS AFTER LOCK-OFF.
13. FOR BOTTOM UP INSTALLATION, BACKFILL UP TO A MINIMUM HEIGHT OF 4' ABOVE THE ANCHOR LOCATION, OR AS REQUIRED TO DEVELOP SUFFICIENT PASSIVE PRESSURE PRIOR, TO ANCHOR INSTALLATION AND STRESSING.
14. REQUIRE ADDITIONAL CLEAR CONCRETE COVER TO REINFORCEMENT BARS TO ACCOUNT FOR IMPRESSIONS OF AESTHETIC SURFACE TREATMENT.
15. AFTER PLACEMENT OF CEMENT CONCRETE IN CONCRETE EMBEDMENT, PLACE LEAN CEMENT CONCRETE OR FLOWABLE FILL IN REMAINDER OF DRILLED HOLE UP TO TOP OF GROUND. REMOVE LEAN CEMENT CONCRETE OR FLOWABLE FILL DURING EXCAVATION TO PLACE TIMBER LAGGING. [FOR TOP DOWN INSTALLATION] FLOWABLE FILL PER PUB 408 SECTION 220.2, TYPE A OR B.
16. DESIGN FOR THE PRESENCE OF WATER BEHIND THE WALL AS REQUIRED BY SITE CONDITIONS AND DRAINAGE. DRAINS ARE TO OUTLET AT MAXIMUM INTERVALS OF 100'.
17. IF CAST-IN-PLACE CAP BEAM IS USED, CONSTRUCT THE CAP BEAM AFTER LOCK OFF OF ANCHORS.
18. IF PRECAST LAGGING IS USED, LOCK OFF ANCHORS PRIOR TO INSTALLATION.
19. UNLESS SPECIFICALLY REQUIRED TO ADDRESS DRAINAGE NEEDS FOR SPECIFIC SITE CONDITIONS AVOID PLACEMENT OF INLETS, UTILITY HOLES, AND DRAINAGE FACILITIES IN THE BACKFILL OF THE ANCHORED WALL TO AVOID DAMAGE TO ANCHORS DUE TO INSTALLATION OR MAINTENANCE ACTIVITIES.
20. EVALUATE ACCESSIBILITY OF DRILLING RIG TO PILE LOCATIONS. ENSURE THAT A RELATIVELY LEVEL AREA CAN BE ACCOMMODATED ADJACENT TO THE PILE LOCATIONS FOR DRILLING OPERATIONS.
21. USE REDUCED SECTION PROPERTIES AT OPENINGS FOR ANCHOR PENETRATIONS IN THE PILE SECTIONS AT ANCHOR LOCATIONS.
22. SIZE TIMBER LAGGING IN ACCORDANCE WITH CONSTRUCTION HANDBOOK FOR BRIDGE TEMPORARY WORKS BY AASHTO.
23. DESIGN CONCRETE AND REINFORCEMENT FOR A TEST LOAD OF 125% TO 150% OF UNFACTORED LOAD PER AASHTO ARTICLE 11.9.8.1 IN ADDITION TO THE STRENGTH CONDITION. FOR SINGLE PILE W/C.I.P. WALL AND COLUMN ALTERNATIVE, DESIGNER MUST EVALUATE BOTH A MINIMUM AND MAXIMUM STRUCTURAL BACKFILL DENSITY TO LIMIT WALL DEFLECTION TOWARDS BACKFILL DUE TO ANCHOR STRESSING OPERATIONS. INCLUDE STRUCTURAL BACKFILL DENSITY RANGE ON CONTRACT DRAWINGS AND SPECIAL PROVISIONS.

GENERAL ANCHOR WALL DESIGN METHODOLOGY

1. ESTABLISH PROJECT REQUIREMENTS INCLUDING ALL GEOMETRY, EXTERNAL LOADING CONDITIONS (TEMPORARY AND/OR PERMANENT, ETC.), CONSTRUCTION CONSTRAINTS AND PERFORMANCE CRITERIA IN ACCORDANCE WITH THE SPECIAL PROVISIONS.
2. EVALUATE SITE SUBSURFACE CONDITIONS AND RELEVANT PROPERTIES OF IN SITU SOIL AND ROCK.
3. ESTABLISH ANCHOR INCLINATION ANGLES. INCLINE ANCHORS TO MINIMIZE ANCHOR LENGTH, TO AVOID UTILITIES AND OTHER UNDERGROUND OBSTRUCTIONS, TO STAY WITHIN RIGHT-OF-WAY, AND TO OPTIMIZE ANCHOR FORCE.
4. DETERMINE EARTH PRESSURE DISTRIBUTIONS, INCLUDING SURCHARGES, FOR WALL WITH APPROPRIATE LOAD FACTORS AND LIMIT STATES AS PER DESIGN MANUAL PART 4 AND THIS STANDARD. RESISTING PASSIVE PRESSURE BEGINS AT BOTTOM OF WALL.
5. EVALUATE GLOBAL STABILITY FOR ANCHORED SYSTEM USING LIMIT EQUILIBRIUM ANALYSES. REVISE ANCHOR GEOMETRY IF NECESSARY.
6. FOR ANALYSIS AND DESIGN OF WALL ELEMENTS, EMBEDMENT DEPTHS AND ANCHOR FORCES, USE EITHER AASHTO METHOD OR TWO DIMENSIONAL BEAM FINITE ELEMENT COMPUTER MODEL.
7. DESIGN WALL ELEMENTS FOR THE RESULTING FORCES (MOMENT, SHEAR AND AXIAL) AND DEFLECTION. WITH THE EXCEPTION OF SINGLE PILE DESIGN W/C.I.P. WALL AND COLUMN ALTERNATIVE, LIMIT DEFLECTION TO A MAXIMUM OF 1" UNLESS THE SENSITIVITY OF ADJACENT STRUCTURES OR FACILITIES REQUIRES A LESSER LIMIT. WALL DEFLECTION IN EXCESS OF 1" IS PERMITTED WITH APPROVAL OF THE CHEIF BRIDGE ENGINEER. INCLUDE MONITORING PROGRAM TO VERIFY THAT DEFLECTION OF ADJACENT STRUCTURES DOES NOT EXCEED 1" LIMIT. DEFLECTION TOWARDS THE BACKFILL FOR SINGLE PILE DESIGN W/C.I.P. WALL AND COLUMN DESIGNS SHALL BE PREVENTED IN ORDER TO AVOID OR REDUCE CRACKING ON EXPOSED FACE.
8. DETERMINE REQUIRED PILE EMBEDMENT OR CONCRETE EMBEDMENT DEPTH FOR SHAFT AND ROCK SOCKET.
9. ESTIMATE NUMBER OF STRANDS OR BAR DIAMETER REQUIRED TO RESIST ANCHOR FORCES. ESTIMATE ANCHOR BOND LENGTH AND PULLOUT CAPACITY. FINAL DETERMINATION OF THE SIZE AND NUMBER OF STRANDS OR BAR DIAMETER, ANCHOR BOND DIAMETER, GROUTING METHOD, GROUTING PRESSURE AND ANCHOR BOND LENGTH IS THE RESPONSIBILITY OF THE ANCHOR SPECIALTY CONTRACTOR.
10. CHECK AXIAL LOAD RESISTANCE OF THE CONCRETE EMBEDMENT OR DRIVEN PILE.
11. CHECK TEMPORARY CONDITIONS (CONSTRUCTION STAGING) FOR THE STR-I LIMIT STATE.
12. CHECK DEFLECTION FOR THE WORST CASE SER-I LIMIT STATES.
13. CHECK WALL COMPONENTS SUCH AS BEARING PLATE ASSEMBLY, CORBEL, AND WALER.
14. CHECK WALL REDUNDANCY AS PER WALL REDUNDANCY PROCEDURE ON SHEET 2.
15. CHECK THAT SUFFICIENT PASSIVE PRESSURE CAN BE DEVELOPED BEHIND THE WALL AT THE UPPERMOST ANCHOR TO RESIST THE ANCHOR TEST LOAD.

Ref. PennDOT Pub. 218M Std. Detail BD-626M.

BD-627M	MOMENT SLABS
BC-736M	REINFORCEMENT BAR FABRICATION DETAILS
BC-751M	BRIDGE DRAINAGE
BC-757M	STEEL PILE TIP REINFORCEMENT & SPLICES
RC-12M	BACKFILL AT STRUCTURES
RC-54M	BARRIER PLACEMENT AT OBSTRUCTIONS

REFERENCE DRAWINGS

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION
BUREAU OF PROJECT DELIVERY

STANDARD
ANCHORED WALLS
NOTES

RECOMMENDED AUG.30, 2019 — ACTING CHIEF BRIDGE ENGINEER

RECOMMENDED AUG.30, 2019 — ACT. DIR., BUR. OF PROJECT DELIVERY

SHEET 1 OF 9

BD-626M

Figure 12-22 - Typical Detail, Anchored Walls (Sheet 1 of 9)

Commentary Notes.

1. These notes provide high-level guidance for the design of cantilevered and anchored soldier pile walls. These walls can serve as both temporary and permanent retaining structures (e.g., less than and more than 3 years of service life). These notes are not meant to be all-encompassing, but rather to point out some key considerations to detail and construct this type of retaining structure.
2. Provide materials and construct in accordance with PennDOT Pub. 408 and Pub. 218M Std Dwg. 626M.
3. Consider the required installation tolerance (e.g., within +/- 1-inch of plan location, 1/4-inch per foot of plumb for soldier pile, +/- 2 degrees anchor inclination both laterally and vertically, and +/- 2' horizontal clear distance between adjacent soldier pile flanges over the entire exposed height of soldier pile),
4. Assess the project needs and site constraints including, but limited to:
    a. Avoidance features (e.g., utilities, railroads lateral deflection restrictions in proximity to sensitive structures),
    b. Critical load cases (e.g., landslide load due to seasonal groundwater fluctuation and surface water infiltration at tension cracks, and change in surcharge load),
    c. Rate of loading (e.g., impact on available shear strength, peak shear strength versus residual shear strength versus undrained shear strength),
    d. Anticipated construction sequence (e.g., top-down versus bottom-up),
    e. Required corrosion protection,
    f. Long-term need to sustain minimum anchor prestress load, which may influence whether steel bars or 7-wide low relaxation strand anchors are more appropriate,
    g. Need for temporary lagging for temporary excavation support,
    h. Constructability considerations (e.g., access and required clearance for installation including tail swing and overhead),
    i. Sloping the ground surface to mitigate against ponding and provide positive drainage away from the retaining wall, and
    j. Miscellaneous items like management of wall drainage, and need for the permanent casing, utility support/protection, and architectural treatment (if required).
5. Complete lateral capacity analyses, equilibrium analysis, and overall (global) stability analyses (shallow- and deep-seated) to determine the required:
    a. Pile size, reinforcement, spacing, socket diameter, and embedment length,
    b. Post-tensioned anchor bond stress, design load, lock-off load, test load, anchor performance, proof, and creep test procedures, acceptance criteria, and
    c. Wall facing (e.g., precast concrete panels, cast-in-place concrete wall facing, timber lagging).

WALL REDUNDANCY

1. ANCHOR LOADS: USE EXT III LIMIT STATE TO DETERMINE ANCHOR REDUNDANCY LOADS ASSUMING ONE ANCHOR FAILS. USE ANCHOR RESISTANCE EQUAL TO ANCHOR PROOF TEST LOAD, 0.8 GUTS.
2. WALL ELEMENTS AND FOUNDATION FOR REDUNDANCY: DESIGN WALL ELEMENTS AND FOUNDATION FOR EXT-III.
3. DEFLECTION CHECKS ARE NOT REQUIRED FOR REDUNDANCY.
4. DESIGN ANCHOR WALL TO PROTECT FROM CATASTROPHIC FAILURE DUE TO THE FAILURE OF ANY ONE ANCHOR AS FOLLOWS:
   a.) WALL WITH CAST-IN-PLACE FACING: DESIGN THE FACING TO DISTRIBUTE LOAD TO ADJACENT SOLDIER PILES AND ANCHORS IN THE EVENT ANY ONE ANCHOR FAILS.
   b.) WALL WITH PRECAST LAGGING: DESIGN A POSITIVE MEANS OF REDUNDANCY IN THE EVENT ANY ONE ANCHOR FAILS USING ONE OR MORE, BUT NOT LIMITED TO, THE FOLLOWING METHODS:
   - PROVIDE CONTINUOUS REINFORCED CAST-IN-PLACE CONCRETE CAP BEAM
   - PROVIDE HORIZONTAL STEEL TIE RODS BETWEEN PILES
   - PROVIDE ADDITIONAL ANCHORS
   - DESIGN ADJACENT ANCHORS TO RESIST ADDITIONAL LOAD REDISTRIBUTED FROM THE FAILED ANCHOR

TYPICAL LATENT ANCHOR CONSTRUCTION PROCEDURE

LATENT ANCHORS: THE USE OF LATENT ANCHORS MAY BE CONSIDERED FOR STRENGTHENING OR LIMITING DISPLACEMENT OF EXISTING WALLS, WITH OR WITHOUT A NEW CONCRETE BLANKET WALL. THE CASING IS DESIGNED AS A STRUCTURAL MEMBER, TRANSFERING THE ANCHOR LOAD TO THE CASING EMBEDMENT (BOND) LENGTH. THE ANCHOR IS NOT ENGAGED BY THE WALL UNTIL THERE IS A TENDENCY FOR WALL MOVEMENT.

1. DRILL THROUGH WALL, FILL AND INTO THE ROCK TO THE REQUIRED STEEL CASING EMBEDMENT LENGTH. HOLE MUST BE OF SUFFICIENT DIAMETER TO ACCEPT ALL COMPONENTS AND PROVIDE SPECIFIED COVER.
2. FILL HOLE WITH SUFFICIENT GROUT AND PLUNGE CASING TO THE FULL DEPTH OF EMBEDMENT LENGTH TO ENSURE FULL GROUTING OF ANNULAR SPACE BETWEEN CASING AND ROCK.
3. AFTER SUFFICIENT CURE OF THE GROUT, WITH A MINIMUM COMPRESSIVE STRENGTH OF 1000 PSI AND A MINIMUM OF 16 HOURS, DRILL ANCHOR BOND LENGTH, INSTALL ANCHOR, AND PRESSURE GROUT THE BOND LENGTH.
4. INSTALL HOT DIPPED GALVANIZED BEARING PLATE, FIELD WELD THE PLATE TO THE STEEL CASING, AND FIELD GALVANIZE. CLEAN AND FIELD GALVANIZE THE EXPOSED END OF CASING.
5. PRIOR TO TESTING THE ANCHORS, WEDGE THE STEEL CASING IN THE HOLE THROUGH THE WALL TO FIRMLY SECURE IT. AFTER ANCHOR BOND GROUT HAS CURED, PERFORM LOAD TESTS AND PROOF TESTS ON THE ANCHORS AND LOCK OFF LOAD TO THE DESIGN LOAD IN ACCORDANCE WITH THE SPECIAL PROVISIONS. JACK AGAINST BEARING PLATE. DO NOT APPLY JACKING LOADS TO THE WALL.
6. GROUT THE STEEL CASING UNTIL GROUT EMERGES FROM END OF CASING. GROUT ANNULAR SPACE BETWEEN CASING AND THE WALL.
7. CONSTRUCT ANCHOR CORBELS. ENSURE FULL CONSOLIDATION OF CONCRETE BEHIND BEARING PLATE.
8. CUT EXCESS TENDON LENGTH. INSTALL GREASE FILLED GALVANIZED CAP, AND CAULK JOINT BETWEEN BEARING PLATE AND CONCRETE ON ALL SIDES WITH APPROVED ELASTOMERIC CAULKING COMPOUND.
9. OTHER METHODS OF CONSTRUCTION MAY BE CONSIDERED WITH APPROVAL OF THE CHIEF BRIDGE ENGINEER.

LIMIT STATES AND LOADING

1. USE THE FOLLOWING LOAD FACTORS:

	LOAD FACTORS FOR ANCHOR WALL DESIGNS					
	LOAD TYPES					
LIMIT STATE	DC	DD	DW	EH	LS	LL
STR-I ••	1.25	1.25	1.50	1.35	1.75	1.75
EXT-III •	1.25	1.25	1.50	1.05	1.15	1.15
SER-I	1.00	1.00	1.00	1.00	1.00	1.00

• - FOR REDUNDANCY ANALYSIS

•• - USE A LOAD FACTOR FOR EH = 1.5 FOR DIFFICULT GEOLOGY, SUCH AS AREAS PRONE TO LANDSLIDES

DC - SELF WEIGHT OF WALL COMPONENTS AND VERTICAL COMPONENT OF ANCHOR LOAD

DD - DOWNDRAG ACTING UPON DRILLED CAISSON OR PILE PER DESIGN MANUAL PART 4.

DW - WEIGHT OF ATTACHED UTILITIES AND WEIGHT OF WALL-SUPPORTED MOMENT SLAB

EH - HORIZONTAL EARTH PRESSURE PER DESIGN MANUAL PART 4.

LS - LIVE LOAD SURCHARGE PER DESIGN MANUAL PART 4.

LL - LIVE LOAD TRANSMITTED DIRECTLY TO STRUCTURE FROM WALL-SUPPORTED MOMENT SLAB (DOES NOT INCLUDE IMPACT)

NOTE 1: EH LOADS MAY HAVE VERTICAL AND HORIZONTAL COMPONENTS ACTING SIMULTANEOUSLY.

2. LIMIT STATE DESCRIPTIONS:

STR-I BASIC LOAD COMBINATION FOR DESIGN OF ANCHOR WALL ELEMENTS AND FOUNDATION ELEMENTS.

EXT-III REDUNDANCY LOAD COMBINATION FOR DESIGN OF ANCHOR WALL ELEMENTS AND FOUNDATION (USE THIS LIMIT STATE WITH OR WITHOUT GLOBAL STABILITY LOAD CONSISTENT WITH THE GOVERNING STRENGTH LIMIT STATE)

SER-I BASIC LOAD COMBINATION FOR DEFLECTION CHECK AND ANCHOR DESIGN

WALL ELEMENTS CONSIST OF SOLDIER PILE, CONCRETE EMBEDMENT, LAGGING, WALER, WALL FACING, CORBEL, AND BEARING PLATE ASSEMBLY.

FOUNDATION ELEMENTS CONSIST OF SIDE RESISTANCE AND BEARING STRESS OF CONCRETE EMBEDMENT AND PILES.

ANCHOR DESIGN CONSISTS OF STRAND SELECTION AND BOND ZONE.

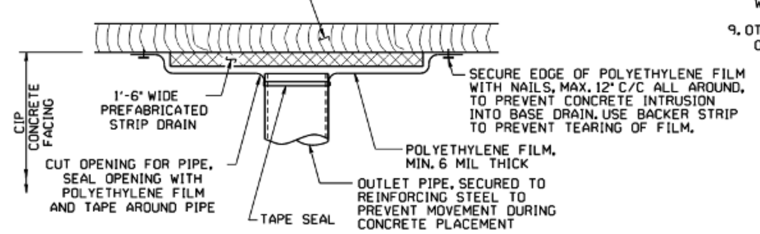
DEFLECTION CHECK IS FOR HORIZONTAL WALL DISPLACEMENTS.

3. ANCHOR RESISTANCE:

ANCHOR LOAD RESISTANCE	
SER-I	0.6 GUTS
STR-I	0.75 GUTS
EXT-III	0.8 GUTS

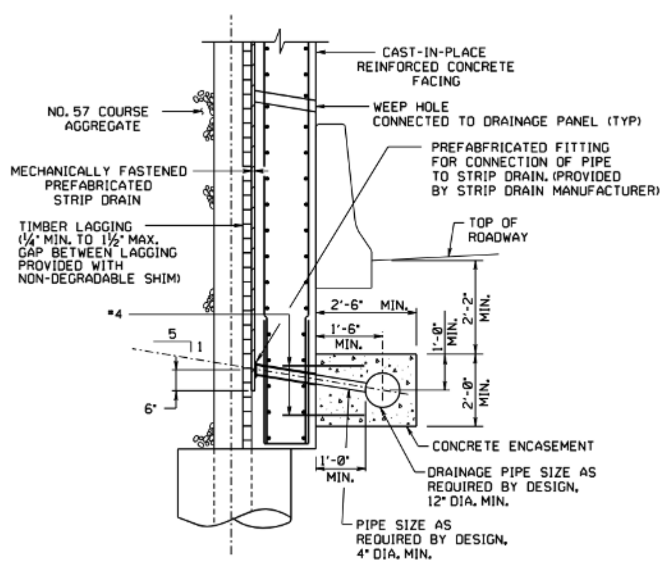
GUTS - GUARANTEED ULTIMATE TENSILE STRENGTH

RESISTANCE FACTORS: USE RESISTANCE FACTORS FOR WALL ELEMENTS AND FOUNDATION ELEMENTS IN ACCORDANCE WITH AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS AS SUPPLEMENTED BY DESIGN MANUAL 4.



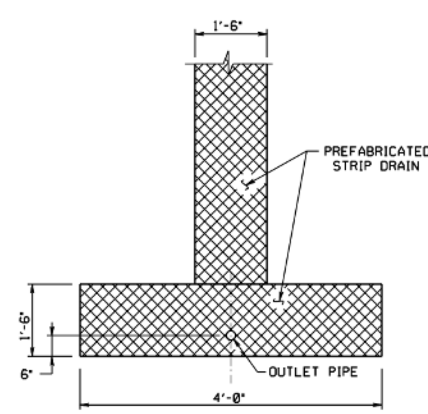
ALTERNATE FITTING FOR CONNECTION OF PIPE TO STRIP DRAIN

WITH C.I.P. WALL SHOWN, WITH PRECAST LAGGING SIMILAR



SECTION AT DRAINAGE PANEL

WITH C.I.P. WALL SHOWN, WITH PRECAST LAGGING SIMILAR



ELEVATION AT DRAINAGE PANEL

Ref. PennDOT Pub. 218M Std. Detail BD-626M.

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION
BUREAU OF PROJECT DELIVERY

STANDARD

ANCHORED WALLS
NOTES AND DRAINAGE DETAILS

RECOMMENDED AUG.30, 2019	RECOMMENDED AUG.30, 2019	SHEET 2 OF 9
ACTING CHIEF BRIDGE ENGINEER	ACT. DIR., BUR. OF PROJECT DELIVERY	BD-626M

Figure 12-23 – Typical Detail, Anchored Walls (Sheet 2 of 9)

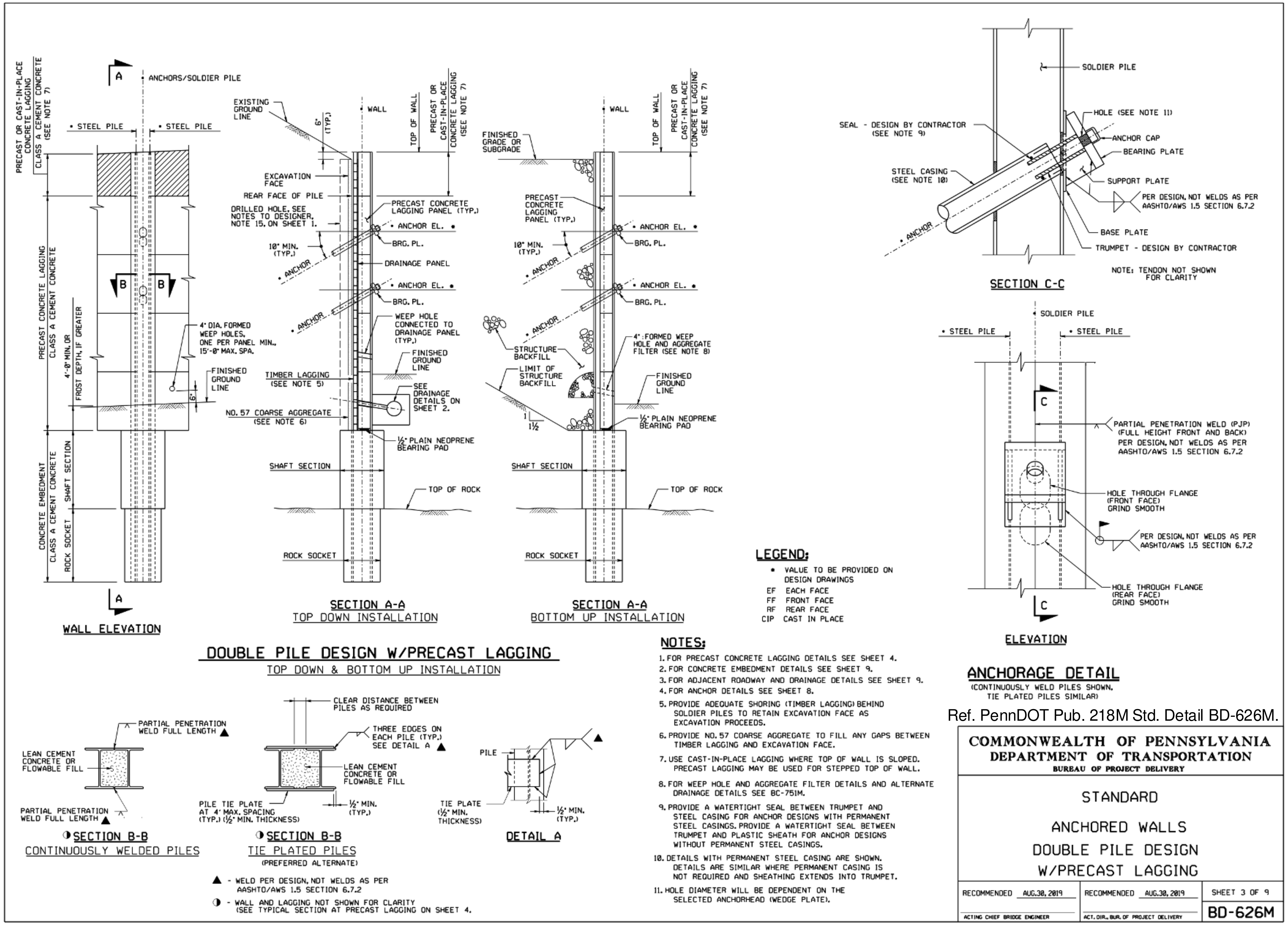


Figure 12-24 - Typical Detail, Anchored Walls (Sheet 3 of 9)

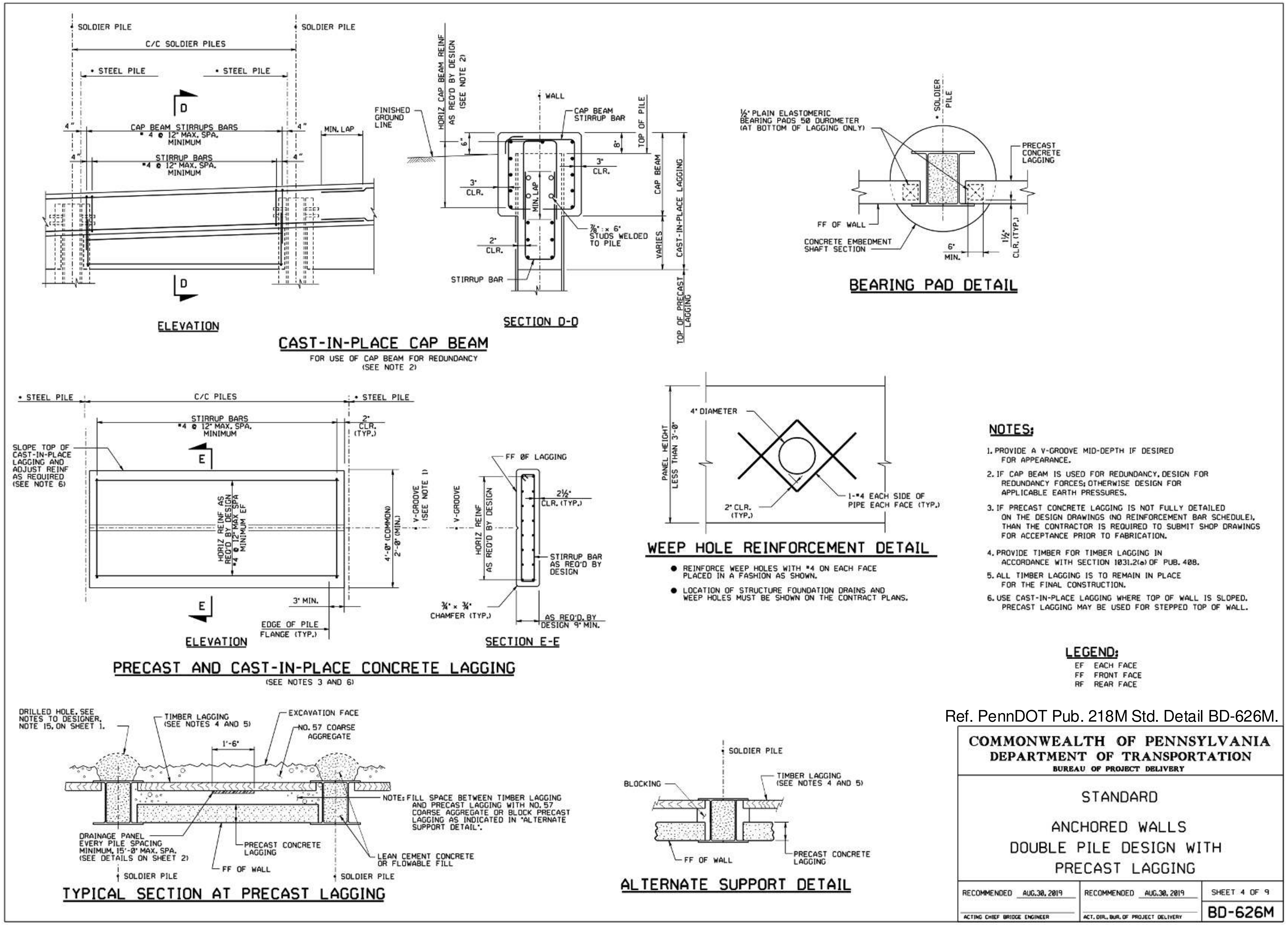


Figure 12-25 - Typical Detail, Anchored Walls (Sheet 4 of 9)

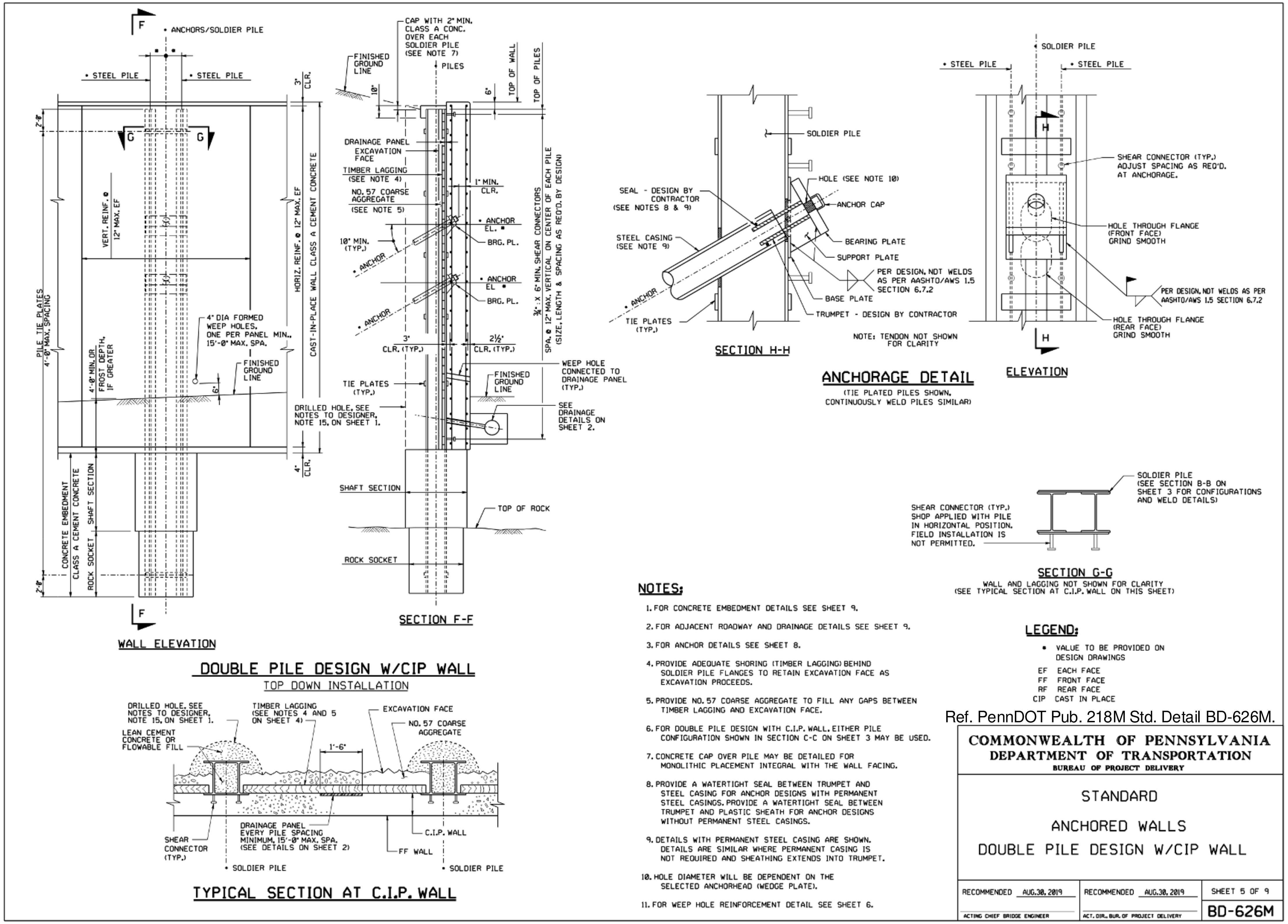
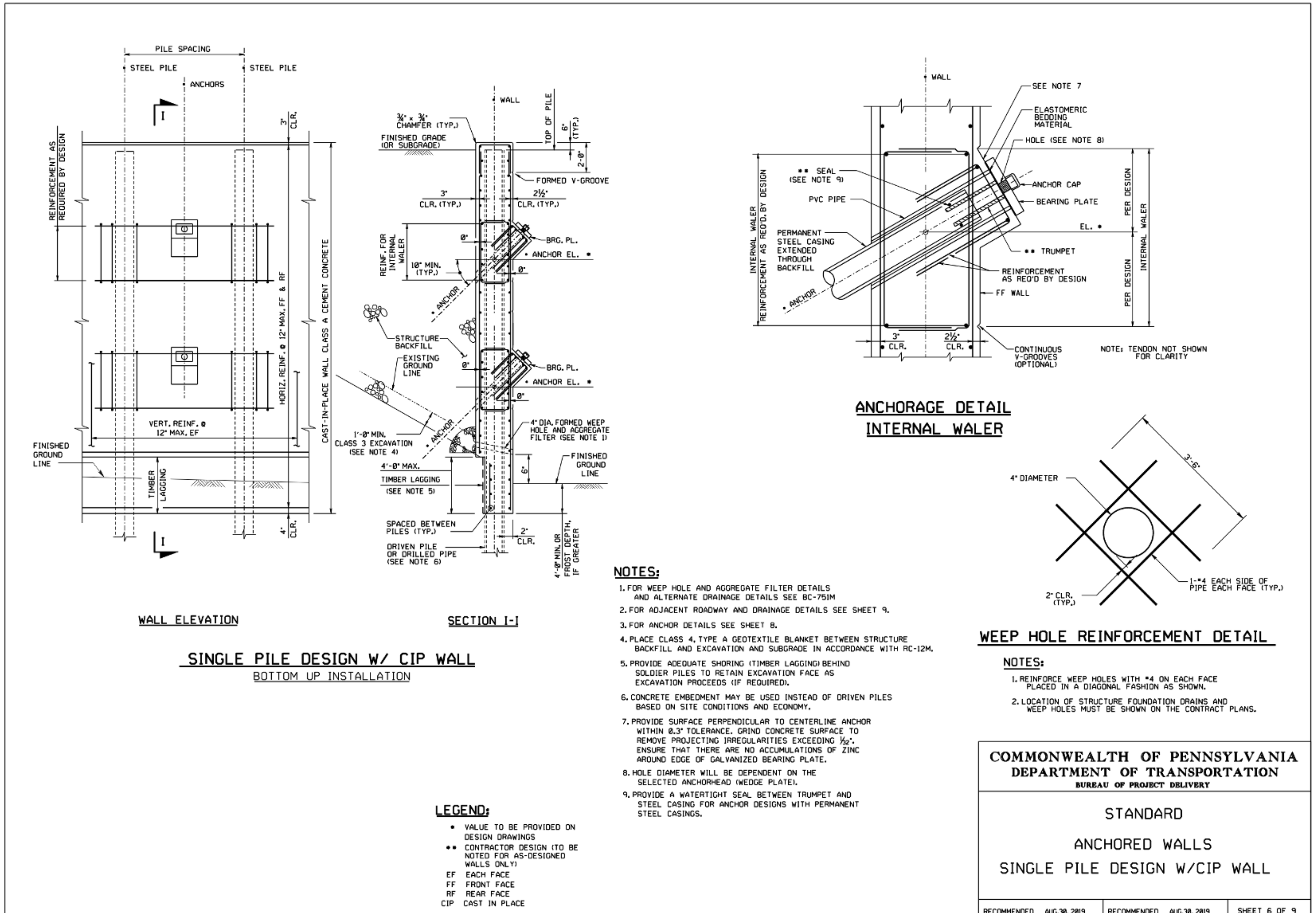


Figure 12-26 - Typical Detail, Anchored Walls (Sheet 5 of 9)



Ref. PennDOT Pub. 218M Std. Detail BD-626M.

Figure 12-27 - Typical Detail, Anchored Walls (Sheet 6 of 9)

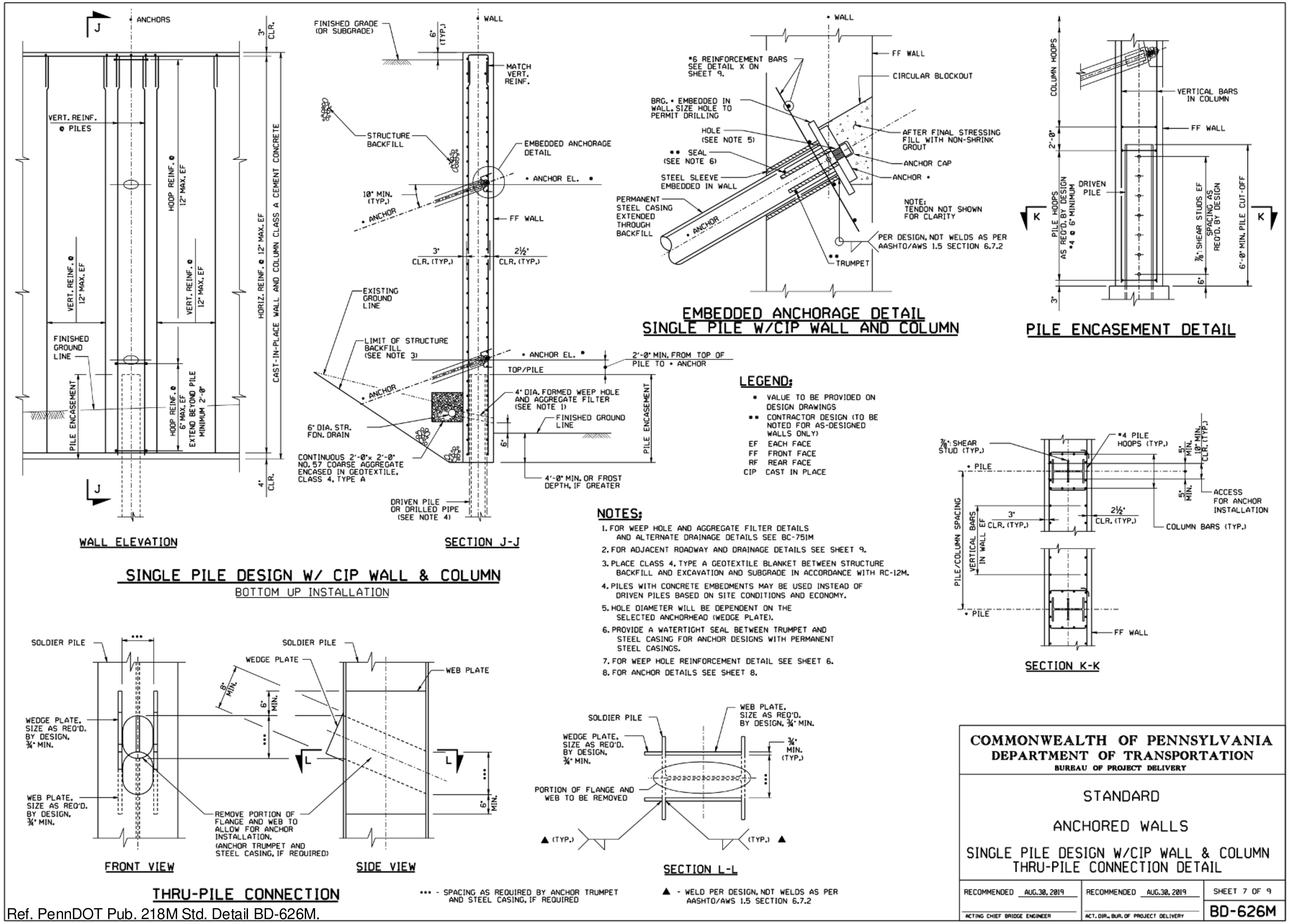


Figure 12-28 - Typical Detail, Anchored Walls (Sheet 7 of 9)

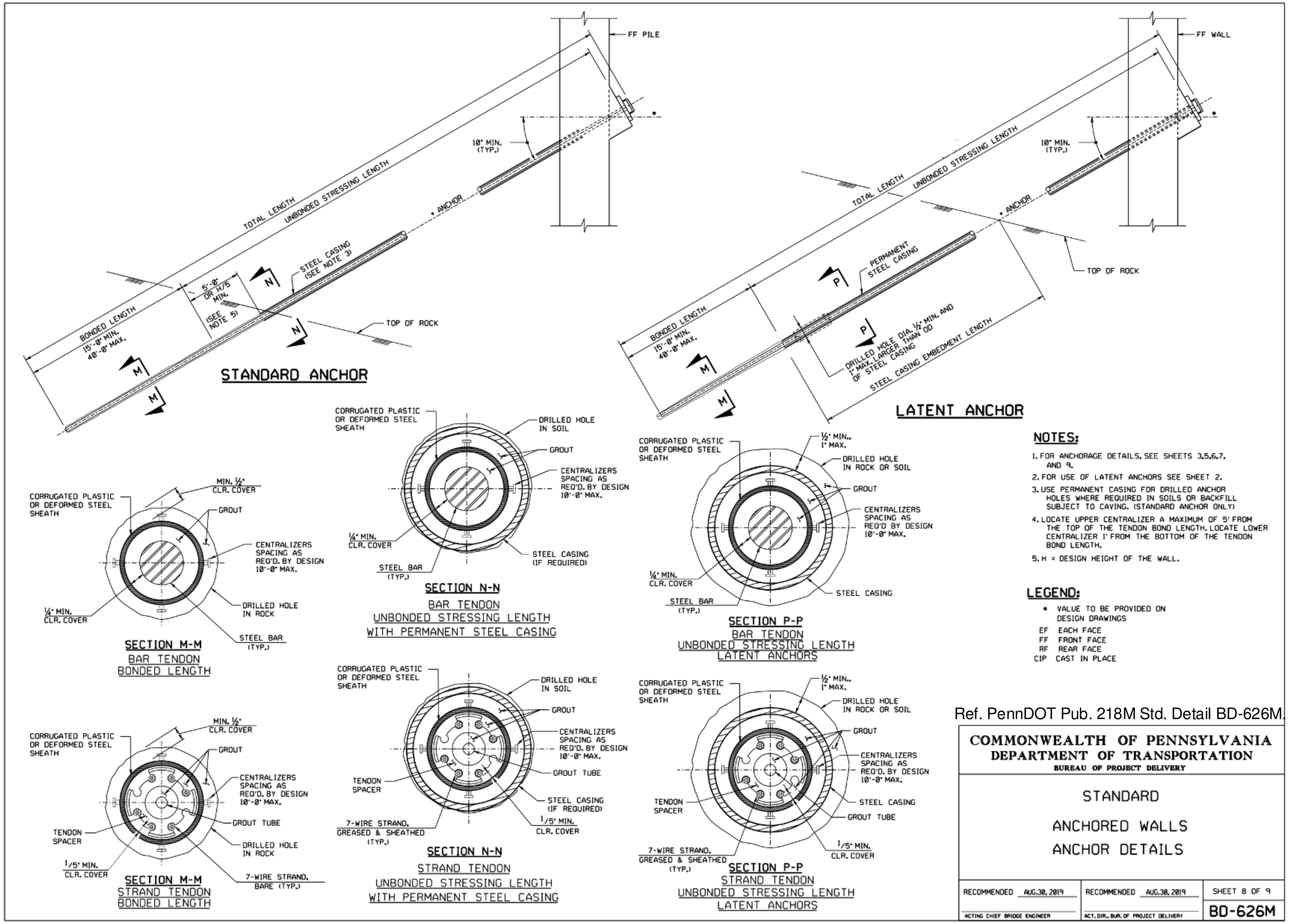


Figure 12-29 - Typical Detail, Anchored Walls (Sheet 8 of 9)

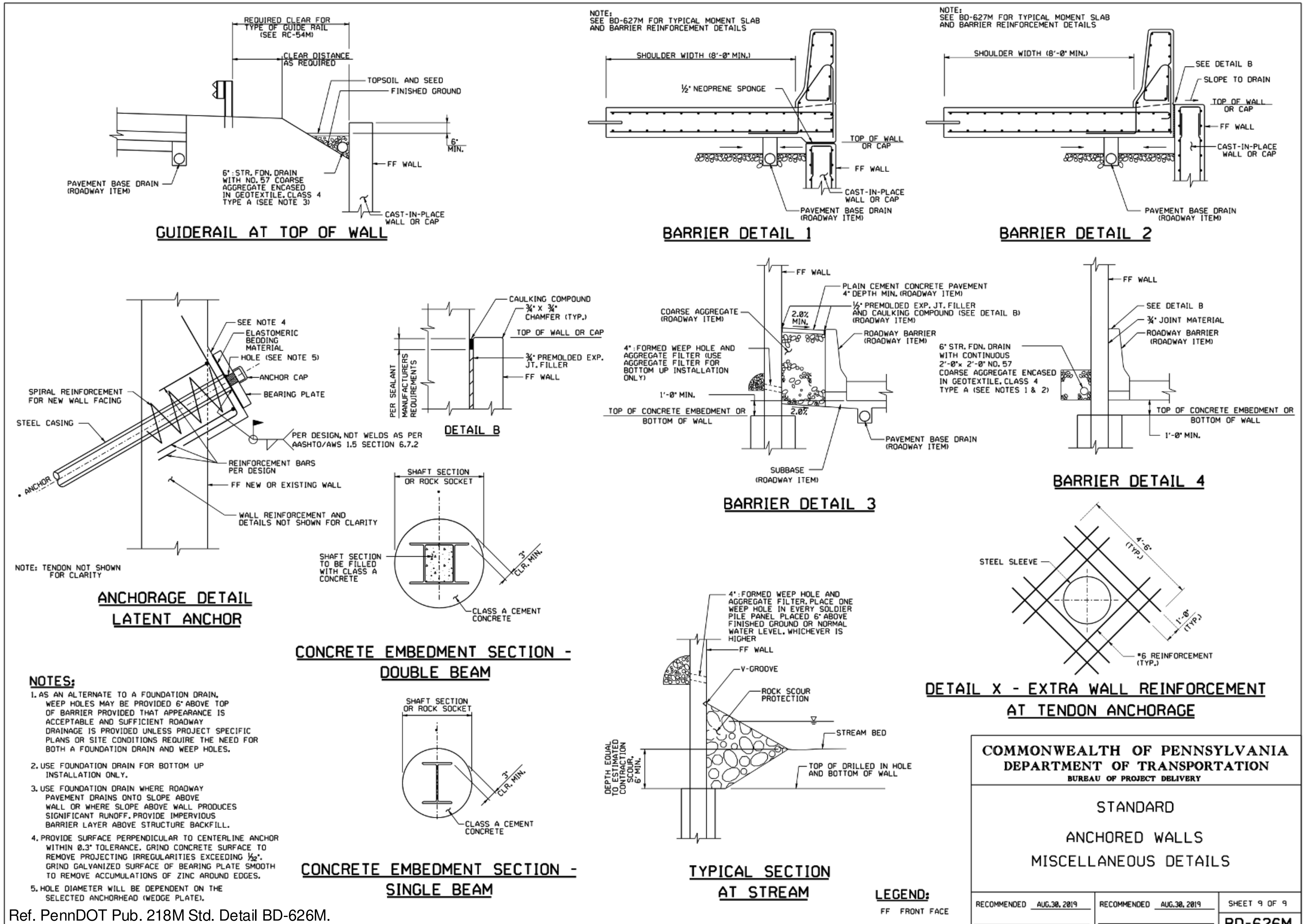
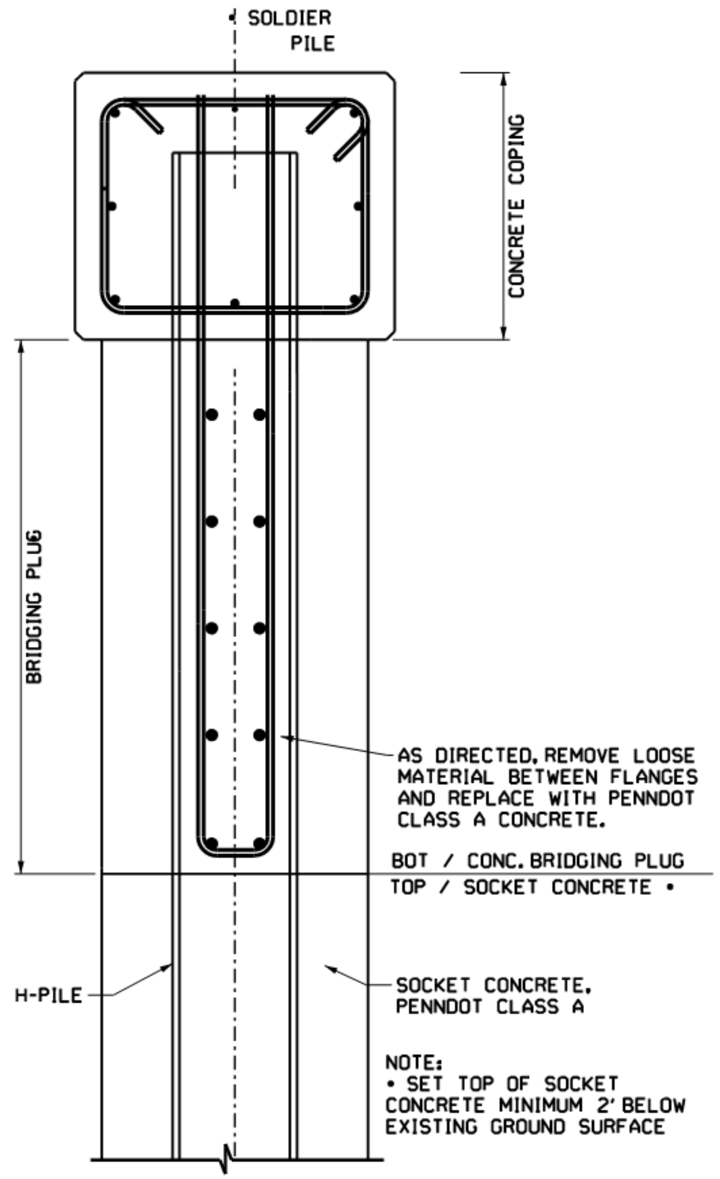
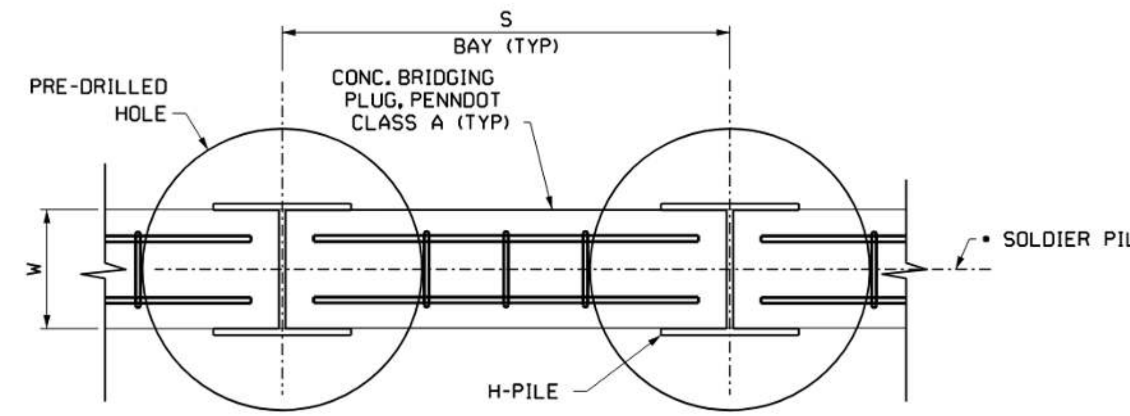


Figure 12-30 - Typical Detail, Anchored Walls (Sheet 9 of 9)



SECTION



PLAN

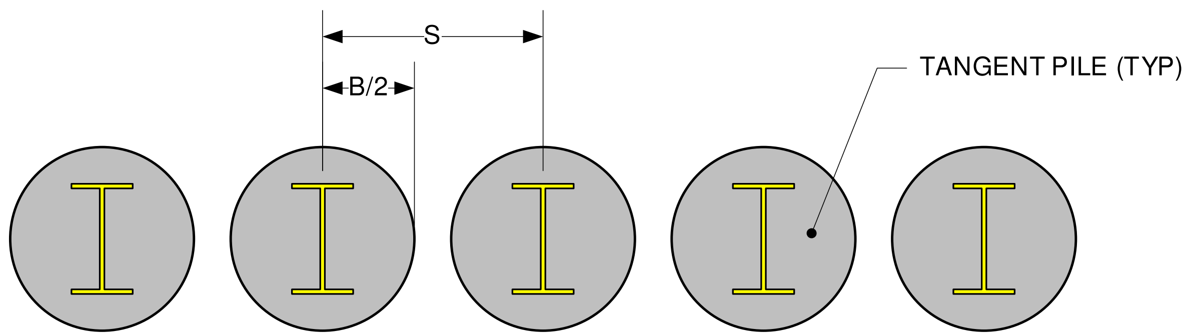
TYPICAL DETAIL – SOLDIER PILE WALL WITH BRIDGING PLUG

(NOT TO SCALE)

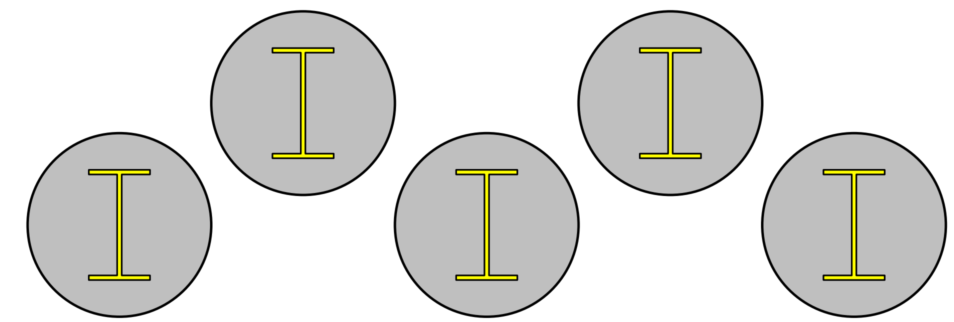
NOTES.

1. Provide materials and construct in accordance with PennDOT Pub. 408.
2. Consider vacuum excavation as an option to excavate between the H-Piles after the soldier piles are installed.
3. Then, consider placing concrete for bridging plug neat against the existing soils.
4. Complete lateral capacity analysis, equilibrium analysis, and overall (global) stability analyses (shallow- and deep-seated) to determine the required:
    a. Soldier pile size, reinforcement, spacing (S), socket diameter, and embedment length,
    b. Bridging plug depth and reinforcement.

Figure 12-31 - Typical Detail, Bridging Plug with Concrete Coping



ALTERNATE A, PLAN VIEW – TYPICAL LAYOUT
TANGENT PILE WITH BENEFIT OF ARCHING



ALTERNATE B, PLAN VIEW – TYPICAL LAYOUT
TANGENT PILE WITHOUT BENEFIT OF ARCHING

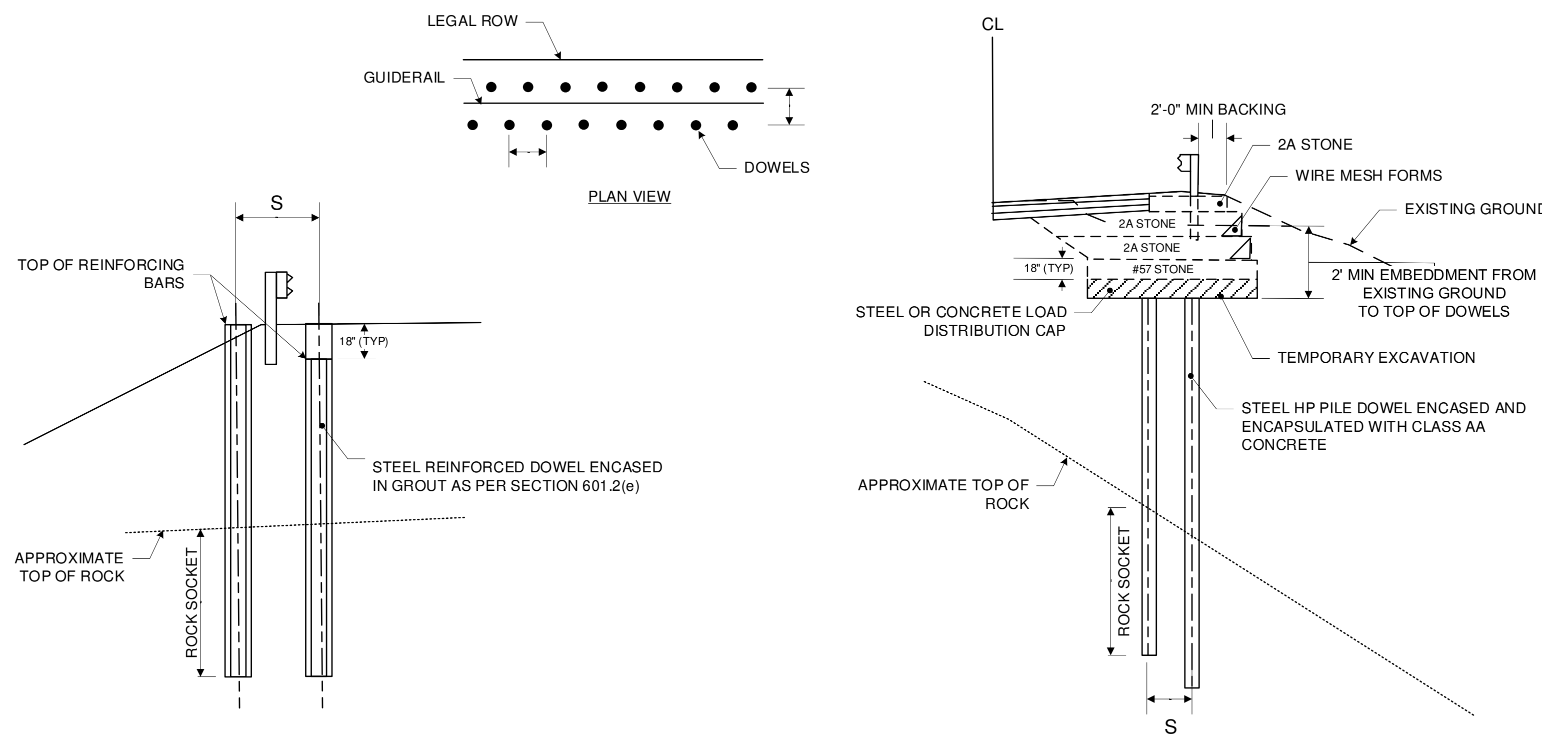
TYPICAL DETAIL – TANGENT PILE

(NOT TO SCALE)

NOTES.

1. Provide materials and construct in accordance with PennDOT Pub. 408.
2. Consider concrete-filled tangent piles (with embedded steel rolled shape) to provide the opportunity to eliminate the need for wall facing. This is of particular interest at landslides that are at or near equilibrium, where additional ground disturbance could induce significant slope movement.
3. Tangent piles provide the benefit of leaving gaps through which groundwater can flow through the wall alignment and reduce the potential buildup of excess pore pressure.
4. Complete lateral capacity analyses, equilibrium analysis, and overall (global) stability analyses (shallow and deep-seated) to determine the number and size of tangent piles that are required to maintain wall stability and stabilize the landslide. It is critical to use the stability analysis results to determine the depth to the critical plane of rupture (e.g., failure surface) so that the tangent piles are installed deep enough to mobilize the required lateral resistance. Reduction in the lateral resistance due to the close spacing of the pile should also be considered for design.
5. To start, consider 30-inch diameter (B) tangent piles that are spaced at 36 to 48 inches (S), with an embedded W18x rolled shape with a 10- to 12-inch wide flange. Check the nominal clear distance from the corner of the flange to the edge of the predrilled hole to affirm that there is sufficient concrete cover.
6. Make allowance for the pile installation tolerance (e.g., +/- 1 inch) and possible over-drilling (e.g., +2 inches). With this said, consider a pile spacing (S) equal to the theoretical pile diameter plus 6 inches, or 36 inches (S) center-to-center for 30-inch diameter (B) predrilled holes.
7. Consider the existing subsurface conditions to assess whether or not arching can be relied on to provide lateral resistance to stabilize the landslide mass. If arching can be relied on (typically cohesive soil), then consider Alternate A (illustrated above) to lay out the tangent piles. If arching cannot be relied on (e.g., soils are soft and saturated sufficiently that the soils are prone to flow around the tangent piles), then consider Alternate B (illustrated above) to lay out the tangent piles.
8. When in doubt about variable subsurface conditions and/or when a redundant solution is required, consider the addition of a structural cap beam to provide an alternate load path between adjacent tangent piles.

Figure 12-32 - Typical Detail, Tangent Piles



PIPE DOWELS TO GROUND SURFACE

PIPE DOWELS WITH CAP

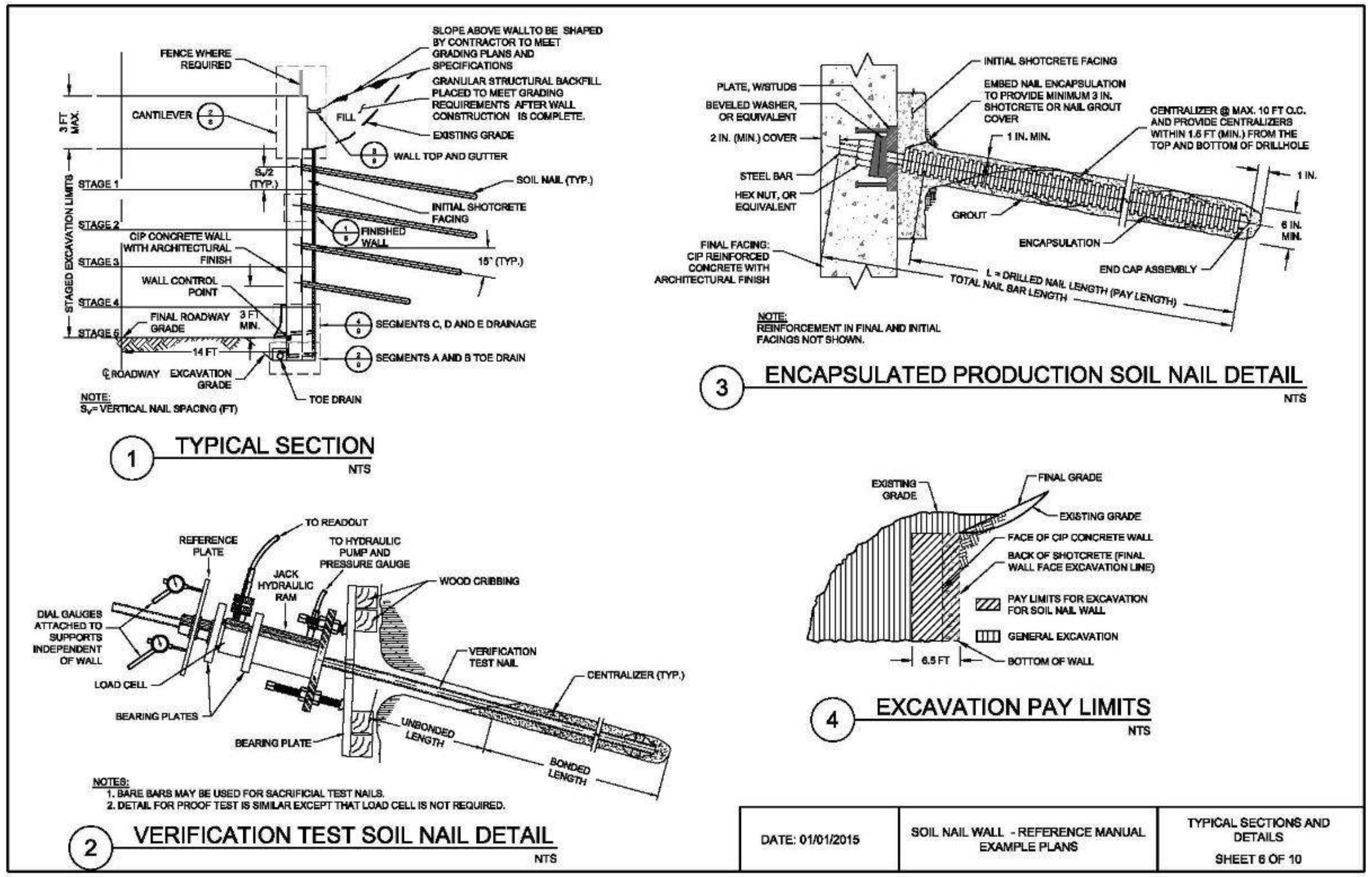
TYPICAL DETAIL – PIPE DOWELS

(NOT TO SCALE)

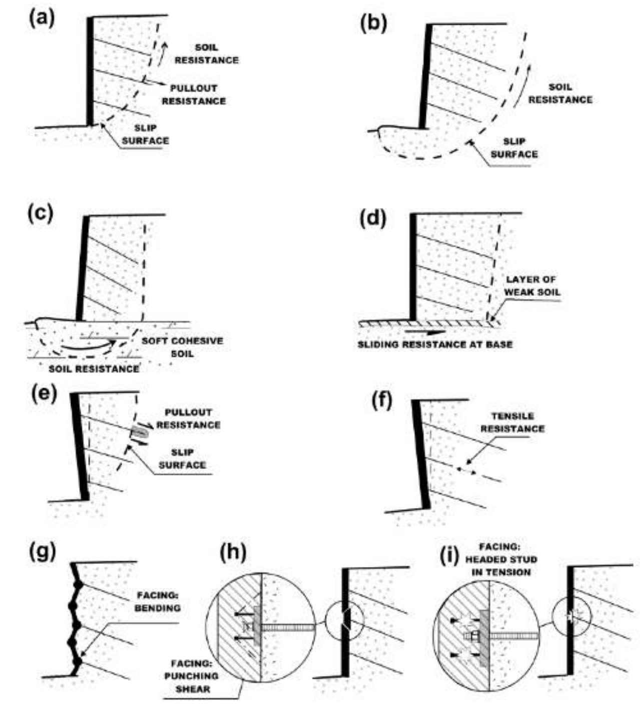
NOTES.

1. Provide materials and construct in accordance with PennDOT Pub. 408.
   Pipe dowels provide the benefit of leaving gaps through which groundwater can flow through the wall alignment and reduce the potential buildup of excess pore pressure.
2. Complete lateral capacity analyses, equilibrium analysis, and overall (global) stability analyses (shallow and deep-seated) to determine the number, spacing, and size of piles that are required to maintain wall stability and stabilize the landslide. It is critical to use the stability analysis results to determine the depth to the critical plane of rupture (e.g., failure surface) so that the piles are installed deep enough to mobilize the required lateral resistance.
3. To start, consider a minimum 6"-inch diameter (B) piles that are spaced at 36 inches (S); final number, size and spacing of pile is dependent on site-specific stability analysis. Steel reinforcement may consist of steel pipe (ASTM A53, Type E), deformed reinforcing steel, steel H-Pile, or rail steel. Check the nominal clear distance from the corner of the flange to the edge of the predrilled hole to affirm that there is sufficient concrete cover for the selected reinforcement.
4. Make allowance for the pile installation tolerance (e.g., +/- 1 inch) and possible over-drilling (e.g., +2 inches).
5. Consider the existing subsurface conditions to assess whether or not arching can be relied on to provide lateral resistance to stabilize the landslide mass.
6. When in doubt about variable subsurface conditions and/or when a redundant solution is required, consider the addition of a structural cap beam to provide an alternate load path between adjacent piles.
7. Typical structural elements consisting of a concrete pile with steel reinforcement are shown for the purposes of providing this typical detail; however, the actual structural elements used will be based on availability of materials and the final engineering design which may vary from what is presented.

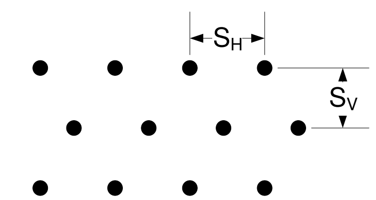
Figure 12-33 - Typical Detail, Pipe Dowels



Ref. FHWA NHI-14-007 (GEC 007), 2015.



POTENTIAL LIMIT STATES
(a) Internal stability (slip- surface intersecting soil and nails)
(b) global stability (slip surface not intersecting nails)
(c) global stability: basal heave
(d) geotechnical strength: lateral sliding
(e) geotechnical strength: pullout
(f) structural strength: nail in tension
(g) facing structural strength: bending
(h) facing structural strength: punching shear
(i) facing structural strength: headed stud in tension.



ELEVATION VIEW
TYPICAL SOIL NAIL SPACING

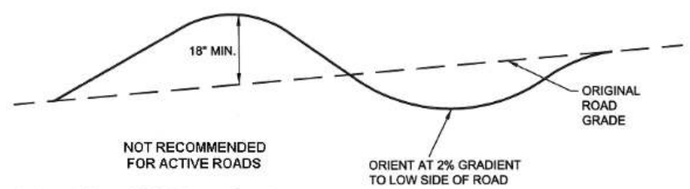
TYPICAL DETAIL – SOIL NAIL WALL/SLOPE TREATMENT

(NOT TO SCALE)

NOTES.
1. Provide materials and construct in accordance with PennDOT Pub. 408.
2. Soil nailing typically involves a mesh facing or shotcrete facing and potentially an additional concrete wall facing, depending on the design requirements and the engineered soil nail design. However, if surficial stability is not a concern and there is sufficient nail embedment length in the slope mass to mobilize the tensile strength of the soil nail, then it is possible to install the soil nails without wall facing. However, this scenario is not typical.
3. Soil nails are typically staggered horizontally in a triangular pattern as illustrated above. Soil nails are typically not staggered vertically, to reduce the number of benches that need to be excavated to install the soil nails.
4. Consider the method of grout placement to develop soil nail details. Some soil nails utilize a hollow bar through which cement grout can be injected.
5. Complete an assessment of all potential limit states to determine the number and size of soil nails required to stabilize the landslide. It is critical to use the stability analysis results to determine the depth to the critical plane of rupture (e.g., failure surface) so that the soil nails are installed deep enough to mobilize the required lateral resistance. Refer to GEC 7 [46] for a more comprehensive discussion about the modes of failure that should be considered.

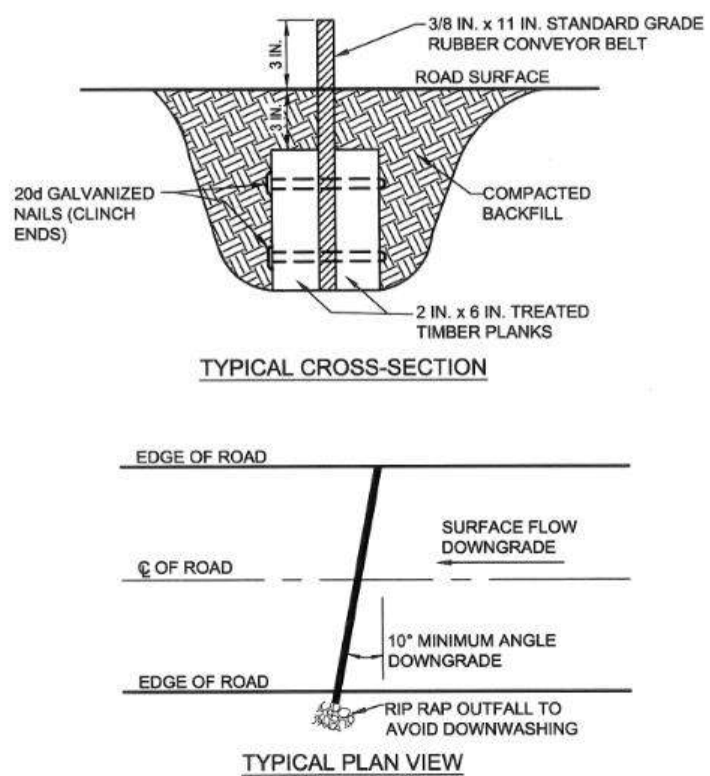
Figure 12-34 - Typical Detail, Soil Nail Wall/Slope

STANDARD CONSTRUCTION DETAIL #3-5
Waterbar



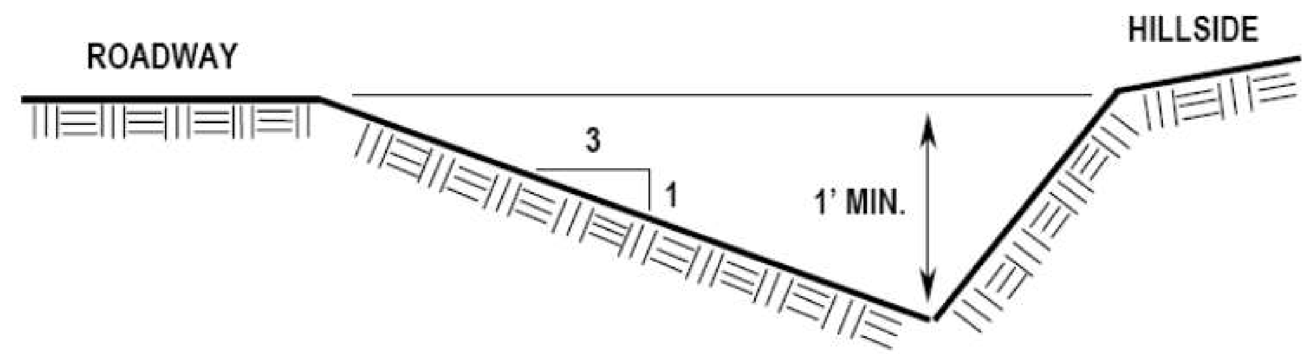
Adapted from USDA Forest Service

STANDARD CONSTRUCTION DETAIL #3-9
Water Deflector



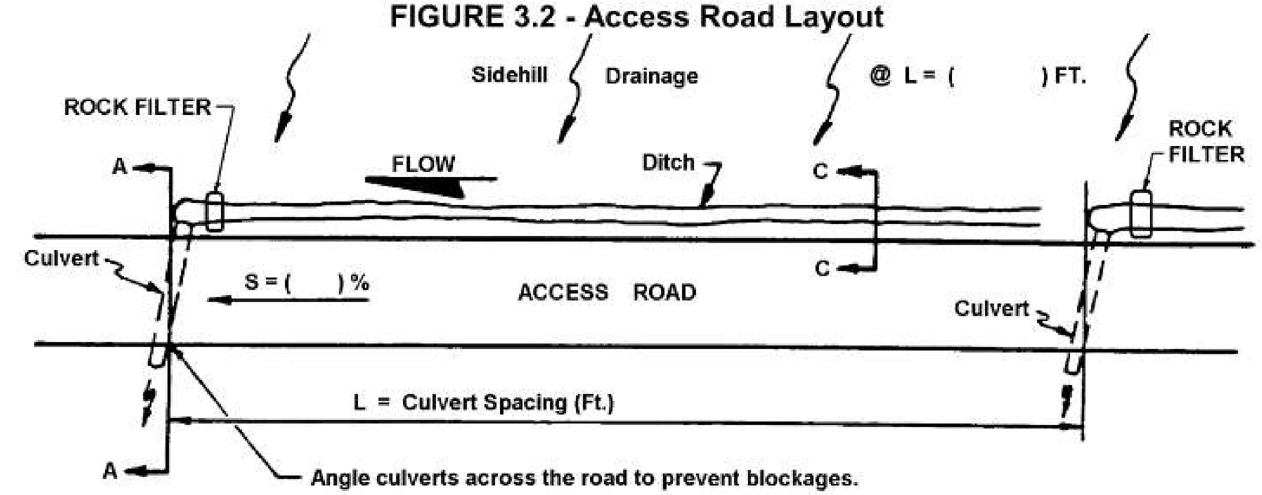
USDA Forest Service

FIGURE 3.1 - Typical Roadside Ditch Section



USDA Forest Service

FIGURE 3.2 - Access Road Layout



USDA Forest Service

Ref. PA DEP Erosion & Sediment Polution Control Program Manual (2012)

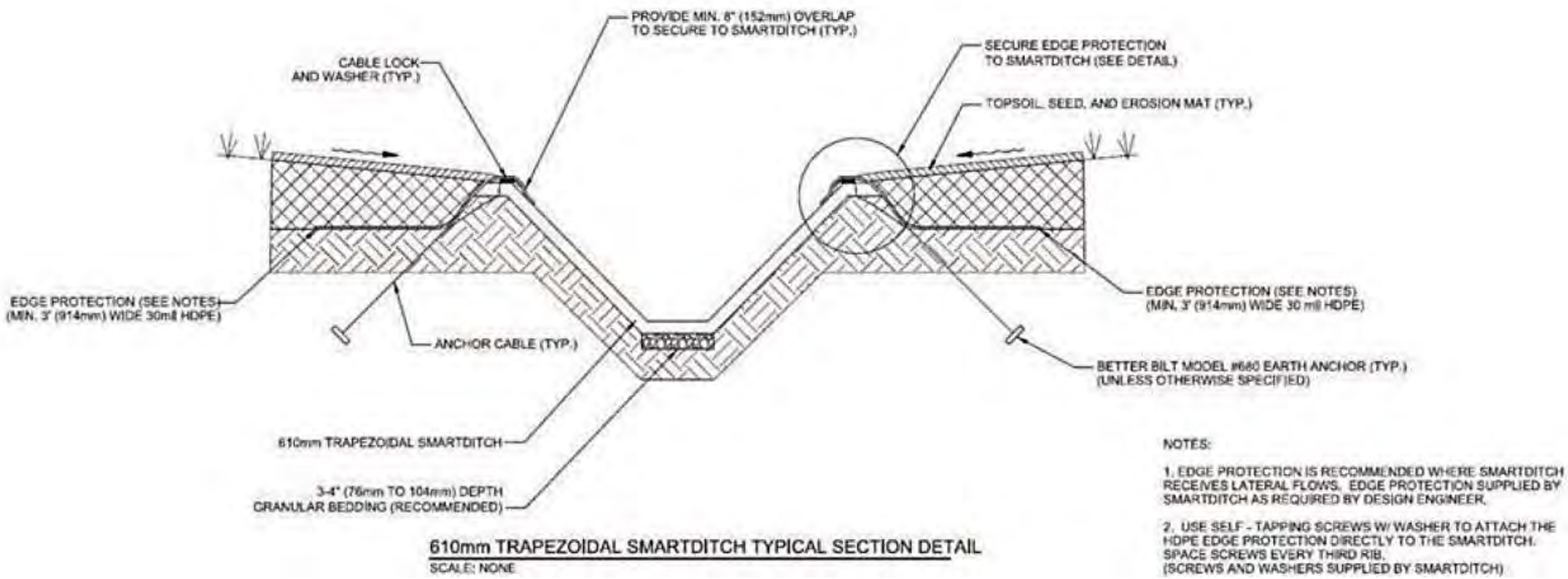
TYPICAL DETAIL – UPSLOPE DIVERSION

(NOT TO SCALE)

NOTES.

1. Provide materials and construct in accordance with PennDOT Pub. 408.
2. Upslope diversion typically entails developing a detail to conform to existing site conditions. Ultimately, the goal is to divert surface water away from areas that are sensitive to slope movement.
3. Several details that are commonly used are illustrated on this sheet, which was copied from the PA DEP Erosion & Sediment Control Program Manual.

Figure 12-35 - Typical Detail, Upslope Interception and Diversion

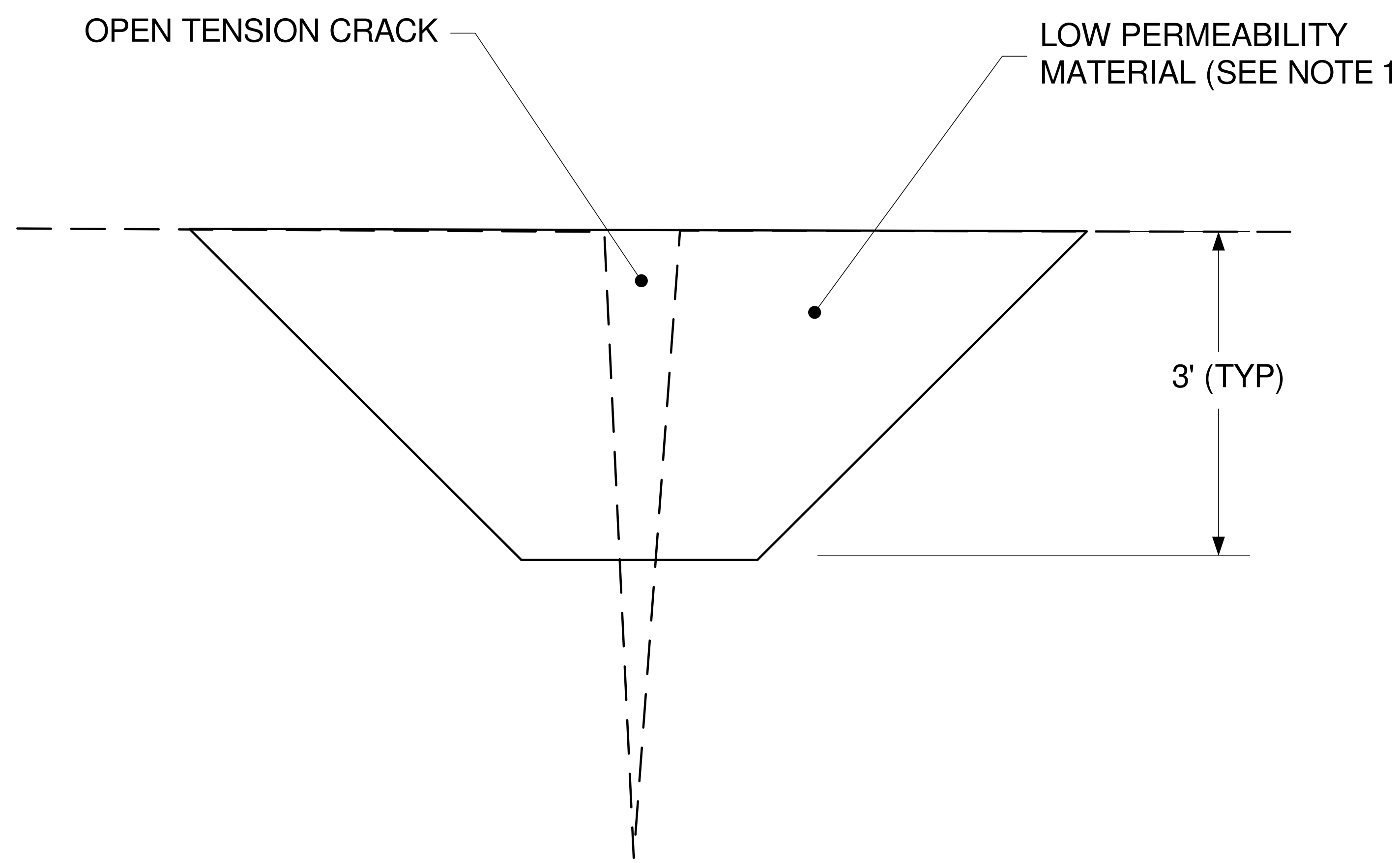


Ref. Smart Ditch Technical Manual

NOTES.

1. Provide materials and construct in accordance with PennDOT Pub. 408.
2. Consider ditch lining to manage surface water infiltration at landslides.
3. Use shiplap splices to join successive sections of ditch liners.
4. Consider edge protection where lined ditches receive lateral flow.

Figure 12-36 - Typical Detail, Smart Ditch

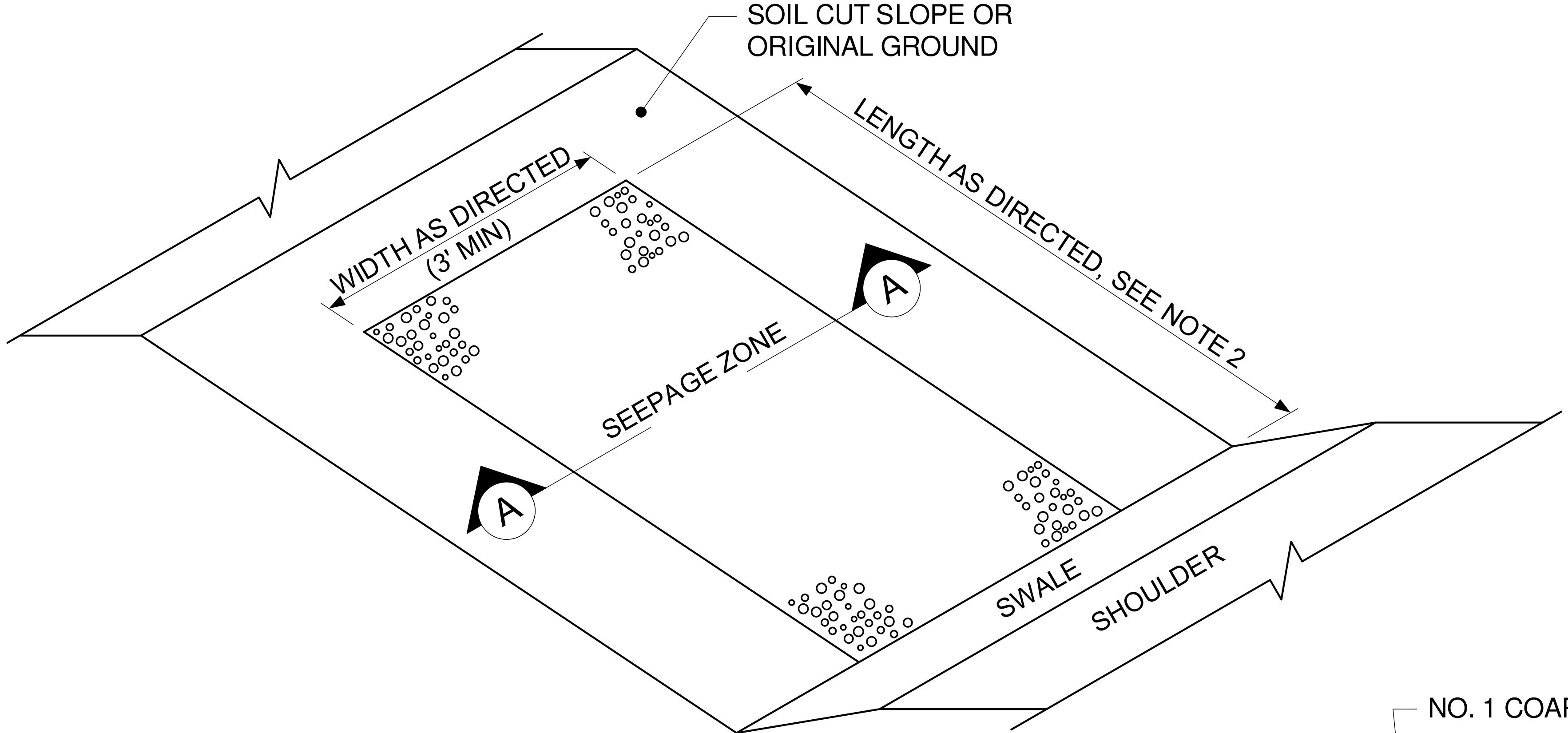


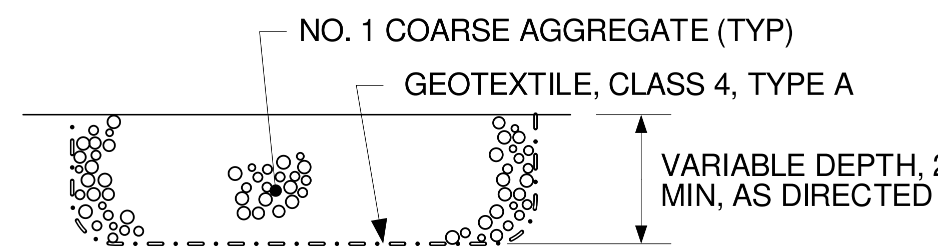
TYPICAL DETAIL – CRACK SEALING

NOTES.

1. Low permeability material may consist of compacted on-site cohesive soils, bentonite clay, grout, or asphalt (for pavement applications).

Figure 12-37 - Typical Detail, Crack Sealing





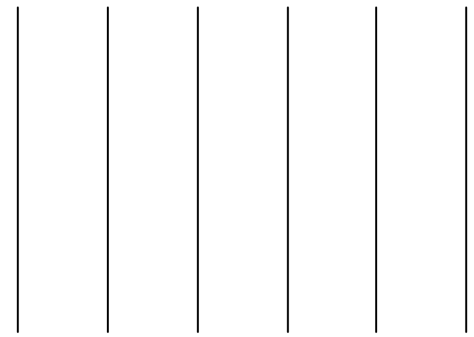
TYPICAL DETAIL
SEEPAGE INTERCEPTOR DRAIN

SECTION A-A
(NOT TO SCALE)

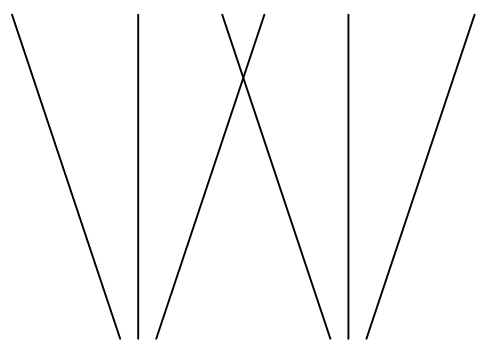
NOTES.

1. Provide materials and construct as specified in PennDOT Pub. 408, Sec. 612 for subgrade drains. Refer to PennDOT Pub. 72M Std. Dwg. RC-30M.
2. At soil cuts, extend the interceptor drain to the swale ditch; under the embankment, extend the interceptor drain to the outlet; and at undisturbed natural soil slopes, extend the interceptor drain to the collector pipe at a lower limit of seepage area.
3. Size of the seepage interceptor drain is dependent on the extent of seepage.
4. Provide continuous geotextile coverage using a 3-foot lap splice(s), where required.
5. Maintain a stable slope. As a general guide, consider 2H:1V, or flatter for the preliminary grading layout, subject to the final design. Assess slope stability including, but not limited to, a shallow veneer (sheet flow) mode of failure. Compensate for sliding resistance along the aggregate-geotextile interface.

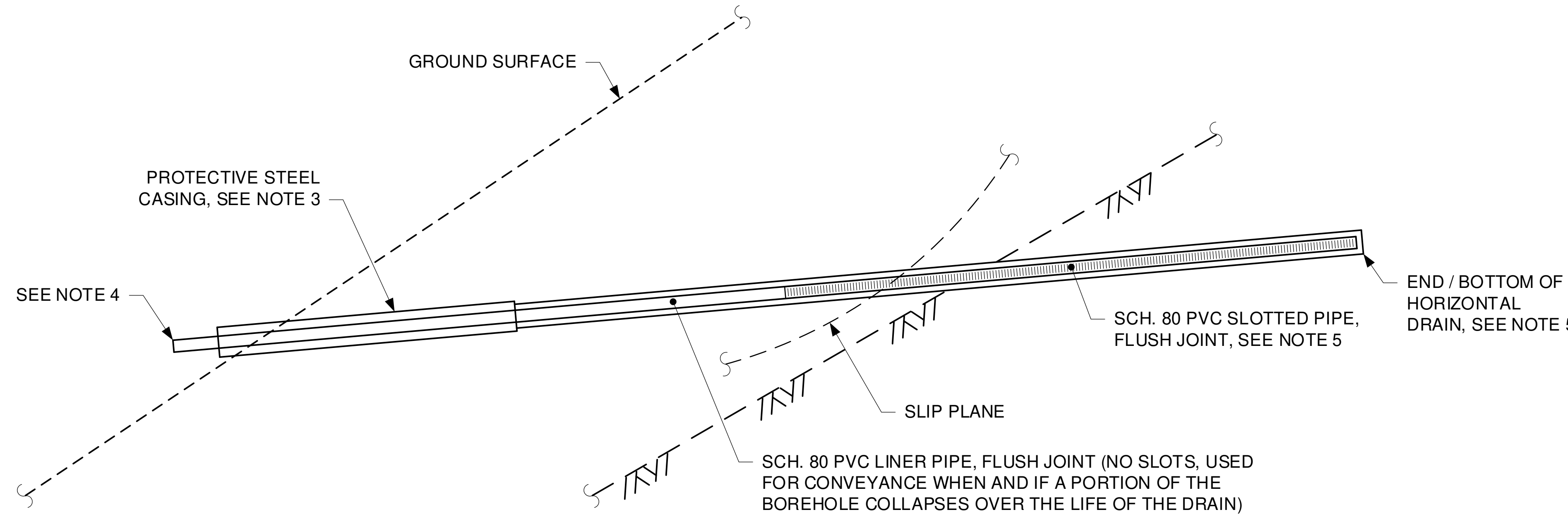
Figure 12-38 - Typical Detail, Seepage Interceptor Drain



PLAN VIEW – ALTERNATE A



PLAN VIEW – ALTERNATE B



SECTION VIEW

Ref. WSDOT WA-RD 787.1, Design Guidelines for Horizontal Drains Used for Slope Stabilization (2013), and TRB 783, Royston, D, Horizontal Drains and Horizontal Drilling (1980)

TYPICAL DETAIL – HORIZONTAL DRAIN

(NOT TO SCALE)

NOTES.

1. Provide materials and construct in accordance with PennDOT Pub. 408.
2. Consider drilling at a slight incline upward (e.g., 10 degrees) to promote positive drainage by gravity.
3. Typically, protective steel casing is installed near the ground surface to maintain borehole stability and reduce the potential of saturating the surficial soils.
4. Consider the option of adding a collector pipe and burying the end of the horizontal drain to avoid clogging with ice during cold weather.
5. Use the results of the subsurface investigation to identify the location of the plausible rupture plane, the location and depth of underlying weathered rock, and possible seepage pathways. Use those results to determine the location, length, outfall elevation, and inclination of the horizontal drains required. In addition, use the results to assess the size (e.g., width) of slots for the screened section of PVC pipe to minimize potential blinding. Consider the flow capacity of the slotted pipe section.
6. Do not obsess about borehole guidance and trajectory, preciseness; the locations, length, outfall elevation, and inclination of the horizontal drain may need to be modified in the field based on the actual conditions encountered. Horizontal drilling for drainage purposes has not reached a point of development at which all these things can be precisely known or controlled, nor do they need to be. Do not overly refine or set rigid requirements to install horizontal drains. The choice of spacing and length of horizontal drains has been done, in practice, largely based on trial and adjustment. Key factors to consider are the quantity of water tapped in the first few installations, the predicted internal drainage system, the height and volume of the potentially unstable area, the soil permeability, and the termination limit concerning the location of the head scarp and the top of slope.

Figure 12-39 - Typical Detail, Horizontal Drains

CHAPTER 13

Legal Liabilities

13.1 GENERAL

This chapter highlights some of the legal liabilities that are associated with the stabilization and repair of landslides. The discussion in this chapter concerning legal liabilities should not be substituted for the engagement and judgment of a licensed attorney.

Safety is of utmost importance. An active landslide poses the potential to place individuals, property, and infrastructure at risk. “Many studies have shown that most damaging landslides are human-related. Thus … the potential for hazard may be reduced by the introduction of countermeasures such as improved grading procedures, land use controls, and drainage or runoff controls.”[49]

It is essential that the practitioner act in a responsible manner that causes no harm to their client or a third party. The practitioner must perform his work carefully, apply a standard of care using his(her) professional knowledge, experience, skill and engineering judgment in matters involving landslide mitigation and repair, and use that to act as a faithful agent to provide professional advice and recommendations to his(her) client.

In the case of landslide mitigation, the practitioner is reminded that he (or she) is obligated to alleviate landslide hazards in a responsible manner that does not place adjacent property at risk. The practitioner should not assume that protection of adjacent land will automatically apply to the structures that reside on that adjacent land.

Adjacent side-by-side property owners are obligated to laterally support each other’s property in their respective natural state. A property owner has the right to excavate on his property, provided that such action does not jeopardize the stability of the adjacent neighboring land. The practitioner is reminded that these two concepts (e.g., providing lateral support and the right to excavate) represent mutual rights.

When it becomes evident that the proposed land disturbance will risk the potential loss of stability at the adjacent neighboring property, the landowner should notify the adjacent property owner in advance of such action.

It is important to realize that such legal liabilities apply to not only construction activity, but also to maintenance, and design activity.

In closing, the practitioner is advised to “cause no harm” in the performance of his (or her) work to mitigate or repair a landslide in a professional manner that is consistent with standard practice.

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[3] AASHTO Designation T-236, p. T 236-2.

[4] AASHTO Designation T-236, Standard Method of Test for Direct Shear Test of Soils under Consolidated Drained Conditions. 2008, editorially revised 201, p. T 236-7.

[5] ASTM STP 361, p. 401.

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GLOSSARY

Active Landslide – landslide that is currently moving [26]

Alluvial Terrace - One, or a series of flat-topped landforms in a stream valley that flank and are parallel to the stream channel, originally formed by a previous stream level, and representing remnants of an abandoned flood plain, stream bed, or valley floor produced during a past state of fluvial erosion or deposition (i.e., currently very rarely or never flooded; inactive cut and fill, scour and fill, or both processes). Erosional surfaces cut into bedrock and thinly mantled with stream deposits (alluvium) are called "strath terraces." Remnants of constructional valley floors thickly mantled with alluvium are called alluvial terraces. [135]

Angle Of Repose - The maximum angle of slope (measured from a horizontal plane) at which loose, cohesionless material will come to rest. [135]

Bedding Plane - A planar or nearly planar bedding surface that visibly separates each successive layer of stratified sediment or rock (of the same or different lithology) from the preceding or following layer; a plane of deposition. It often marks a change in the circumstances of deposition, and may show a parting, a color difference, a change in particle size, or various combinations. [135]

Benefit/Cost Ratio - The estimated amount of reduction in the estimated total amount of losses after the implementation of loss-reduction measures, divided by the cost of applying the measures. [81]

Borehole A circular hole drilled into the earth, often to a great depth for exploratory purposes. [75]

Check Dams Check dams are small sediment storage dams built in the channels of steep gullies to stabilize the channel bed. A common use is to control channelized debris flow frequency and volume. Check dams are expensive to construct and are therefore usually only built where important installations or natural habitat (such as a camp or unique spawning area) lies downslope. [75]

Colluvium - A general term applied to loose and incoherent deposits, usually at the foot of a slope or cliff and brought there chiefly by gravity. [75]

Consequence (e.g., Impact) - The effect of the occurrence of a hazard on people and community infrastructure. [81]

Debris Flow – The mass movement process, associated sediments (debris flow deposit), or resultant landform characterized by a very rapid type of flow dominated by a sudden downslope movement of a mass of rock, soil, and mud (more than 50% of the particles are > 2mm), and whether saturated or comparatively dry, behaves much as a viscous fluid when moving. [135]

Degradation - The wearing down or away, and the general lowering of the land surface by natural processes of weathering and erosion (e.g., the deepening by a stream of its channel); it may infer the process of transportation of sediment. [135]

Depression - Any relatively sunken part of the earth's surface; especially a low-lying area surrounded by higher ground. A closed depression has no natural outlet for surface drainage (e.g., a sinkhole). An open depression has a natural outlet for surface drainage. [135]

Digital Elevation Model (DEM) - A digital elevation model (DEM) is a digital file consisting of terrain elevations for ground positions at regularly spaced horizontal intervals. (A commercial definition – new technology) Digital Terrain Model (DTM) The term used by United States Department of Defense and other organizations to describe digital elevation data. [75]

Dip [structural geology] - The maximum angle that a structural surface, (e.g., a bedding or fault plane) makes with the horizontal, measured perpendicular to the strike of the structure and in the vertical plane; used in combination with "dip" to describe the orientation of bedrock strata. [135]

Drawdown - Lowering of water levels in rivers, lakes, wells, or underground aquifers due to withdrawal of water. Drawdown may leave unsupported banks or poorly packed earth that can cause landslides. [75]

Extensometer - An instrument for measuring small deformations, as in tests of stress. [75]

Factor of Safety - The factor of safety, also known as Safety Factor, is used to provide a design margin over the theoretical design capacity to allow for uncertainty in the design process. The uncertainty could be any one of a number of the components of the design process including calculations and material strengths for example. Commonly, a factor of safety of less than 1, for instance, on an engineered slope indicates potential failure, where a factor of safety of greater than 1, indicates stability. [75]

Fracture - Brittle deformation due to a momentary loss of cohesion or loss of resistance to differential stress and a release of stored elastic energy. Both joints and faults are fractures. [75]

Geographic Information System (GIS) - A computer program and associated data bases that permit cartographic information (including geologic information) to be queried by the geographic coordinates of features. Usually the data are organized in "layers" representing different geographic entities such as hydrology, culture, topography, and so forth. A geographic information system, or GIS, permits information from different layers to be easily integrated and analyzed. [75]

Geologic Hazard - A naturally occurring or man-made geologic condition or phenomenon that presents a risk or is a potential danger to life and property. [81]

Gully - A small channel with steep sides caused by erosion and cut in unconsolidated materials by concentrated but intermittent flow of water usually during and immediately following heavy rains or ice and snow melt. A gully generally is an obstacle to wheeled vehicles and too deep (e.g., > 0.5 m) to be obliterated by ordinary tillage; (a rill is of lesser depth and can be smoothed over by ordinary tillage). [135]

Hazard Rating – a numerical assessment of the relationship of the risk of a slope failures (e.g., likelihood) and consequences if failure occurred. Establishing a Hazard Rating is a tool in Slope Maintenance systems to establish criteria for Threat Tolerance.

Head Scarp - The steep surface on undisturbed ground at the upper edge of a landslide, caused by movement of displaced material away from the undisturbed ground; it is visible a part of the surface of rupture (rupture plane). [135]

Hummocky – uneven or rolling terrain characterized by small ridges in the soils, often associated with ground disturbance and/or soil creep.

Inclinometer - Instrument for measuring inclination to the horizontal. [75]

Landslide - A general, encompassing term for most types of mass movement landforms and processes involving the downslope transport and outward deposition of soil and rock materials, caused by gravitational forces and that may or may not involve saturated materials. Names of landslide types generally reflect the dominant process, the resultant landform, or both. The main operational categories of mass movement are fall (rockfall, debris fall, soil fall), topple (rock topple, debris topple, soil topple), slide (rotational landslide, block glide, debris slide, lateral spread), flow (rockfall avalanche, debris avalanche, debris flow (e.g., lahar), earthflow, (creep, mudflow)), and complex landslides. [135]

Landslide Inventory Maps - Inventories identify areas that appear to have failed by landslide processes, including debris flows and cut-and-fill failures. [75]

Landslide Susceptibility Map - This map goes beyond an inventory map and depicts areas that have the potential for landsliding. These areas are determined by correlating some of the principal factors that contribute to landsliding, such as steep slopes, weak geologic units that lose strength when saturated, and poorly drained rock or soil, with the past distribution of landslides. [75]

Landslide Hazard Map - Hazard maps show the areal extent of threatening processes: where landslide processes have occurred in the past, where they occur now, and the likelihood in various areas that a landslide will occur in the future. [75]

Landslide Risk Map - Landslide hazards and the probability that they will occur, expressed in statistical recurrence rates; risk maps may show cost/benefit relationships, loss potential and other potential socioeconomic effects on an area and (or) community. lithology The physical character of a rock, generally as determined at the microscopic level, or with the aid of a low-power magnifier; the microscopic study and description of rocks. [75]

Liquefaction - The transformation of saturated, loosely packed, coarse-grained soils from a solid to a liquid state. The soil grains temporarily lose contact with each other, and the particle weight is transferred to the pore water. [75]

Mine Spoil - Randomly mixed, earthy materials artificially deposited as a result of either surficial or underground coal mining activities. [135]

Mitigation - Activities that reduce or eliminate the probability of occurrence of a disaster and (or) activities that dissipate or lessen the effects of emergencies or disasters when they actually occur. [75]

Mudflow - A general term for a mass-movement landform and process characterized by a flowing mass of predominately fine-grained earth material possessing a high degree of fluidity during movement. The water content may range up to 60 percent. [75]

Outcrop - That part of a geologic formation or structure that appears at the surface of the earth. [135]

Overburden - The upper part of a sedimentary deposit, compressing and consolidating the materials below. [135]

Perched Ground Water - Unconfined ground water separated from an underlying main body of ground water by an unsaturated zone. [75]

Piezometer - An instrument for measuring pressure head in a conduit, tank, or soil—it is a small diameter water well used to measure the hydraulic head of ground water in aquifers. [75]

Pore-Water Pressure - A measure of the pressure produced by the head of water in a saturated soil and transferred to the base of the soil through the pore water. This is quantifiable in the field by the measurement of free water-surface level in the soil or by direct measurement of the pressure by means of piezometers. Pore-water pressure is a key factor in failure of a steep slope soil and operates primarily by reducing the weight component of soil shear strength. [75]

Pore Water - Subsurface water in an interstice, or pore. [75]

Reconnaissance Geology/Mapping - A general, exploratory examination or survey of the main features of a region, usually preliminary to a more detailed survey. It may be made in the field or office, depending on the extent of information available. [75]

Relief - The difference in elevation between the high and low points of a land surface. [75]

Rill - A very small channel with steep sides caused by erosion and cut in unconsolidated materials by concentrated but intermittent flow of water, usually during and immediately following moderate rains or after ice or snow melt. Generally, a rill is not an obstacle to wheeled vehicles and is shallow enough (e.g., < 0.5 m) to be obliterated by ordinary tillage. [135]

Risk - The probability of occurrence or expected degree of loss, as a result of exposure to a hazard. [75]

Rupture Plane - A landslide displacement surface, often slickensided and striated, or brecciated, and sub planar. It is best exhibited in argillaceous materials and in those materials that are highly susceptible to clay alteration when granulated. [135]

Scour - The powerful and concentrated clearing and digging action of flowing air, water, or ice, especially the downward erosion by stream water in sweeping away mud and silt on the outside curve of a bend, or during the time of a flood; a process. [135]

Seepage - An area, generally small, where water outflows slowly at the land surface indicated by moist areas on open slopes, and seepage sites along road cuts. The locations of these areas of concentrated subsurface flow should be noted on maps and profiles as potential sites of active, unstable ground. [75] [135]

Shear - A deformation resulting from stresses that cause contiguous parts of a body to slide relative to each other in a direction parallel to their plane of contact. [75]

Slickenside – A polished and striated rock surface that results from friction along a fault or bedding plane.

Soil Mechanics - The application of the principles of mechanics and hydraulics to engineering problems dealing with the behavior and nature of soils, sediments, and other unconsolidated accumulations; the study of the physical properties and utilization of soils, especially in relation to highway and foundation engineering. [75]

Stress - In a solid, the force per unit area, acting on any surface within it, and variously expressed as pounds or tons per square inch, or dynes or kilograms per square centimeter; also, by extension, the external pressure that creates the internal force. [75]

Subsidence - Sinking or downward settling of the Earth's surface, not restricted in rate, magnitude, or area involved. Subsidence may be caused by natural geologic processes, such as solution, compaction, or withdrawal of fluid lava from beneath a solid crust or by human activity such as subsurface mining or the pumping of oil or ground water. [75]

Surficial Geology - Geology of surficial deposits, including soils; the term is sometimes applied to the study of bedrock at or near the Earth's surface. [75]

Tensile Stress - A normal stress that tends to pull apart the material on the opposite sides of the plane on which it acts. [75]

Threat (e.g., Vulnerability) - The susceptibility or exposure to injury or loss from a hazard. [81]

Toe of Landslide - The lower margin of the disturbed material of a landslide pushed over onto the undisturbed slope. [81]

Uncertainty – A function of the amount of data available and the unknown variables to be accounted for in design.

Weathering - The destructive process by which earth and rock materials exposed to the atmosphere undergo physical disintegration and chemical decomposition resulting in changes in color, texture, composition, or form. Processes may be physical, chemical, or biological. [75]

> **Weathering, Differential** - When weathering across a rock face or exposure occurs at different rates mainly due to variations in the composition and resistance of the rock. This results in an uneven surface with the more resistant material protruding. [75]
>
> **Weathering, Mechanical** - The physical processes by which rocks exposed to the weather change in character, decay, and crumble into soil. Processes include temperature change (expansion and shrinkage), freeze-thaw cycle, and the burrowing activity of animals. [75]

APPENDIX A

FORMS

A.1

SLOPE MOVEMENT FIELD VISIT CHECKLIST

SLOPE MOVEMENT FIELD VISIT CHECKLIST

* Refer to measurement and terminology "References" for consistency/clarity.

CLIENT & PROJECT NAME: ____

COUNTY: ____________________ MUNICIPALITY (IES): ____________________

ADDRESS: ____

LATITUDE: ____________________ LONGITUDE: ____________________

DATE OF FIELD VISIT: ______________ WEATHER: ____________________

INSPECTOR: ____

PROJECT DESCRIPTION: ____

____

____

1 Site Visit Preparation

1.1 Site Visit Preparation Checklist

Did you review all relevant literature materials? ____________

- Site history
- Geologic setting
- Hydrogeologic setting
- Available aerial photography

Do you have equipment to help document and inspect the site? _____________

- Tape Measure/Ruler
- Measuring Wheel
- Camera
- Notepad

Do you have a device to record GPS coordinates and elevations of observed features? __________

2 General Site Inspection

2.1 Inspect and describe notable surface features within the slide mass and in the areas adjacent to the slide mass (including seepage, sinkholes, settlement or possible evidence of minesubsidence, benching, hummocky ground, depressions, tension cracks, bulges, etc.).

____

____

____

2.2 Inspect area for any rock outcrops or evident rock types and/or bedding planes.

____

____

____

2.3 Inspect existing on-site drainage and associated drainage structures (culverts, pipes, inlets, ditches, etc.) or natural water features. Describe and provide the location of any concerns about existing drainage features. Note any significant erosion, washout areas, evidence of runoff, scour at toe of the slope from stream, etc.

____

____

____

2.4 Note previous construction, if any, at the site (widening, relocation, structure replacement, slope mitigation activities, etc.).

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2.5 Note the presence of all utilities at site, both above and below ground (electric, gas, telephone, cable, water, sewer, etc.).

____

____

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2.6 Note the presence of all buildings/structures in the vicinity, and note presence of any damage such as “stair-step cracking” or displacement.

____

____

____

2.7 Interview, if possible, any local residents, project/design engineers, municipal/state employees (police, fire, maintenance, etc.), utility company employees, etc. Note results of interview(s). Collect contact information for any future inquiries/follow-ups.

____

____

____

2.8 Note locations of any existing borings and instrumentation, such as inclinometers, piezometers, and/or survey pins, e.g., to monitor crack movement.

____

____

____

2.9 Photograph significant features and include a sketch of the site below (note direction in sketch and include the locations and directions of photos taken). Include measurements as necessary. Label major landslide features including the crown, head scarp, toe and flanks.

2.10 Note the date of any previous site visits and record any change of conditions from last visit (if applicable).

____

____

____

3 Slope Movement Valuation

Terminology to be used when describing items in this section with relation to the slope movement:

* Unknown/Absent
* Upper/lower limits of landslide
* Upper/lower limits of landslide and adjacent areas
* Entire landslide
* Entire landslide and adjacent areas
* Adjacent Areas

3.1 General Description of Damage/Impact: ____

____

____

3.2 Effect of Movement on:

a. Roadway:____

____

Unknown None Berm Lane Closed

b. Structures:____

____

Unknown None Hairline Cracks Major Separation/Tilting

Collapse

c. Other - Utility Lines, Private Property, etc.:____

____

Unknown None Relatively Unaffected Use Limited

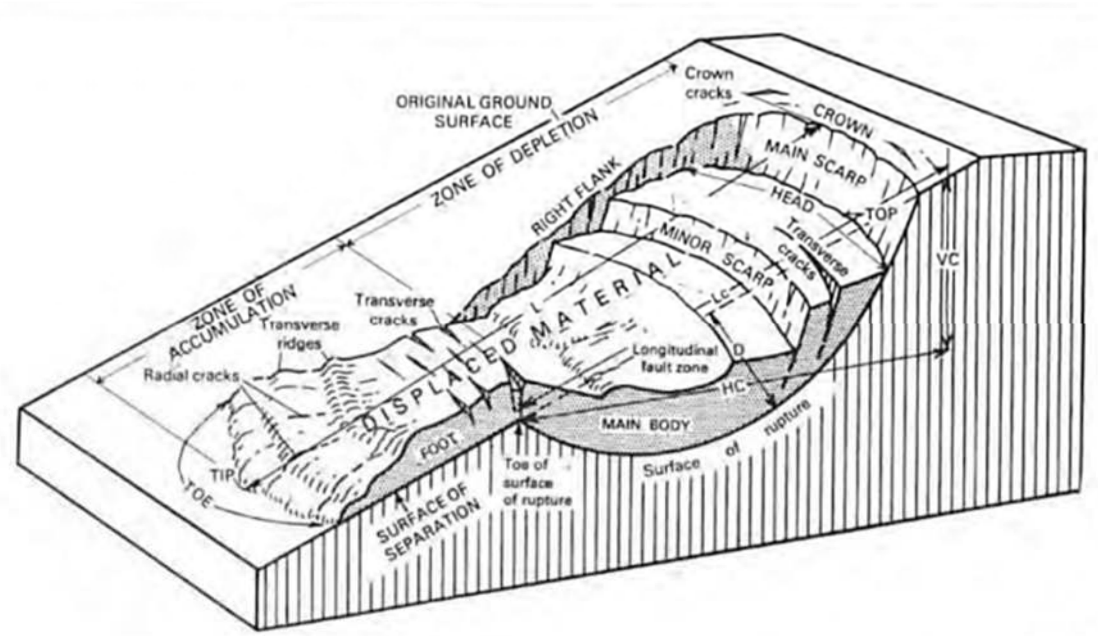
Use Disrupted

d. Stream:____

Unknown None Blockage (less than 1/2 channel)

Blockage (greater than 1/2 channel)/Diversion Total Blockage

3.3 General Measurements of Slide Mass:



a. Height from crown to toe:____Ft.

Type of Measurement:

_________ Unknown _______ Measured _______ Estimated

b. Height of Main Scarp:____Ft.

Type of Measurement:

_________ Unknown _______ Measured _______ Estimated

c. Length of Zone of Depletion:_______________________________________Ft.

Type of Measurement:

_________ Unknown _______ Measured _______ Estimated

d. Length of Zone of Accumulation:____Ft.

Type of Measurement:

__________ Unknown ________ Measured ________ Estimated

e. Width from Left Flank to Right Flank:____________________________________Ft.

Type of Measurement:

__________ Unknown ________ Measured ________ Estimated

f. Depth to Surface of Rupture:____Ft.

Type of Measurement:

__________ Unknown ________ Measured ________ Estimated

g. Volume of Displaced Material:____CF.

Range:

______Small (1000) ________ Moderate (100,000) ______ Large (>1,000,000)

Type of Measurement:

__________ Unknown ________ Measured ________ Estimated

3.4 Dates of Major Movement:

Date of First Observed Movement: ____

By:____

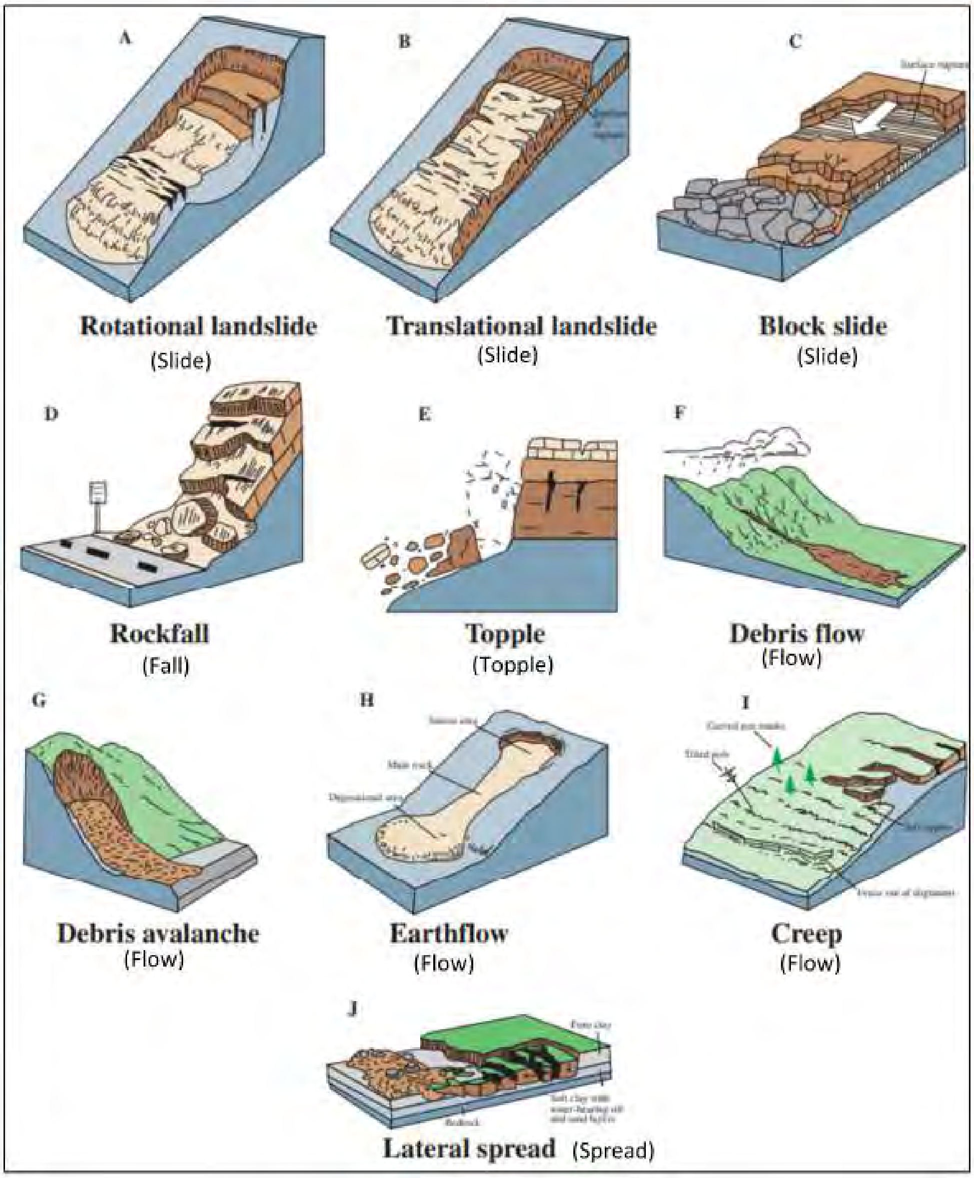
Degree of Certainty: Unknown Actual

Date of Last Observed Movement: ____

By:____

Degree of Certainty: Unknown Actual

3.5 Identify and Describe Type of Movement:



3.6 Type of Material:

Rock Soil Spoil/Mine Waste Man Made Debris

3.7 Activated Movement in Natural Material:

Unknown Yes No

3.8 Movement Triggered by:

Unknown Nature Man Both

3.9 Probability of Additional Movement:

3.10 Stream Undercutting Present?

3.11 Note Any Recent Observed Human Activity (excavation, vibration, surcharge load, retaining structure, etc.):

3.12 Miscellaneous Comments:

A.2

ABBREVIATED SITE VISIT CHECKLIST

ABBREVIATED SLOPE MOVEMENT FIELD VISIT CHECKLIST

CLIENT & PROJECT NAME: ____________________

COUNTY: ____________________ MUNICIPALITY (IES): ____________________

ADDRESS: ____________________

LATITUDE: ____________________ LONGITUDE: ____________________

DATE OF FIELD VISIT: ____________________ WEATHER: ____________________

INSPECTOR: ____________________

PROJECT DESCRIPTION: ____________________

____

____

REASON FOR SITE VISIT: ____________________

____

____

FIELD OR REMOTE INSPECTION: ____________________

1 Site Inspection

1.1 Inspect and describe notable surface features within the slide mass and in the areas adjacent to the slide mass (settlement or possible evidence of minesubsidence, benching, hummocky ground, depressions, tension cracks, bulges, tree tilt, etc.).

____

____

____

1.2 Record the approximate slope ratio and location of the head scarp.

1.3 Inspect and describe notable surface features within the slide mass and in the areas adjacent to the slide mass (settlement or possible evidence of minesubsidence, benching, hummocky ground, depressions, tension cracks, bulges, tree tilt, etc.).

1.4 Inspect area for any rock outcrops or evident rock types and/or bedding planes.

1.5 Note any significant erosion from drainage features (i.e. washout areas, evidence of runoff, scour at toe of the slope from stream etc.)

1.6 Note any seepage or free water on the slope observed.

1.7 Note previous construction, if any, at the site (widening, relocation, structure replacement, slope mitigation activities, etc.).

____

____

____

1.8 Note the presence of all buildings/structures, utilities, roadways at site and their proximity to the slope or slide mass.

____

____

____

1.9 Note the effect of slope movement (if any) on the aforementioned buildings/structures, utilities, roadways. Does the slope movement pose any immediate risk to the public?

____

____

____

1.10 Note the date of any previous site visits and record any change of conditions from last visit (if applicable).

____

____

____

1.11 Note the date of any previous site visits and record any change of conditions from last visit (if applicable).

____

____

1.12 Photograph significant features and include a sketch of the site below (note direction in sketch and include the locations and directions of photos taken). Note any conclusion or recommendations for next steps at the site.

A.3

SLOPE MAINTENANCE CHECKLIST

SLOPE MAINTENANCE CHECKLIST

CLIENT & PROJECT NAME: ______________________________

COUNTY: ______________________ MUNICIPALITY (IES): ______________________

ADDRESS/ROADWAY: ______________________________

LATITUDE: ______________________ LONGITUDE: ______________________

DATE OF SITE VISIT: ______________ WEATHER: ______________________

INSPECTOR: ______________________________

FIELD OR REMOTE INSPECTION: ______________________________

NOTE ANY SIGNIFICANT PRECIPITATION EVENTS IN THE PAST WEEK: ______________________

____

REVIEW OBSERVATIONS AND RECORD DATE OF LAST SITE VISIT: ______________________

1 CONCLUSIONS

RECOMMENDATIONS FOR IMMEDIATE RESPONSE OR INCREASE INSPECTION FREQUENCY BASED ON

OBSERVATIONS MADE: ______________________________

____

____

RECOMMENDED MAINTENANCE AND URGENCY OF NEED: ______________________

____

____

2 Surface Drainage

2.1 Inspect existing on-site surface drainage features to ensure proper function. All drainage channels and ditches should be clear of obstructions, excessive vegetation overgrowth, and debris; additionally, any aggregate used as part of the drainage system should be free draining and unclogged from accumulated siltation.

◻ Operational ◻ Deficient ◻ Immediate Action Required

2.2 Inspect slope for drainage gullies and/or water along or at the toe of slope may indicate improper drainage. All surface flow should be properly conveyed off the slope with the existing surface drainage features.

◻ Operational ◻ Deficient ◻ Immediate Action Required

2.3 Note any significant erosion from drainage features (i.e. washout areas, evidence of runoff, scour at toe of the slope from stream etc.)

◻ Operational ◻ Deficient ◻ Immediate Action Required

3 Subsurface Drainage

3.1 Inspect drainage pipes for cracks and separated joints which can cause eroded subgrade. Inspect drainage inlets for water backup, which could be indicative of obstructed flow or inadequate pipe size; in the case that clogs or obstructions are observed, they should be documented and removed to promote unobstructed flow.

□ Operational □ Deficient □ Immediate Action Required

____

____

____

3.2 Inspect drainage outlets to ensure water is being properly conveyed off the slope. Note any excessive erosion near the outlets.

□ Operational □ Deficient □ Immediate Action Required

____

____

____

4 Subgrade Drainage

4.1 Inspect slope for seepage and/or areas of vegetation known to thrive in saturated conditions. Examples of these plants in southwestern PA include, but are not limited to cattails, Japanese knotweed, skunk cabbage, and briars.

□ Acceptable □ Deficient □ Immediate Action Required

____

____

____

5 Surface Features

5.1 Inspect containment structures (i.e., slide fences, rockfall fences, catchment walls); ensure they are clear of material buildup.

□ Acceptable □ Deficient □ Immediate Action Required

5.2 Inspect slope for areas exhibiting bare soil, evidence of cracked, rutted, or damaged slope surfaces.

□ Acceptable □ Deficient □ Immediate Action Required

5.3 Inspect slope for areas exhibiting depressions or areas of ponded water.

□ Acceptable □ Deficient □ Immediate Action Required

5.4 Inspect slope for shallow/surficial slides/failures or visible surface erosion. Note any areas of excessively hummocky (i.e. lumpy) ground or curved trees.

□ Acceptable □ Deficient □ Immediate Action Required

5.5 Inspect site infrastructure for structural irregularities such as tension cracks in pavement, leaning or sagging guiderails, and/or wall cracks in nearby structures.

☐ Acceptable ☐ Deficient ☐ Immediate Action Required

5.6 Inspect toe of slope for visible material loss and note the cause (manmade, drainage channel erosion, or natural waterway scour).

☐ Acceptable ☐ Deficient ☐ Immediate Action Required

5.7 Inspect crest of slope and note any new (or unapproved) surcharge loading.

☐ Acceptable ☐ Deficient ☐ Immediate Action Required

6 **Photograph significant features and include a sketch of the site below (note direction in sketch and include the locations and directions of photos taken).**

APPENDIX B

DESIGN EXAMPLE

Unstable Slope is Identified – Time to Respond (Section B.1)

Site Reconnaissance (Section B.3)

- Complete Field Checklist
- Identify scope and urgency of project
- Identify key geologic and topographic features
- Identify site constraints such as existing infrastructure, utilities, proximity to ROW/property line, etc.
- Determine likely mode of failure

Desktop Study (Section B.2)

- Review available geologic, historic, landslide, mining and topographic information
- Identify any presence of problematic geologic units or colluvium (historic slides)
- Review historic aerial photographs to document history of slope movement
- Request maintenance records if available
- Identify sensitive features in proximity to slide

Subsurface Investigation & Laboratory Testing (Sections B.4 and B.5)

- Prepare/execute an Exploration Plan
- Drill test borings and excavate test pits to characterize subsurface conditions and assess the extent of the slide mass
- Perform a laboratory investigation to aid in classification of soils encountered and determine engineering properties for analysis and design

Instrumentation and Monitoring (Sections B.6)

- Depending on the urgency or impact of the unstable slope, this may be performed prior to the subsurface investigation
- Install instrumentation to monitor piezometric conditions and/or slope movement

Analysis and Design (Sections B.8)

- Use data obtained to develop subsurface section(s), define the extent of the slide mass and mode of instability, and assess rate and magnitude of movement
- Perform engineering analyses and conduct a detailed alternatives analysis
- Complete design and prepare construction documents

Identify Preferred Solution(s) (Sections B.7 and B.9)

- Select the preferred alternative to repair/stabilize the slope with the client considering: Effectiveness; Acceptable Risk; Economic Constraints; Impact to the public and environment; and Time constraints (e.g., urgency)

Construction (Section B.10)

- Execute plans to mitigate repair/stabilize landslide
- Monitor the site closely to ensure construction activities are not triggering additional slope movement
- Ensure surface water and groundwater are managed during construction

Post Construction Monitoring (Section B.11)

- Use instrumentation and/or regular site visits to verify successful execution to stabilize slope
- Confirm implemented stabilization or repair is performing as designed

Design Example

B.1 Time to Respond

Slope movement from a deep-seated landslide occurred in a side-hill embankment (i.e., fill slope) that was threatening operations that were critical for the client.

Best Practice Concept. Crack sealing is an effective means of reducing water infiltration into tension cracks and pore water pressure within the unstable soil mass. See Chapter 8 for further detail on maintenance procedures.

The client first observed evidence of slope movement three years after the construction of the embankment; maintenance and operations staff had recorded a tension crack near some of their operating equipment during a routine maintenance inspection. The maintenance crew had attempted to seal the tension crack at the ground surface with Portland cement grout to reduce surface water infiltration; however, the tension crack continued to widen and re-appear.

Photograph 1. Observed Tension Crack

Based on the observed surficial movement, slope monitoring was implemented by the client; however, continued slope movement was measured as part of the slope monitoring program.

Due to concerns about continued slope movement and threat to operations, the client engaged the practitioner to assess the active landslide, conduct a subsurface investigation, and develop construction plans to stabilize the embankment slope.

B.2 Desktop Study

Purpose/Problem Statement

Review available mapping to assess the geologic setting and review available historic aerial photography to inform the practitioner of the site history.

Given

- Site location and general area of observed instability (provided by the client).
- Project History Report (provided by the client).

Assumptions

Based on prior study reports, it is understood that the side-hill embankment was benched into existing soils; the site was regraded and existing soil was used to construct the side-hill embankment.

References

(exact mapping/location not listed for confidentiality purposes; refer to Chapter 3 for links to these documents for various counties within the region.)

- USGS Topographic Mapping, 7.5-minute quadrangle
- Web Soil Survey
- Bedrock Geology Map
- Stratigraphic Column
- Structure Contour Map
- Mine Map
- Landslide Susceptibility Map
- Shaded Relief (i.e., hillshade) Orthoimages
- Historical Aerial Photographs (via Google Earth)

Methodology

Review mapping to identify site features that may be contributing to instability at the project site.

Result

The following conclusions were made based on the available mapping and site data.

- Soils at the project site have been significantly modified as a result of the original construction of the side-hill embankment.
- Based on surficial soil data that was obtained from the Natural Resources Conservation Service (NRCS) Web Soil Survey (WSS), native soils at the site are from the Gilpin and Guernsey series; the Guernsey series in this area is noted to have formed from colluvial parent material and are prone to slippage.
    - Based on previous reports (by others), it is understood that the side-hill embankment was possibly benched into colluvial soil; most of these soils are anticipated to have been removed from below the side-hill embankment and reincorporated as engineered fill.

- Bedrock beneath the project site consists of the Permian and Pennsylvanian-aged Dunkard Group.
    - Claystone, shale, and siltstone associated with the Dunkard Group are known to weather into weak soils that are subject to deterioration when exposed to moisture.
    - Coal and limestone beds, and their associated underclay(s), frequently act as water-bearing units that are associated with seeps and springs where outcropping (e.g., down-dip) along hillsides.
    - The Waynesburg coal bed is believed to underlie the project site.
- The Waynesburg coal is not shown to have been mined near the project site.
- Based on structure contour maps, bedrock at the site is believed to dip to the southeast at approximately 0.4 degrees (e.g., towards the slope).
- Several combination, old, and active landslides have occurred in the vicinity of the project site, but none within the project limits. One "combination landslide" is shown to have occurred just northwest of the project limits. One active or recently active landslide is shown to have occurred just east of the project limits.
- Historically wet soils at the toe of slope (indicated by darker-colored features) and a previous landslide near the toe of slope (circa 2004, before placement of the embankment) were observed during the review of historic aerial photographs.

> Best Practice Concept. Identification of landslide-prone geologic units, historic landslide activity, and pertinent aerial features at the site are essential to forming an initial hypothesis to inform the site reconnaissance and surface investigation. See Chapters 3 and 4 for more detail.

Additionally, the Project History Report for the site was reviewed; major takeaways include:

- Construction of the embankment was completed in May 2014.
- A large crack in the southeast corner of the side-hill embankment was observed in September 2016 (see Section B.1.1).
- Inclinometers were installed in December 2017 to monitor the southeast fill slope movement; data from the piezometers was provided as part of the report (see Exhibits 2-1 and 2-2).
- It was concluded that the southeastern fill slope is still moving, albeit at a slow rate. The movement is deep-seated and will continue to be monitored for changes in the rate of movement.

B.3 Site Reconnaissance

Purpose/Problem Statement

Perform a project kickoff site visit (scoping visit) to assess pertinent site features and the extent of the slide mass. A detailed site reconnaissance was not warranted at this site due to the availability of a detailed site history report (provided by the client), access to site construction plans, and existing inclinometer data.

Given/References

- Project History Report (provided by the client), see Section B.1.2.

Result

During the scoping visit, a cursory site reconnaissance was made, including discernment of the general extent of the slide mass and other surface features such as seepage, soil bulging, and tension cracks. An abbreviated slope movement field visit checklist was completed during the scoping visit (see Exhibit 3-1).

A preliminary subsurface section was derived to include the existing subsurface conditions as reported by the site history report, available inclinometer data, and site reconnaissance observations (see Exhibit 3-2).

> Best Practice Concept. Site reconnaissance efforts are performed to further define the understanding of the site, identify conditions not apparent during the desktop study, and identify site constraints that may affect mitigation design. A field checklist can assist with ensuring all relevant observations are documented. See Section 4.5 for further detail.

Due to the potential consequence of further slope movement (e.g., significant financial loss for the client), additional site visits were conducted periodically throughout the landslide investigation and as needed during design. These visits were made to assess the site, progress of slope movement, and determine if the conditions observed warranted immediate corrective action.

SITE VISIT PHOTOGRAPHS

Photograph 1. Typical Section of Southeastern Slope; inclinometer locations visible

Photograph 2. Embankment Toe, Erosion Rill Looking West

B.4 Subsurface Investigation

Purpose/Problem Statement

Perform a subsurface investigation to obtain supplemental soil data needed for analysis and support the development of mitigation alternatives. Viable mitigation alternatives include a structural option (e.g., tangent pile retaining wall) and an earthwork option (e.g., toe buttress with rock toe key).

Existing boring data consists of two borings (drilled and sampled near the existing inclinometers to obtain supplemental subsurface data) and an additional boring at the toe of the engineered fill (drilled and sampled beyond the toe of the side-hill embankment, pre-construction). These borings were terminated approximately 10 to 13 feet below the top of rock; cored bedrock consisted of claystone and shale.

The supplemental subsurface investigation was proposed to provide:

- Additional detail to develop an accurate subsurface section along the slope,
- Determination of bearing strata for the tangent pile option,
- Assessment of the slip plane material properties,
- Assessment of a possible rupture (i.e., slip) plane that extends further downslope from the side-hill embankment toe key,
- Assessment of the mode of failure,
- Extended monitoring of groundwater conditions, and
- Samples for subsequent laboratory testing.

Given/References

- Geotechnical Report (including boring investigation performed before embankment construction)
- USCS logging criteria
- PennDOT Publication 222
- Standard Penetration Test (SPT) Sampling (ASTM D1586)

Assumptions

The site is accessible.

Methodology

Two additional test borings were drilled and sampled to enhance subsurface characterization, fill in data gaps in the subsurface section, and complement the existing boring data that was provided by the client.

Borings were drilled and sampled through the soil overburden generally in accordance with PennDOT Publication 222 Section 202. Representative soil samples were completed at generally 1.5-foot intervals (i.e., continuous sampling) and Standard Penetration Tests (SPTs) were obtained generally in accordance with ASTM

> Best Practice Concept. Boring placement should consider placement at the crown, mid-slope, and near the toe of landslide mass at a minimum in order to develop a representative subsurface section; boring placement should also consider evaluation of in place material relative to the sliding mass. Depending on the size of the slide mass multiple subsurface sections may be required. See Sections 4.6 and 4.7 for further detail.

D1586. Hand penetrometer readings were obtained for fine-grained soil samples to estimate undrained shear strength. Rock coring was performed using an NQ2 double-tube wire line core barrel with a diamond-impregnated bit. See Exhibit 4-1 for the boring plan.

Best Practice Concept. Sampling at a continuous interval (i.e., SPTs at 1.5-foot center to center) is essential for landslide investigations to assess the soil moisture profile and identify potential slip planes. See Section 4.6 for further detail.

Below the toe of slope, test pits were excavated to characterize the slide mass (i.e. colluvium), identify if the slip plane extended beyond the toe of the side-hill embankment slope (consistent with the 2004 landslide area, see Section B.1.2), and provide supplemental data about the depth, type, and condition of weathered rock. Test pits were considered favorable in this area due to the shallow depth to the top of rock and the opportunity to visually inspect the soil profile. Test pits were completed in slots (no wider than the bucket) using conventional excavation methods. Excavated material, as well as visible in-situ soils, were visually classified and photographed by a geologist. Relatively undisturbed Shelby tube samples of the slip plane material were obtained for subsequent shear strength testing in the laboratory. See Exhibit 4-1 for the boring plan.

Best Practice Concept. Where test pits are preferred, it is important to limit the excavation to "slots" in the direction of slope movement in order to minimize risk of accelerating slope movement. See Section 4.6 for further detail.

Detailed notes regarding evidence of the slip plane, evidence of seeps, and groundwater observations were recorded for the borings and the test pits.

Additional test pits were conducted at onsite stockpile areas to collect samples for laboratory testing and determine if suitable borrow material is available onsite.

Execution

Two test borings were drilled in alignment with the existing borings to further refine the subsurface section. The test borings were drilled at the approximate crest and toe of the pre-existing embankment slope. The upslope boring (MB-102) was drilled near an existing boring (I-1) and augered to the top of rock to confirm existing boring data. For the downslope boring (MB-101), continuous SPT samples were collected and twenty feet of bedrock was cored. Both borings were backfilled with tremie-placed grout upon completion.

Best Practice Concept. It is important to tremie grout the boreholes upon completion. Tremie grouting ensures that no voids or depressions form at the borehole locations which could provide additional pathways for water to infiltrate into the slope. See Section 4.6 for further detail.

Standpipe piezometers were installed in the test borings for subsequent groundwater monitoring. The piezometers were installed generally in accordance with PennDOT Publication 222 Subchapter 5E Section 206.

Seven test pits were completed near the existing toe bulge for the landslide mass (see Exhibit 4-1) to depths ranging from 4.7 to 10.0 feet; the test pits were generally terminated upon bucket refusal in weathered rock.

Five additional test pits were completed at the existing stockpile areas; bulk samples were collected.

Result

Subsurface conditions encountered generally included fill, underlain by colluvium, residuum (e.g., bedrock that has weathered to a soil-like state), and deeper bedrock; bedrock encountered consisted of claystone and sandstone.

Slickensides (e.g., slip plane) were identified in the test pits; the material was logged in the field as gray lean to fat clay. Several Shelby tube samples within the depth interval of the observed slip plane were collected for laboratory testing.

The depth to the top of rock was established.

Subsurface Section A-A was developed (see Exhibit 4-2). A high level of detail was able to be obtained using data from the four test borings and two test pits, as well as monitoring data from the piezometers and existing inclinometers.

B.5 Laboratory Testing

Purpose/Problem Statement

Perform laboratory testing on select soil and rock samples collected during the subsurface investigation to determine shear strength parameters, index properties, moisture profile (e.g., piezometric conditions), and unconfined compressive strength of rock for analysis and design.

References

- ASTM Test Methods
- USACE EM1110-2-1906

Methodology and Execution

The following tests were completed in the laboratory:

- Moisture content (ASTM D2216) was determined to develop a detailed moisture profile and assist with the determination of the phreatic surface within the slide mass.
- Soil Classification (ASTM D2487) was completed to confirm the field descriptions and obtain Atterberg limits for parameter correlations and assessment of the Liquidity Index.
  - Note: classification included Sieve Analysis (ASTM D6913), Hydrometer Analysis (ASTM D422), and Multi-Point Atterberg Limits (ASTM D4318).
- Direct shear, peak shear strength (ASTM D3080) was performed to obtain the long-term shear strength parameters for the colluvium.
  - Direct shear tests were performed on colluvial soil that was retrieved from the Shelby tube samples. Normal loads specified for testing ranged from 0.25 to 3.0 tons per square foot (tsf) to capture the range of effective overburden pressure involved.
- Reverse Direct Shear, residual shear strength (COE EM1110-2-1906) was performed to obtain the residual shear strength parameters for the slip plane material.
  - Reverse direct shear tests were performed on colluvial soil that was extracted from the Shelby tubes. Normal loads specified for testing ranged from 0.25 to 3.0 tsf to capture the range of effective overburden pressure involved.
- Shelby tube unit weight (ASTM D7263)

Best Practice Concept. Moisture content should be performed with each Atterberg Limit test to complement determination of index properties. Due to the low cost and high value of moisture content results along the soil profile, a continuous moisture profile within the soils of interest add valuable for analysis and design. See Section 7.3 for further detail.

Best Practice Concept. Site specific testing criteria must be defined for strength testing. See Section 7.4 for further detail.

Best Practice Concept. Where significant deformation has occurred, peak shear strengths should not be used for design; it is important to include residual shear strength parameters for the material within the slip plane(s). See Section 7.6 for further detail.

- Specific gravity (ASTM D854)
- Soil Loss on Ignition (ASTM D2974) was performed on potential borrow stockpile material to estimate organic content and determine if that material was suitable for re-use as embankment fill.
- Unconfined compressive strength of rock (ASTM D7012, Method C) was performed to estimate the strength of the bedrock onsite for drilled shaft design and estimate the shear strength of the rock mass.

Result

Soil classification and shear strength (peak and residual) data were obtained for use in parameter development.

The materials observed within the test pits at both the north and southwest stockpiles generally consisted of lean clay (CL) and clayey sand (SC) with minor amounts of larger-sized rock fragments. The moisture content of the soils typically exceeded the Plastic Limit by approximately 5% to 6%.

The Loss on Ignition testing, used as a rough approximation of the organic content of the soil, did not reveal any excessive organic material (less than 3.5%) within the sampled soil materials from the soil stockpiles.

B.6 Instrumentation and Monitoring

Purpose/Problem Statement

Perform continuous slope monitoring to track and assess the rate of slope movement.

Perform a field topographic survey, both of the immediate side-hill embankment and the slope area that extends further downslope from the existing embankment toe key to enhance the historic topographic mapping that was provided by the client.

Collect piezometer data to support analysis and final design.

Given

The client has provided historical inclinometer data and monthly reading updates for the two existing inclinometers at the project site.

Due to inclinometer data and visual assessment of significant slope deterioration reported by the client, emergency slope unloading consisting of the excavation of soils at the slope crest was recently performed due to the risk of compromising critical operations.

Slope movement is isolated to the southeastern slope.

Piezometers were installed at the boring locations during the subsurface investigation.

References

- Instrumentation Manuals
- PennDOT Publication 222
- Federal Geographic Data Committee (FDGC) standards for Second Order Class II differential closed-level loop survey

Methodology and Execution

The topographic survey was performed using static LiDAR and conventional survey methods. Existing benchmarks and control monuments were located to verify project survey control.

Considering the existing inclinometer and piezometer data, slope monitoring activities consisted of weekly surface deformation monitoring and crack measurements to track the relative rate of movement while minimizing instrumentation costs.

> Best Practice Concept. Monitoring instrumentation should be tailored to the site-specific needs and budget of each project; these can vary widely. See Sections 6.2 and 6.4 for further detail on instrumentation type, function, and estimated cost.

During the field topographic survey, six (6) surface monitoring points (MP-1 through MP-6) were installed; these monitoring points were positioned near the top of slope, mid-slope, and the toe of slope (see Exhibit 6-1). Surface deformation monitoring was performed using a differential closed-level loop survey. The differential level surveys were tied to three benchmarks that were located outside the apparent limit of slope movement to identify possible survey outliers and provide assurance that the survey benchmarks were not compromised due to slope movement. Weekly measurements of the monitoring points were collected to establish the relative rate and extent of slope movement. Based on the difference between readings, approximate horizontal and vertical movement of the monitoring points was determined.

Crack measurements were performed using relative measurements at set control points. Two control points were set using sand-filled bags along the existing crack at the crown of the landslide. Relative measurements of these points from the vertical and horizontal, as well as the width and visible length (i.e., persistence) of the crack, were recorded. Measurements were used to track tension crack-growth and corroborate slope monitoring data by comparing crack measurements to the slope movement observed as part of the surface deformation monitoring.

Open standpipe piezometers were installed during the subsurface investigation (see Section B.1.4). Readings were recorded periodically as part of the monitoring program using a water level meter to assess static groundwater levels and potential fluctuations in groundwater.

Result

The results of the surface deformation monitoring confirmed that the slope was still moving in a southeasterly direction. A rate of horizontal movement from about 0.10 to 0.42 inches per day was recorded starting in May 2019 (see Exhibits 6-2 and 6-3). Based on the increased rate of movement, the client decided to perform an emergency short-term stabilization response.

The observed changes in the tension cracks and continued toe roll movement (see Exhibit 6-4) are consistent with the measured overall movement of the slope.

Water levels measured at inclinometers I-1 and I-2, which are extended into the sandstone, indicate a total piezometric head that was higher than what was measured in the soil overburden at Borings MB-101 and MB-102. This is likely indicative of a confined aquifer and elevated head within the sandstone bedrock based on the data obtained (e.g., piezometer data, understanding of local structural geology, fracture stains, and presence of overlying beds of weathered claystone and clayshale).

B.7 Emergency Short Term Stabilization Response

Purpose/Problem Statement

Based on the accelerated deformation rate measured, an emergency response is required to slow the rate of slope movement to allow time to complete final design and implement long-term mitigation measures.

Assumptions

The solution must be implemented quickly to slow the rate of movement and reduce the adverse impact on operations at the top of the slope.

Earthwork disturbance should be undertaken with caution to minimize the risk of accelerating the rate of slope movement; no excavation is permitted near the toe of slope for the emergency response measure(s).

Emergency short-term stabilization response will provide a service life of up to 3-months and will be replaced by long-term mitigation measures after the final design is completed and executed.

Effectiveness will be measured through slope monitoring; a more robust short-term stabilization response may be required if the rate of slope movement does not diminish to a satisfactory rate.

> Best Practice Concept. Tailoring an effective response is contingent upon economic factors, consequences, degree of risk, and the magnitude of loss. See Sections 9.6.7 and 9.6.8 for further detail.

Methodology and Execution

Previous efforts to unload (e.g. excavate) the crest of the slope had previously been performed by the client, but were unsuccessful in reducing slope movement to an acceptable rate; thus reducing the driving forces was not considered for emergency response efforts.

The construction of a temporary stability berm at the toe of slope (e.g., toe berm) was deemed a viable option to enhance overall slope stability by increasing the resisting forces, and therefore decreasing the rate of slope movement. Considering the available onsite fill material identified during the subsurface investigation (Sections B.1.4 and B.1.5), this option was selected based on effectiveness over the assumed service life, cost, and availability of on-site borrow for immediate implementation.

Result

A temporary stability berm was placed using onsite borrow material. The stability berm was able to be constructed within two days; approximately 8 days after the increased rate of movement was recorded.

Continued surface deformation monitoring indicated that the addition of the temporary stability berm resulted in a reduction in the rate of slope movement from about 0.45 to 0.14 inches per day.

B.8 Analysis and Final Design

Purpose/Problem Statement

Identify long-term mitigation measures available to remediate the active landslide and restore the southeastern slope.

Perform geotechnical analyses to assess mitigation alternatives to provide an acceptable factor of safety for long-term performance. Use the summation of data obtained from the previous steps (B.1.1 through B.1.7) to inform the analysis.

Assumptions

The triggering cause of the landslide movement is believed to be excess pore water pressure and weak colluvium present from historic landslide activity.

A minimum calculated factor of safety (FS) of 1.5 is acceptable for the final condition based on the high consequence of slope failure (i.e. low risk tolerance).

> Best Practice Concept. The required minimum factor of safety against global stability must account for the amount of uncertainty in site conditions, risk, and consequence of failure. See Sections 9.6.4 for further detail.

References

- AASHTO LRFD Bridge Design Specifications, 8th Edition
- Hoek-Brown Failure Criterion (2002)

Methodology

A limiting equilibrium slope stability analysis was performed using the Rocscience Slide 2 computer program to determine the minimum calculated factor of safeties for the various cases considered.

The subsurface profile, piezometric conditions, and failure surface, derived from the boring and test pit data in conjunction with inclinometer readings, are as shown on the subsurface section (Exhibit 4-2).

> Best Practice Concept. Where the mode of failure is well-defined, back-calculation of parameters used in conjunction with boring and laboratory data can be an effective means to assess the reasonability of the subsurface model and parameters. See Section 7.6 for further detail.

Residual shear strengths were considered for the colluvial and residual soils, as well as the claystone, within the estimated slip (i.e. rupture) plane.

Performed model calibration by back-calculating the site conditions that were necessary to sustain the movement of the historic landslide and the current active landslide.

No short-term (i.e. temporary excavation) condition was analyzed. To retain earthwork as a viable mitigation option, phased construction and "slot" excavations were utilized considering the slope in 3-dimensions. Since the proposed construction sequence is intended to utilize the 3-dimensional stability of the overall slope, a 2-dimensional slope stability model was not considered to be

> Best Practice Concept. Risk tolerance will be dependent on the client/site-specific requirements and service life of the condition. See Section 7.6 for further detail.

representative. Due to the reasoning stated above, and the short-term duration of the open slot excavations (see Section B.1.9), this was considered as an acceptable risk using engineering judgment. In addition to construction controls, slope monitoring will also be utilized to track and manage the rate of movement during construction. The client was agreeable to this approach.

Execution

Parameter Selection

- Direct shear test results were utilized, along with the subsurface data and published literature for materials in Southwestern Pennsylvania, to estimate soil parameters for the colluvium.
- Where direct shear testing was not available, soil parameters were derived based on boring data, empirical correlations with SPT N-values, hand penetrometer readings, and laboratory test results.
- Considering the source of the proposed fill material was not able to be controlled (limited to the use of onsite stockpile material), the effective friction angle was conservatively modeled as 28 degrees.
- Bedrock parameters were based on the shear strength of rock mass and assessment of rock discontinuities in accordance with the Hoek-Brown Failure Criterion (2002) and published literature for materials in Southwestern PA.

Earthwork Option, Rock Toe Key with Toe Buttress

I. Slope Stability Model Calibration

Slope stability model calibration was performed using the current site conditions and the pre-construction site conditions for the 2004 landslide. A targeted FS of 1.0 was obtained to reflect the conditions present in order to mobilize earth movement. Janbu's method of analysis with a fully specified failure surface through the estimated rupture plane was utilized for analysis.

> Best Practice Concept. Back-analysis (i.e., model calibration) can be an effective means to refine the parameters derived from the data collected. Calibrating the model before analysis of mitigation alternatives can also serve as a baseline for the minimum required FS to quantify improvements made to existing stability. See Sections 7.6 and 10.3 for further detail.

- Step 1: A model calibration was performed using the current site conditions (after the emergency slope unloading, see Section B.1.6). Surface deformation monitoring determined that there was continued rate of movement therefore a target FS of 1 was considered. The soil parameters obtained from the parameter selection were further refined to achieve the target FS of 1.0. See Exhibit 8-1.

- Step 2: An additional analytical model was created to confirm the calibration obtained in the previous step using the ground surface near 2004 (pre-construction, pre-historic landslide). The piezometric surface for this analytical model was slightly raised near the toe to account for the seepage observed as part of the desktop study (Section B.1.2) before slope failure in 2004. A FS of 1.0 was achieved with the parameters calculated in Step 1 and the revised piezometric surface. See Exhibit 8-2.

II. Slope Stability Analysis

Slope stability analyses were performed using the refined soil parameters from the Slope Stability Model Calibration to determine the toe key and toe buttress geometry that is required to result in a minimum calculated FS of 1.5.

The bottom elevation of the rock toe key was established on competent sandstone to provide a stable base, disrupt the landslide rupture plane, and provide a positive drainage outlet for groundwater present in the lower portion of the slope. A 360 psf equivalent soil live load surcharge was modeled at the top of slope to account for operations equipment and traffic at the site.

- Step 1: The base width of the rock toe key and the vertical extent of the rock required above the toe of slope were iteratively adjusted to obtain a minimum FS of 1.5. Janbu's method of analysis with a fully specified failure surface passing through the rupture plane of the deep-seated landslide was used for this analysis considering effective stress soil parameters (see Exhibit 8-3).
- Step 2: Based on the established rock toe key dimensions established during Step 1; a compacted embankment fill toe buttress was modeled at a two and one-half horizontal to one vertical (2.5H:1V) slope and iteratively increased in elevation to result in a minimum FS of 1.5. Additional rupture planes to include global (top of embankment to rock toe) and local failures (within the toe buttress) were considered. Both circular and block-type failures were analyzed using effective stress soil parameters. For failures within the toe buttress, total stress soil parameters were also analyzed to account for the end of construction condition under the new buttress loading. See Exhibits 8-4 through 8-7 for select slope stability output plots.

Structural Option, Tangent Pile Wall

A tangent pile wall was determined to be a feasible alternative for the stabilization of the southeast slope; however, this alternative was not selected due to the high relative cost compared to the earthwork option. Since the earthwork option was determined to be feasible, no detailed analysis or design was performed for the structural option.

Result

Based on the analyses performed, a toe buttress and a rock toe key near the toe of the side-hill embankment will improve the stability of the southeast slope and arrest slope movement by the addition of resisting forces to counteract existing driving forces. The toe key and toe buttress should be utilized along with horizontal toe drains to maintain positive relief of porewater pressure from within and beneath the landslide mass.

Construction of a 2.5H:1V buttress consisting of compacted embankment material supported by a durable rock toe key founded on competent sandstone near the toe of the existing slope was determined to be feasible within the available Limit of Disturbance (L/D). A bottom width of 22 feet for the rock toe key and a top elevation of 1240.0 feet for the toe buttress was determined to be required to achieve a minimum calculated FS of 1.5 for all of the cases analyzed.

B.9 Identify Preferred Long-Term Mitigation Option

Purpose/Problem Statement

Provide a best-value solution to mitigate the active landslide at the southeast slope based on the results of the analyses performed.

Assumptions

The preferred mitigation option must account for time-to-implement, cost, and constructability.

Excavation and site disturbance must be kept to a minimum so as not to initiate a slope failure.

Due to ongoing operations by the client at the top of the embankment slope, over-excavation of the slip plane and slide mass is not feasible.

Methodology

Michael Baker evaluated multiple remedial design alternatives considering efficacy, relative construction cost, constructability, general permitting effort, and potential impact on ongoing site operations.

A tangent pile wall was determined to be a feasible alternative for the stabilization of the southeast slope.

Construction of a toe buttress consisting of compacted embankment material supported by a durable rock toe key founded on competent sandstone near the toe of the existing side-hill embankment was determined to be feasible within the available L/D.

Horizontal drains will be considered as part of the selected mitigation option to provide positive relief of excess pore water pressure from within and beneath the landslide mass.

> Best Practice Concept. It is preferred to take a proactive (versus reactive) approach to mitigate landslides, with due consideration of cost, reasonableness, and effectiveness. Simple steps like improving drainage can reap large benefits. See Section 10.3.2 for further detail.

Execution and Result

Conceptual level cost estimation was completed to compare alternatives:

- Earthwork Option - $1,300,000
  - $475,000; includes rock toe key with toe buttress consisting of about 15,000 cubic yards of fill. Approximately 9,000 cubic yards of fill material were available onsite; foreign (off-site sourced) borrow material was required for rock toe.
  - $200,000; includes an additional allowance for moisture conditioning with hydrated lime due to the potential for elevated moisture in the available fill material onsite.
  - $325,000; includes installation of 3,000 linear feet of horizontal drains
  - Approximately 30% of the total cost was considered as an allowance for miscellaneous items (e.g. E&S controls, clearing, site access, site restoration, etc.).
- Tangent Pile Wall Option - $1,852,500
  - $1,100,000; includes tangent piles socketed in bedrock consisting of 40-ft long, 36-inch diameter drilled shafts with W24x162 steel beams. For estimation purposes, the piles were spaced at 5-ft centers for a total of 57 piles along 280-foot of wall.

  - $325,000; includes installation of 3,000 linear feet of horizontal drains
  - Approximately 30% of the total cost was considered as an allowance for miscellaneous items (e.g. E&S controls, clearing, site access, site restoration, etc.).

The earthwork option provides faster construction time, onsite availability of fill material, and lower relative cost compared to the tangent pile wall option; therefore, the earthwork option including installation of horizontal drains and subsequent construction of the toe buttress and rock toe key was preferred for the final design.

Considering the slope is an active landslide, the following construction controls were derived to limit the risk of slope failure during construction :

- The installation of horizontal drains will be performed before construction of the toe buttress and slope regrading to relieve excess pore pressure, and therefore improve stability, before any excavation occurs.
- Excavation of the rock toe key will be completed in discreet [maximum] 50-foot widths (e.g., slots) to protect the integrity of the existing embankment. The excavated "slots" must be filled before the next slot excavation is attempted. Continuous slope monitoring will be performed to track and assess slope movement during construction.
- The toe buttress will consist of a compacted earthen embankment slope of moisture-conditioned soils (obtained primarily from onsite excavations and soil stockpiles) on a 2.5H:1V slope. The toe buttress will be founded on a rock toe key consisting of imported durable rock that is bearing on competent sandstone. The bottom elevation of the rock toe key was established at top of competent sandstone which was confirmed using test pits as part of the subsurface investigation (Section B.1.4).

B.10 Construction and Construction Monitoring

Purpose/Problem Statement

Plan set was derived based on the conclusions made during the geotechnical analysis and final design.

Perform continuous slope monitoring to assess the slope performed during horizontal drain installation and "slot" excavations.

Perform construction monitoring in the field.

Execution

Plan set for construction was finalized based on the preferred long-term mitigation option. See Exhibit 10-1 for the typical details used for the rock toe key and horizontal drains.

Full-time construction monitoring was performed under the supervision of an engineer. Daily field reports and photographs were submitted daily to track progress and monitor slope conditions

Surface deformation monitoring (see Section B.1.6) continued during construction to monitor slope movement (Exhibit 10-2).

Result

The preferred long-term mitigation solution consisting of horizontal drains with a toe buttress and rock toe key was successfully implemented at the project site.

B.11 Post Construction Monitoring

Purpose/Problem Statement

This monitoring is desired to verify the performance and success of the construction completed to mitigate the deep-seated slope movement and to monitor fluctuations in excess pore pressure.

Given

The existing two inclinometers (I-1 and I-2) have been rendered out of service due to severe distortion of the inclinometer casing from the deep-seated landslide movement. The standpipe piezometers installed during the subsurface investigation (MB-101 and MB-102) were rendered out of service due to site earthwork activities and deep-seated landslide movement.

References

- Instrumentation Manuals
- PennDOT Publication 222

Methodology

The Post Construction Slope Monitoring Plan was implemented after site restoration to monitor the performance of the reconstructed slope.

The instrumentation locations were selected to avoid interference with the horizontal drains and other subsurface drainage features, e.g. embankment bench drains. The inclinometer and piezometer locations were staked out to avoid drainage features before installation to avoid interference during drilling.

Elevated seasonal groundwater levels are typically experienced in southwestern PA through the fall and late winter/early spring seasons, with resultant potential for adverse impact on landslide-prone slopes. Based on this, a minimum of ten (10) monthly slope monitoring events were recommended following the completion of the toe buttress landslide remediation work to validate that the southeast slope has been satisfactorily stabilized. Additionally, the piezometer installation will be furnished with remote monitoring equipment so that continuous readings can be obtained.

Best Practice Concept. Due to the variability of subsurface conditions between the seasons in Southwestern PA, monitoring programs may span multiple seasons with a particular focus on monitoring slope conditions during the wet seasons. Refer to Section 6.1 for further detail.

Execution

Piezometers and inclinometers were installed in accordance with PennDOT Publication 222 Subchapter 5E Sections 206 and 207, respectively.

Three (3) piezometers (MB-P-1, MB-P-2, and MB-P-3) and three (3) inclinometers (MB-I-1, MB-I-2, and MB-I-3) were installed in the reconstructed southeast slope. Each piezometer was installed adjacent to an inclinometer (see Exhibit 11-1).

Best Practice Concept. Inclinometers and piezometers are typically installed in pairs to obtain site-specific data about both existing groundwater conditions and rate/magnitude of movement. Refer to Section 6.2 for further detail.

MB-P-1 and MB-I-1 were installed at the top of slope, MB-P-2 and MB-I-2 were installed on the mid-slope bench, and MB-P-3 and MB-I-3 were installed on the lower slope of the toe buttress. The locations and surface elevations of the installed inclinometers and piezometers were subsequently surveyed.

Inclinometer data was processed using equipment-specific software and used to produce graphical reading plots along the A and B axes. Based on the data provided by the software, data verification checks were performed including the computation of checksums. See Exhibit 11-3 and 11-4 for sample inclinometer output and checksums.

Best Practice Concept. Data verification should be performed for all data collection events to confirm the reliability of the results. Refer to Sections 6.4.2.1 and 6.5 for further detail.

Result

Water levels beneath the southeast embankment slope have decreased slightly at all three piezometers (see Exhibit 11-2). This trend was suggestive of the continued positive performance of the horizontal drains, bench drains, and toe key drain to provide positive drainage and relief of excess pore water pressure from the slope and mitigate the effects of seasonal fluctuation in groundwater.

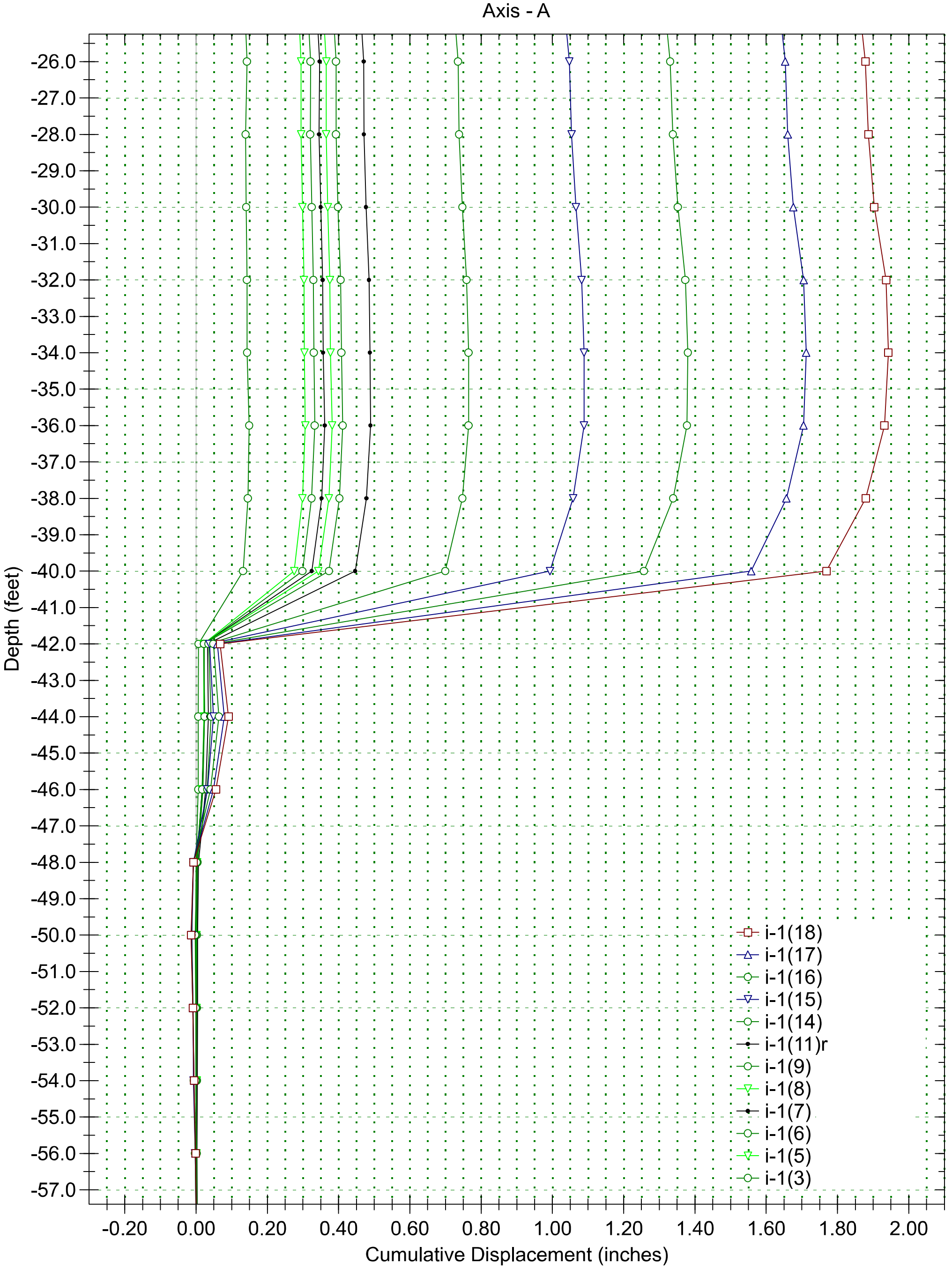
Based on the evaluation of the inclinometer data, the maximum horizontal movement measured over 10 months (0.10 inches at MB-1-2) was considered to be a negligible rate of movement and not suggestive of active landslide movement. Based on these factors, the long-term mitigation measures were considered to be successful, and the southeast embankment slope was considered to be performing satisfactorily.

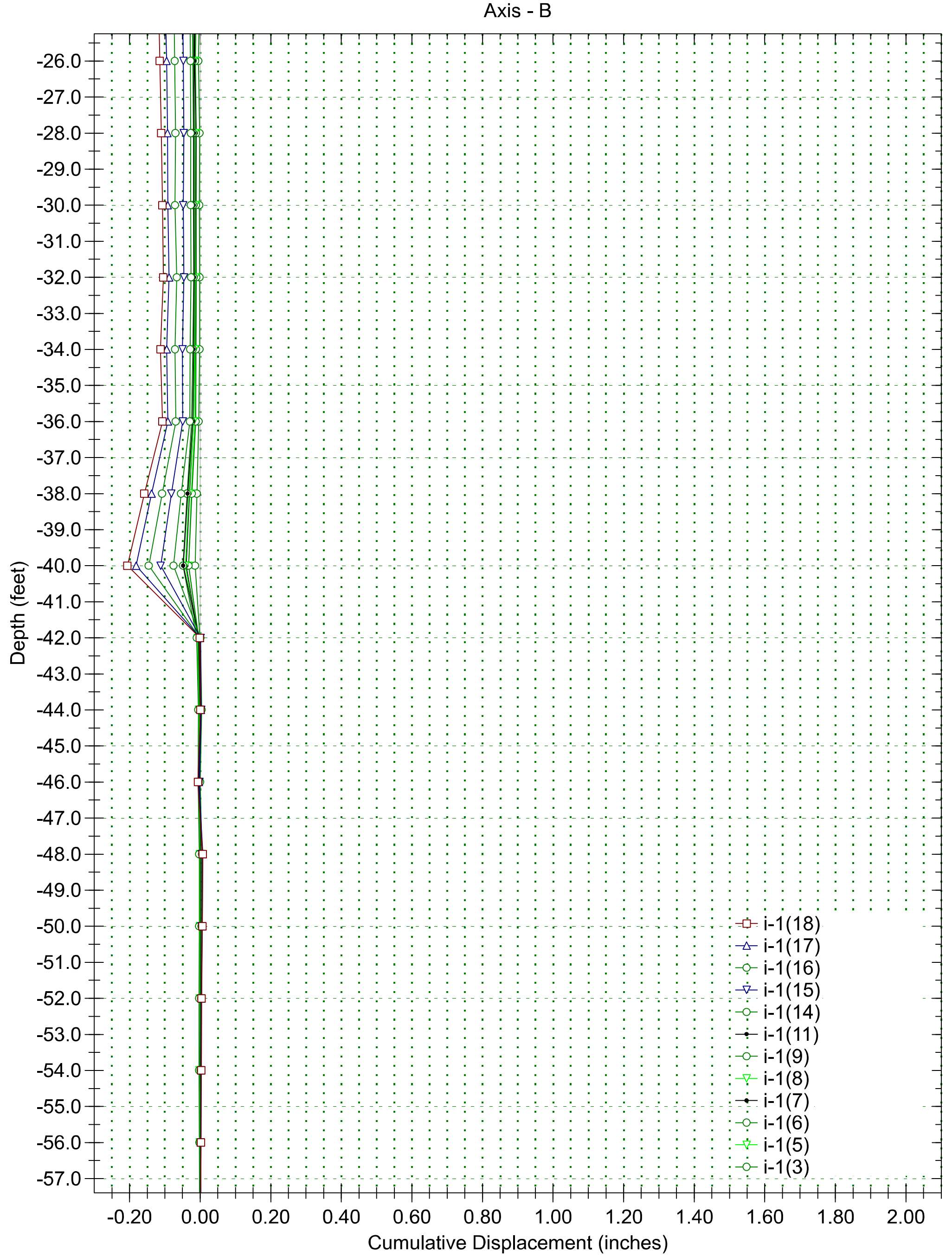
Biannual slope monitoring equipment readings and monthly surface observation were recommended for long-term maintenance.

B.12 Exhibits

CUMULATIVE DISPLACEMENT

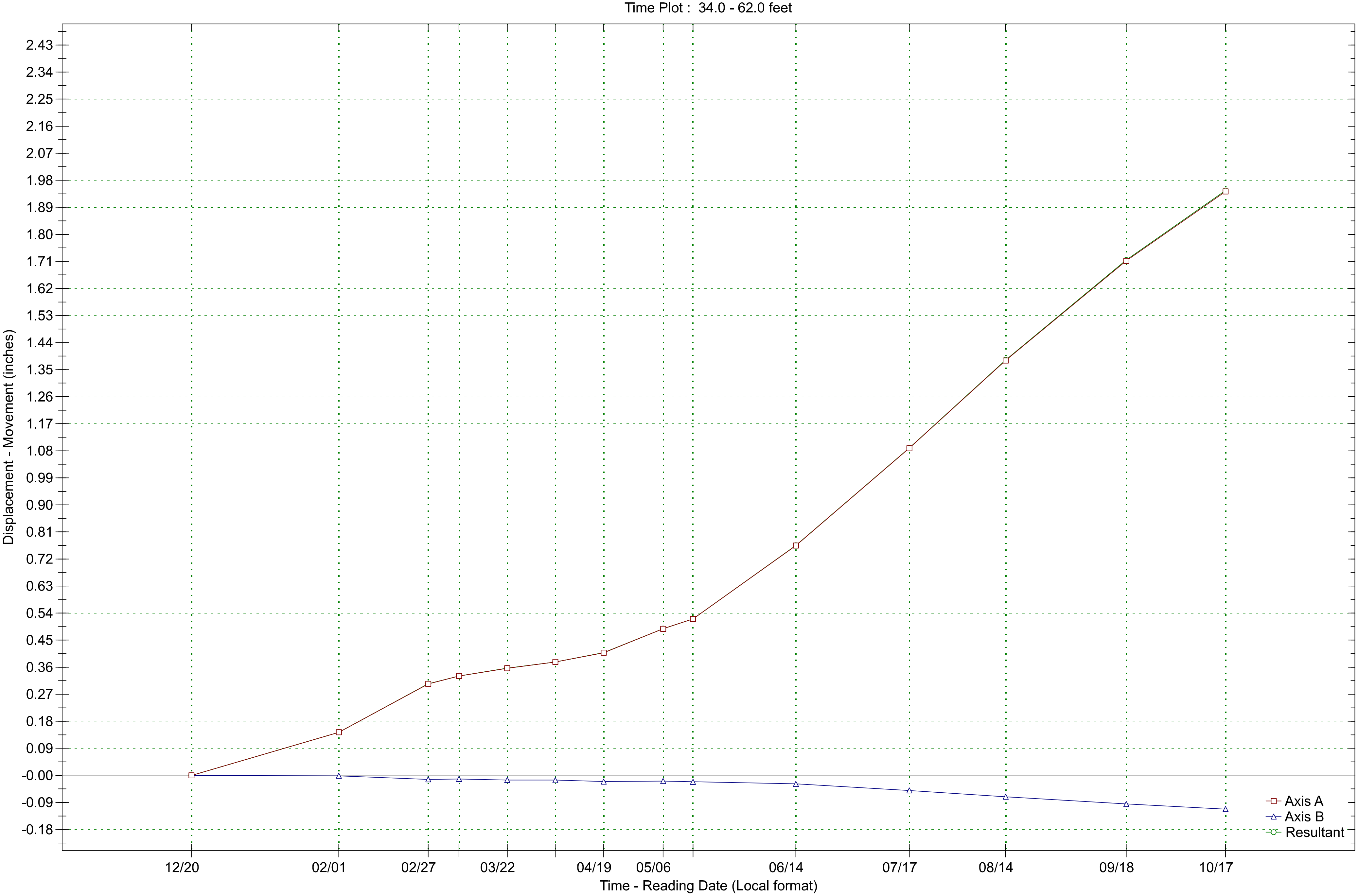
Exhibit 2-1





TIME PLOT
Displacement vs. Time

Exhibit 2-2



ABBREVIATED SLOPE MOVEMENT FIELD VISIT CHECKLIST

CLIENT & PROJECT NAME: ______________________________

COUNTY: ______________ MUNICIPALITY (IES): ______________

ADDRESS: ______________________________

LATITUDE: ______________ LONGITUDE: ______________

DATE OF FIELD VISIT: ______________ WEATHER: ______________

INSPECTOR: ______________________________

PROJECT DESCRIPTION: Southeast slope movement.

____

REASON FOR SITE VISIT: Conduct a general site visit to obtain site observations to supplement the site history documents provided by the client.

____

FIELD OR REMOTE INSPECTION: Field

1 Site Inspection

1.1 Inspect and describe notable surface features within the slide mass and in the areas adjacent to the slide mass (settlement or possible evidence of minesubsidence, benching, hummocky ground, depressions, tension cracks, bulges, tree tilt, etc.).

Toe roll/bulge observed beyond the toe of the sidehill embankment approximately 100 feet to the southeast – this appears to be the limits of landslide movement. Approximately seven lateral tension cracks were observed along the face of the southeast slope. Head scarp (5') and primary tension crack were observed at the top of the slope.

Exhibit 3-1a

1.2 Record the approximate slope ratio and location of the head scarp.

Sidehill embankment is at an approximate 2H:1V; beyond the toe of the sidehill embankment is at an approximate 5H:1V

1.3 Inspect and describe notable surface features within the slide mass and in the areas adjacent to the slide mass (settlement or possible evidence of minesubsidence, benching, hummocky ground, depressions, tension cracks, bulges, tree tilt, etc.).

Hummocky ground noted near toe roll/bulge beyond the toe of the sidehill embankment is at an approximate 5H:1V. Approximately 150 feet southeast of the embankment toe of slope, an over-steepened eroded stream bank was observed. It is plausible that the deep seated slope movement extends toward and involves the eroded stream bank. Stream water may continue to erode away the stream bank.

1.4 Inspect area for any rock outcrops or evident rock types and/or bedding planes.

None

1.5 Note any significant erosion from drainage features (i.e. washout areas, evidence of runoff, scour at toe of the slope from stream etc.)

None

1.6 Note any seepage or free water on the slope observed.

Multiple seeps observed along slope.

1.7 Note previous construction, if any, at the site (widening, relocation, structure replacement, slope mitigation activities, etc.).

Previous slope mitigation efforts evident; detailed documentation of these efforts to be provided by client.

1.8 Note the presence of all buildings/structures, utilities, roadways at site and their proximity to the slope or slide mass.

Critical operations for the client are active at the top of the slope. Additional movement poses a significant risk to the ongoing operations.

Exhibit 3-1b

1.9 Note the effect of slope movement (if any) on the aforementioned buildings/structures, utilities, roadways. Does the slope movement pose any immediate risk to the public?

Additional movement poses an immediate risk to the ongoing operations.

1.10 Note the date of any previous site visits and record any change of conditions from last visit (if applicable).

None.

1.11 Photograph significant features and include a sketch of the site below (note direction in sketch and include the locations and directions of photos taken). Note any conclusion or recommendations for next steps at the site.

See attached.

SITE OBSERVATIONS

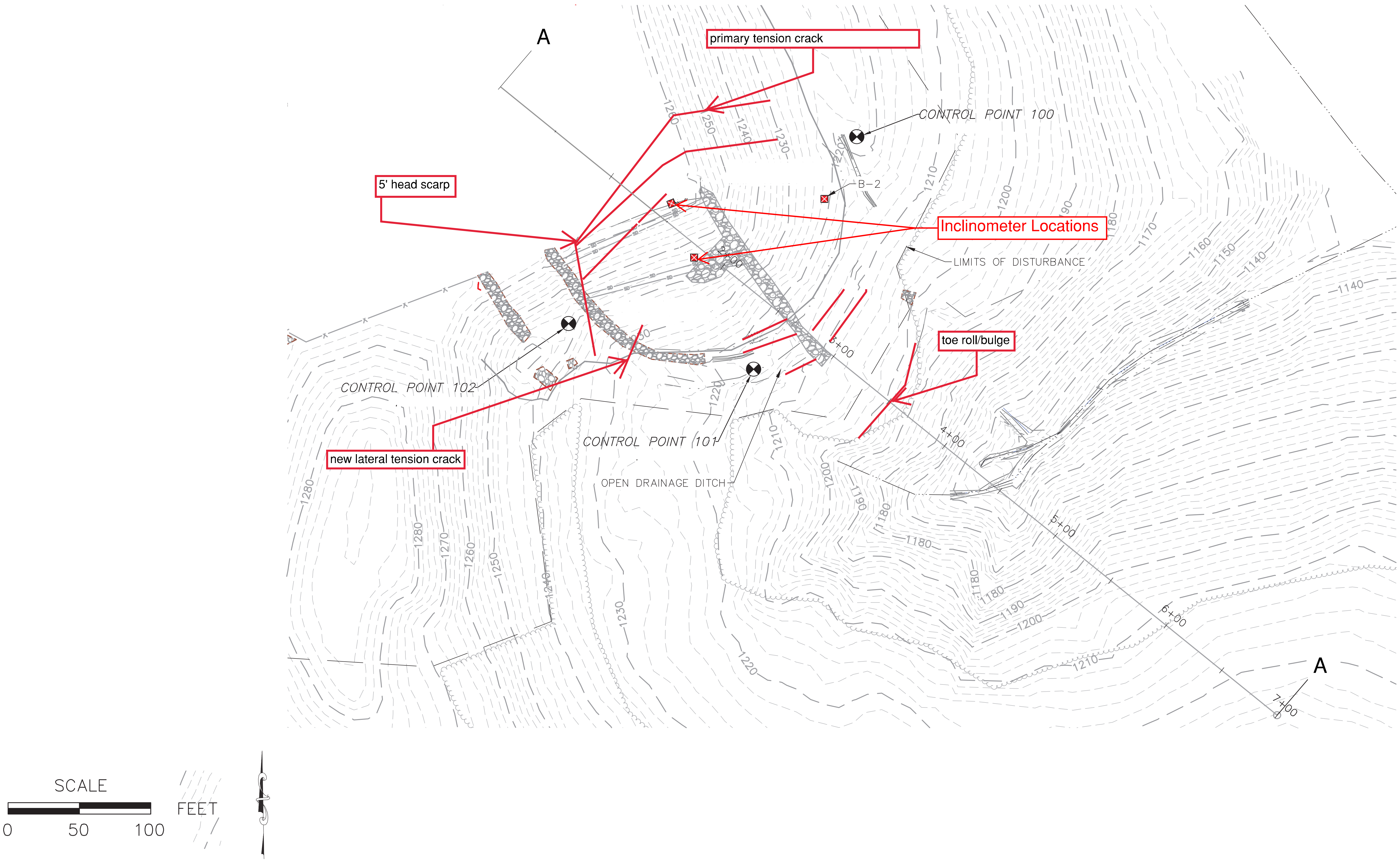
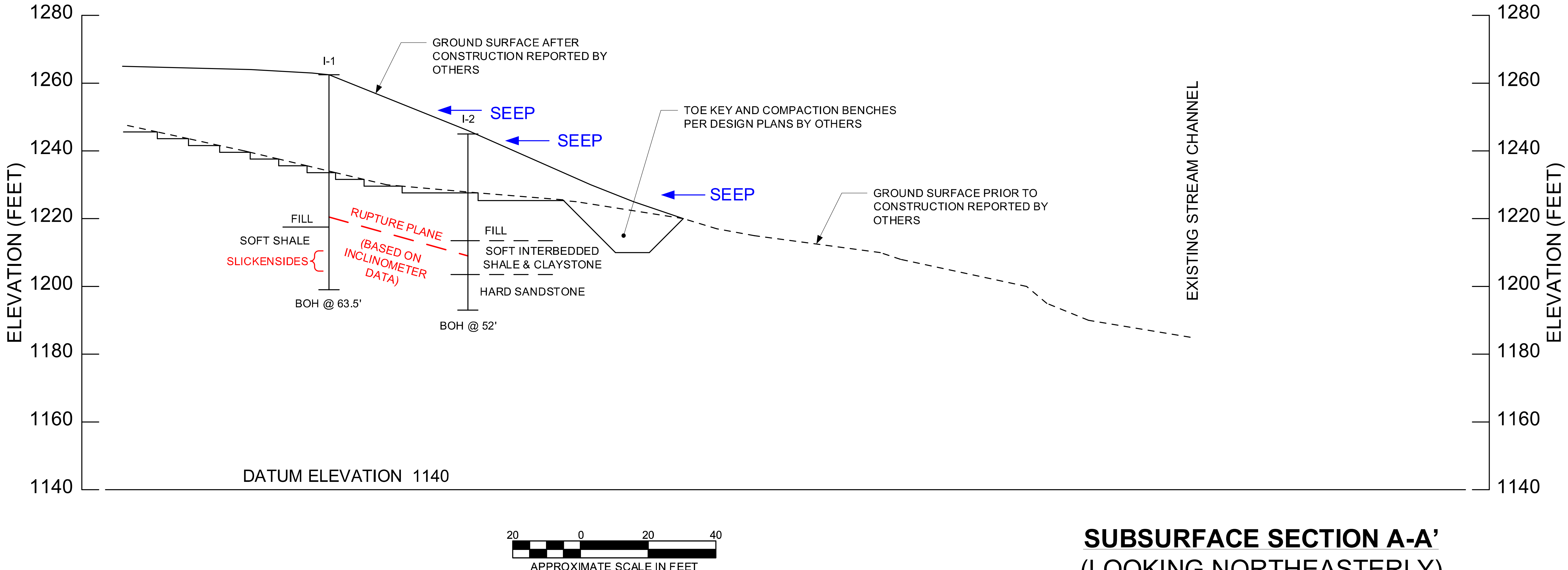


Exhibit 3-1d



SUBSURFACE SECTION A-A'
(LOOKING NORTHEASTERLY)

Exhibit 3-2

BORING PLAN

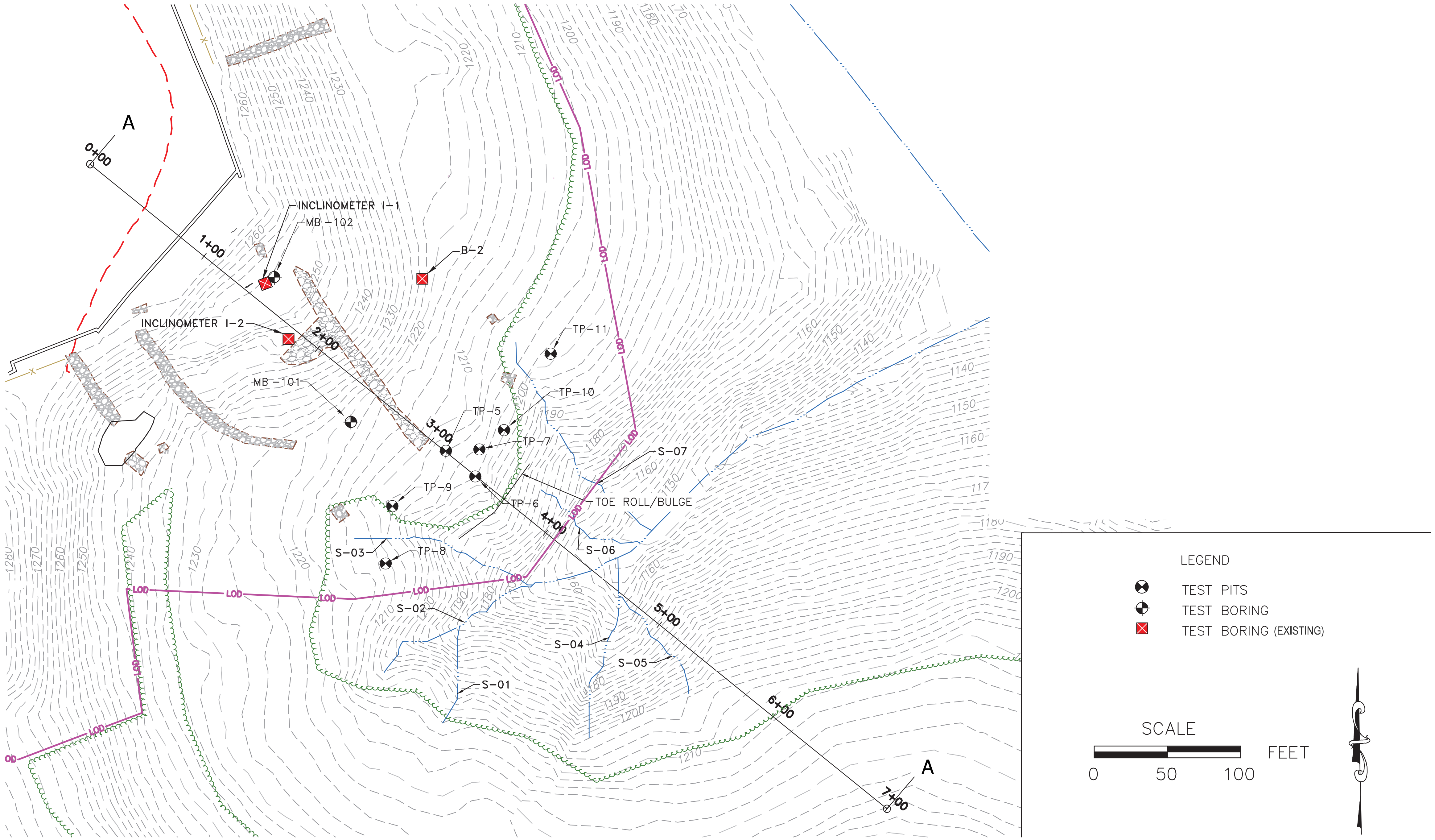
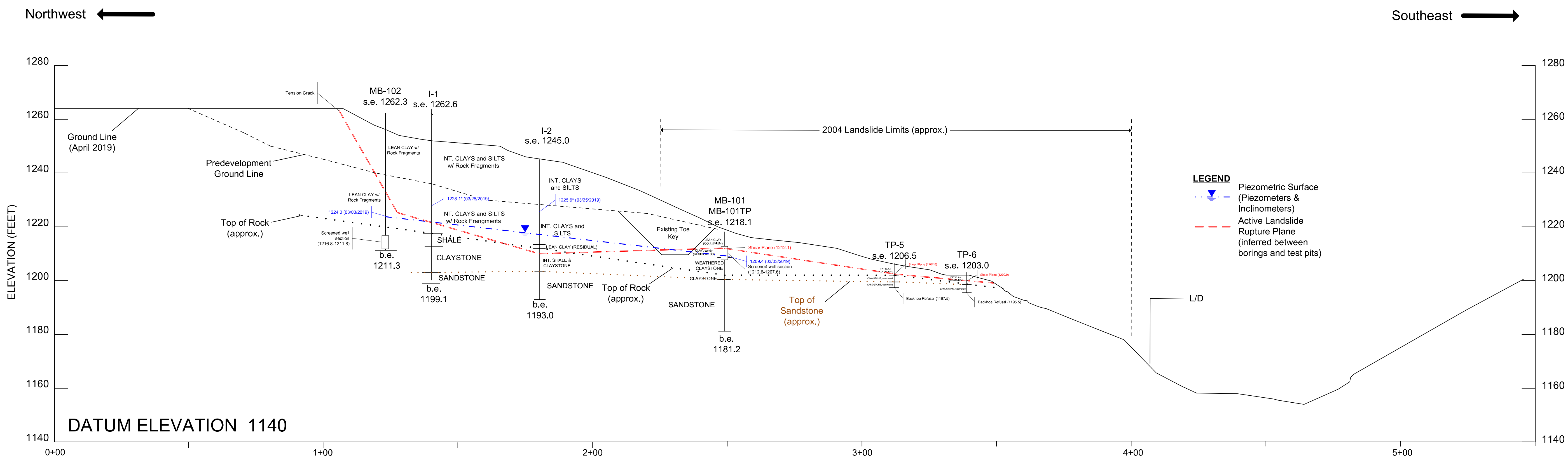


Exhibit 4-1



SE Embankment Active Landslide Subsurface Profile A-A

20 0 20 40
SCALE IN FEET

Measured Water Levels

Boring	Elevation	Date	Remarks
MB-101	1216.2	12/8/2018	After Coring
MB-101	1207.8	12/8/2018	0-hr
MB-101	1209.5	02/25/2019	---
MB-101	1209.4	03/03/2019	---
MB-101	---	03/25/2019	obstructed at 4.9' bgs
MB-101TP	Dry	04/15/2019	seeps at 3.0' bgs
MB-102	1213.3	12/6/2018	0-hr
MB-102	1222.6	12/7/2018	22-hr
MB-102	1224.5	02/25/2019	---
MB-102	1224.0	03/03/2019	---
MB-102	---	03/25/2019	obstructed at 37.9' bgs
I-1	Dry	11/20/2017	Initial
I-1	1255.6	11/20/2017	At Completion
I-1	1228.1*	03/25/2019	---
I-2	Dry	11/21/2017	Initial
I-2	1238.8	11/21/2017	At Completion
I-2	1236.3	11/22/2017	After 24-hrs
I-2	1225.6*	03/25/2019	---
I-2	1223.5	05/20/2019	---
I-2	1224.7	06/08/2019	---
I-2	1224.1	06/17/2019	---

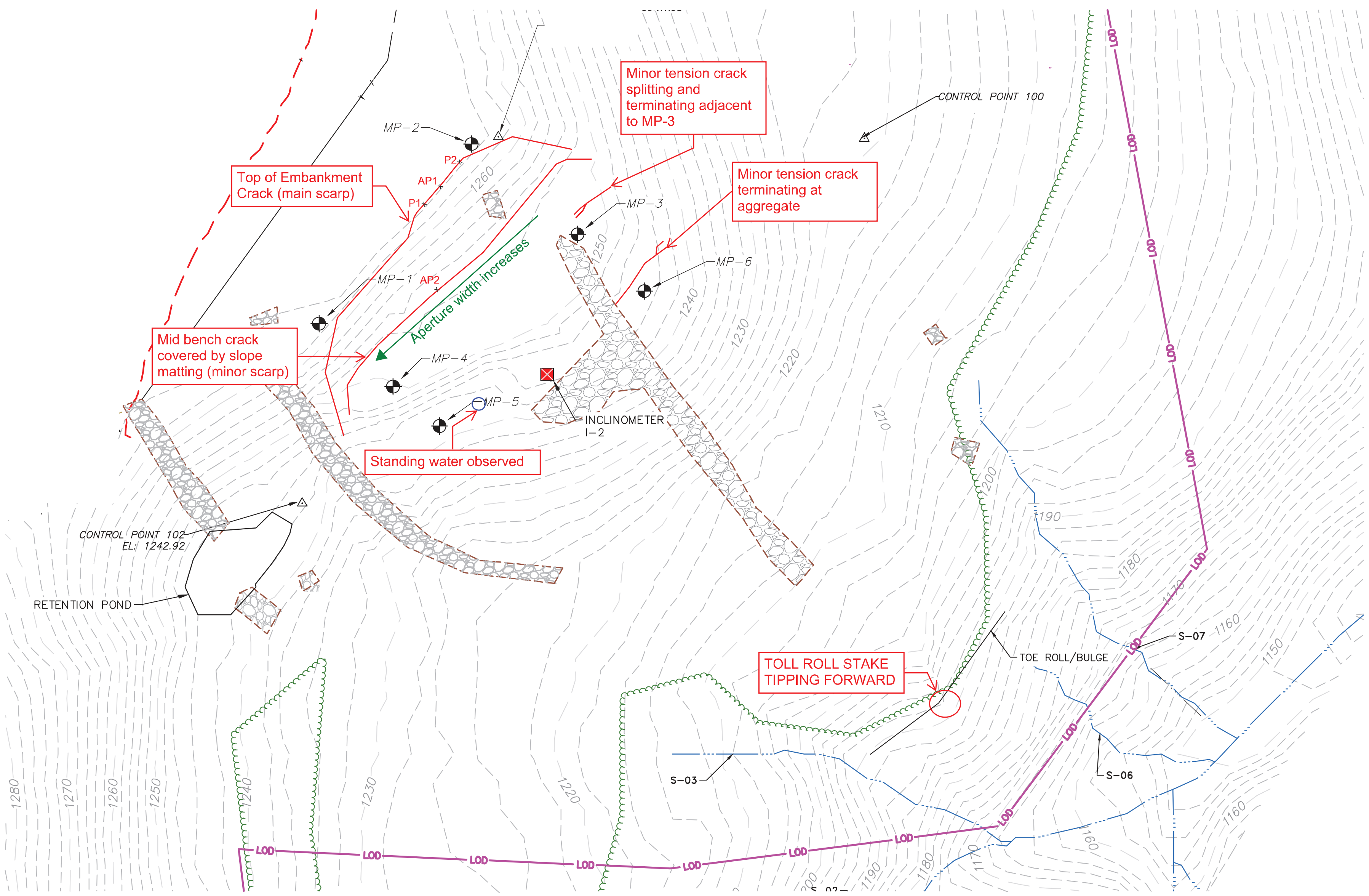
*Water level readings were determined to reflect elevated head within the sandstone bedrock unit; not representative of static piezometric conditions

Inclinometer Rupture Plane Elevations

Boring	Elevation	Depth (ft)
I-1	1221.6	41.0
I-2	1210.0	35.0

Exhibit 4-2

SURFACE MONITORING PLAN



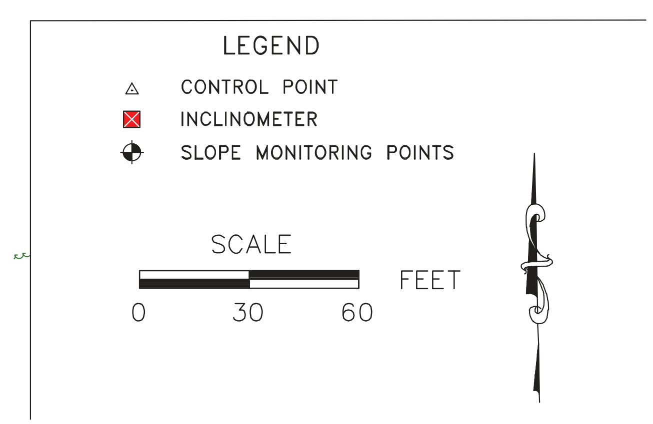
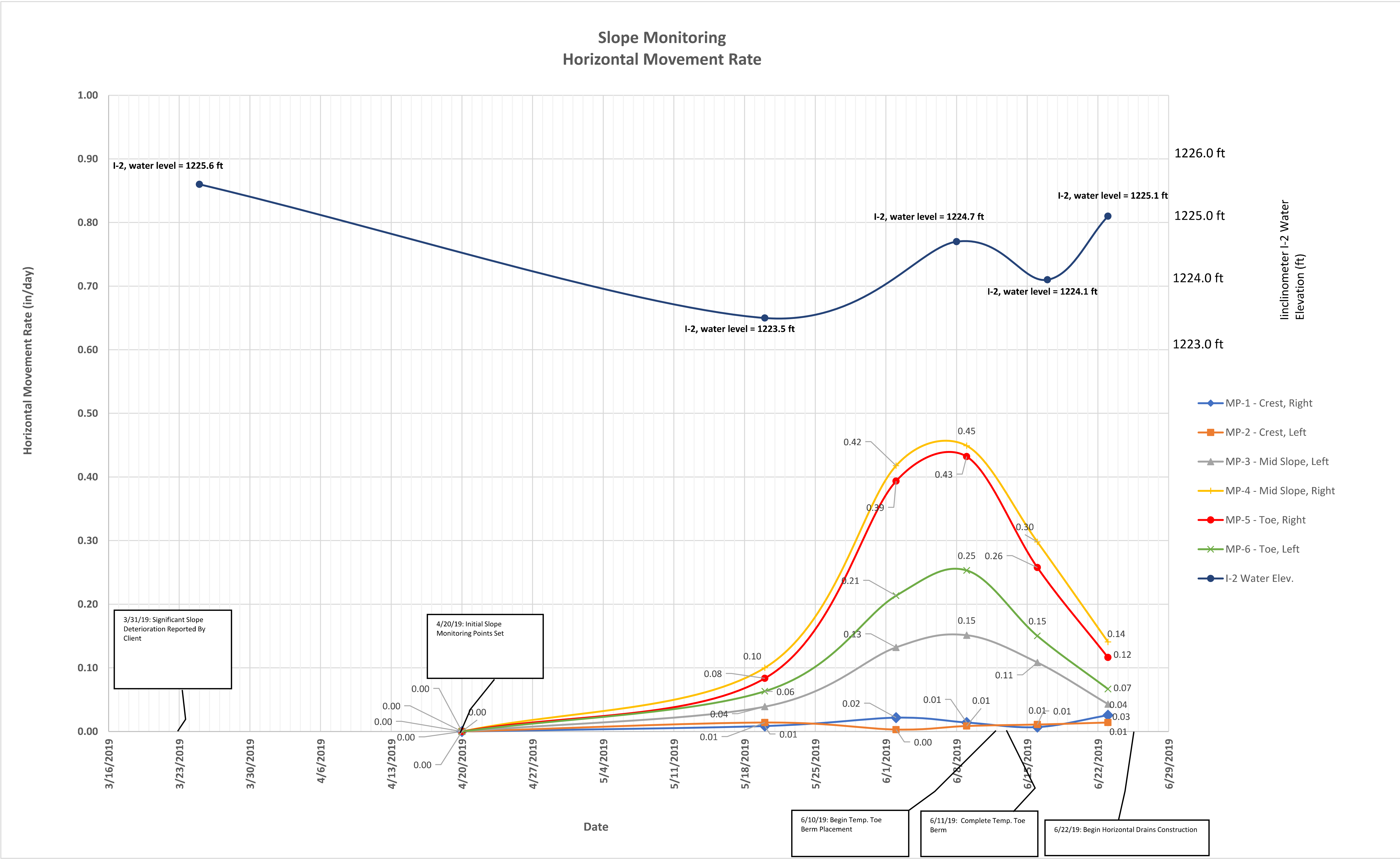
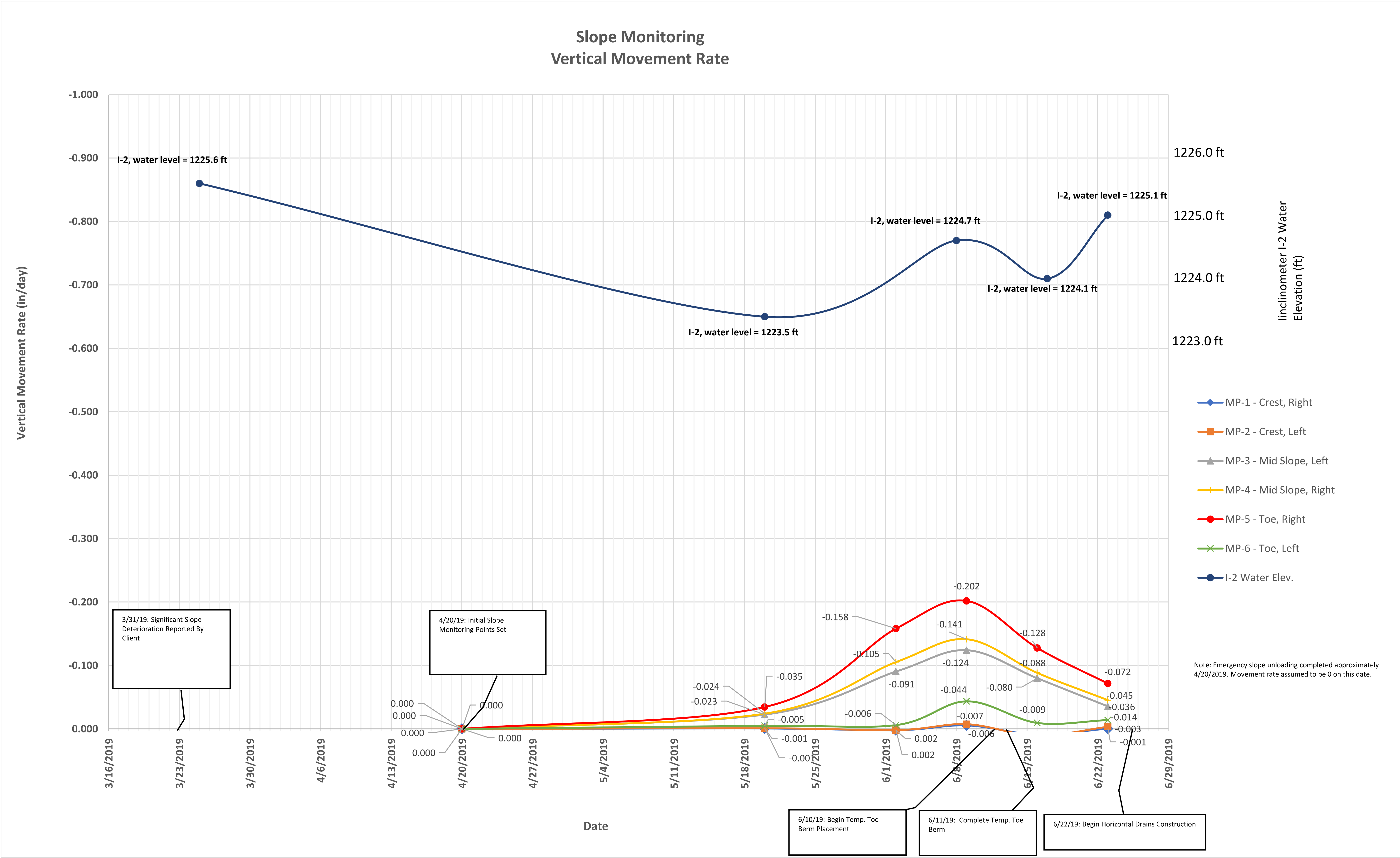


Exhibit 6-1



Slope Monitoring
Horizontal Movement Rate
Horizontal Movement Rate (in/day)
Inclinometer I-2 Water Elevation (ft)
Date
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
1226.0 ft
1225.0 ft
1224.0 ft
1223.0 ft
3/16/2019
3/23/2019
3/30/2019
4/6/2019
4/13/2019
4/20/2019
4/27/2019
5/4/2019
5/11/2019
5/18/2019
5/25/2019
6/1/2019
6/8/2019
6/15/2019
6/22/2019
6/29/2019
I-2, water level = 1225.6 ft
I-2, water level = 1223.5 ft
I-2, water level = 1224.7 ft
I-2, water level = 1224.1 ft
I-2, water level = 1225.1 ft
MP-1 - Crest, Right
MP-2 - Crest, Left
MP-3 - Mid Slope, Left
MP-4 - Mid Slope, Right
MP-5 - Toe, Right
MP-6 - Toe, Left
I-2 Water Elev.
3/31/19: Significant Slope Deterioration Reported By Client
4/20/19: Initial Slope Monitoring Points Set
6/10/19: Begin Temp. Toe Berm Placement
6/11/19: Complete Temp. Toe Berm
6/22/19: Begin Horizontal Drains Construction
0.42
0.45
0.43
0.39
0.30
0.25
0.26
0.21
0.15
0.15
0.13
0.14
0.12
0.11
0.10
0.08
0.06
0.07
0.04
0.04
0.02
0.01
0.01
0.01
0.01
0.03
0.01
0.01
0.01
0.00
0.00
0.00
0.00
0.00
0.00
0.00

Exhibit 6-2



Slope Monitoring
Vertical Movement Rate
Vertical Movement Rate (in/day)
-1.000
-0.900
-0.800
-0.700
-0.600
-0.500
-0.400
-0.300
-0.200
-0.100
0.000
1226.0 ft
1225.0 ft
1224.0 ft
1223.0 ft
Iinclinometer I-2 Water Elevation (ft)
I-2, water level = 1225.6 ft
I-2, water level = 1223.5 ft
I-2, water level = 1224.7 ft
I-2, water level = 1224.1 ft
I-2, water level = 1225.1 ft
3/16/2019
3/23/2019
3/30/2019
4/6/2019
4/13/2019
4/20/2019
4/27/2019
5/4/2019
5/11/2019
5/18/2019
5/25/2019
6/1/2019
6/8/2019
6/15/2019
6/22/2019
6/29/2019
Date
3/31/19: Significant Slope Deterioration Reported By Client
4/20/19: Initial Slope Monitoring Points Set
6/10/19: Begin Temp. Toe Berm Placement
6/11/19: Complete Temp. Toe Berm
6/22/19: Begin Horizontal Drains Construction
-0.202
-0.158
-0.141
-0.128
-0.105
-0.124
-0.088
-0.072
-0.035
-0.024
-0.023
-0.091
-0.044
-0.080
-0.045
-0.036
-0.005
-0.006
-0.007
-0.009
-0.014
-0.001
-0.003
-0.006
-0.002
0.002
0.002
-0.001
MP-1 - Crest, Right
MP-2 - Crest, Left
MP-3 - Mid Slope, Left
MP-4 - Mid Slope, Right
MP-5 - Toe, Right
MP-6 - Toe, Left
I-2 Water Elev.
Note: Emergency slope unloading completed approximately 4/20/2019. Movement rate assumed to be 0 on this date.

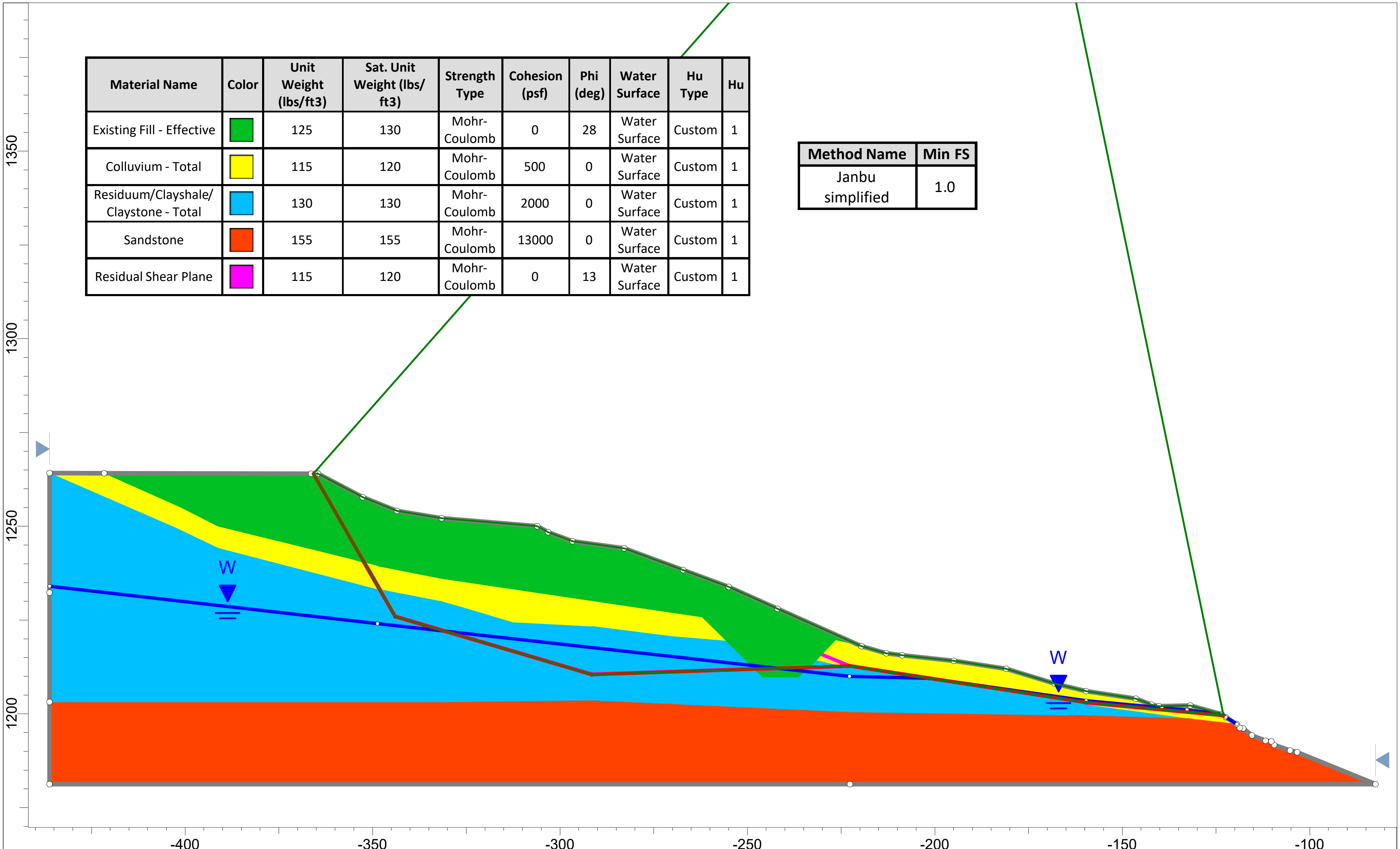
Exhibit 6-3

CRACK MEASUREMENTS

Top of Emb. Crack					
Distance to P1	23.50'	23.50'	23.60'	23.60'	24.00'
Vertical Displacement P1	1 - 7/16"	1 - 7/16"	4"	4-7/8"	6-7/16"
Depth of Crack P1 (from pad side of crack)	--	--	8-1/16"	8-1/2"	10-1/2"
Distance to P2	23.61'	23.59'	23.70'	23.74'	23.90'
Vertical Displacement P2	0"	0"	1-3/4"	3"	4-5/16"
Depth of Crack P2 (from pad side of crack)	--	--	7-7/8"	8"	10-1/2"
AP1 (aperature) width	3/8"	7/8"	2-1/4"	3-1/4"	4"
Visible Length of Crack	--	29.1'	37.8'	37.8'	--

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Existing Fill - Effective		125	130	Mohr-Coulomb	0	28	Water Surface	Custom	1
Colluvium - Total		115	120	Mohr-Coulomb	500	0	Water Surface	Custom	1
Residuum/Clayshale/ Claystone - Total		130	130	Mohr-Coulomb	2000	0	Water Surface	Custom	1
Sandstone		155	155	Mohr-Coulomb	13000	0	Water Surface	Custom	1
Residual Shear Plane		115	120	Mohr-Coulomb	0	13	Water Surface	Custom	1

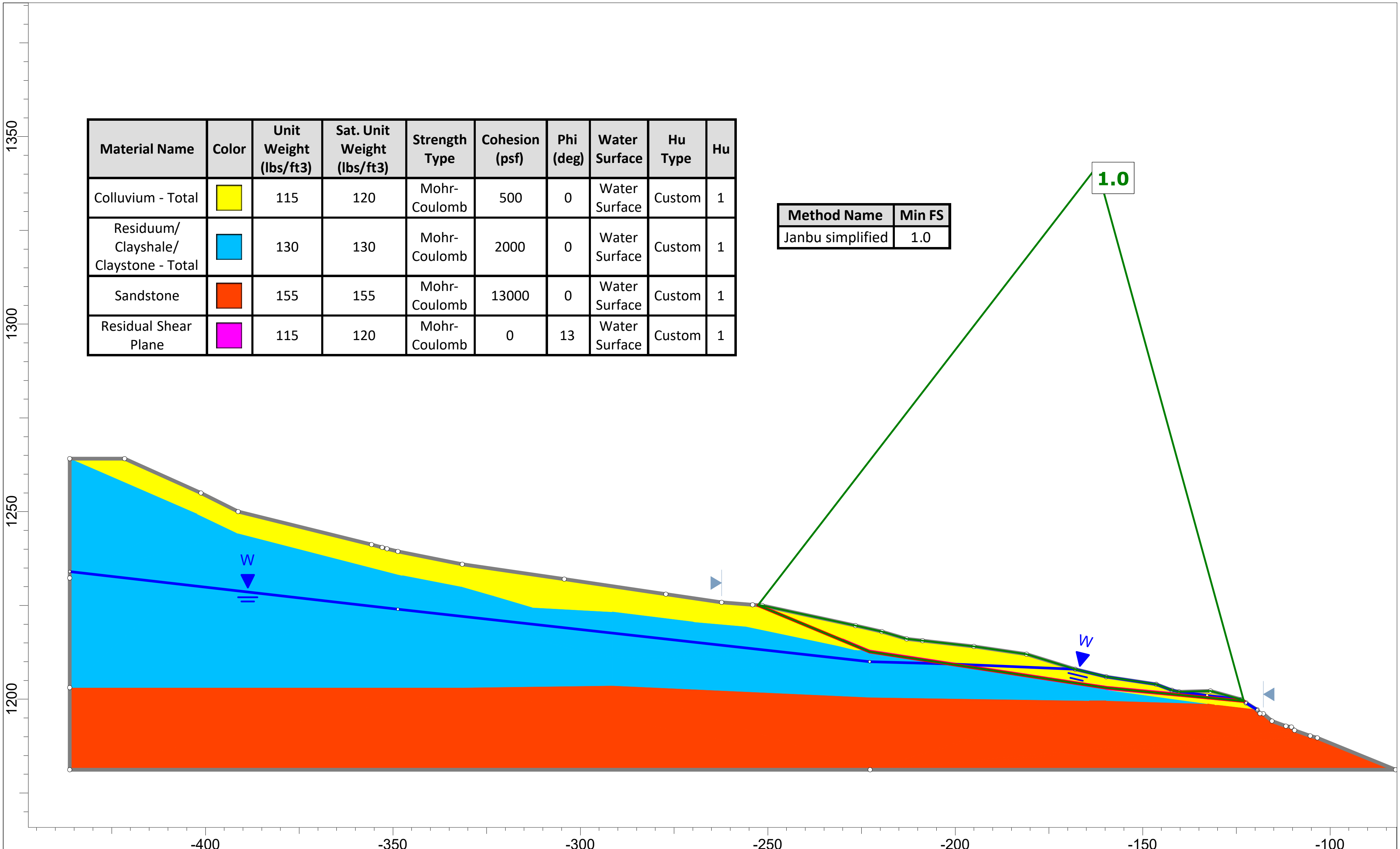
Method Name	Min FS
Janbu simplified	1.0



rocscience	*Project*	Design Example
	Analysis	Model Caibration - Original Construction
	Figure	**8-1**
	Company	IRISE
SLIDEINTERPRET 9.025	*File Name*	8-1 Original Construction.slim

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Colluvium - Total		115	120	Mohr-Coulomb	500	0	Water Surface	Custom	1
Residuum/ Clayshale/ Claystone - Total		130	130	Mohr-Coulomb	2000	0	Water Surface	Custom	1
Sandstone		155	155	Mohr-Coulomb	13000	0	Water Surface	Custom	1
Residual Shear Plane		115	120	Mohr-Coulomb	0	13	Water Surface	Custom	1

Method Name	Min FS
Janbu simplified	1.0



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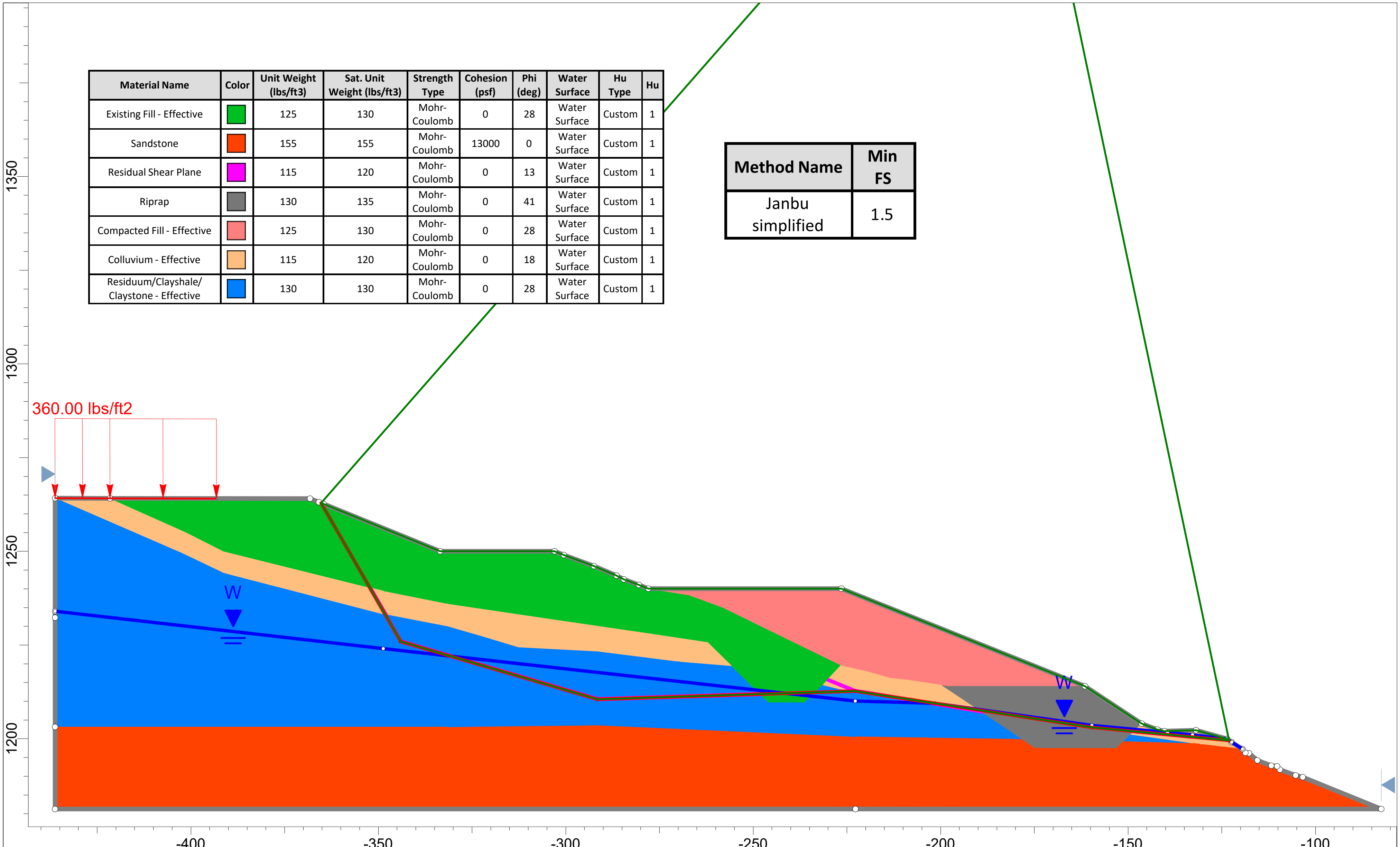
SLIDEINTERPRET 9.025

Project	Design Example
Analysis	Model Caibration - Pre-Construction (2004 Landslide)
Figure	**8-2**
Company	IRISE
File Name	8-2 Pre-Construction.slim

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Existing Fill - Effective		125	130	Mohr-Coulomb	0	28	Water Surface	Custom	1
Sandstone		155	155	Mohr-Coulomb	13000	0	Water Surface	Custom	1
Residual Shear Plane		115	120	Mohr-Coulomb	0	13	Water Surface	Custom	1
Riprap		130	135	Mohr-Coulomb	0	41	Water Surface	Custom	1
Compacted Fill - Effective		125	130	Mohr-Coulomb	0	28	Water Surface	Custom	1
Colluvium - Effective		115	120	Mohr-Coulomb	0	18	Water Surface	Custom	1
Residuum/Clayshale/ Claystone - Effective		130	130	Mohr-Coulomb	0	28	Water Surface	Custom	1

Method Name	Min FS
Janbu simplified	1.5



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SLIDEINTERPRET 9.025

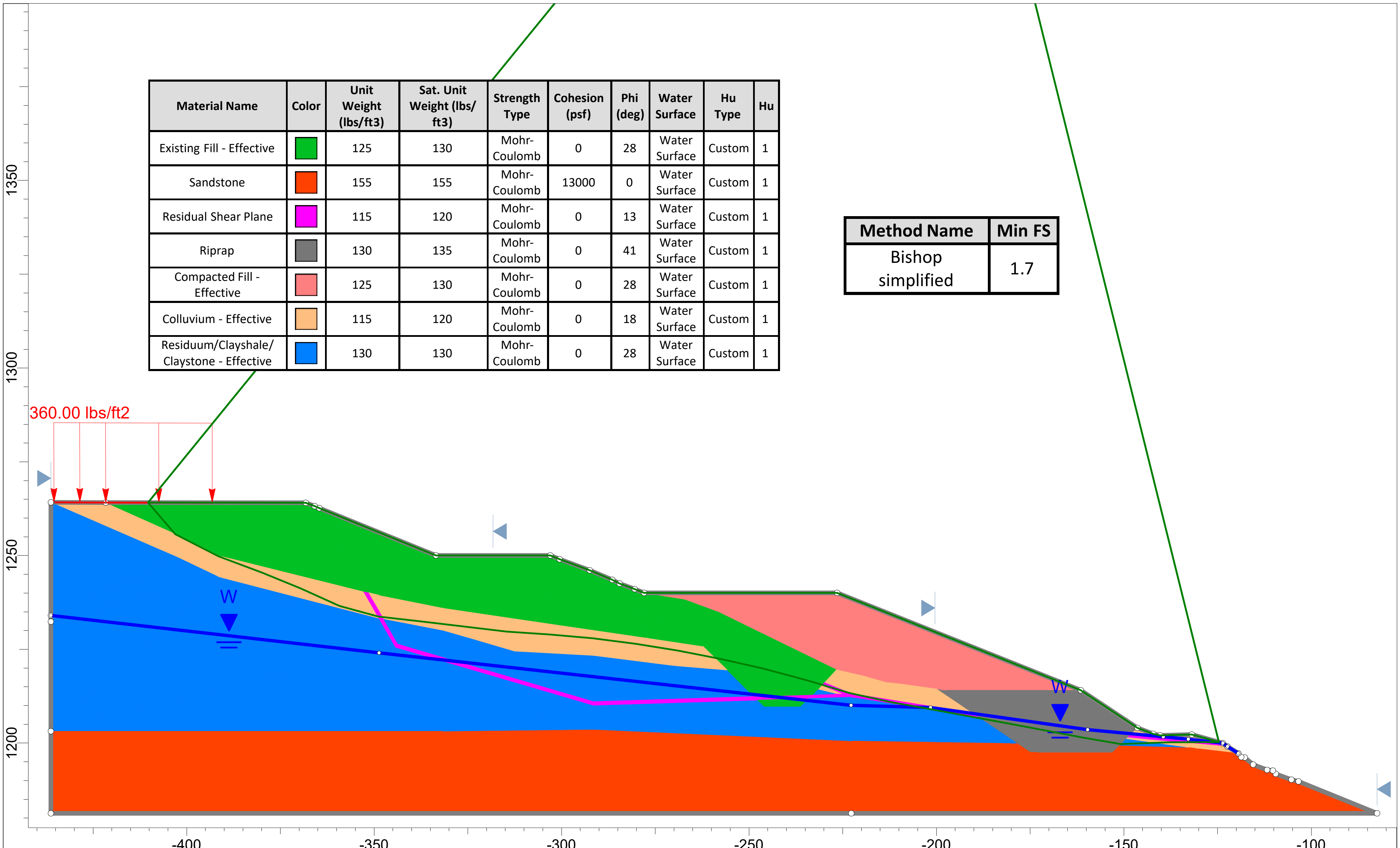
Project Design Example

Analysis Toe Buttress - Fully Specified

Figure **8-3**

Company IRISE

File Name 8-3 Toe Buttress.slim



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Existing Fill - Effective		125	130	Mohr-Coulomb	0	28	Water Surface	Custom	1
Sandstone		155	155	Mohr-Coulomb	13000	0	Water Surface	Custom	1
Residual Shear Plane		115	120	Mohr-Coulomb	0	13	Water Surface	Custom	1
Riprap		130	135	Mohr-Coulomb	0	41	Water Surface	Custom	1
Compacted Fill - Effective		125	130	Mohr-Coulomb	0	28	Water Surface	Custom	1
Colluvium - Effective		115	120	Mohr-Coulomb	0	18	Water Surface	Custom	1
Residuum/Clayshale/ Claystone - Effective		130	130	Mohr-Coulomb	0	28	Water Surface	Custom	1

Method Name	Min FS
Bishop simplified	1.7

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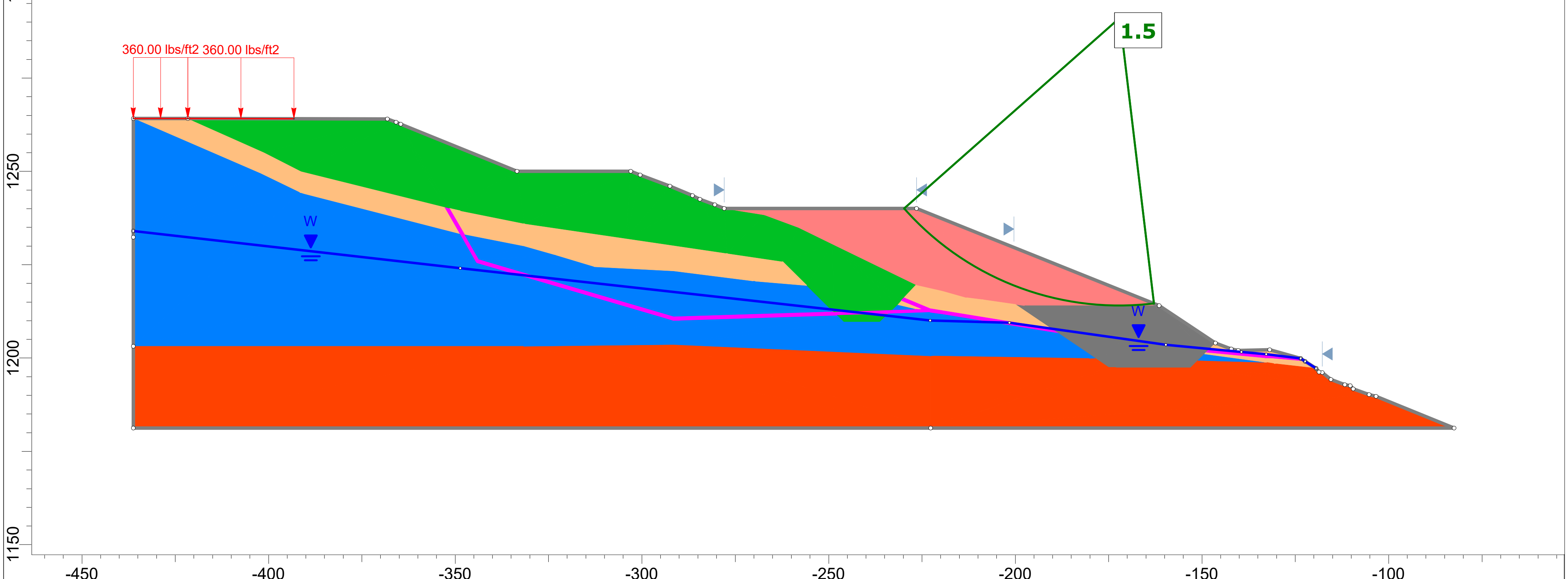
SLIDEINTERPRET 9.025

Project	Design Example
Analysis	Toe Buttress - Global Search
Figure	**8-4**
Company	IRISE
File Name	8-4 Toe Buttress.slim

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Existing Fill - Effective		125	130	Mohr-Coulomb	0	28	Water Surface	Custom	1
Sandstone		155	155	Mohr-Coulomb	13000	0	Water Surface	Custom	1
Residual Shear Plane		115	120	Mohr-Coulomb	0	13	Water Surface	Custom	1
Riprap		130	135	Mohr-Coulomb	0	41	Water Surface	Custom	1
Compacted Fill - Effective		125	130	Mohr-Coulomb	0	28	Water Surface	Custom	1
Colluvium - Effective		115	120	Mohr-Coulomb	0	18	Water Surface	Custom	1
Residuum/Clayshale/ Claystone - Effective		130	130	Mohr-Coulomb	0	28	Water Surface	Custom	1

Method Name	Min FS
Bishop simplified	1.5



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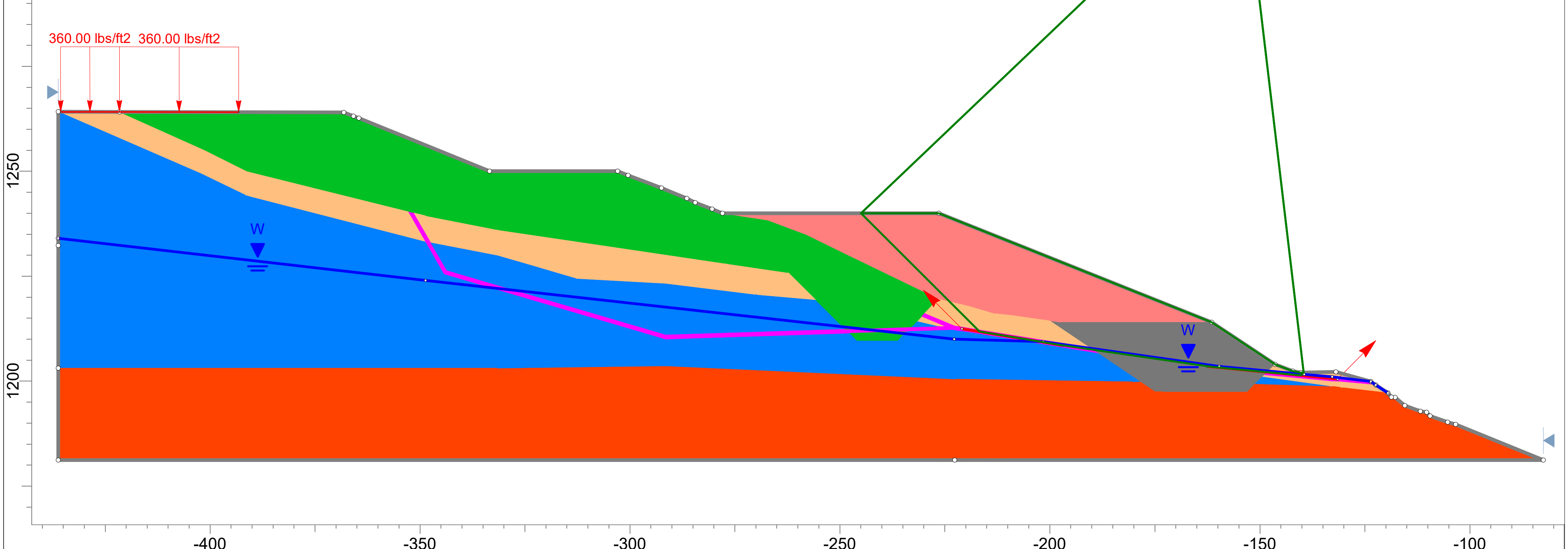
SLIDEINTERPRET 9.025

Project	Design Example
Analysis	Toe Buttress - Local Search - Effective Stress
Figure	**8-5**
Company	IRISE
File Name	8-5 Toe Buttress.slim

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Existing Fill - Effective		125	130	Mohr-Coulomb	0	28	Water Surface	Custom	1
Sandstone		155	155	Mohr-Coulomb	13000	0	Water Surface	Custom	1
Residual Shear Plane		115	120	Mohr-Coulomb	0	13	Water Surface	Custom	1
Riprap		130	135	Mohr-Coulomb	0	41	Water Surface	Custom	1
Compacted Fill - Effective		125	130	Mohr-Coulomb	0	28	Water Surface	Custom	1
Colluvium - Effective		115	120	Mohr-Coulomb	0	18	Water Surface	Custom	1
Residuum/Clayshale/Claystone - Effective		130	130	Mohr-Coulomb	0	28	Water Surface	Custom	1

Method Name	Min FS
Janbu simplified	1.5



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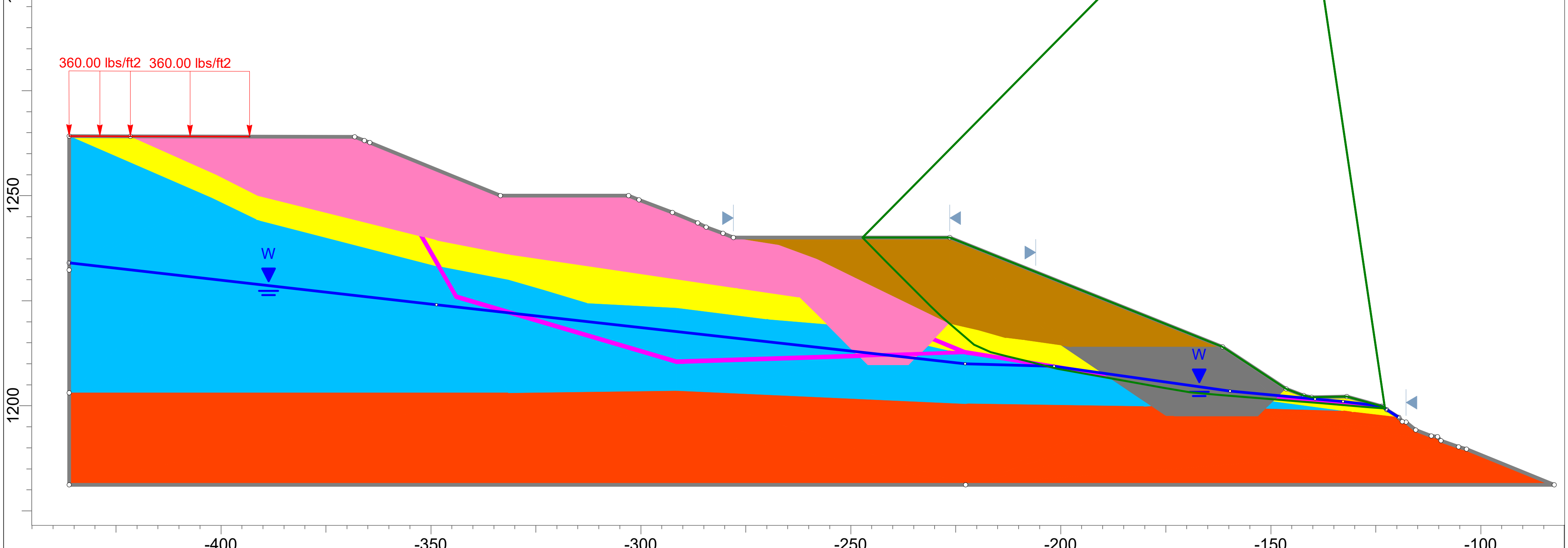
SLIDEINTERPRET 9.025

Project	Design Example
Analysis	Toe Buttress - Local Search - Effective Stress (Block)
Figure	8-6
Company	IRISE
File Name	8-6 Toe Buttress.slim

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Colluvium - Total		115	120	Mohr-Coulomb	500	0	Water Surface	Custom	1
Residuum/Clayshale/Claystone - Total		130	130	Mohr-Coulomb	2000	0	Water Surface	Custom	1
Sandstone		155	155	Mohr-Coulomb	13000	0	Water Surface	Custom	1
Residual Shear Plane		115	120	Mohr-Coulomb	0	13	Water Surface	Custom	1
Compacted Fill - Total		125	130	Mohr-Coulomb	1500	0	Water Surface	Custom	1
Riprap		130	135	Mohr-Coulomb	0	41	Water Surface	Custom	1
Existing Fill - Total		125	130	Mohr-Coulomb	1500	0	Water Surface	Custom	1

Method Name	Min FS
Bishop simplified	2.0



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SLIDEINTERPRET 9.025

Project

Design Example

Analysis

Toe Buttress - Local Search - Total Stress

Figure

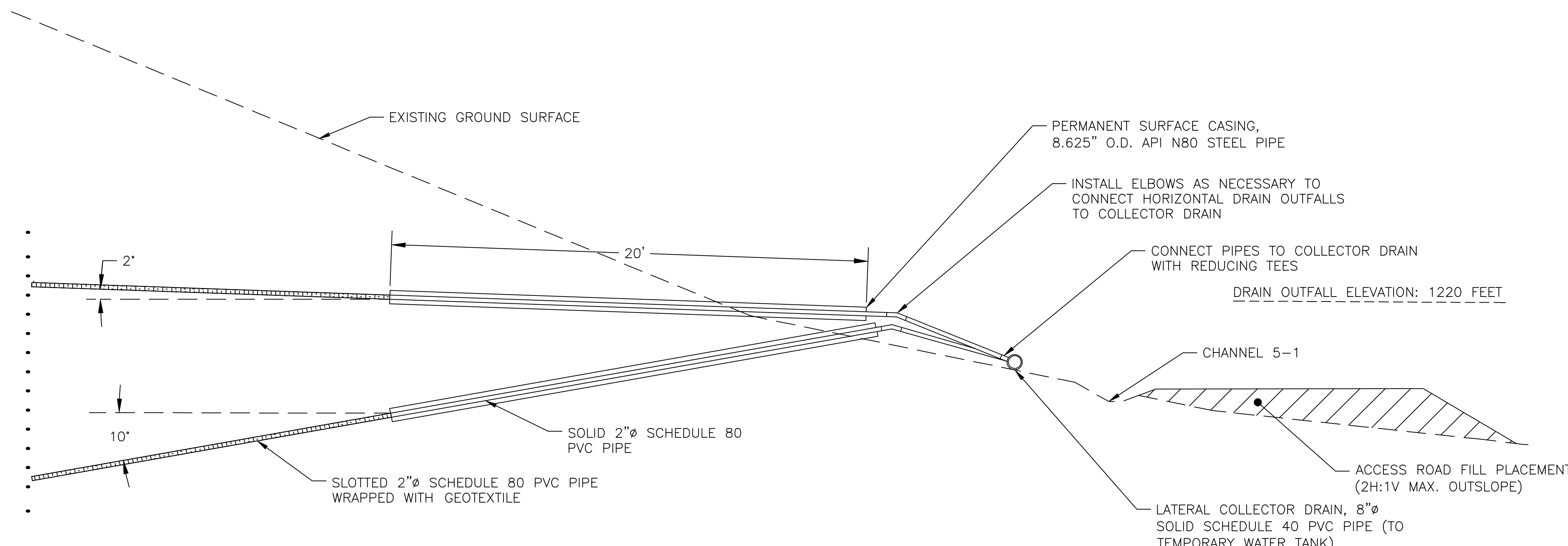
8-7

Company

IRISE

File Name

8-7 Toe Buttress.slim



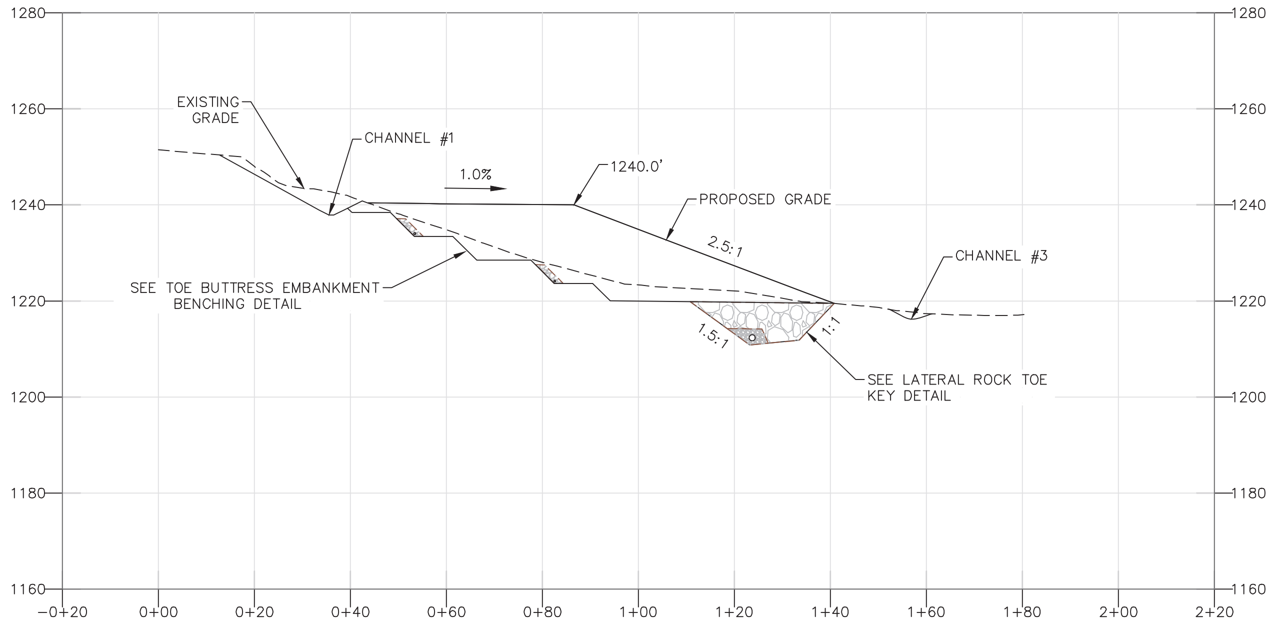
HORIZONTAL DRAIN INSTALLATION TYPICAL DETAIL

NOT TO SCALE

SCHEDULE OF HORIZONTAL RELIEF DRAINS

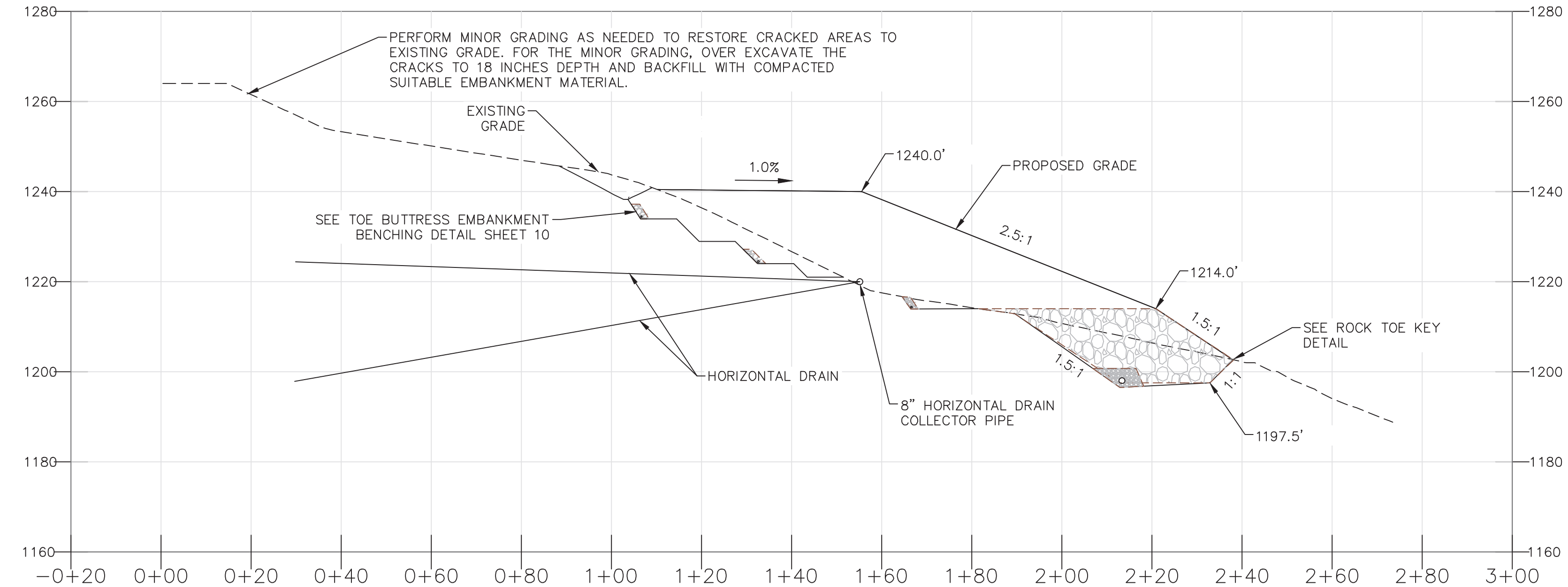
DRAIN #	OUTFALL ELEVATION (FEET-NAVD 88)	ESTIMATED HORIZONTAL LENGTH (FEET)	AZIMUTH (DEGREES)	VERTICAL INCLINATION FROM HORIZONTAL (DEGREES)
HD-1A	1220.0	148.0	309	-10
HD-1B	1220.0	145.0	309	+2
HD-2A	1220.0	147.0	309	-10
HD-2B	1220.0	147.0	309	+2
HD-3A	1220.0	147.0	309	-10
HD-3B	1220.0	149.0	309	+2
HD-4A	1220.0	150.0	309	-10
HD-4B	1220.0	150.0	309	+2
HD-5A	1220.0	147.0	309	-10
HD-5B	1220.0	145.0	309	+2

Exhibit 10-1a



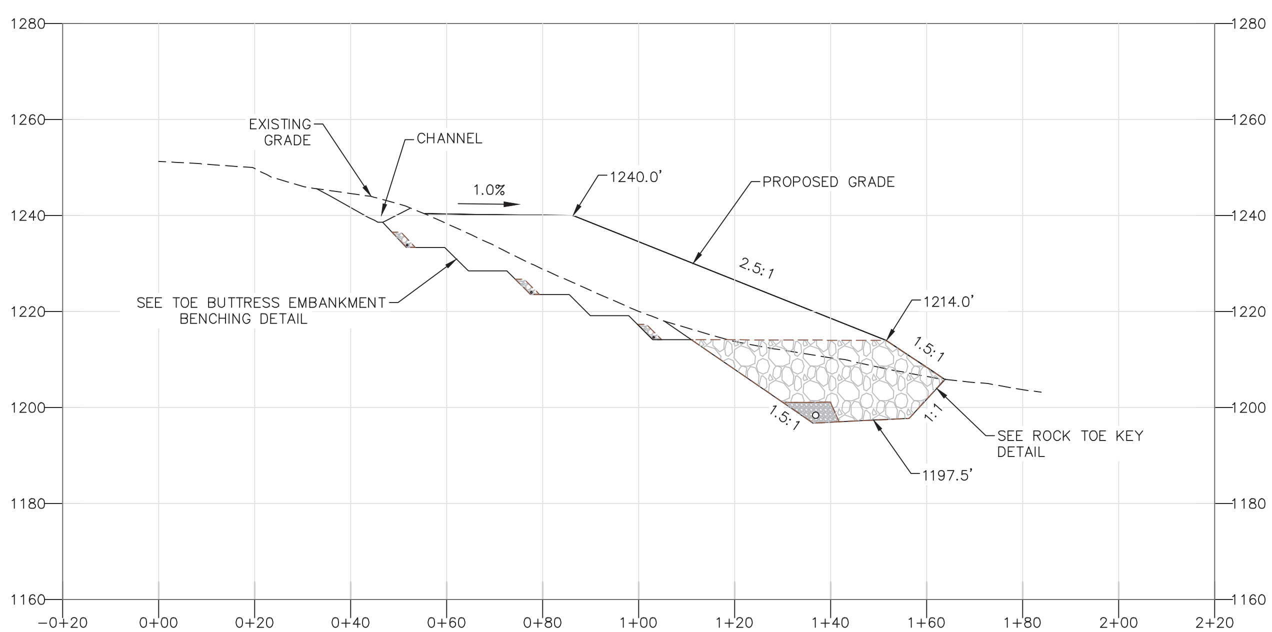
SECTION A-A

SCALE: 1" = 20' HORZ. & VERT.



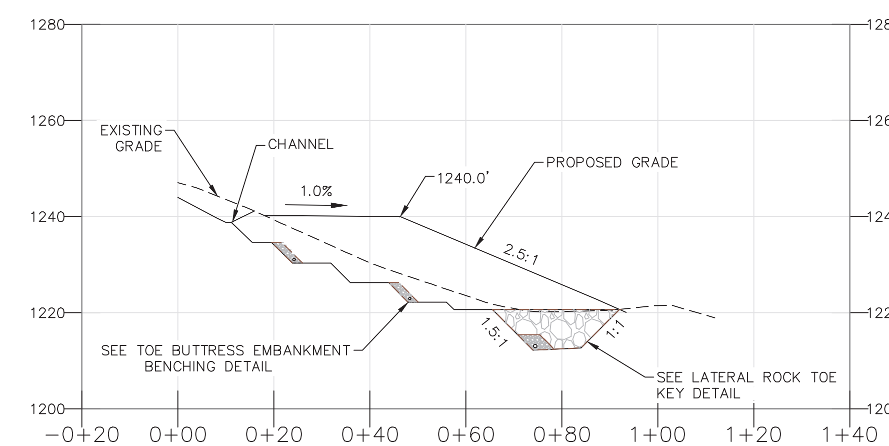
SECTION B-B

SCALE: 1" = 20' HORZ. & VERT.



SECTION C-C

SCALE: 1" = 20' HORZ. & VERT.



SECTION D-D

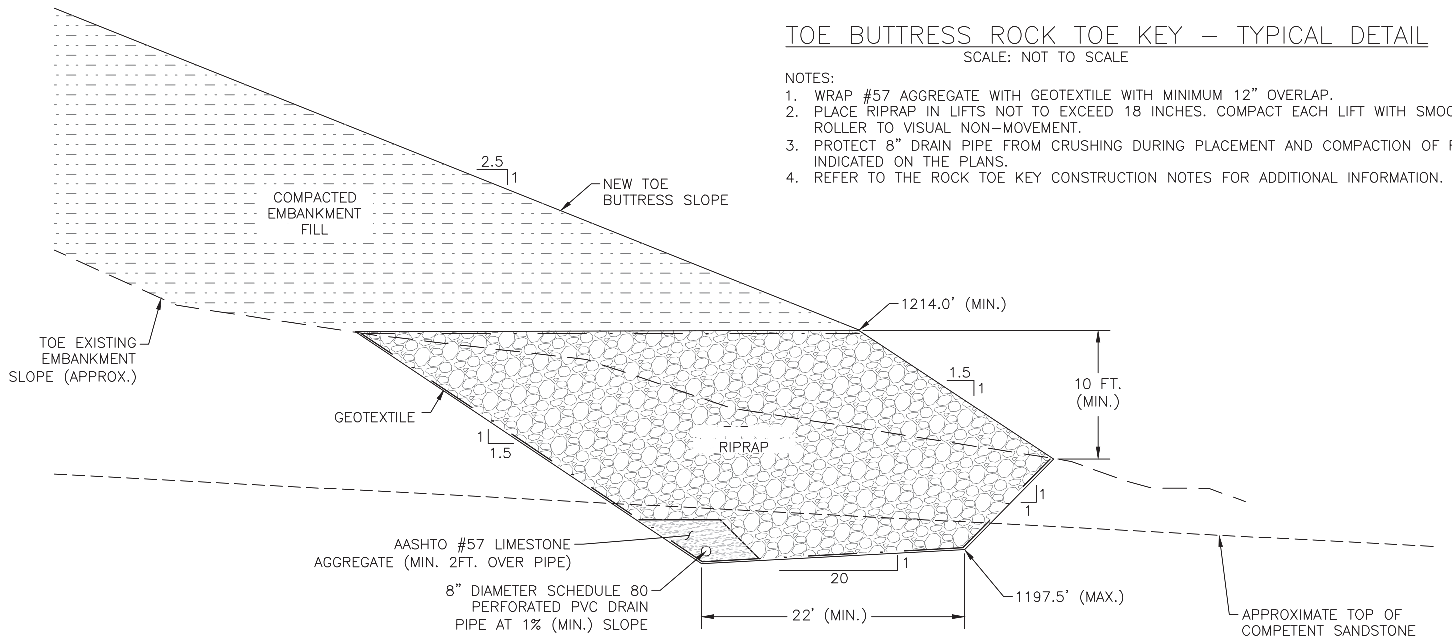
SCALE: 1" = 20' HORZ. & VERT.

Exhibit 10-1b

TOE BUTTRESS ROCK TOE KEY – TYPICAL DETAIL

SCALE: NOT TO SCALE

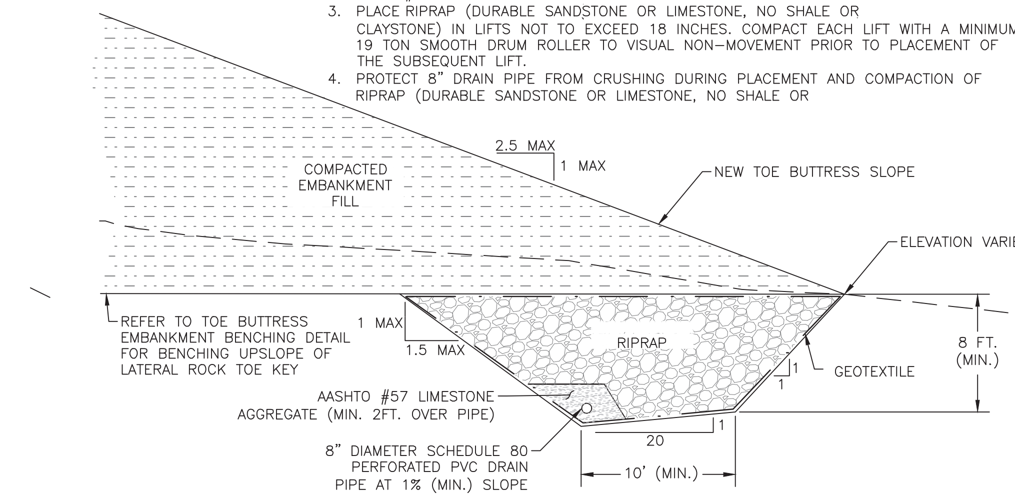
NOTES:
1. WRAP #57 AGGREGATE WITH GEOTEXTILE WITH MINIMUM 12" OVERLAP.
2. PLACE RIPRAP IN LIFTS NOT TO EXCEED 18 INCHES. COMPACT EACH LIFT WITH SMOOTH DRUM OR VIBRATORY ROLLER TO VISUAL NON–MOVEMENT.
3. PROTECT 8" DRAIN PIPE FROM CRUSHING DURING PLACEMENT AND COMPACTION OF RIPRAP. OUTLET PIPE AS INDICATED ON THE PLANS.
4. REFER TO THE ROCK TOE KEY CONSTRUCTION NOTES FOR ADDITIONAL INFORMATION.

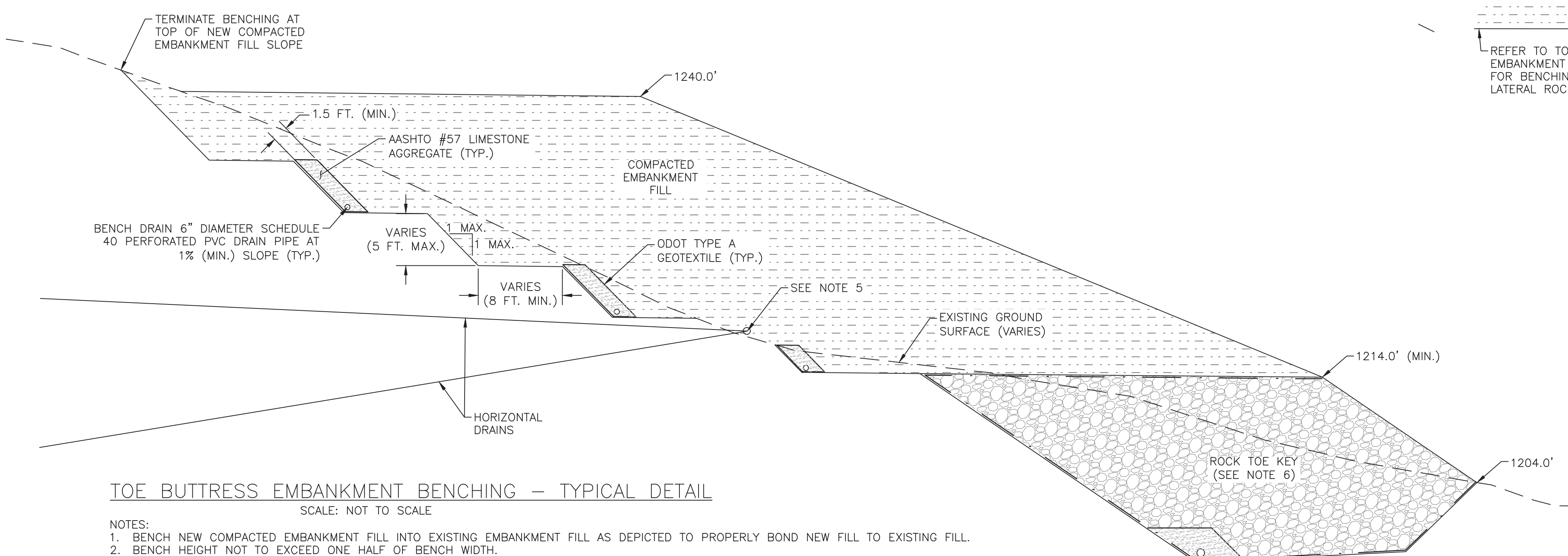


LATERAL ROCK TOE KEY – TYPICAL DETAIL

SCALE: NOT TO SCALE

NOTES:
1. CONSTRUCT AT LOCATIONS INDICATED ON THE PLANS AND TIE INTO TOE BUTTRESS ROCK TOE KEY AT BOUNDARY.
2. WRAP #57 AGGREGATE WITH ODOT TYPE A GEOTEXTILE WITH MINIMUM 12" OVERLAP.
3. PLACE RIPRAP (DURABLE SANDSTONE OR LIMESTONE, NO SHALE OR CLAYSTONE) IN LIFTS NOT TO EXCEED 18 INCHES. COMPACT EACH LIFT WITH A MINIMUM 19 TON SMOOTH DRUM ROLLER TO VISUAL NON–MOVEMENT PRIOR TO PLACEMENT OF THE SUBSEQUENT LIFT.
4. PROTECT 8" DRAIN PIPE FROM CRUSHING DURING PLACEMENT AND COMPACTION OF RIPRAP (DURABLE SANDSTONE OR LIMESTONE, NO SHALE OR





TOE BUTTRESS EMBANKMENT BENCHING – TYPICAL DETAIL

SCALE: NOT TO SCALE

NOTES:
1. BENCH NEW COMPACTED EMBANKMENT FILL INTO EXISTING EMBANKMENT FILL AS DEPICTED TO PROPERLY BOND NEW FILL TO EXISTING FILL.
2. BENCH HEIGHT NOT TO EXCEED ONE HALF OF BENCH WIDTH.
3. INSTALL BENCH DRAINS AT LOCATIONS WHERE SEEPAGE/SPRINGS ARE INTERCEPTED DURING EMBANKMENT BENCHING, OR AT MAXIMUM 10 FEET VERTICAL INTERVALS, AND OUTLET IN ACCORDANCE WITH THE PLANS. PROTECT BENCH DRAINS FROM CRUSHING DURING FILL PLACEMENT AND COMPACTION.
4. FOR EACH BENCH DRAIN, WRAP #57 AGGREGATE WITH GEOTEXTILE WITH MINIMUM 12" OVERLAP.
5. CONNECT HORIZONTAL DRAINS TO 8" DIAMETER SCHEDULE 80 SOLID PVC COLLECTOR PIPE AND OUTLET AT A MINIMUM 1% IN ACCORDANCE WITH THE PLANS. PROTECT HORIZONTAL DRAINS AND COLLECTOR PIPE FROM CRUSHING DURING FILL PLACEMENT AND COMPACTION.
6. REFER TO THE TOE BUTTRESS ROCK TOE KEY DETAIL FOR FURTHER INFORMATION.
7. REFER TO THE CONSTRUCTION NOTES FOR ADDITIONAL INFORMATION REGARDING EMBANKMENT FILL PLACEMENT.

Exhibit 10-1c

SURFACE DEFORMATION MONITORING

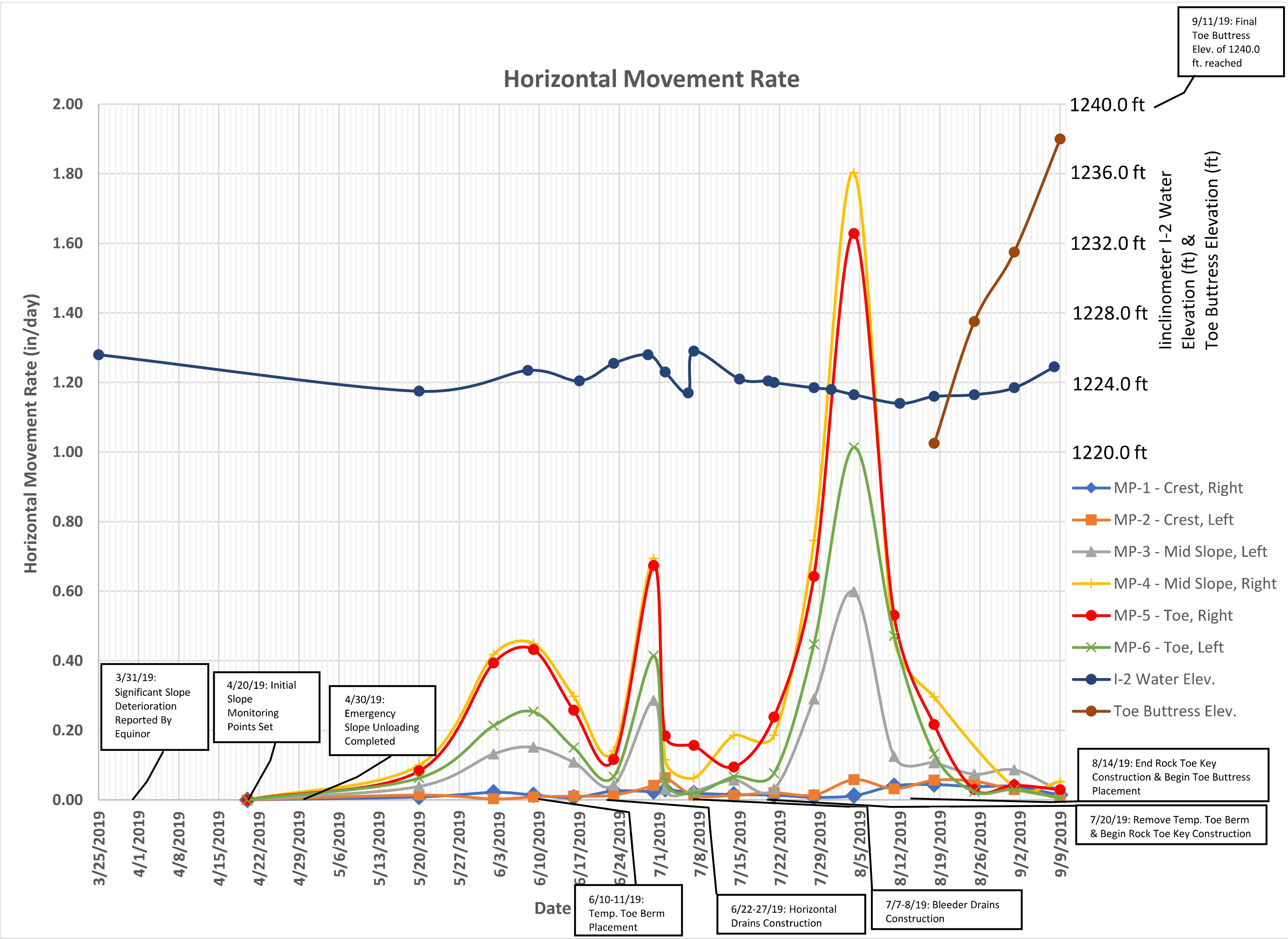


Exhibit 10-2a

SURFACE DEFORMATION MONITORING

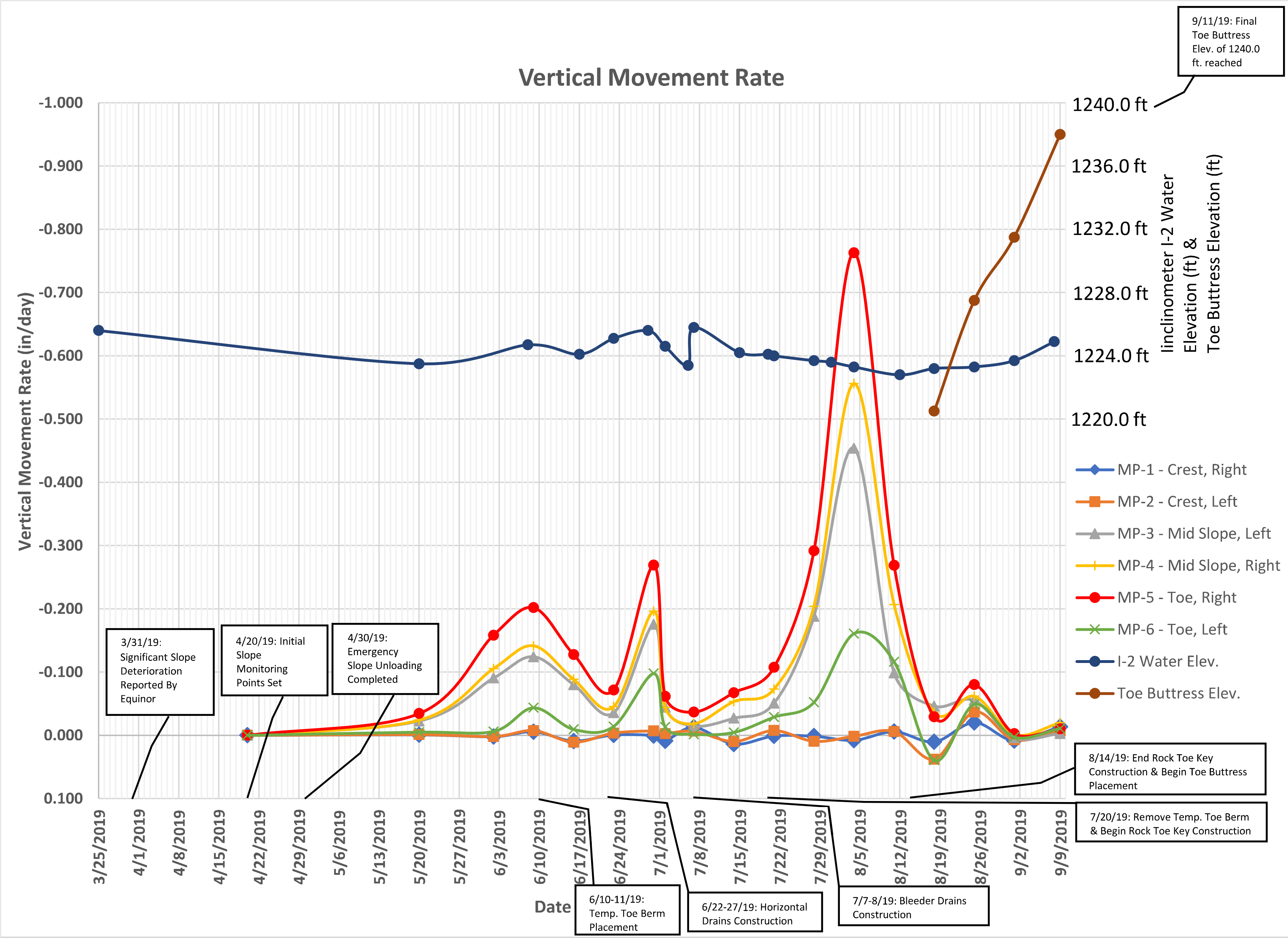
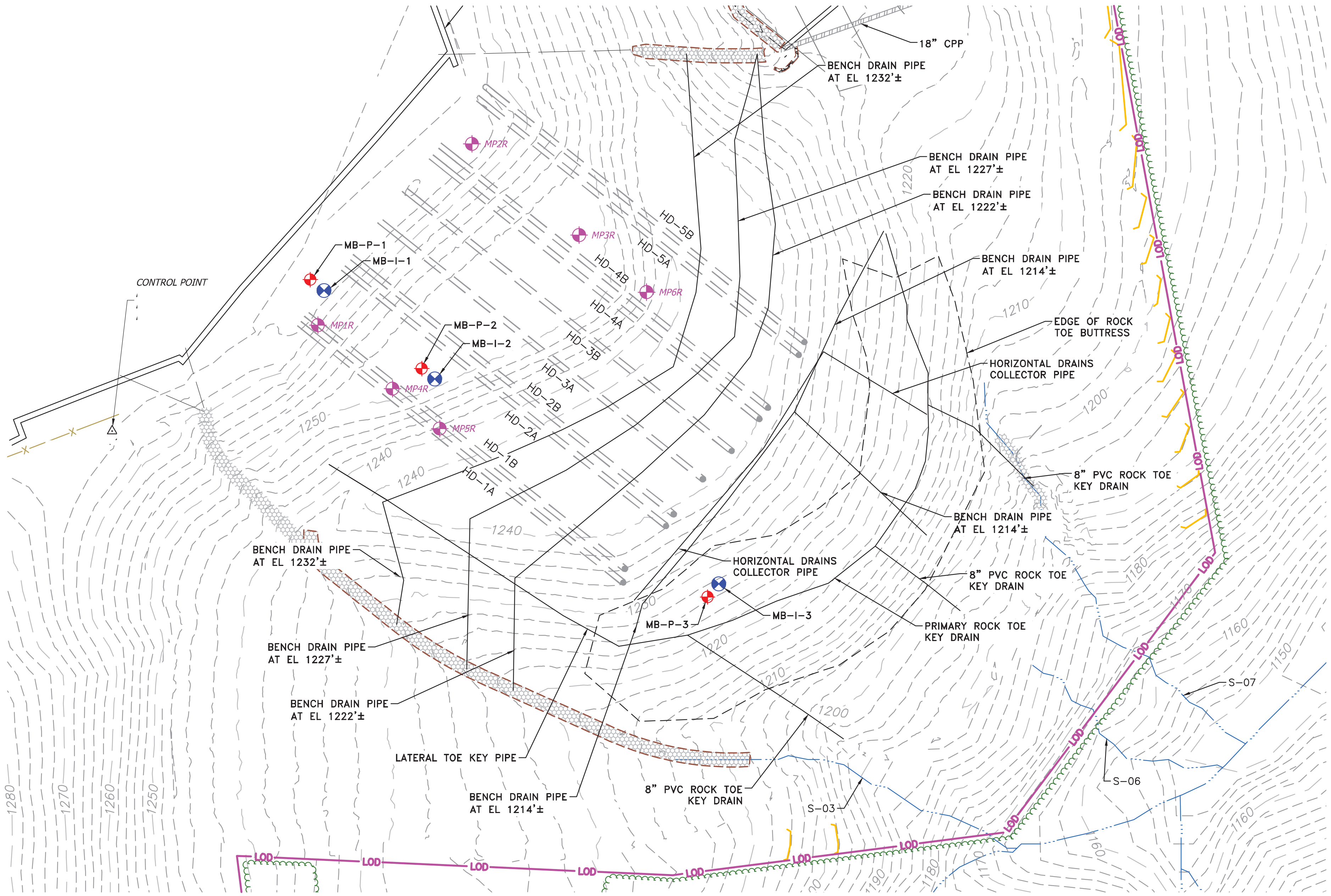


Exhibit 10-2b

POST CONSTRUCTION MONITORING PLAN

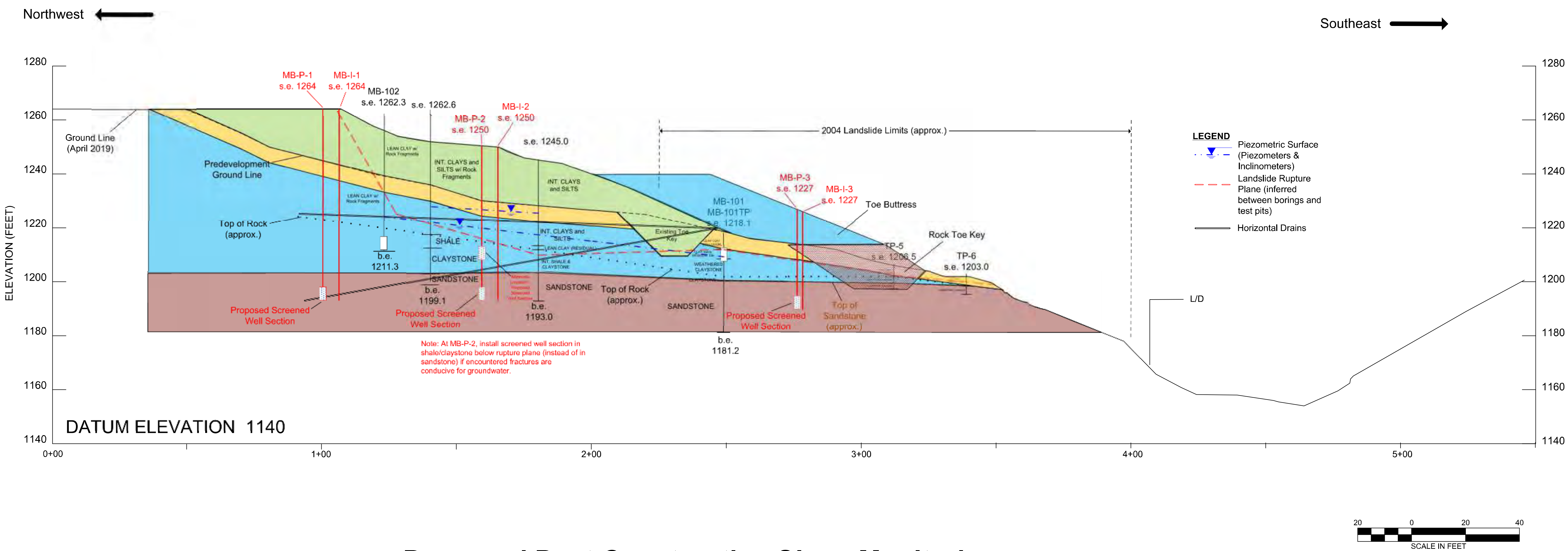


LEGEND

- SURFACE MONITORING POINT
- PROPOSED INCLINOMETER
- PROPOSED PIEZOMETER
- 24" CFS
- LIMIT OF DISTURBANCE
- CONTROL POINT
- STREAM
- MAJOR CONTOUR
- MINOR CONTOUR
- FENCE
- TREE LINE
- HORIZONTAL RELIEF DRAINS AND HOLE LOCATIONS
- FLEXAMAT LINING

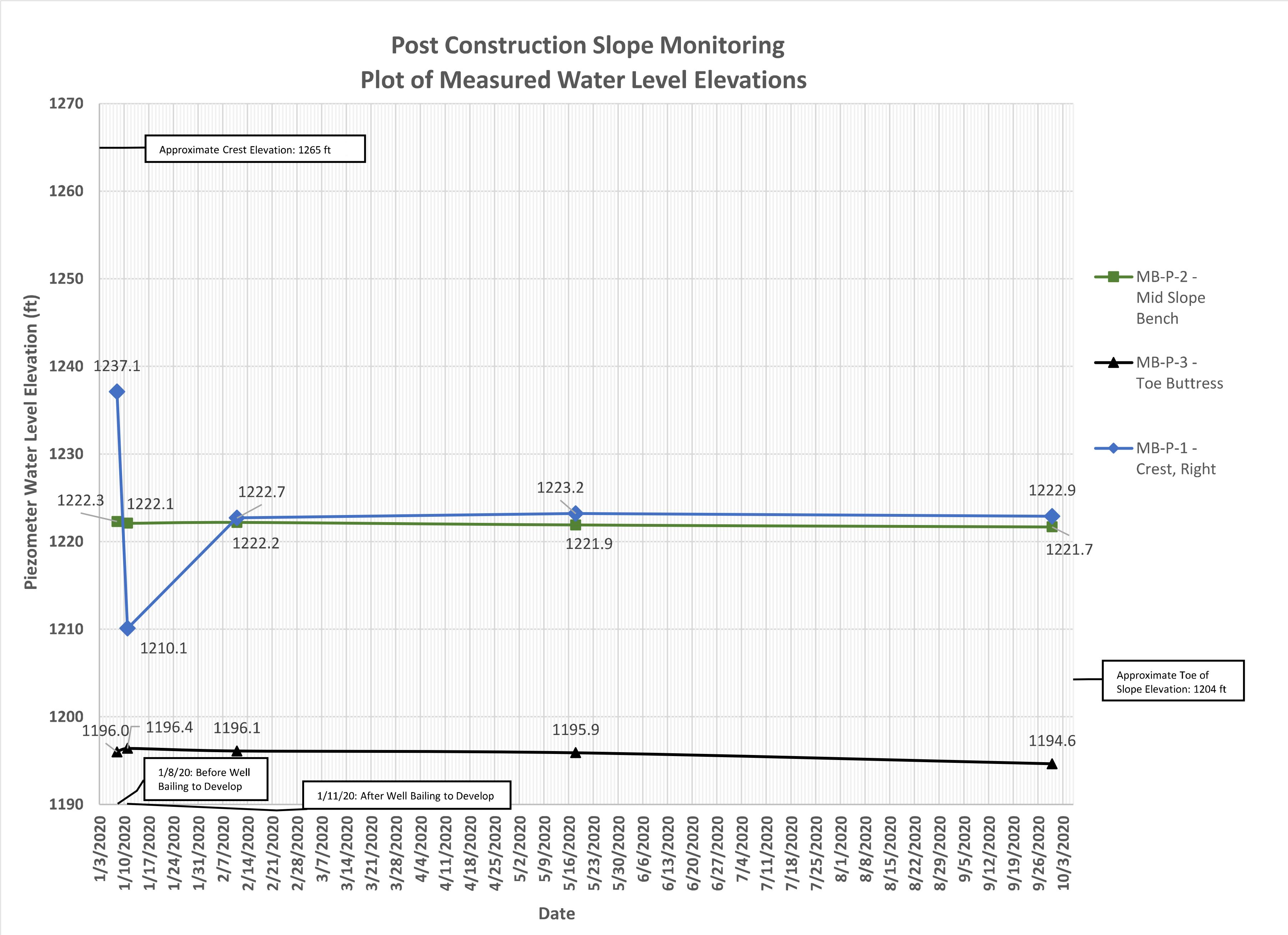
SCALE
0 30 60 FEET

Exhibit 11-1a



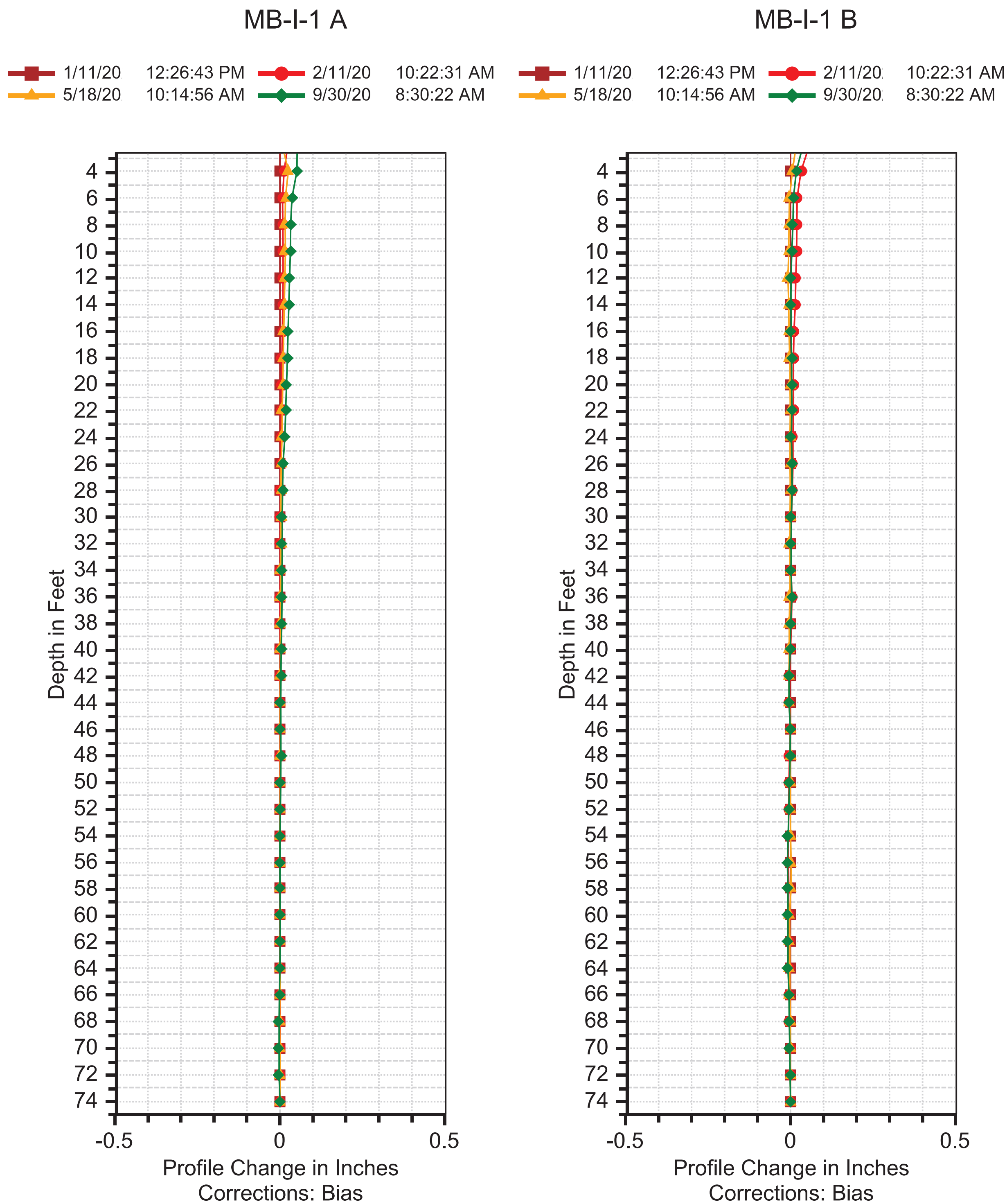
Proposed Post-Construction Slope Monitoring

Exhibit 11-1b



Post Construction Slope Monitoring
Plot of Measured Water Level Elevations
Piezometer Water Level Elevation (ft)
Date
Approximate Crest Elevation: 1265 ft
Approximate Toe of Slope Elevation: 1204 ft
1/8/20: Before Well Bailing to Develop
1/11/20: After Well Bailing to Develop
MB-P-2 - Mid Slope Bench
MB-P-3 - Toe Buttress
MB-P-1 - Crest, Right
1237.1
1222.3
1222.1
1210.1
1222.7
1222.2
1223.2
1221.9
1222.9
1221.7
1196.0
1196.4
1196.1
1195.9
1194.6
1270
1260
1250
1240
1230
1220
1210
1200
1190
1/3/2020
1/10/2020
1/17/2020
1/24/2020
1/31/2020
2/7/2020
2/14/2020
2/21/2020
2/28/2020
3/7/2020
3/14/2020
3/21/2020
3/28/2020
4/4/2020
4/11/2020
4/18/2020
4/25/2020
5/2/2020
5/9/2020
5/16/2020
5/23/2020
5/30/2020
6/6/2020
6/13/2020
6/20/2020
6/27/2020
7/4/2020
7/11/2020
7/18/2020
7/25/2020
8/1/2020
8/8/2020
8/15/2020
8/22/2020
8/29/2020
9/5/2020
9/12/2020
9/19/2020
9/26/2020
10/3/2020

Exhibit 11-2



Notes:
1.The depicted depth represents depth below top of inclinometer casing.
2. Bias error is the most common type of error in inclinometer surveys. It is the value returned to the probe when it is absolutely vertical. In theory it should be zero, but in practice it is non-zero. It is normal for the bias of the probe to change. Any bias not eliminated during the data reduction process is bias-shift error. Bias-shift error is then corrected for on the plots. Bias-shift error can typically be identified when minor movement is shown at the base of the inclinometer, where the casing is anchored in stable ground. Any appearance of movement there is generally a bias error in the data. (DigiPro2 Manual, 2018/6/07; Slope Indicator Data Reduction and Error Correction, 2000/10/30)

Exhibit 11-3

CHECKSUMS

Inclinometer:		MB-I-1		Survey Date:		
Depth	A0	A180	B0	B180	A Sum	B Sum
(feet)	(unit)	(unit)	(unit)	(unit)	(unit)	(unit)
2	2153	-2105	475	-431	48	44
4	785	-783	626	-570	2	56
6	-37	44	857	-785	7	72
8	-286	314	709	-640	28	69
10	-359	369	502	-446	10	56
12	-245	261	284	-222	16	62
14	-89	101	98	-51	12	47
16	49	-30	109	-62	19	47
18	233	-208	0	50	25	50
20	307	-292	-120	158	15	38
22	251	-235	-234	279	16	45
24	176	-164	-286	349	12	63
26	347	-331	-112	160	16	48
28	738	-708	-237	295	30	58
30	489	-472	141	-76	17	65
32	241	-226	289	-241	15	48
34	255	-245	362	-297	10	65
36	379	-365	612	-562	14	50
38	85	-60	602	-551	25	51
40	-279	294	566	-520	15	46
42	-365	385	604	-554	20	50
44	-376	394	563	-546	18	17
46	-367	374	709	-663	7	46
48	-282	305	667	-621	23	46
50	-293	307	618	-572	14	46
52	-363	376	546	-513	13	33
54	-235	242	709	-632	7	77
56	103	-92	909	-823	11	86
58	28	-6	663	-575	22	88
60	-72	88	337	-296	16	41
62	-84	97	129	-71	13	58
64	-110	117	150	-85	7	65
66	-126	142	356	-307	16	49
68	-53	84	377	-318	31	59
70	277	-262	629	-574	15	55
72	834	-819	1138	-1099	15	39
				Mean	16.7	53.8
				StDev	8.4	13.9
					OK	OK

Note:In general, the standard deviation of checksums should not exceed 10 for the A axis and 20 for the B axis, per the manufacturer's recommendation. Therefore, the accuracy and precision of the positioning data for B-direction in this survey should not be considered ideal. The plot for the B-direction at MB-I-2 is still displayed as it may indicate movement at any given time.

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