November 15, 2024

Valerie Capels
Town Administrator
Town of Bristol
1 South Street, PO Box 249
Bristol, VT 05443

Re: 734189-DR4720VT – Bristol – Briggs Hill Road
Summary of Cost Opinions

Dear Valerie,

As requested, DuBois & King, Inc. completed an evaluation of costs for the proposed repair of two slope failures on Briggs Hill Road. This letter provides a summary of the following:

1. A summary of the applicable in-kind standards being applied
2. HydroCAD modeling results for culvert sizing
3. Design plans for the recommended repair (in-kind with codes and standards repair of the damages with mitigation
4. Costs for:
   a. In-kind pre-disaster repair of the damages
   b. In-kind with codes and standards repair of the damages
   c. In-kind with codes and standards repair of the damages with mitigation

## Applicable In-Kind Standards

Town of Bristol Road and Bridge Standards, July 22, 2019, are attached. Applicable in-kind standards that apply to this project include:

1. All new or substantially reconstructed paved roads shall have at least 15 inches thick gravel sub-base.
2. Section 6 – Guardrail standard.
3. Paved/ditched roads shall be crowned where repaving involves removal of existing paving: minimum 1/8 inch per foot or 1%; recommended: 1%-2%
4. Road drainage for roads with slopes 8% or greater: stone-lined ditch; recommended 2-foot ditch depth from top of stone-lined bottom. For slopes 8-10%, minimum 6 to 8-inch minus stone or the equivalent; for slopes >10%, minimum 6 to 8-inch minus stone or the equivalent, recommended 12-inch minus stone or the equivalent.

5. Roadway culverts minimum size 18-inch; end treatment or headwall required on slopes 5% or greater; stabilize outlet such that there will be no scour erosion; stone aprons or plunge pools required for new construction on road slopes 5% or greater.
6. Catchbasin outlet stabilization: all catchbasin outlets shall be stabilized to eliminate all rill and gully erosion, including stone-lined ditch, stone apron, check dams and culvert header/headwall.
7. Stone check dam specification.

The attached Basis of Stabilization Design Memorandum from GEODesign, Inc., geotechnical engineering subcontractor for the project indicates the following codes and standards apply for the required slope stabilizations:

1. State of Vermont's *Geotechnical Engineering Instruction on Soil Slope Stability Investigation and Evaluation* publication (VTrans GEI 14-01, Oct. 2014). These standards establish Factor of Safety (FS) values of 1.3 for slopes not directly supporting structures such as bridges and retaining walls and 1.5 for slopes directly supporting bridges and retaining walls.
2. US Army Corps of Engineers Slope Stability Engineer Manual (EM 1110-2-1902, Oct. 2003), which includes acceptable FS values consistent with #1.

According to the geotechnical slope stability modeling, the pre-slide slope conditions do not meet codes and standards because the FS requirements listed above are not met with an "in-kind" repair to pre-disaster conditions (see attached slope stability modeling results). The slope stabilization design measures that were developed required flattening (with revegetation) where possible, stone fill armoring, and retaining wall support to mitigate the unstable slide areas and meet FS codes and standards values, as shown on the attached slope stability modeling results.

## HydroCAD Modeling Results

As indicated above, Town codes and standards require minimum 18-inch roadway culverts. D&K completed HydroCAD modeling of the proposed culverts, the results of which are attached. The modeling results indicate the proposed 18-inch roadway culverts are sufficient to pass a 50-year design storm, which meets VTrans roadway standards.

## Design Plans

The attached design plans depict the proposed project, which includes in-kind plus codes and standards repairs, plus the following mitigation measures:

1. Catchbasin inlets for the proposed roadway culverts to reduce the roadway culvert slopes, reducing the erosive velocities of stormwater as compared to the current steeply-sloping roadway culverts.
2. Retaining walls and associated site work to achieve the slope geometry required to meet FS requirements.

**Cost Opinions**

Preliminary Opinions of Probable Construction Costs (OPCCs) for the following scenarios are attached:

1. In-kind repair only: $1,064,288 + 20% contingency = $1,277,288
2. In-kind repair with codes and standards: $1,930,009 + 20% contingency = $2,316,010
3. Mitigation: 85,950 + 20% contingency = $103,140
4. In-kind repair with codes and standards plus mitigation: $2,514,389

Please feel free to contact me with any questions.

Sincerely,

Jonathan B. Ashley, PE
Senior Vice President

Enclosures



DuBois & King Inc.

FEMA SUMMARY

734189-DR4720VT

Bristol – Briggs Hill Road

# Town Codes and Standards

**Town of Bristol**
P.O. Box 249
1 South Street
Bristol, VT 05443
(802) 453-2410
www.bristolvt.org

# TOWN OF BRISTOL
## TOWN ROAD AND BRIDGE STANDARDS

The *Selectboard* of *the Town of Bristol* hereby adopts the following Town Road and Bridge Standards which shall apply to the construction, repair, and maintenance of town roads and bridges.

The standards below are considered minimums. Municipalities that have construction standards / specifications in place that exceed the minimum standards: indicate adoption date and include as Appendix C. **Date of Adoption:** *Not applicable.*

Municipalities must comply with all applicable state and federal approvals, permits and duly adopted standards when undertaking road and bridge activities and projects.

Any new road regulated by and/or to be conveyed to the municipality shall be constructed according to the minimum of these standards.

Circle or underline **YES** or **NO** below to indicate town adoption of that section of the Standards

| **Road and Bridge Standards Sections** | **Hydrologically-connected road segments*** | **Non-hydrologically-connected road segments**** |
|---|---|---|
| Section 1 – Municipal Road Standards | YES (Required by Act 64) | YES NO |
| Section 2 – Class 4 Road Standards | YES (Required by Act 64) | YES NO |
| | **Town wide** | |
| Section 3 - Perennial stream- bridge and culvert standards | YES (Required by DEC Stream Alteration Standard) | |
| Section 4 – Intermittent stream crossings | YES NO | |
| Section 5 - Roadway construction standards | YES NO | |
| Section 6 - Guardrail standard | YES NO | |
| Section 7 - Driveway access standard | YES NO | |

**Road segments** – ANR Resources Atlas includes a map layer of all of Vermont's municipal roads divided into 100-meter (328 foot) segments, each with a unique identification number.

***Hydrologically-connected road segments** - are those municipal road segments and catch basin outlets, Class 1-4, as shown on the ANR Natural Resources Hydrologically-connected municipal road segment layer (http://anrmaps.vermont.gov/websites/anra5/) or the Road Erosion Inventory Scoring (MRGP Implementation Table portal) layer (https://anrweb.vt.gov/DEC/IWIS/MRGPReportViewer.aspx?ViewParms=True&Report=Portal)
.

****Adoption of standards on non-hydrologically-connected road segments** does not indicate that these road segments are then subject to the Municipal Roads General Permit (MRGP). Municipalities may also find additional resources in the latest version of the *Vermont Better Roads Manual.*
https://vtrans.vermont.gov/sites/aot/files/highway/documents/ltf/Better%20Roads%20Manual%20Final%202019.pdf

## Road and Bridge Standards Sections

**Section 1 – Municipal Road Standards** - See Appendix A.

These standards are required by Act 64 and the DEC Municipal Roads General Permit (MRGP) for hydrologically-connected roads only.

Municipalities may adopt Section 1 Road standards by road type for non-hydrologically-connected roads/segments/catch basins.

**Section 2 – Class 4 Road Standards** - See Appendix A 2.

**Section 3 - Perennial stream - bridge and culvert standards**

Bridge and culvert work on perennial stream crossings must conform with the statewide DEC Stream Alteration Standard.

***"Perennial stream"*** means a watercourse or portion, segment, or reach of a watercourse, generally exceeding 0.25 square miles in watershed size, in which surface flows are not frequently or consistently interrupted during normal seasonal low flow periods. Perennial streams that begin flowing subsurface during low flow periods, due to natural geologic conditions, remain defined as perennial. All other streams, or stream segments of significant length, shall be termed intermittent. A perennial stream shall not include the standing waters in wetlands, lakes, and ponds.

Streambank stabilization and other in-stream work must conform with the statewide DEC Stream Alteration Standard.

For River Management Engineer Districts:
https://dec.vermont.gov/sites/dec/files/wsm/rivers/docs/RME_districts.pdf.

**Section 4 – Intermittent stream crossings** – See Appendix B for sizing table and graphic. These standards are above and beyond the culvert standards in Section 1.

***"Intermittent streams"*** are defined as streams with beds of bare earthen material that run during seasonal high flows but are disconnected from the annual mean groundwater level.

**Section 5 - Roadway construction standards** – Sub-base and gravel standards

All new or substantially reconstructed gravel roads shall have _at least 12_ inches* thick gravel sub-base, with an additional inches* top course of crushed gravel.

All new or substantially reconstructed paved roads shall have _at least 15_ inches* thick gravel sub-base.

*Municipalities shall indicate their own construction criteria.

**Section 6 - Guardrail standard**

When a roadway, culvert, bridge, or retaining wall construction or reconstruction project results in hazards such as foreslopes, drop offs, or fixed obstacles within the designated clear-zone, the AASHTO Roadside Design Guide will govern the analysis of the hazard and the subsequent treatment of that hazard. For roadway situations, an approved barrier system may be steel beam guardrail with 6-foot posts and approved guardrail end treatment. If there is less than 3 feet from the rail to the hazard, then steel beam guardrail with 8-foot posts shall be used. The G-1D is an example of an approved guardrail end treatment. For bridge rails systems, VTrans bridge rail standards shall be referenced

**Section 7 - Driveway access standard**

The municipality has a process in place, formal or informal, to review all new drive accesses and development roads where they intersect town roads, as authorized under 19 V.S.A. Section 1111. Municipality may reference VTrans Standard *A-76 Standards for Town & Development Roads* and *B-71 Standards for Residential and Commercial Drives*; the VTrans *Access Management Program Guidelines*; and the latest version of the *Vermont Better Roads Manual* for other design standards and specifications.

**Passed and adopted by the _Selectboard_ of _the Town of Bristol_, State of Vermont, on _July 22_, 20_19_:**

| | |
|---|---|
| *Joel Bouvier*, **Chair** | [signature] |
| *Peter Coffey* | [signature] |
| *John Heffernan* | [signature] |
| *Theodore Lylis* | [signature] |
| *Michelle Perlee* | [signature] |

# Appendix A

## Section 1: MUNICIPAL ROAD STANDARDS

The following standards constitute the minimum required Best Management Practices (BMPs) for municipal roads. These standards shall apply to the construction, repair, and maintenance of all town roads and bridges.

It is the municipality's responsibility to maintain all practices after installation. Roads not meeting these standards must implement the BMPs listed below in order to meet the required town's standards.

## Feasibility

Municipalities shall implement these standards to the extent feasible. In determining feasibility, municipalities may consider the following criteria: The implementation of a standard listed in of this documentation does not require the acquisition of additional state of federal permits or noncompliance with such permits, or noncompliance with any other state or federal law. The implementation of a standard does not require the condemnation of private property; impacts to significant environmental and historic resources, including historic stone walls, historic structures, historic landscapes, or vegetation within 250 feet of a lakeshore; impacts to buried utilities; and excessive hydraulic hammering of ledge.

## Standards for All Construction and Soil Disturbing Activities

Following construction and soil disturbance on a road, all bare or unvegetated areas shall be revegetated with see and mulch, hydroseeded, or stone lined within 5 days of disturbance of soils, or, if precipitations is forecast, sooner.

## Standards for Gravel and Paved Roads with Ditches

### Baseline Standards for Gravel and Paved Roads with Ditches

The following are the standards for all gravel and paved municipal roads with drainage ditches, whether or not erosion is present. These standards also apply to all new construction and significant upgrades of stormwater treatment practices.

A. Roadway/Travel Lane Standards
    1. Roadway Crown
        a. Gravel roads shall be crowned, in or out-sloped:
        Minimum: ¼ inch per foot
        Recommended: ¼ inch to ½ inch per foot or 2% - 4%
        b. Paved/ditched roads shall be crowned during new construction,
        redevelopment, or repaving where repaving involves removal of the existing paving.
        Minimum: 1/8 inch per foot or 1%
        Recommended: 1% - 2%
    2. Shoulder berms (also called Grader/Plow Berm/Windrows)
    Shoulder berms shall be removed to allow precipitation to shed from the travel lane into the road drainage system. Roadway runoff shall flow in a distributed manner to the drainage ditch or filter area and there shall be no shoulder berms or evidence of a "secondary ditch". Shoulder berms may remain in place if the road crown is in-sloped or out-sloped to the opposite side of the road from berm side of road. The shoulder berm standard only applies to gravel roads with drainage ditches.

B. Road Drainage Standards

Roadway runoff shall flow in a distributed manner to grass or a forested area by lowering road shoulders or conversely by elevating the travel lane level above the shoulder. Road shoulders shall be lower than travel lane elevation. If distributed flow is not possible, roadway runoff may enter a drainage ditch, stabilized as follows:

1. For roads with slopes between 0% and 5%: At a minimum, grass-lined ditch, no bare soil. Geotextile and erosion matting may be used instead of seed and mulch. Alternatively, ditches may be stabilized using any of the practices identified for roads with slopes 5% or greater included in subpart B.2 below.

   Recommended shape: trapezoidal or parabolic cross section with mild side slopes; 2 foot horizontal per 1 foot vertical or flatter and 2-foot ditch depth.

2. For roads with slopes 5% or greater but less than 8%:
   a. Stone-lined ditch: minimum 6 to 8-inch minus stone or the equivalent for new practice construction. Recommended 2-foot ditch depth from top of stone-lined bottom,
   b. Grass-lined ditch with stone check dams[1], or
   c. Grass-lined ditch if installed with disconnection practices such as cross culverts and/or turnouts to reduce road stormwater runoff volume. There shall be at least <u>two</u> cross culverts or turnouts per segment disconnecting road stormwater out of the road drainage network into vegetated areas or spaced every 160 feet.

3. For roads with slopes of 8% or greater: Stone-lined ditch.
   a. For slopes greater than or equal to 8% but less than 10%: minimum 6 to 8-inch minus stone or the equivalent for new construction. Recommended 2-foot ditch depth from top of stone-lined bottom.
   b. For slopes greater than 10%: minimum 6 to 8-inch minus stone. Recommended 12-inch minus stone or the equivalent. Recommended 2-foot ditch depth from top of stone-lined bottom.

4. If appropriate, bioretention areas, level spreaders, armored shoulders, and sub-surface drainage practices may be substituted for the above road drainage standards.

C. Drainage Outlets to Waters & Turnouts

Roadway drainage shall be disconnected from waterbodies and defined channels, since the latter can act as a stormwater conveyance, and roadway drainage shall flow in a distributed manner to a grass or forested filter area. Drainage outlets and conveyance areas shall be stabilized as follows:

1. Turn-outs – all drainage ditches shall be turned out to avoid direct outlet to surface waters.

2. There must be adequate outlet protection at the end of the turnout, based upon slope ranges below. Turnout slopes shall be measured on the bank where the practice is located and not based on the road slope.
   a. For turnouts with slopes of 0% or greater but less than 5%: stabilize with grass at minimum. Alternatively, stabilize using the practices identified in subpart b – c below, when possible.
   b. For turnouts with slopes 5% or greater: stabilize with stone.
   c. For slopes greater than 5% but less than 10%: minimum 6-inch to 8-inch minus stone or the equivalent for new construction.
   d. For slopes greater than 10%: minimum 6 to 8-inch minus stone or equivalent for new construction. Recommend 12-inch minus stone or the equivalent.

[1] See check dam installation specifications.

### Drainage and Intermittent Stream Culvert Standards

The following are the required culvert standards for all gravel and paved roads with ditches where rill or gully erosion is present. These standards also apply to new construction and significant upgrades of stormwater treatment practices.

1. Municipal Culverts (Drainage and Intermittent Streams)
    1. Culvert end treatment or headwall required for areas with road slopes 5% or greater if erosion is due to absence of these structures. End treatment or headwall is required for new construction on slopes 5% or greater.

    2. Stabilize outlet such that there will be no scour erosion, if erosion is due to absence or inadequacy of outlet stabilization. Stone aprons or plunge pools required for new construction on road slopes 5% or greater.

    3. Upgrade to 18-inch culvert (minimum), if erosion is due to inadequate size or absence of structure.

    4. A French Drain (also called an Underdrain) or French Mattress (also called a Rock Sandwich) sub-surface drainage practice may be substituted for a cross culvert.

2. Driveway Culverts within the municipal ROW
    1. Culvert end treatment or headwall required for areas with road slopes of 5% or greater, if erosion is due to absence of these structures. End treatment or headwall is required for new construction.

    2. Stabilize outlet such that there will be no scour erosion, if erosion is due to absence or inadequacy of outlet stabilization. Stone aprons or plunge pools required for new construction.

    3. Upgrade to minimum 15-inch culvert, 18-inch recommended, if erosion is due to inadequate size or absence of structure.

## Standards for Paved Roads with Catch Basins

Catch Basin Outlet Stabilization: All catch basin outlets shall be stabilized to eliminate all rill and gully erosion. Catch basin outfall stabilization practices include: stone-lined ditch, stone apron, check dams and culvert header/headwall.

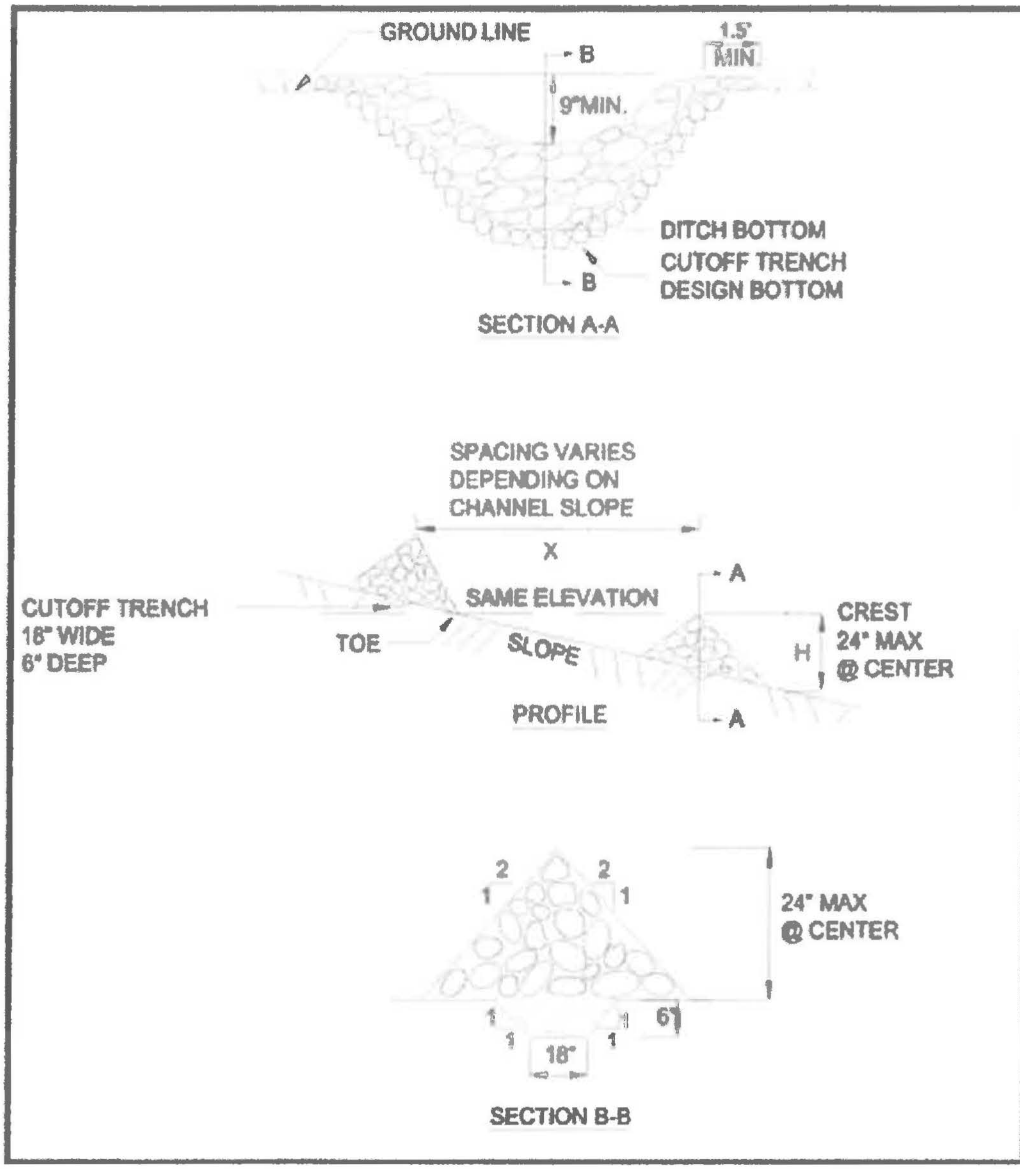
## Stone Check Dam Specification

- Height: No greater than 2 feet. Center of dam should be 9 inches lower than the side elevation
- Side slopes: 2:1 or flatter
- Stone size: Use a mixture of 2 to 9-inch stone
- Width: Dams should span the width of the channel and extend up the sides of the banks
- Spacing: Space the dams so that the bottom (toe) of the upstream dam is at the elevation of the top (crest) of the downstream dam. This spacing is equal to the height of the check dam divided by the channel slope.

  $$\text{Spacing (in feet)} = \frac{\text{Height of check dam (in feet)}}{\text{Slope in channel (ft/ft)}}$$

- Maintenance: Remove sediment accumulated behind the dam as needed to allow channel to drain through the stone check dam and prevent large flows from carrying sediment over the dam. If significant erosion occurs between check dams, a liner of stone should be installed.

**Check Dam Specification:**




## Section 2: STANDARDS FOR CLASS 4 ROADS

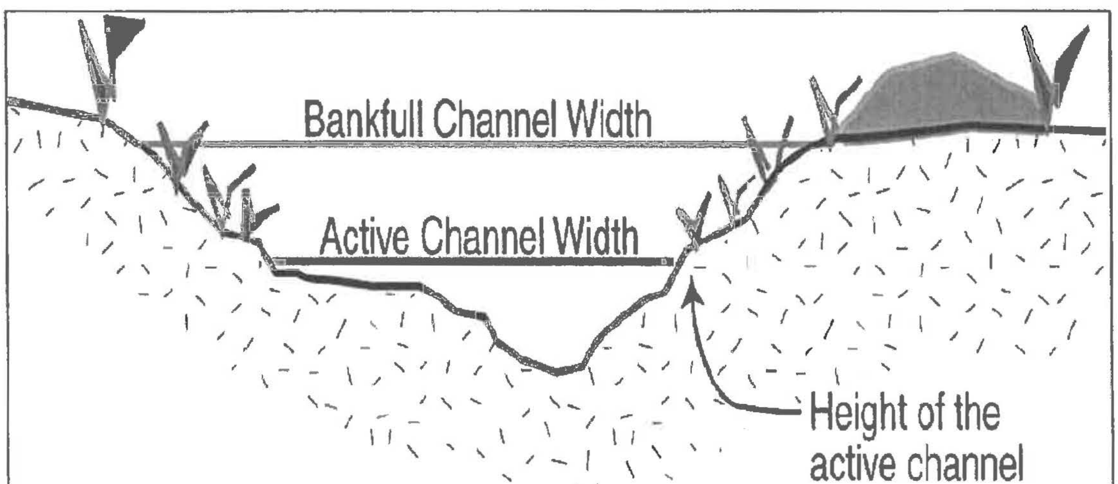
Stabilize any areas of gully erosion with the practices described above or equivalent practices. Disconnection practices such as broad-based dips and water bars may replace cross culverts and turnouts.

# Appendix B
## Active Channel Culvert Sizing for Intermittent Stream Crossings

Choose the drainage area closest to your crossing site drainage area

| Drainage Area *(Acres)* | Minimum Diameter for Culverts on Intermittent Streams *(inches)* |
|---|---|
| 4 | 15 |
| 8 | 18 |
| 16 | 24 |
| 20 | 30 |
| 40 | 36 |
| 50 | 42 |
| 80 | 48 |
| 120 | 60 |
| 160 | 66 |
| 200 | *Streams with drainage areas of 160 acres or greater are likely to be perennial. Adhere to the VTDEC Technical Guidance for Identification of Perennial Streams* |
| 320 | |
| 350 | |
| 450 | |
| 640 | |

## Active Channel Width




**Active Channel Width** means the limits of the streambed scour formed by prevailing stream discharges, measured perpendicular to streamflow. The active channel is narrower than the bankfull width (approximately 75%) and is defined by the break in bank slope and typically extends to the edge of permanent vegetation.

**Culvert sizing for crossings on intermittent streams:** Determine the Active Channel Width by field measurements, ***the culvert size should meet or exceed the Active Channel Width***. To obtain the measurements go to the crossing location and obtain several upstream Active Channel Width measurements in riffle (fast moving water) narrower channel locations. The selected channel width should be a representative average of the field measurements. In the absence of field measurements, the drainage areas in the table can be used.

FEMA SUMMARY

734189-DR4720VT

Bristol – Briggs Hill Road

# Geotechnical Codes and Standards

GEODesign, Inc.
85 Granite Shed Lane, Unit #1
Montpelier, VT 05602
(802) 674-2033

# MEMORANDUM

**TO:** Jonathan Ashley, P.E. – Dubois & King (D&K)

**FROM:** **GEODesign, Inc. –** Jason Gaudette / Jacob Wimett, P.E.

**DATE:** August 12, 2024

**RE:** Basis of Stabilization Design
Briggs Hill Road Slope Instability – Bristol, VT

**FILE NO.:** 0804-52

This memorandum addresses GEODesign's basis of stabilization design for the captioned project. Our geotechnical input so far has been provided in the form of sketches and markups on D&K's progress plans based on our stability evaluations. A summary report will be provided separately.

## BASIS OF STABILIZATION DESIGN (FACTORS OF SAFETY)

Appropriate Factors of Safety (FS) are determined using experience and considering uncertainty and confidence in slope stability input parameters and consequences of failure. Consistent with the AASHTO LRFD Bridge Design Specifications, we use the State of Vermont's *Geotechnical Engineering Instruction on Soil Slope Stability Investigation and Evaluation* publication (VTrans GEI 14-01, Oct. 2014). FS values of 1.3 and 1.5 are considered acceptable for evaluation and design of slopes not supporting structures and slopes supporting structures, respectively. FS values of 1.1 and 1.2 may be acceptable for extreme (e.g. seismic) and temporary (e.g. construction stages, storm surges, high groundwater) conditions. These minimum acceptable FS values for slopes are also consistent with the US ACOE Slope Stability Engineer Manual (EM 1110-2-1902, Oct. 2003).

## EXISTING & PROPOSED (REPAIR) SLOPE CONDITIONS

Slope conditions at the two Briggs Hill Road slide areas (Upper Region and Lower Region) do not meet acceptable FS values in their present condition, as supported by our stability modeling. We attempted to calibrate our existing models to match observations of the actual slide conditions (i.e., develop FS values of approximately 1.0). Since our initial involvement following the July/August 2023 heavy rain events, FS values have likely worsened due to the continued and measurable downslope movements that have occurred. Stabilization design measures were therefore developed that required slope flattening (with revegetation) where possible, stone fill armoring, and retaining wall support to mitigate the unstable slide areas and meet appropriate FS values.

We understand our evaluation and basis of stabilization design also meet requirements outlined in the Town of Bristol's Town Road and Bridge Standards (July 22, 2019) and minimum required Best Management Practices (BMPs) for municipal roads.

**Geotechnical Engineering Instruction on**

# SOIL SLOPE STABILITY INVESTIGATION & EVALUATION

VERMONT

**Approved:** Christopher C. Benda **Date:** 10/10/14

Christopher C. Benda, P.E.
Geotechnical Engineering Manager

VTrans GEI 14-01 October 2014

# Table of Contents

# 1.0 Introduction

## 1.1 Target Audience

This guidance document is intended to be used by VTrans Geotechnical Engineering staff. The secondary audience is the Consultant community who provide geotechnical engineering services to VTrans and whose services are reviewed by the Geotechnical Engineering Section.

## 1.2 Purpose

The primary purpose of this document is to provide a consistent approach for VTrans' geotechnical engineers and engineering geologists to follow when providing engineering services for <u>soil slopes only</u>. The recommended geological and geotechnical engineering procedures for rock slope projects will be provided under a separate document. Specific guidance is provided herein for the completion of soil slope stability evaluations to be used in the design and construction of existing and proposed slopes and embankments as well as in the remediation of failed slopes. Slope stability evaluations may include one or more of the following activities; site visits, subsurface investigations, laboratory testing and analyses, instrumentation and monitoring, engineering analyses and design and report documentation. Also outlined herein are VTrans' design preferences and interpretations, general slope remediation strategies and typical design details for slope remediation projects.

This document is also meant to provide the Consultant community with a description of VTrans' approach to slope stability evaluations so that they may tailor their services to meet VTrans' goals and objectives. It is not intended to be a comprehensive summary of all available techniques for slope stability analysis and mitigation design. This document may also serve as a checklist for VTrans to verify that a Consultant meets the desired minimum level of service. It is the intent that this document will be chapter in the Agency's developing Geotechnical Engineering Design Manual.

## 1.3 Project Definition, Type and Scope

A project is defined within the request for geotechnical engineering services, see Section 2.1. The entity requesting engineering services may be an Operations District or a Design (Structures, Highway Safety & Design, LTF, etc) Section. Projects, or attributes, requiring slope stability evaluations include, but are not limited to, highway embankments, proposed or existing side slopes adjacent to existing transportation facilities, retaining walls and failed soil slopes.

Typically slope failures are emergency projects managed by Operations Bureau personnel while proposed and existing slopes, or non-emergent projects, are typically managed by the Project Delivery Bureau.

Slope stability evaluations are performed for a wide variety of geotechnical engineering applications, including, but not limited to the:

- determination of short and long term stability of both temporary and permanent cut and fill slopes including staged embankment construction;

- assessment of overall stability of retaining walls, including global and compound stability (including permanent systems and temporary shoring systems);

- assessment of overall stability of shallow and deep foundations for structures located on slopes or over soft soils,

- analysis of slope stability due to rapid drawdown caused by lowering of the groundwater table and/or the removal of stabilizing effects of water adjacent to slopes;

- stability assessment of landslides (mechanisms of failure and determination of design properties through back-analysis), and design of mitigation techniques to improve stability, and;

- evaluation of instability due to construction equipment vibration or seismic induced liquefaction.

**1.4 Document Overview**

This document provides an overview of the stages for a typical slope stability project including the initial site assessment, project documentation and project management. This guide also outlines the steps followed for the geotechnical investigation from the initial scoping activities to the subsurface investigation. The laboratory testing program details what tests are routinely conducted and for what soil types. The analysis and design procedures documented herein describe the procedures used to model the in-situ soil profile and groundwater conditions as well as the required factor of safety and methods typically utilized to conduct various slope stability analyses.

In Chapter 6, a description of both structural and non-structural remediation strategies is presented along with a discussion of the mitigation selection process, cost estimating and VTrans' preferred design alternative. This section is followed by a discussion of what items need to be included in the project's geotechnical report, what details and drawings need to be prepared, specification review and any construction or post-construction monitoring and inspection requirements. A list of references used to develop this document is included in Chapter 11 just prior to the Appendices.

**1.5 Document Limitations**

- ❖ As mentioned previously this document does not cover the investigation, analysis and design of rock slopes.

- ❖ This document will not provide adequate information for every aspect of every slope stability evaluation. Therefore the user is cautioned and encouraged to consult additional resources to ensure that a sound engineering approach is utilized and that all risk factors have been adequately considered. Therefore, when necessary, it is expected that additional reference and textbooks both within and outside the

Geotechnical Engineering Section library will be consulted to gain a better understanding of the geotechnical engineering fundamentals outlined herein.

- ❖ Consultants are encouraged to use this document to understand VTrans' general procedures, however, they are responsible for their own field, design and reporting procedures. The Consultants assume all liability for their designs and evaluations. Consultants should use this guide as a litmus test to ensure that they have provided a level of service equal to or greater than the level of service described herein.

- ❖ This manual is intended for use by VTrans personnel and recommended for consideration by its Consultants and is not intended to be a literary document. Sections within this report are referenced directly from the sources identified with only limited recognition of the source. In other words, this document is a compilation of a review of current VTrans procedures and documents developed and produced by other States (particularly WSDOT and NYSDOT), FHWA, and Southern California Earthquake Center (SCEC).

- ❖ This document was developed based on the understanding that it would be updated every 5 years to accommodate the knowledge, skills and abilities of Geotechnical Engineering personnel and to ensure that the use of current technologies was integrated into the Section's procedures.

# 2.0 Project Determination, Assessment and Management

## 2.1 Requests for Geotechnical Assistance

Requests for geotechnical assistance are made to the Geotechnical Engineering Manager. These requests are used as the basis to allocate and prioritize State resources. Requests for geotechnical assistance with regards to slopes generally involve a slope failure although guidance can also be given for constructing new slopes or embankments.

Requests for geotechnical assistance to review a slope's stability may come from many different stakeholders. Internally, within VTrans, requests are most frequently received from Districts within Operations and Design Units within the Program Development Division. Externally, requests are most often received from towns or municipalities. The entity requesting assistance, or the requestor, should complete a Geotechnical Services Request Form. The requestor should provide any available pictures, project description and background information as well as any other pertinent information of the issue to convey an initial condition or assessment as to the scope of the problem. When possible, the requestor should also include a statement about the current level of severity of the problem and whether the situation is emergent.

## 2.2 Initial Site Assessment Report

Upon receipt of the geotechnical services request form, the Geotechnical Engineering Section will schedule an initial site visit. The Section will develop an Initial Site Assessment Report based on the findings of the initial site visit.

The Initial Site Assessment Report shall identify the location of the site and the owner of the facility. For infrastructure that is maintained solely by a Municipality, initial site assessments are typically provided by the Geotechnical Engineering Section as a professional courtesy. Once the report on the initial site visit has been prepared by the Geotechnical Engineering Section, the Municipality is provided with a short list of geotechnical consultants with whom they are encouraged to coordinate and facilitate the remaining project development process steps. Once the Municipality has contracted with a Consultant the Agency is no longer involved in the design process, however, the municipality may request the Agency to review the Consultant's analysis and design.

The goals of the Initial Site Assessment Report are:

- to identify the project owner, as this has a direct effect on the scope of involvement and level of engagement by Geotechnical Engineering Section;

- to determine the potential scope of the project and to determine whether or not the geotechnical issue is sufficiently complex to warrant a geotechnical investigation; and

- to identify operational needs and concerns, specifically related to the mobility and safety of the transportation system users.

Simple slope failures within VTrans' ROW that have limited impact and a well defined failure surface may be assessed by VTrans on-site, without the need for an in-depth geotechnical investigation. If this is the case, recommendations will be provided in person with written follow-up documentation performed via e-mail. Documentation may include a sketch or typical section (See Section 8.2) for the Operations' District personnel or Contractor to use during construction.

For slopes that require a geotechnical investigation, the Project Manager or Sponsor should be decided whether the project will be treated as an emergency repair scenario and managed by the District, or will be an Agency construction project as programmed through the efforts of the various design sections such as the Operation's Rail Section and the Highway Safety & Design Section (HS&D). This decision depends on project variables such as the scope of the remediation, cost of the repair and the construction or design schedule of an adjacent project. The decision making process is typically a collaborative effort with participation from personnel representing both the Highway Safety and Design Section and the local District based on input from the Geotechnical Engineering Section.

From a maintenance and safety viewpoint the Initial Site Assessment Report should address questions such as,

- "What is the likelihood that the slope continues to move and what are the impacts to both Agency personnel and public safety?"

- "Can the site be traversed and drilled in a safe manner?"

- "Can the road remain open with appropriate precautions, e.g. signs, temporary barriers?" "Should the road be reduced down to a single lane or closed completely?"

The Geotechnical Project Engineer (Engineer) shall complete the site condition survey form in Appendix A to be used as a basis for determining the necessity of a more thorough geotechnical investigation. If a geotechnical investigation is deemed necessary then the Engineer may elect to conduct the official site investigation immediately or schedule a future date to return to the site and perform a more detailed site investigation.

### 2.3 Project Documentation

The Engineer shall establish the digital and paper file folders necessary to store project information. The Engineer shall place the site condition survey form in the appropriate project folder. All paper files shall also be stored electronically in the appropriate project file folders in the Geotechnical Engineering Section folder on the Z:/drive.

Multiple photographs of the site should be taken to document the extent of the problem, including but not limited to: the roadway approach from both directions, the uphill and downhill direction of the slide, any nearby water features (including culverts, drainage ditches and drop inlets), and the toe of the slope. The photos should be downloaded and placed in the appropriate digital folder established for the project. Sketches depicting the site layout and location of important features should be retained in the project documentation.

An e-mail shall be prepared documenting the findings of the Initial Site Assessment. The e-mail shall include key photographs or sketches so that the extent of the issue(s) can be communicated effectively. The distribution list may include one or more of the following:

- the Geotechnical Engineering Manager,
- District Transportation Administrator or Highway Safety & Design Project Manager,
- and any other stakeholders present at the time of the Initial Site Assessment.

### 2.4 Project Management

Once it has been decided whether the project will be managed as an Operations sponsored repair project, or a “District Needs” project programmed through the HS&D Section (as described in Section 2.2 above), the role of Geotechnical Engineering Section will become more clearly defined.

If the project is to be programmed via HS&D, a transportation project manager from HS&D will be assigned to the project, and will coordinate all activities. The Agency project manager, typically someone from within the District or HS&D, will coordinate all requests to various sections and maintain the administrative control over the project, with Geotechnical Engineering personnel providing technical assistance.

For HS&D programmed projects, Agency design squads will interpret the geotechnical report and develop construction plans and specifications with guidance and reviews performed by the Geotechnical Engineering Section.

If the project is managed by the Operations Bureau, then the Geotechnical Engineering Section will provide an increased supportive role; often providing technical assistance in the preparation of construction plans and specifications.

Regardless of who is responsible for the plan development, the Engineer will direct the subsurface investigation and subsequent analyses. The Engineer will provide a geotechnical report with remediation recommendations and projected costs. In addition, the Engineer must review the design details and typical sections coupled with a general review of the developed plans to ensure that the contract documents accurately reflect the intended design.

# 3.0 Geotechnical Investigation

## 3.1 Initial Scoping Activities

Initial scoping activities address some of the same considerations as the Initial Site Assessment. The primary difference between the two activities is that the initial scoping activities include a desktop review of geotechnical, geological and environmental data which cannot be completed in the field, such as a review of water well information and determinations of critical, protected environmental resources. In addition, based on the results of the Initial Site Assessment Report the Agency may decide not to conduct a geotechnical investigation in which case scoping activities would not be performed.

Initial scoping activities are initiated once the geotechnical investigation has been authorized by the Geotechnical Engineering Manager based on the Request for Geotechnical Assistance and the Initial Site Assessment Report. Initial scoping activities may include identification and compilation of potential resources pertaining to any assets existing adjacent to or within the slide location, including but not limited to historical and archeological assets, environmental resources (wetlands, endangered species, river/lake gauge information, groundwater characterization), right-of-way (ROW), aerial photographs, record plans and subsurface information from commonly used resources such as past projects, Agency's map of subsurface boring information and Agency of Natural Resources – Natural Resources Atlas. Information may also be obtained from Town officials or VTrans Operations personnel during the site visit and/or with abutting property owners. Refer to Section 4.2 of VTrans' MREI 11-01, for additional information.

## 3.2 Resource Identification (Survey, Hydraulics, Environmental, ROW)

If a topographical survey has not been completed for the slope stability project, the Engineer shall submit a request to the Agency's Project Manager as soon as possible. Topographical surveys may be requested by the Geotechnical Engineering Section in an effort to coordinate efficient mobilization of resources as the Survey Unit often obtains boring location information and their efforts can be synced to limit Agency efforts. The Project Manager should be copied on these requests to keep them aware of the project status.

Important features to consider when recommending limits for survey include the grade of any uphill/downhill slope, the full geometry of a river or waterway, nearby drainage features, and the existing condition of the road or adjacent infrastructure. The survey should capture any scarps, cracks, depressions, bumps, humps or bulges evident within the project area. At a minimum the areal extent of the topographical survey should extend 100 ft around the physical perimeter of the slide.

In order to complete a thorough analysis of remediation options, it may be necessary to obtain a hydraulic study, identify environmental resources (wetlands, etc.), and identify the limits of the state owned ROW. Therefore, ROW information should be requested at the same time as the field survey. ROW information should be plotted on the final field survey. It is the responsibility of the Project Manager to initiate these requests and coordinate with the appropriate sections; however the Engineer should be aware that the requisition of these resources will impact the project delivery process including the design time and project construction timeframes. The Engineer shall work with the Project Manager to ensure the completion of the aforementioned activities.

### 3.3 Site Reconnaissance

The primary purpose of the site reconnaissance is to determine the field location of the geotechnical borings based on existing site conditions. As part of the site reconnaissance the Engineer should document accessibility of drilling and investigation equipment and make an initial determination of what type of equipment might be best suited for the site conditions. If site preparation is necessary, the Engineer shall document the type of equipment, such as a dozer or excavator that may be needed to construct safe access for the desired drilling equipment. The Engineer shall describe any potential conflicts with overhead and/or underground utilities, site access, private property or other obstructions.

Utility clearances are required prior to initializing the actual drilling process. Utility locations will influence where explorations can be performed. The availability and distance to water should be noted as well as whether coring or mud rotary drilling methods are anticipated. The Engineer shall note any traffic control items required to accomplish the field exploration program safely while considering the practical aspects of the proposed drilling plan with regard to public impacts.

The site reconnaissance should describe the material composition of any streambeds including the size of any cobbles or boulders which may have an impact on drilling or construction activities such as the installation of steel sheeting, H-piles, soil nails or ground anchors. Notes should be made as to which type of drilling is best suited to the site. The site reconnaissance is also the appropriate stage to note any special sampling or testing equipment needs, such as undisturbed sampling or vane shear equipment. Sites with difficult access or a large areal extent or complexity, as determined by the Engineer, may require site reconnaissance participation by the drill crew supervisor so as to take advantage of his/her technical expertise.

### 3.4 Drilling & Subsurface Investigation

The primary goal of any subsurface investigation is to accurately document and report soil and groundwater conditions. The development of the subsurface investigation program should consider the results of the initial site assessment report, initial scoping activities, resource identification and the site reconnaissance. Geotechnical investigations such as borings and test pits, coupled with field survey information are used to estimate the three-dimensional geometry of a site. Selection of the number, location, and depth of borings are critical decisions that should be carefully considered during the development of the field work order and prior to conducting any field investigations. A comprehensive subsurface investigation should be conducted to enable the determination and characterization of the upper and lower limits of the failure zone as well as the soils above and below the failure zone, including soils which have not been subjected to any movement.

If the failure mechanism for a "simple" [a "simple" failure is typically defined as a crescent shaped failure surface at the top of slope combined with a known failure point at the toe of slope] rotational slope failure or shallow translational failure can be reliably established in the field, then it may be possible to avoid conducting an in-depth field investigation and use simplified design charts, see Section 5.7.3.2. In this case the Engineer may decide to collect "grab samples" to be tested for index properties to confirm the physical properties of the surficial soil layers.

### 3.4.1 Field Work Order

The Engineer will complete a Field Work Order to identify the required subsurface information to be obtained in order to characterize the site conditions and failure mechanisms. The field work order provides instruction to the drill crew(s) so that they can obtain all necessary field information in a timely and efficient manner. Depending upon the complexity of the instability, the geotechnical investigation may include multiple field work orders and subsurface investigations. A thorough understanding of the possible soils that may be encountered will enable the Engineer to better predict field and laboratory sampling and testing requirements for the project. Whenever feasible, drilling methods should be specified that limit the use of drilling fluids including water.

Continuous sampling is often required to locate zones of potential weakness. For most projects, samples should be obtained at maximum vertical intervals equal to 5 feet. For "follow-up" borings drilled in the general area of the initial borings, it is preferable to identify major changes in material types and perform additional field and laboratory testing to develop a better understanding of the soil strata. This may require multiple borings in one area to identify weak soil layers, and to facilitate the sampling, testing and instrumentation needs of the field investigation.

The Field Work Order should restrict the use of water as a drilling fluid for slope stabilization projects. It is the Agency's preference to maximize the use of hollow stem augers for its slope stabilization projects.

### 3.4.2 Boring Locations and Depths

The number of borings drilled for each project is a function of the project's magnitude and scope, information from previous investigations, and the complexity of the geologic features being investigated. Sound geologic and engineering judgment is required to estimate the number of borings required for a specific site. Preliminary requirements for the number of borings and corresponding depths may be found in VTrans' *MREI 11-01 "Geotechnical Guidelines for Subsurface Investigations*," with additional information found in FHWA's *Engineering Circular No. 5 (GEC 5), Evaluation of Soil and Rock Properties*.

For slope failures less than 500 ft wide at the toe of slope, a single row of borings, drilled perpendicular to the highway's centerline, is typically specified as shown in Figure 1. Borings are located to coincide with the apex of the slide failure surface with one boring above the slide area and one boring at the crest and toe of slope. Normally a single monitoring well and inclinometer are installed for slides of this magnitude as shown in Figures 1 and 2. The monitoring well is placed outside the failure zone while the inclinometer is typically placed behind the guard rail near the top of the slope. If desired and practicable, a third boring is beneficial and, if used, is routinely installed at the toe of the failed slope.

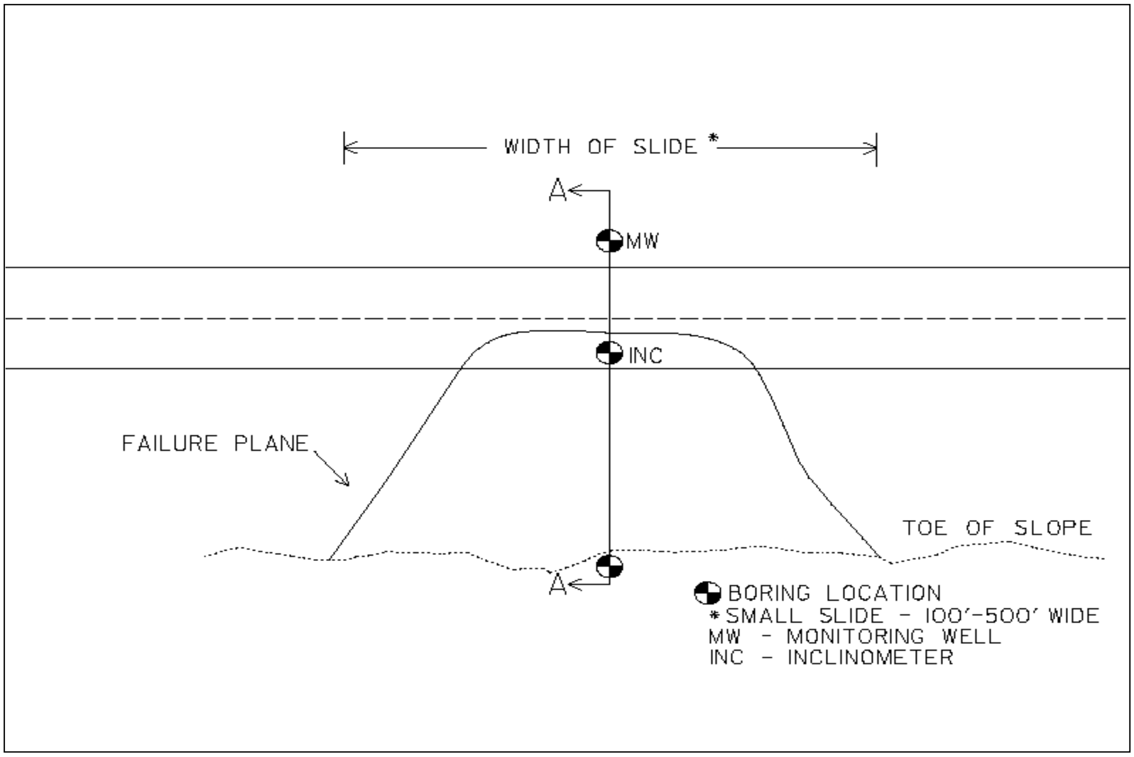



**Figure 1: Small Slide Boring Layout**

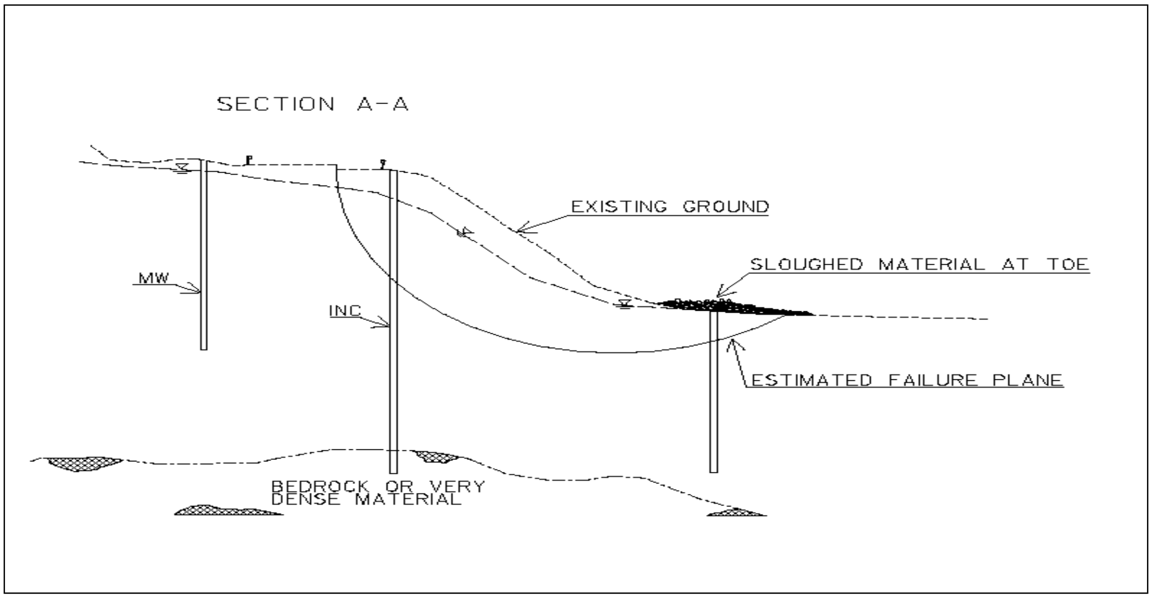



**Figure 2: Cross Section A-A**

When the slope failure length (normal to the roadway), measured at the base of the slide, exceeds 500 feet additional inclinometers and monitoring wells should be considered. For larger slides, 500 to 1000 feet in width, an initial boring plan similar to the one shown in Figure 3 is utilized. These additional borings are generally spaced 200 to 400 feet apart. Borings in slide areas should establish the full geological cross section necessary for stability analyses. Borings should be positioned such that extrapolation of geologic conditions is minimized within the areas of interest. For highway fill slopes, or embankments, standard practice is to extend the boring to a depth of twice the fill height whereas borings for cut slopes are extended at least 15 feet below the depth of the cut.

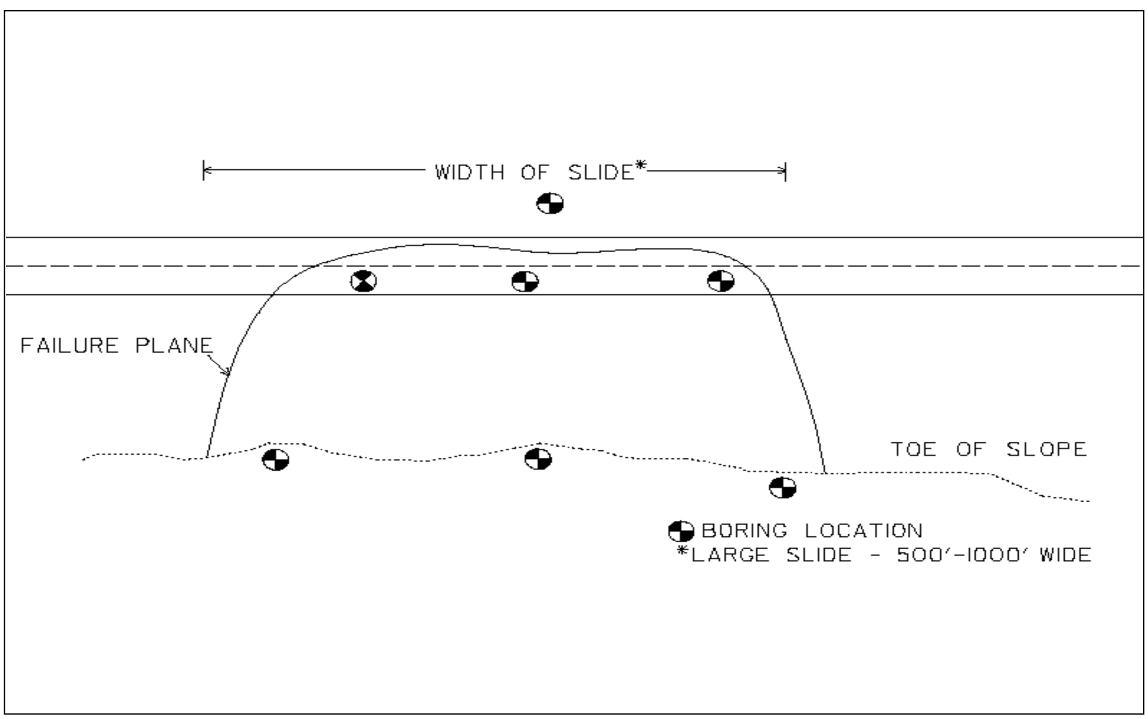



**Figure 3: Large Slide Boring Layout**

Borings can be expected to be added as the subsurface investigation progresses. In addition, boring depths may be increased pending the results of the instrumentation installation and subsequent design analyses. In general, the depth of the borings should be greater than any potential estimated failure surfaces. Additional borings are typically required for many cases where the geology or failure mechanisms are complex. The extent to which additional borings are needed may not be known for several months to a year after the installation of instrumentation and the analysis of the initial data has been completed.

### 3.4.3 Groundwater Determination

The majority of slides are caused by issues related to water; either surface or subsurface water (often referred to as groundwater). The presence of groundwater in a slope can reduce effective stresses when positive pore water pressures develop, causing a reduction in shear resistance. Groundwater can also increase the de-stabilizing forces in the slope due to the additional weight associated with a saturated soil mass or seepage forces.

Groundwater levels depend on a number of geotechnical, hydrological, and hydrogeological factors, including soil permeability, geology, original profile of the groundwater level, intensity and duration of rainfall, amount of antecedent rainfall, rate of surface irrigation, rate of evapotranspiration, rate of waste water disposal and groundwater flow from adjacent areas.

Groundwater elevations should be noted at the beginning of each day's drilling activities. If an artesian ground water condition is encountered, every effort should be made to determine the height (pressure head) of the water discharging above the ground elevation. This can be performed by placing additional casing lengths above the ground surface. The depth and thickness of the soil layer at which the artesian condition was encountered during drilling should be noted as it has a direct impact on the analysis and type of mitigation selected.

### 3.4.4 Field Sampling & Testing

Field tests conducted for slope stability analyses include one or more of the following tests; standard penetration testing (SPT), cone penetrometer testing (CPT), borehole shear tests (BST), vane shear testing (VST) in combination with the laboratory testing of undisturbed (Shelby) samples when applicable. These sampling and testing activities are intended to assist the Engineer in the determination of the soil's in-situ shear strength and density. Extracted undisturbed soil samples are transported to the Agency's laboratory for testing. Although field sampling methods and techniques are not covered in depth in this document, a number of resources are available in the list of references at the end of the document. Pocket penetrometer and shear torvane methods should only be used as indices to provide the Engineer with an approximate range of the consistency and strength of the cohesive soil.

Selection of sampling techniques should consider the effects of shear strain to the soil. Commonly available sampling techniques include driving thick-walled samplers advanced by means of hammer blows, pushing thin-walled (Shelby) tube samplers advanced by static force, and hand-carving test pit samples.

There are two types of thick-walled driven samplers that are most often used in practice; the SPT split spoon sampler, which has a 2.0-inch outside diameter and 5/16-inch wall thickness, and the California sampler, which typically has a 3.0- to 3.3-inch outside diameter, 1/4- to 3/8-inch wall thickness, and internal space for brass sample tubes (which typically are stacked in 1.0-inch increments). Pushed thin-walled tube samplers are typically 3 to 5 inches in diameter with approximately 1/16 to 1/8-inch-thick walls. When configured with a 3.0-inch outside diameter and advanced with a simple static force, they are referred to as Shelby tubes (ASTM D1587).[1]

The selection of a sampling method for any given soil type should take into consideration the disturbance associated with field sampling, including those disturbances associated with transportation/shipping and handling. Tube samplers require specimen extrusion and trimming, whereas the brass rings used in California samplers can be directly inserted into direct shear or consolidation testing equipment.

Some general sampling guidelines are provided below:

1. The strength of clean granular soil (except gravel) is generally best estimated with correlations from normalized standard penetration resistance (SPT blow counts). CPT tip resistance values can be used to supplement, but should not replace, SPT blow counts for use in correlations. Blow counts from California samplers are not an acceptable substitute for SPT blow counts.

2. Thick deposits of soft to medium stiff clay (i.e., Lake Champlain or varved Connecticut Valley clays) should be sampled with pushed thin-walled tubes or a hydraulic piston sampler. Such soil is readily amenable to laboratory specimen extrusion. Hand-carved specimens are an acceptable substitute for tube samples.

3. Stiff to hard cohesive soil and clayey bedrock materials (claystone, shale) can be sampled with California samplers, Pitcher tube samplers, or cored. Soil strengths established from drained laboratory testing of such specimens are likely to be conservatively low with respect to in-situ conditions. Hand-carved specimens are a desirable substitute for tube and driven samplers.

4. Intact rock should be sampled by coring. Jointed or bedded bedrock often contains planes or zones of weakness, such as slickensided surfaces, gouge zones, discontinuities, relict joints, clay seams, etc., which control the strength and, therefore, the stability of the deposits. Sampling must be carefully performed so that the thin planes or zones of weakness are not missed. For more detail on rock sampling and testing contact the Agency's Geologist.

5. A conservative estimate of shear strength along unweathered joint surfaces in rock masses can be obtained by pre-cutting an intact rock specimen in the laboratory and then shearing the sample in a direct shear device along the smooth

[1] http://www.scec.org/resources/catalog/LandslideProceduresJune02.pdf

cut surface. The strength obtained from the pre-cut sample is generally a conservative estimate because actual joint surfaces have asperities not present in the lab specimen. Alternatively the rock may be repeatedly sheared without precutting the sample. The objective in sampling for this type of testing is therefore an intact rock specimen.

6. Hand carving of clay block samples may be required to preserve sample integrity.

7. For newly compacted fills, bulk samples of borrow materials can be obtained for re-molding and compacting in the laboratory.

8. For soil containing significant gravel, correlations with penetration resistance can be used to estimate strengths and relative densities. Correlations with penetration resistance are based on SPT blow counts and engineering judgment which needs to be exercised for materials with high gravel contents.

Detailed procedures for sample handling and transporting, developing testing procedures, and sample testing can be found in the Agency's MREI 12-01 – Geotechnical Guidelines for Sample Handling, Testing and Data Reporting. Important notes from this MREI regarding sampling are reproduced below;

> *It is important that soil and rock samples used for design should not be averaged across multiple strata (samples with differing properties should not be comingled). Therefore, when there is a significant change in strata, the retrieved sample may need to be segregated into two separate lab samples for testing. All samples shall be collected using care to preserve sample integrity. Care should be used to avoid subjecting the sample to conditions which might alter the properties of the material such as freezing, excessive heating or contamination. Improperly handled samples can lead to poor test data that does not reflect actual geotechnical conditions.*

### 3.4.5 Geotechnical Instrumentation

Geotechnical instrumentation functions can be divided into two primary applications for slope stability assessment: investigation and monitoring. Although the surficial sloughing of slopes can be readily identified in the field, many other failure mechanisms are not as easily defined. Thus the Agency utilizes inclinometers to monitor movement of soil mass 1) at suspected locations and verify the behavior, location and depth of the failure surface against theoretical predictions(s) and 2) to monitor constructed solutions (embankments, fill and cut slopes, and retaining walls). Routine instrumentation installations include both monitoring wells and inclinometers.

The installation of remote sensing equipment should be considered when there is a concern for catastrophic movement which requires early detection of ground displacement OR the purchase of the instrumentation can be justified due to travel expenses and extended monitoring duration.

#### 3.4.5.1 Inclinometers

It should be noted that the movement of landslides can occur across multiple soil layers and multiple failure surfaces. Accordingly, locating the shallowest potential slide plane at a site may not be sufficient as there may be potential for a deeper failure surface underneath. Inclinometers should sufficiently penetrate (30 ft) a stable soil mass (such as a very dense glacial till) to limit the risk of misinterpreting or not identifying a shallow failure surface.

Inclinometers are constructed from solid pieces of PVC casing. The PVC casing has 4 longitudinal grooves spaced 90 degrees apart in the inside of the casing. During installation, these four longitudinal grooves should be positioned both perpendicular and parallel to the highway. A probe connected to a data acquisition box is lowered into these grooves to determine the verticality and subsequent displacement of the borehole.

Inclinometers detect lateral slope movements and are installed to ascertain the depth and layer(s) where the potential failure surface(s) reside. For smaller slides with widths of approximately 100’ to 500’ and heights less than 40’, one inclinometer is generally sufficient. The general location for a single inclinometer is the top of the slope; see Figure 1, perhaps just behind existing guardrail but within the failure zone. Larger and more complex slides may require multiple inclinometers, with a minimum of one inclinometer at the top and bottom (toe) of the slope.

The Engineer establishes the inclinometer monitoring frequency for each project; however, typically slopes are monitored frequently at the discovery stage of the slope instability when little information is known about the subsurface soils and the time history of the failure. Monitoring frequency tends to decreases with time depending upon measured movements and the likelihood of a failure. For construction projects, instrumentation monitoring frequency is initially established at a high frequency although the overall schedule can be highly variable. The monitoring frequency is dependent upon several factors including construction activities, weather and time. For more information on inclinometers review the Transportation Research Circular entitled *Use of Inclinometers for Geotechnical Instrumentation on Transportation Projects*.

In addition to conventional slope inclinometers, in-place inclinometers, shape acceleration arrays and rod extensometers can be used for more advanced monitoring of unstable slopes. These instruments are expensive specialty equipment not frequently used by the Agency, and should only be used on high value (i.e. significant cost or high risk) projects. In addition to inclinometers, tiltmeters can also be used to monitor the displacement of structures.

### 3.4.5.2 Monitoring Wells

Monitoring wells are boreholes that are sleeved with a combination of slotted and solid PVC pipe. These wells are backfilled with uniformly graded sand in the areas of the slotted pipe and native material or bentonite in the areas of the solid pipe. Monitoring wells are typically used to measure variation in seasonal groundwater depths. Monitoring wells are usually installed on the uphill side of the slide and outside the apparent failure zone. Typically monitoring wells are of the open standpipe variety where the top of the well is at atmospheric pressure. The groundwater table can be measured manually with a tape and probe set-up. To gain a better understanding of groundwater fluctuations, water level data loggers can be installed within the standpipe to record water levels at discrete time intervals. The data can be uploaded periodically from the field eliminating the need for frequent site visits.

Other types of groundwater instrumentation include piezometers which can be utilized to obtain pore water pressures which are then correlated to groundwater levels. Monitoring wells can be utilized to identify the groundwater characteristics for a project including artesian conditions. Products can be purchased that allow for the measurement of multiple groundwater depths (multi-channel tubing).

Field testing of monitoring wells can include assessments of hydraulic conductivity (infiltration study or well pump tests) and soil permeability testing; data which could be useful in determining the influence of groundwater on the overall factor of safety.

## 4.0 Laboratory Testing

Specific information on laboratory testing, including standard tests performed and associated AASHTO methods can be found in the Agency's MREI 12-01 – Geotechnical Guidelines for Sample Handling, Testing and Data Reporting.

VTrans utilizes a combination of in-house (mostly) and outside testing (less often) capabilities to determine soil shear strength parameters for both cohesionless and cohesive soils. Cohesionless soils generally undergo a particle size analysis and possibly some screening for Atterberg Limits; see Section 4.2 for more details.

Consolidated-undrained (CU) and consolidation testing shall be conducted for each major cohesive soil layer in order to reliably characterize each layer's soil strength parameters. The Engineer shall request CU, direct shear and consolidation testing directly from the Soils Laboratory Technician via the Geotechnical Engineering Manager. Cohesive soils are much more complex and the shear strength of these soils are a function of strain rate, drainage conditions during shear, effective stresses acting on the soil prior to shear, the stress history of the soil, stress path, and any changes in water content and density that may occur over time. These factors influence the degree to which the soil undergoes a contractive or dilatant reaction to applied shear. This reaction strongly influences soil strength and the stress-deformation response.

What is significant about these factors from the standpoint of soil sampling is that the factors may be changed or lost as a result of sample disturbance, causing the properties of laboratory specimens to deviate from those of in-situ soil. Therefore, the degree to which these factors are adequately represented in strength testing is also a function of the sample disturbance associated with the chosen sampling procedures. Due to the strong dependence of soil strength on these factors, methods of soil sampling and testing (which can potentially alter the above conditions for a tested sample relative to in-situ conditions) should undergo careful selection and review.

Residual friction values are more appropriate for use for soils that have undergone significant strain. The clay particles align with increased movement creating slickened-sides along the shear plane. Once a slope failure has occurred and a continuous slickensided failure surface has developed then only the soil's residual friction remains to resist the movement. In other words, as the shearing resistance decreases, and the movement increases, the soil approaches the residual friction angle. Direct shear, direct simple shear and torsional ring shear testing have been used by the Agency to assess the residual strength of soils.

## 4.1 Moisture Content, Particle Size Analysis and Soil Classification

At a minimum, soil classification and index testing should be performed on soil and rock samples extracted from subsurface borings drilled during slope stability investigations. The mass of a soil sample is measured prior to conducting a particle size analysis to assist in the determination of the in-situ water content. Once the sample is dry the material is placed in a set of sieves and the particle size is analyzed. Based on the results of this test the material is classified in accordance with the AASHTO and USCS soil classification systems.

## 4.2 Atterberg Limits and Soil Behavior

Laboratory testing for Atterberg Limits includes plastic and liquid limit tests used to determine the soil's plasticity index. As mentioned in Section 4.1, the natural water content of extracted soil samples is also determined as part of the Agency's routine testing; this allows for the determination of the liquidity index. The following tests provide insight on how a soil with plastic characteristics will perform or behave.

### 4.2.1 Plastic Limit

The plastic limit (PL) is the moisture content at which a soil transitions from being in a semisolid state to a plastic state. For additional references, see AASHTO T90 - *Standard Method of Test for Determining the Plastic Limit and Plasticity Index of Soils*.

### 4.2.2 Liquid Limit

The liquid limit (LL) is defined as the moisture content at which a soil transitions from a plastic state to a liquid state. For additional references, see AASHTO T89 - *Standard Method of Test for Determining the Liquid Limit of Soils*.

### 4.2.3 Plasticity Index

The larger the percentage of clay minerals and the more active the clay mineral, the more complex and difficult the behavior of the soil can be to predict. The plasticity index, or PI, is a useful indicator to screen for potential problems including swelling, creep, strain softening and changes in behavior due to physiochemical effects. In general, higher values of PI are more indicative of poor performing soils.

The plasticity index (PI) is defined as the difference between the liquid limit and the plastic limit of a soil. The PI represents the range of moisture contents within which the soil behaves as a plastic solid.

| PI Range | Description |
|---|---|
| 0 | Nonplastic |
| 1 – 5 | Slightly Plastic |
| 5 – 10 | Low Plasticity |
| 10 – 20 | Medium Plasticity |
| 20 – 40 | High Plasticity |
| > 40 | Very High Plasticity |

**Table 1: PI Range**

### 4.2.4 Liquidity Index

The liquidity index (LI) is used for scaling the natural water content of a soil sample to the limits. LI is a good indicator of geologic history and relative soil properties. It can be calculated as a ratio of difference between natural water content, plastic limit, and liquid limit. The liquidity index is a measure of the relative consistency of a cohesive soil in its natural state. If the in-situ moisture content, $w_n$ is equal to the LL then LI = 1 or if $w_n$ is equal to PL then LI = 0. Therefore, for a soil in a plastic state (LL > $w_n$ > PL the LI varies from 1 to 0. Sensitive clays are soils, in an undisturbed state, whose $w_n$ > LL but if the soil becomes disturbed could transform into a liquid state; thus a sensitive clay would have a LI > 1.

## 4.3 Soil Shear Strength

Determining the design soil shear strength is one of the most important steps in the stability analyses of slopes. However, this analysis is only reliable if the shear strengths used in design accurately reflect the in-situ conditions. Typically, laboratory testing to determine soil shear strength is performed for cohesive soils while empirical correlations are utilized to assess the behavior and strength parameters for cohesionless soils.

Cohesive soils require a determination of whether or not undrained and/or drained shear strength parameters are being investigated. The determination of which test to conduct depends on the identified failure mechanism and the sample location. The direct shear test is a cost effective and efficient test to quickly determine residual soil values. Triaxial testing techniques allow the engineer to vary the stress parameters, control specimen drainage and take measurements of pore water pressures. The results of consolidation testing provide a higher level of understanding of the past soil stress history which can be useful in the assessment of the in-situ soil shear strength and therefore are included in the discussion under this section.

### 4.3.1 Strength Tests

As a reminder, additional information is readily available in course reference materials, testing standards or in applicable textbooks and these should be referenced to gain a broader understanding of the intrinsic details. The discussion herein is meant to provide a generalized overview and is not meant to be applicable to each and every situation.

The shear strength is the internal resistance per unit area that the soil can handle before failure and is expressed as a stress. There are two components of shear strength, a cohesive element (expressed as the cohesion, c, in units of force/unit area) and a frictional element (expressed as the angle of internal friction, $\phi$). These parameters are expressed in the form of total stress (c, $\phi$) or effective stress ($c'$, $\phi'$). The total stress on any subsurface element is produced by the overburden pressure plus any applied loads. The effective stress equals the total stress minus the pore water pressure. The common methods of ascertaining cohesion and friction in the laboratory are discussed below. Although all of these tests are normally performed on undisturbed samples these tests may also be performed on remolded samples.

#### 4.3.1.1 Unconfined Compression Test

The unconfined compression test is a quick method of determining the value of undrained cohesion ($c_u$) for clay soils. The test involves a clay specimen with no confining pressure and an applied axial load. The axial strains are monitored at various stress levels. The stress at failure is referred to as the unconfined compression strength. The undrained cohesion is taken as one-half the unconfined compressive strength, $q_u$. See AASHTO T208 - *Standard Method of Test for Unconfined Compressive Strength of Cohesive Soil*.

#### 4.3.1.2 Triaxial Compression Test

The triaxial compression test is a more sophisticated testing procedure for determining the soil's shear strength. The test involves a soil specimen subjected to an axial load until failure while also being subjected to a confining pressure that approximates the in-situ stress conditions. The three types of tests conducted by the Agency are the Unconsolidated-Undrained (UU), Consolidated-Undrained (CU), and Consolidated-Drained (CD) test. They are described in greater detail below.

##### 4.3.1.2.1 Unconsolidated-Undrained (UU)

In an unconsolidated-undrained test, no drainage is allowed to occur during shearing. The applied load induces a shearing stress which is analyzed for varying confining pressures in order to determine the soil's undrained shear strength, Su. Test results are used primarily in the calculation of immediate embankment stability during quick-loading conditions. The test procedure that the Agency follows is outlined in AASHTO T296 - *Standard Method of Test for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression*.

#### 4.3.1.2.2 Consolidated-Undrained (CU)

The consolidated-undrained test is the most common type of triaxial test. This test allows the soil specimen to be consolidated under a confining pressure that is applied prior to shear. After the pore water pressure has dissipated, the drainage line is closed and the specimen is subjected to an applied vertical load. The applied load induces a shearing stress and an increase in pore pressure which are analyzed for varying confining pressures in order to determine the soil's applicable shear strength parameters. The results can be utilized to determine both total and effective stress parameters. The test procedure that the Agency follows is outlined in AASHTO T297 - *Standard Method of Test for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression*.

#### 4.3.1.2.3 Consolidated-Drained (CD)

The consolidated-drained test is similar to the consolidated-undrained test except that drainage is permitted during shear and the rate of consolidation is very slow. Thus, the buildup of excess pore pressure is prevented. Again, several tests on similar specimens are conducted to determine the shear strength parameters. This test is typically used to determine design parameters for long-term embankment stability. However, since the same results can be determined from CU testing with pore pressure measurements, this test is rarely conducted by the Agency.

### 4.3.1.3 Direct Shear

The direct shear test is the oldest and simplest form of shear test. A soil sample is placed in a metal shear box and undergoes a horizontal force. The soil fails by shearing along a plane when the force is applied. The test can be performed either in stress-controlled or strain-controlled environment. In addition the test is typically performed as a consolidated-drained test on cohesionless soils. The test procedure that the Agency follows is outlined in AASHTO T236 - *Standard Method of Test for Direct Shear Test of Soils Under Consolidated Drained Conditions*.

## 4.3.2 Consolidation Test

The shear strength of soils, $S_u$, can be developed using the SHANSEP (i.e., stress history and normalized soil engineering properties) method developed by Ladd and Foott [1974], based on the results of the laboratory consolidated-undrained (CU) triaxial compression tests and consolidation tests. SHANSHEP is a system to present and characterize the undrained shear strength of soils and is based on the observation that the shear strength of many soils can be normalized with respect to the vertical consolidation pressure. The SHANSEP method can be expressed using the following equation:

$$S_u = S \times \sigma'_{vc} \times OCR^m$$

Where,

$S$ = undrained shear strength ratio under normal consolidation, obtained from CU tests;

$\sigma_{vc}'$ = effective vertical consolidation stress for a given loading;

$OCR$ = over-consolidation ratio, obtained from consolidation tests which is the ratio of the preconsolidation pressure ($p_c'$) to the in-situ vertical effective stress ($\sigma_v'$); and

$m$ = SHANSEP modeling parameter ($m$ = 0.8 for most cohesive soils and typical applications [Ladd and DeGroot, 2003]).

The test procedure that the Agency follows is outlined in AASHTO T216 - *Standard Method of Test for One-Dimensional Consolidation Properties of Soils*.

### 4.3 External Testing Services

There are a number of specialty soil strength tests (e.g. torsional ring shear, direct simple shear) that can be performed via an external testing services contract. In addition, permeability or hydraulic conductivity testing may be necessary to investigate projects with complex groundwater issues. It is important that the Engineer is aware of the scheduling and design implications that an external services contract may have on the delivery time for the final design.

## 5.0 Analysis and Design Procedures

### 5.1 General Approach

The analysis routinely conducted for the majority of Agency slope stability projects typically consists of the following steps:

1. Review site geometry from acquired field surveys, including but not limited to LIDAR, topographic surveys as well as current and historic aerial photographs;

2. Develop the project's soil profile through the process of estimating and assigning soil properties for each soil stratum within the slope using available geologic information, boring logs, topographic surveys, field and laboratory test results, empirical correlations, back-calculations, and any other available information;

3. Review and interpret surface and ground water conditions and apparent fluctuations based on a review of the topographic survey, monitoring well information and documented redoximorphic features within the soil samples;

4. Determine the location and magnitude of existing and future static or dynamic loads (structures, roads, railroads etc) and assess the effect on overall stability (i.e, stabilizing force or destabilizing force);

5. Characterize the existing site conditions through the process of back calculating the soil parameters at failure (FS<1.0), see Section 5.4.7, through an assessment of static, dynamic and seismic loading conditions while accounting for varying groundwater conditions including rapid drawdown and seepage. Identify potential failure locations and modes (circular/block etc.) for 2-D slope stability analyses;

6. Perform a series of slope stability analyses and calculate the design factor of safety for mitigation alternatives using the back calculated soil parameters.

7. Compare the computed design factor of safety to the required factor of safety;

   a. If the computed factor of safety is approximately equal to or slightly greater than the required factor of safety, the design is considered acceptable.

   b. If the factor of safety is significantly greater than the required factor of safety, changes to reduce the computed factor of safety may be considered if significant cost savings can be realized while maintaining an acceptable level of safety.

   c. If the factor of safety is less than the required factor of safety, the Engineer must consider alternative measures to increase the factor of safety and repeat the procedure until an acceptable factor of safety is achieved.

8. Develop geotechnical report, drawings or plans and/or specifications required to successfully implement the selected design alternative.

## 5.2 Development of Subsurface Soil Profile

The ultimate goal of a subsurface investigation is to develop a working model that depicts major subsurface layers exhibiting distinct soil characteristics and behaviors. The end product is the subsurface profile, a two dimensional depiction of the soil stratigraphy. The following steps outline the creation of the subsurface profile:

1. Review the boring log and laboratory test results and develop estimate for the soil properties of various layers. The selection of material properties and ultimately the development of the project's subsurface profile is an iterative process. Test results and boring logs should be revisited several times as the data is developed and analyzed and as new data is acquired before the properties soil layers are finalized.

   Subsurface soil or rock properties are generally determined using one or more of the following methods and are outlined below in Sections 5.3 and 5.4:

   - in-situ testing during the field exploration program,
   - laboratory testing, and
   - back analysis or back calculation methods based on site performance data.

2. Organize the boring logs relative to their respective field locations and compare and match up the different soil and rock units at adjacent boring locations, if possible. Group the subsurface units based on engineering material properties and behaviors. Caution should be exercised when attempting to connect units in adjacent borings, as the geologic stratigraphy does not always fit into well defined layers.

3. A detailed site characterization may include cross sections located at one or more critical stations. Critical cross sections may be initially selected based on a review of the site topography but are often revised during the design process.

   Create cross sections by plotting borings at their respective elevations and positions horizontal to one another with appropriate scales. The cross section(s) should show an interpretation of the entire slope based on the surface mapping, subsurface investigation, and regional geologic maps. The cross sections should identify surface topography, locations of borings, instrumentation, groundwater elevations, and known or hypothesized bedrock elevations. The cross sections may also show the Engineer's interpretation of the subsurface stratigraphy including any design parameters and assumptions utilized.

4. Review the profile to see how it compares with expected results and knowledge of the geologic (depositional) history. Were anomalies and unexpected results encountered during exploration and was testing adequately addressed during the process? Make sure that all of the subsurface features and properties pertinent to design have been addressed.

## 5.3 Determination of Soil and Rock Strength Parameters for Design

### 5.3.1 Cohesionless Soils, c = 0

SPT test results are used to identify the soil parameters such as unit weight, relative density, and internal friction angle along the depth of the borehole. This section outlines the critical relationship between soil density and internal friction angle for SPT blow counts corrected for hammer energy and overburden.

Regarding SPTs, the N-values obtained are dependent on the equipment used and the skill of the operator, and should be corrected for field procedures to standard $N_{60}$ values. This correction is necessary because many past correlations were based on a SPT hammer operated manually using a cathead rope; this method has an overall efficiency rating of 60%. Thus SPT N values should be corrected for hammer efficiency, if applicable to the design method or correlation being used, using the following relationship:

$$N_{60} = (ER/60\%)N$$

Where,

| | | |
|---|---|---|
| $N_{60}$ | = | SPT blow count corrected for hammer efficiency (blows/ft) |
| $ER$ | = | Hammer efficiency expressed as percent of theoretical free fall energy delivered by the hammer |

|  |  |  |
|---|---|---|
|  |  | system actually used. The following values for ER may be assumed if hammer specific data is not available: |
| *ER* | = | 60% for conventional drop hammer rope and cathead |
| *ER* | = | 80% for automatic trip hammer |

VTrans has conducted two separate analyses on the efficiency of their SPT hammers, both manual and automatic, in use on its drilling equipment. The hammer specific values, $C_E$ [$C_E$ =(ER/60%)], for use with AWJ rod area as follows:

| VTrans Drilling Equipment | SPT Hammer Type | $C_E$ |
|---|---|---|
| Large Skid Rig | Automatic | 1.33 |
|  | Manual | 1.15 |
| CME 55 Track Rig | Automatic | 1.46 |
| CME 45 Track Rig | Automatic | 1.34 |

Hammer efficiency (ER) for Consultant hammer systems used in local practice may be used in lieu of the values provided. However, specific hammer system efficiencies shall be developed in general accordance with ASTM D-4945 for dynamic analysis of driven piles or other alternate accepted procedure.

Corrections for rod length, hole size, and use of a liner may also be made if appropriate. In general, these are only significant in unusual cases or where there is significant variation from standard procedures. These corrections may be significant for evaluation of liquefaction. Information on these additional corrections can be found in the following resource: "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils".

When applicable to the design method or correlation being used N-values should also be corrected for overburden pressure. N values corrected for both overburden and the efficiency of the field procedures hall be designated as $N1_{60}$. The overburden correction equation that should be used is:

$$N1_{60} = C_N \times N_{60}$$

Where,

|  |  |  |
|---|---|---|
| $C_N$ | = | correction factor for overburden |
| $C_N$ | = | [0.77 $\log_{10}$ (20/$\sigma'_v$)], $C_N < 2.0$ |
| $N_{60}$ | = | N-value corrected for energy efficiency |
| $\sigma'_v$ | = | vertical effective stress, in TSF |

In general, correlations between SPT N-values and soil properties, see Table 2, such as internal friction angle and unit weight, should only be used for cohesionless soils, and sand in particular. Caution should be used when interpreting the SPT N-values obtained from gravelly soils. Gravel particles can plug the sampler, resulting in increased blow counts and misleading friction angles.

| Description | Very Loose | Loose | Medium | Dense | Very Dense |
|---|---|---|---|---|---|
| Standard Penetration no. N'* | 0 | 4 | 10 | 30 | 50 |
| Approx. angle of internal friction, ($\phi$)degrees** | 25-30 | 27-32 | 30-35 | 35-40 | 38-43 |
| Approx. range of moist unit weight, ($\gamma$)pcf** | 70-100 | 90-115 | 110-130 | 120-140 | 130-150 |

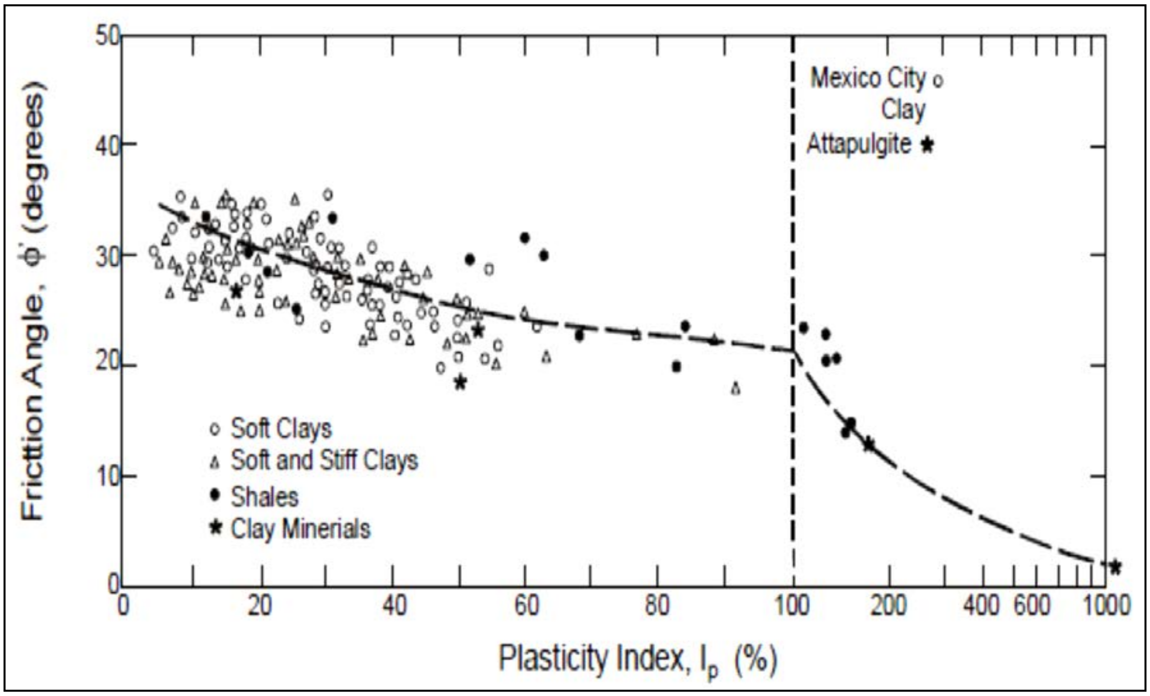
* N' is SPT value corrected for overburden pressure.
** Use larger values for granular material with 5% or less fine sand and silt.
**Source: FHWA Soils & Foundations Workshop Manual (1993)**

**Table 2: Cohesionless Soil Properties Based on SPT Data**

### 5.3.2 Cohesive Soils, $\phi = 0$

Cohesive soils may be assessed using either drained (effective stress) or undrained (total stress) properties. Field testing and sampling is conducted to gain a better understanding of these properties. Often vane shear testing is coupled with the acquisition of shelby tubes and SPT values to develop cohesive soil parameters. A series of empirical correlations are presented herein and have been provided for preliminary design purposes only. Design assumptions and final design values should be obtained via site specific field and laboratory testing.

Typically vane shear tests are conducted in the field to determine the undrained shear strength of the material for use in a total stress analysis. Shelby tube samples are extracted for the purposes of conducting direct shear, consolidation and consolidated-undrained (CU) testing. SPT tests are conducted during an investigation into clayey soils to assist in determining the soils' relative density, soil composition and to provide material for Atterberg Limits testing. SPT N-values shall not be used to determine cohesive soil parameters for design purposes due to the dynamic nature of the test and resulting rapid changes in pore pressures and disturbance within the deposit.




Source: FHWA GEC No. 5

**Figure 4: Relationship between ϕ' and PI**

For clays, empirical correlations have been developed to relate ϕ' to the plasticity characteristics of the soil, see Figure 4. In Figure 4, ϕ' decreases with an increasing PI value. It should be noted that the plasticity index is identified as $I_p$ in Figure 4 vs. PI as identified throughout this document. Considering the overall importance of ϕ' in stability calculations, foundations, and landslide analyses, it is essential to directly assess ϕ' by means of direct shear tests or triaxial tests for final design purposes in most cases involving clay soils.

Figure 5 depicts historic soils data gathered by VTrans and correlating PI to hydrometer test results showing the percent of clay material for various sites across Vermont. Figure 5 can be used to estimate the clay fraction which can then be utilized with Figure 6 to give an approximation of the soil's residual friction angle. This value can be used for preliminary design purposes; however, residual friction angles should be verified through a series of cyclic direct shear tests, direct simples shear or torsional ring shear testing.

Friction angle is one of the strength parameters typically quantified, the other is cohesion. The short-term value of effective cohesion is related to the preconsolidation stress, $\sigma_p'$, and current effective stress state, as shown in Figure 7. However, for long-term analyses involving most insensitive clays, silts, and uncemented sands, it is conservative to adopt $c' = 0$; unless adequate laboratory testing is conducted or sufficient information exists to prove bonding or cementation.

Conservative recommended values of effective cohesion are as follows:

Short Term: $c' = 0.024\ \sigma_p'$ Long Term: $c' = 0$

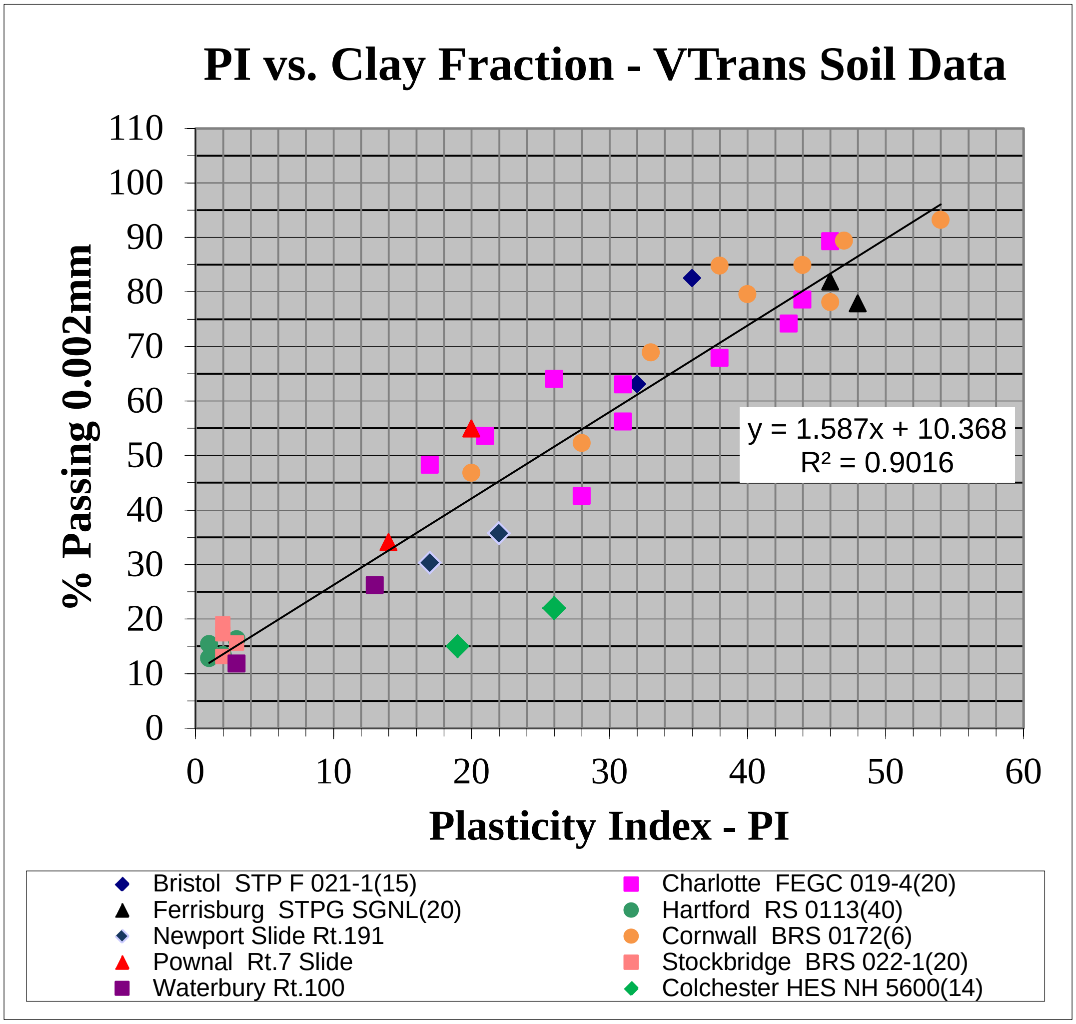
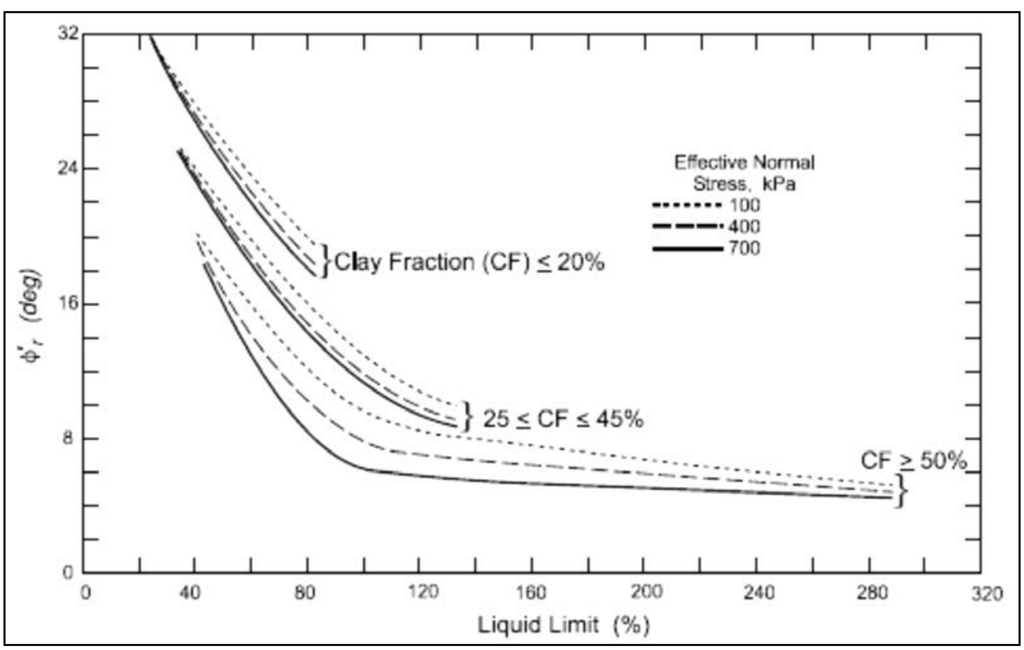
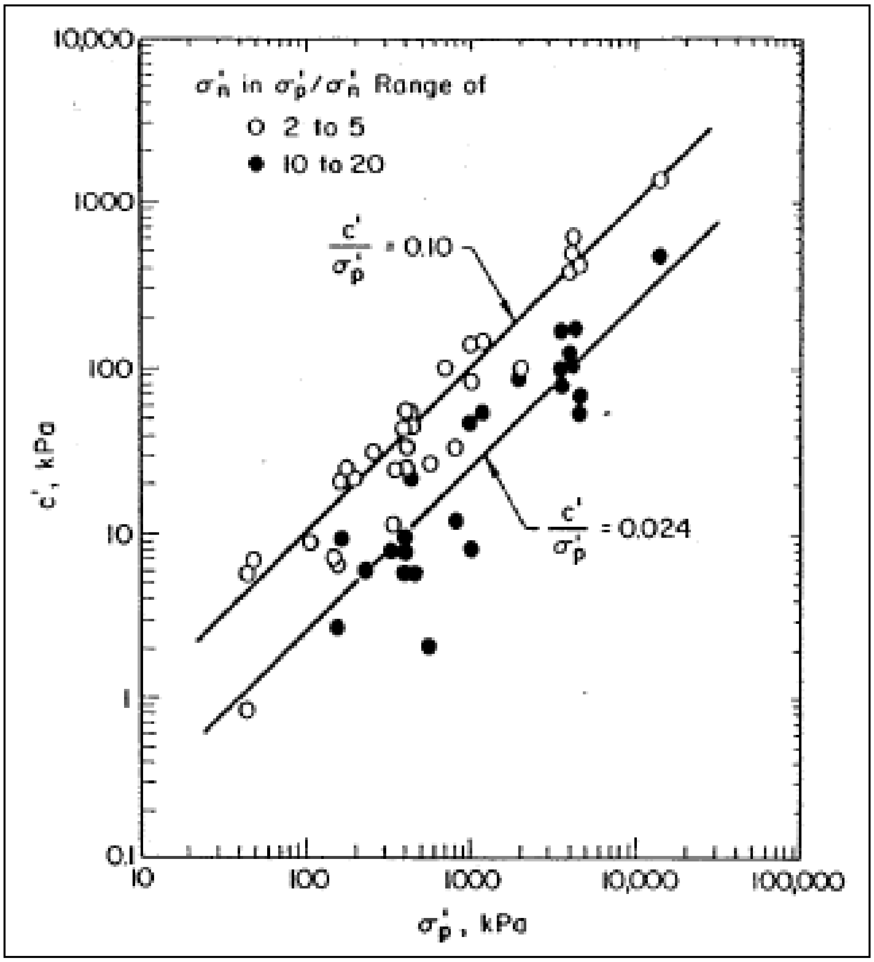



**Figure 5: VTrans' Soil Data: PI vs. % Clay Fraction**




Source: FHWA GEC No. 5

**Figure 6: Residual Friction Angles for Clayey Soils**




Source: FHWA GEC No. 5

**Figure 7: Relationship between c' and $\sigma_p$'**

The undrained shear strength ($S_u$) of cohesive soils (i.e. clay, highly plastic silts and residual soils) can be determined using unconfined compression (UC) tests, unconsolidated undrained (UU) triaxial tests, or consolidated undrained (CU) triaxial tests of undisturbed samples. Typically the total internal friction angle is negligible and assumed equal to zero ($\phi = 0$) and the Mohr-Coulomb shear strength equation for the undrained shear strength ($\tau$) of cohesive soils can be expressed as one half of the shear stress at failure, $\Delta\sigma_f$ , indicated by the following equation:

$$S_u = C = \frac{\Delta\sigma_f}{2}$$

The undrained shear strength of cohesive soils may also be determined by in-situ testing such as Cone Penetrometer Test (CPT), Flat Plate Dilatometer Test (DMT), or Vane Shear Test (VST). As stated previously, the drawback to the use of in-situ field testing methods to obtain undrained shear strengths of cohesive soils is that the empirical correlations are based on a soil database that is material or soil formation specific and therefore the reliability of these correlations must be verified for each project site by substantiated regional experience or by conducting laboratory testing and calibrating the in-situ testing results.

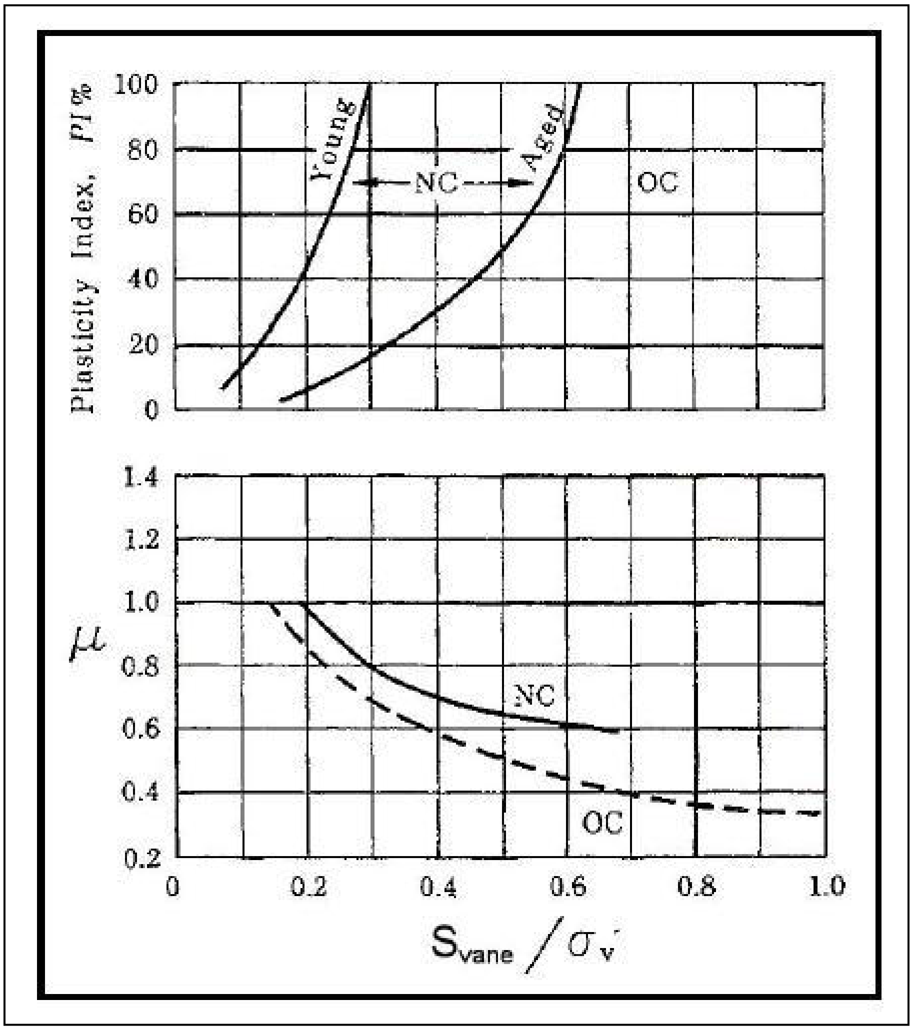
The VST field measured undrained shear strength, $S_{vane}$, should be computed based on the following equation for a <u>rectangular vane only</u> typically used by the Agency with a height to diameter ratio of 2:1;

$$S_{vane} = \frac{6T}{7\pi D^3} \quad \text{for} \quad \frac{H}{D} = 2$$

Where,

| | | |
|---|---|---|
| $T$ | = | VST torque resistance |
| $D$ | = | Diameter of field vane |
| $H$ | = | Height of field vane |

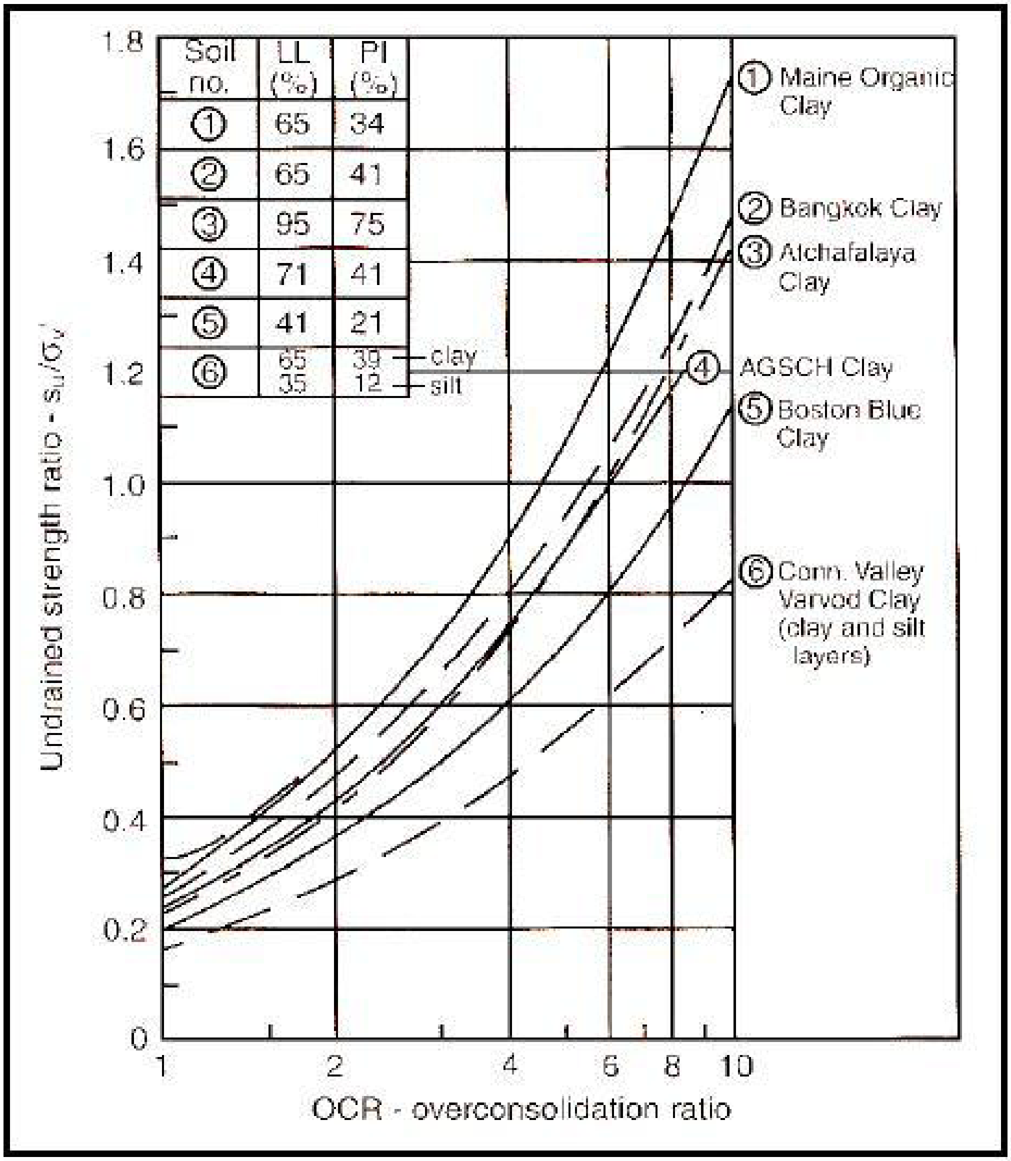
The vane correction factor (μ) is determined from the relationship shown in Figure 9. The vane correction factor (μ) is computed by entering the top chart with PI and ($S_{vane}/\sigma'_{vo}$) to establish whether the clay is within the normally consolidated (NC) range between the limits “young” and “aged”, or overconsolidated (OC). The lower chart is used by entering the ($S_{vane}/\sigma'_{vo}$) and selecting the vane correction factor (μ) for the appropriate NC or OC curves. A maximum vane correction factor (μ) of 1.0 is recommended.




Source: Aas et al., (1986)

**Figure 9: Vane Shear Correction Factor**

Empirical correlations based on SHANSHEP laboratory testing results can be used for preliminary designs and to evaluate the peak undrained shear strength ($S_u$) obtained from laboratory testing or in-situ testing. This method is only applicable to clays without sensitive structure where undrained shear strength increases proportionally with the effective overburden pressure ($\sigma'_{vo}$). The SHANSHEP laboratory test results of Ladd et al. (1977) revealed trends in undrained shear strength ratio ($S_u/\sigma'_v$) as a function of overconsolidation ratio as indicated in Figure 10.




Source: Ladd, et al., (1977)

**Figure 10:** $S_u$ / $\sigma'_v$ **Ratio and OCR Relationship**

The average peak undrained shear strengths (τ) shown in Figure 10 can be approximated by an empirical formula developed by Jamiolkowsi et al. (1985) as indicated by the following equation;

$$S_u = (0.23(OCR)^{0.8})\sigma'_{vo}$$

Where,

| | | |
|---|---|---|
| $S_u$ | = | the undrained shear strength (tsf) |
| $OCR$ | = | overconsolidation ratio |
| $\sigma'_{vo}$ | = | the effective overburden pressure at test depth (tsf) |

The ratio of the undrained shear strength to vertical effective stress is often noted as follows:

For normally consolidated clays (Skempton): $\frac{S_{u\,(VST)}}{\sigma_{vo}}$ ~ 0.11+0.0037 PI

For over-consolidated clays (Chandler): $\frac{S_{u\,(VST)}}{\sigma_{p}}$ ~ 0.11+0.0037 PI

Where,

$\sigma_p$ = the effective preconsolidation stress typically determined from laboratory consolidation testing. It is the maximum past pressure that a soil has been exposed to since disposition.

Using typical SHANSHEP parameters:

$$\frac{S_u}{\sigma'_{vc}} = S(OCR)^m$$

The constant S corresponds to the normalized strength for normally consolidated soils with OCR ~ 1.0. The constant m is known as the strength increase exponent.

The SHANSEP parameters should be used only for fairly homogeneous clay deposits that can be suitably characterized by a normalized strength. The method is not suitable for sensitive or cemented clays.

| Soil Type | Strength Ratio, S | Strength Component, m |
|---|---|---|
| Sensitive marine Clays $I_p$<30%, $I_p$>1 | 0.2 | 1 |
| Homogeneous CL and CH sedimentary clays of low to moderate sensitivity = 20-80% | 0.22 | 0.8 |
| Northeastern U.S. varved clays | 0.166 (DDS mode) | 0.75 |
| Sedimentary deposits of silts and organic soils (Atterberg limits plot below A-line, but excluding peats) and clays with shells. | 0.25 | 0.8 |

**Table 3: SHANSHEP parameters**

The undrained shear strength ($S_u$) can be compared to the remolded shear strength ($S_{ur}$) (residual undrained shear strength, $S_{u\ res}$) to determine the sensitivity ($S_t$) of cohesive soils. Sensitivity is the measure of the breakdown and loss of inter-particle attractive forces and bonds within cohesive soils. Typically in dispersed cohesive soils the loss is relatively small, but in highly flocculated structures the loss in strength can be large. Sensitivity is determined using the following equation:

$$S_t = \frac{S_u}{S_{ur}}$$

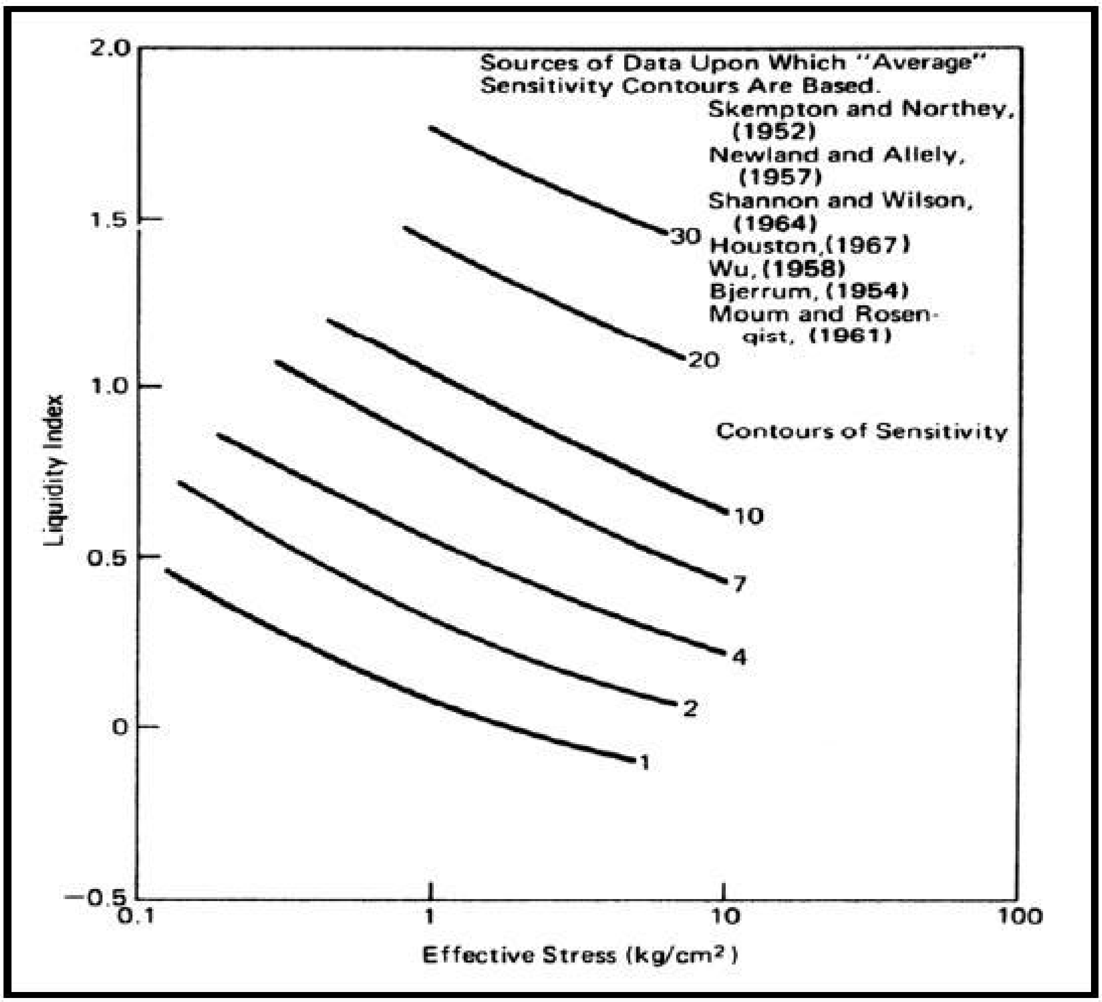
The description of sensitivity is defined in the following table.

| Sensitivity | Descriptive Term |
|---|---|
| <1 | Insensitive |
| 1 – 2 | Slightly Sensitive |
| 3 – 4 | Medium Sensitive |
| 5 – 8 | Sensitive |
| 9 – 16 | Very Sensitive |
| 17 – 32 | Slightly Quick |
| 33 – 64 | Medium Quick |
| >64 | Quick |

Source: modified from Spangler and Handy, (1982)

**Table 4: Sensitivity of Cohesive Soils**

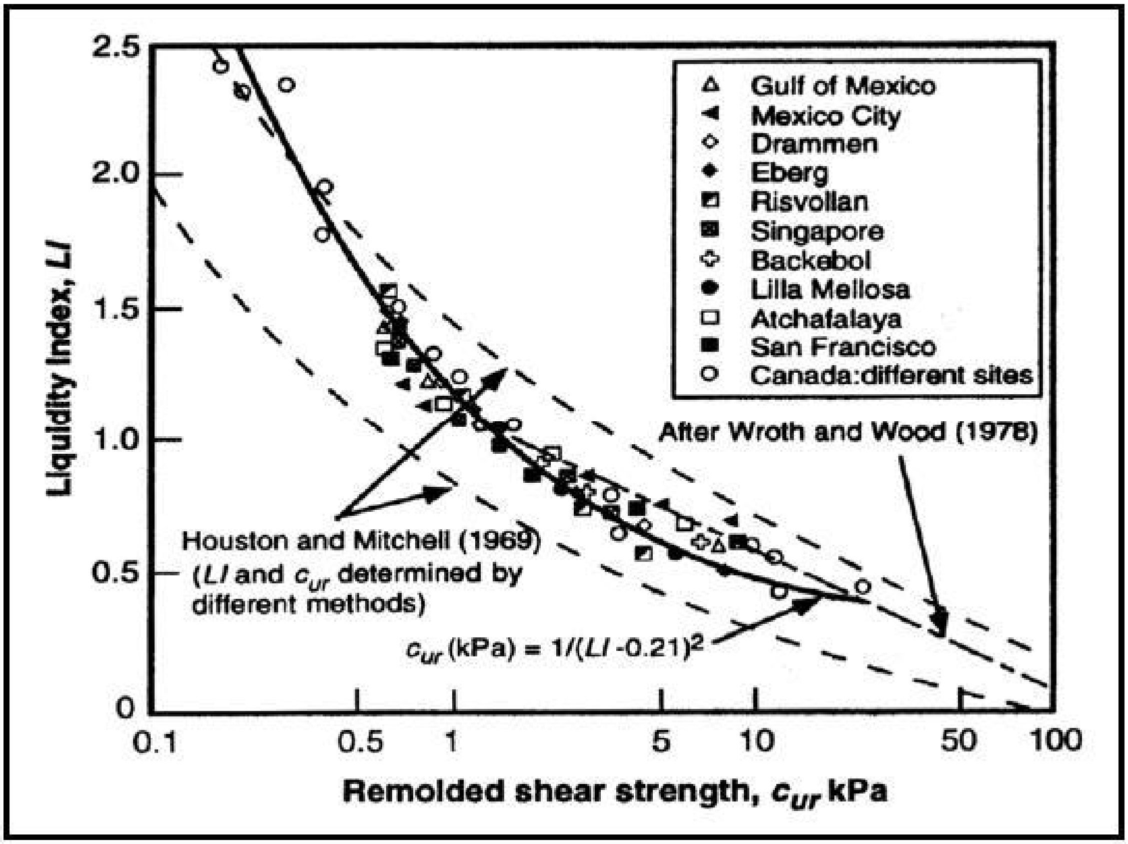
The remolded shear strength of cohesive soils ($S_{ur}$) can be determined from remolded triaxial specimens or from in-situ testing methods (electro-piezocone or field vane). Triaxial specimens should have the same moisture content as the undisturbed sample as well as the same degree of saturation and confining pressure. Further sensitivity can be related to the liquidity index using the following figure.




Source: Mitchell, (1993)

**Figure 11: Sensitivity based on Liquidity Index and $\sigma'_{vo}$**

The Liquidity Index (LI) can also be related to remolded shear strength ($\tau_R = c_{ur} = S_{ur}$) as indicated in the following.




Source: Mitchell, (1993)

**Figure 12: Remolded Shear Strength vs. Liquidity Index**

Where,

1 kPa = 0.0209 ksf

The Liquidity Index (LI) is the relationship between natural moisture content, Plastic Limit (PL), and the Liquid Limit (LL).

The undrained residual shear strength of cohesive soils ($S_t$ < 2) can be estimated for preliminary design and to evaluate the undrained residual shear strength, $S_{u\ res}$, obtained from laboratory testing or in-situ testing. The undrained residual shear strength can be estimated by reducing peak undrained shear strength ($S_u$) by a residual shear strength loss factor ($\lambda$) as indicated in the following equation.

$$S_{ur} = \lambda S_u$$

The residual shear strength loss factor ($\lambda$) typically ranges from 0.50 to 0.67 depending on the type of clay soil. Examples of residual shear strength loss factors ($\lambda$) presented in

Table 5 are based on the results of a pile soil set-up factor study prepared by Rauche et al. (1996).

| Soil Type | | Residual Shear Strength Loss Factor(λ) |
|---|---|---|
| USCS | Description | |
| Low Plasticity Clay | CL-ML | 0.57 |
| Medium to High Plasticity Clay | CL & CH | 0.5 |

**Table 5: Residual Shear Strength Loss Factors**

### 5.3.3 ϕ-c Soils, ϕ > 0 and c > 0

The undrained shear strength of soils that have both ϕ and c components should be determined in the laboratory using the appropriate testing methods. However, if the samples for this type of testing have not been obtained (e.g. during the preliminary exploration), then the soil should be treated as if the soil were either completely cohesive or cohesionless. For soils that are difficult to determine the approximate classification, the undrained shear strength parameters for both cohesive and cohesionless soils should be determined and the more conservative design should be used.

### 5.3.4 Rock Fill Soil Strength Parameters

For design purposes, VTrans assumes a unit weight of 145 to 150 pcf and a friction angle of 45 degrees for rock fill materials.

For unreinforced slopes steeper than 1V:2H VTrans typically uses stone fill slopes. The desired slope angle for stone fill slopes is 1V:1.5H. The slope may be installed at an angle up to 1V:1.25H pending approval by the Geotechnical Engineering Manager. See Section 8 for more design details.

### 5.3.5 "Back-Calculation" or "Back-Analysis" Method

Existing landslides offer the opportunity to estimate the average shear strength properties along the failure surface by mathematical methods. This procedure is generally referred to as back-calculation or back-analysis. The procedure requires the determination of the configuration of the landslide failure surface relative to the topography at the time of failure, variability in earth materials along the failure surface, the subsurface water level at the time of failure, external loading conditions, and the appropriate soil density.

Once the soil parameters have been determined, a slope stability analysis method appropriate to the slide configuration is chosen. The slope stability model is then transferred into and analyzed using a slope stability program developed by RocScience called SLIDE. The slope stability model including the shear strength parameters are then adjusted and the analysis repeated until a factor of safety of 1.0 (FS=1.0) is obtained. This method provides different set of back-calculated values for cohesion, c, and friction angle, ϕ, which satisfy FS = 1.0. The Engineer then selects an appropriate combination of c and ϕ based on a combination of the back-calculation results, laboratory test results, boring log data, empirical relationships, and prior experience. For effective stress analyses, it is common to assume c'= 0, but analyses can be made assuming a small value of cohesion (200 - 400 psf) for first-time slides in over-consolidated clays where appropriate. These strength parameters can then be utilized in the evaluation of a proposed mitigation alternative to assess the long term stability of the slope.

### 5.4 Groundwater Effects

The effects of groundwater should always be considered in slope stability analyses. The presence of groundwater in a slope can reduce effective stresses when positive pore water pressures develop, causing a reduction in shear resistance. Groundwater can also increase de-stabilizing forces in the slope via the additional weight associated with a moist slide mass or via seepage forces. Therefore, engineers and geologists should investigate the presence of groundwater and evaluate potentially adverse future groundwater conditions.

A detailed assessment of the groundwater regime within and beneath the slope/landslide mass is also critical. Detailed piezometric data at multiple locations and depths within and below the slope will likely be needed. The ability to acquire this information is dependent upon the geologic complexity of the stratigraphy and groundwater conditions. Potential seepage at the face of the slope must be assessed and addressed. In some cases, a detailed flow net analysis may be needed. If seepage does exit at the slope face, the potential for soil piping should also be assessed as a slope stability failure mechanism, especially in highly erodible silts and sands.

If groundwater varies seasonally, long-term monitoring of the groundwater levels in the soil should be conducted. If groundwater levels tend to be responsive to significant rainfall events, the long-term groundwater monitoring should be continuous, and on-site rainfall data collection should also be considered.

Analyses of slope stability with a groundwater level located above a portion of the sliding surface can be performed using one of two methods presented by the Southern California Earthquake Center's Guidelines for Analyzing and Mitigating Landslide Hazards in California:

<u>Method 1:</u> By the use of total unit weights and specification of groundwater table location and boundary water pressures. This method is appropriate for effective stress analyses of slope stability and should be used with effective stress strength parameters. [If a total stress analysis is desired, it should be performed with no phreatic surface (i.e., zero pore pressure). Seepage forces should not be included. Total stress (undrained) strength parameters should be used.]

<u>Method 2:</u> By the use of buoyant unit weights and seepage forces below the water table. This method is appropriate for use only with effective stress analyses; it should not be used with total stress analyses.

Method 1 is most commonly selected. In a stability analysis utilizing Method 1, porewater pressures are commonly depicted as an actual or assumed phreatic surface or through the use of piezometric surfaces or heads. The phreatic surface, which is defined as the free groundwater level, is the most common method used to specify groundwater in computeraided slope stability analyses. The use of piezometric surfaces or heads, which are usually calculated during a seepage or subsurface water flow analysis, is generally more accurate, but not as common as identifying the phreatic surface. Computer programs may allow multiple perched water levels to be input within specific units through the specification of piezometric surfaces.

External water pressures acting on the surface of the embankment should be specified because external water pressures are a component of total stress and need be included to satisfy equilibrium in terms of total stress. The depth of water should be varied to account for a variety of conditions including flooding and rapid drawdown.

### 5.4.1 Hydraulic Conductivity

Hydraulic conductivity may be a useful soil parameter to understand when analyzing any seepage related problem or in proposing horizontal or vertical drainage conduits as a mitigation alternative. In general design practice, hydraulic conductivity is estimated based on grain size characteristics of the soil strata. In critical applications, the hydraulic conductivity may be determined through in-situ testing. A discussion of field measurement of permeability is presented in FHWA GEC No. 5. In addition, ASTM D4043 presents a guide for the selection of various field methods. If in-situ test methods are utilized to determine hydraulic conductivity, one or more of the following methods may be used:

- Well pumping tests
- Packer permeability tests
- Seepage tests
- Slug tests
- Piezocone tests

### 5.4.2 Effect of Groundwater and Excess Pore Pressures

Groundwater movement and associated seepage pressures are the most frequent cause of slope instability. The following five groundwater conditions should be carefully considered while assessing slope stability:

1. **Seepage Pressures:** Subsurface water seeping toward the face or toe of a slope produces destabilizing forces which can be evaluated by flow net construction. The piezometric heads which occur along the assumed failure surface produce outward forces which must be considered in the stability analysis.

2. **Construction Pore Pressures:** When compressible fill materials are used in embankment construction, excess pore pressures may develop within the compressible soil and must be considered in the stability analysis. Normally, field piezometric measurements are required to evaluate this condition.

3. **Excess Pore Pressures in Embankment Foundations:** Where embankments are constructed over compressible soils, the foundation pore pressures must be considered in the stability analysis.

4. **Artesian Pressures:** Artesian pressures beneath slopes can have serious effects on slope stability. Should such pressures be found to exist, they must be used to determine effective stresses and unit weights, and the slope and foundation stability should be evaluated by effective stress methods.

5. **Rapid Drawdown:** The rapid drawdown case has long been recognized as one of the most severe loadings conditions that a slope can be subjected to and it is well documented in the literature. The condition is perhaps most commonly associated with the upstream slope of embankment dams; however, failures are also very common in natural and man-made slopes along rivers and man-made drainage channels as a result of flooding.

Flood events can leave water levels high in rivers and drainage channels for significant periods of time and then drop relatively rapidly once the floodwaters recede. The effect of this inundation on the soil in the slope, both prior to and subsequent to drawdown, is the essence of the rapid drawdown loading condition. Therefore, to understand rapid drawdown one must consider what is happening to the soil in the slope, both in terms of soil strength and pore pressure development.

The rapid drawdown condition occurs when totally or partially submerged slopes experience a rapid reduction of external water levels. Failures tend to occur when the excess pore pressures within a fine grained soil do not dissipate as the water levels decrease. Because undrained shear strength is lower than drained shear strength, the excess pore pressures reduce the soil's shear strength. In addition, when the water levels are high, the water acts as a stabilizing force; once rapidly removed the factor of safety decreases.

To model rapid drawdown effectively it is recommended to consult Appendix G of the USACOE Engineering Manual 1110-2-1902.

## 5.5 Drainage Conditions and Total vs. Effective Stress Analysis

### 5.5.1 Drained vs. Undrained Loading Conditions

For a saturated soil subjected to *undrained loading*, (no drainage of pore water from the void spaces can occur), the soil undergoes no change in volume. During undrained conditions, changes in total stress ($\Delta\sigma$) cause the development of either positive excess pore water pressures ($\Delta u > 0$) that will tend to decrease the effective stress in the soil or negative excess pore water pressures ($\Delta u < 0$) that will tend to increase the effective stress in the soil.

The *drained loading* of a saturated soil means that the water in the void spaces is free to move so that no excess pore water pressures develop ($\Delta u = 0$) during loading. There is usually a change (i.e., increase or decrease) in void ratio and a corresponding change in volume. Again, water may be present, but is free to move out of the soil mass (termed *contractive* soil behavior) or into the soil mass (termed dilatant soil behavior). Contractive behavior results in a decrease in volume (e.g., settlement) and dilative behavior causes an increase in volume (e.g., swelling). Coarse grained soils have such a high permeability (e.g., $k > 10^{-3}$cm/s) that, under static loading, they are almost always drained.

Sands, however, will behave in an undrained mode when subjected to rapid loading, such as that imposed by an earthquake whereby the entire deposit is liquified and water is not able drain from the pore spaces. For saturated sands, the pore pressure generated during an earthquake, or shaking, should be estimated with a liquefaction analysis. The undrained residual shear strength should be used if soil liquefies. The residual shear strength can be estimated using available correlations with liquidity index, see Figure 12 in Section 5.3.2. The drained strength should be used if the soil does not liquefy, however, the pore pressure generated during shaking should be estimated, so that the effective stress in the soil can be appropriately reduced.

If a clay soil is loaded slowly enough it will drain and therefore drained loading should be considered in the long term stability analysis of clay slopes. The short term stability of cuts and fill constructed in clay should be represented by undrained loading conditions. Additional discussion about loading is discussed in Section 5.0 – Analysis and Design Procedures.

Soil behavior during drained loading is fundamentally different than during undrained loading. Drained loading implies that loads are applied at a sufficiently slow rate so that no pore pressures are generated in the soil during shear, and volume change is allowed. Undrained loading typically occurs when loads are applied at a sufficiently high rate, relative to the permeability of the material, generating pore pressures within the soil matrix, and volume change is not allowed.

Additional information and discussion on drained versus undrained loading conditions is presented in FHWA's Geotechnical Engineering Circular No. 5 – Evaluation of Soil and Rock Properties.

Guidelines on the appropriate use of drained vs. undrained strength parameters are provided below. The term "loading" refers to a condition in which total normal stresses along potential sliding surfaces are increased, for example the placement of fill, structural loads, etc. Conversely, "unloading" refers to a condition in which total normal stresses are decreased, such as excavations or rapid drawdown of flood or static water levels. In saturated soil, the total stress increase associated with loading tends to increase the pore pressures in the ground, whereas unloading reduces pore pressures. Pore pressures can also increase or decrease as a result of shearing, depending on whether the soil is contractive or dilatant.

### 5.5.2 Total vs. Effective Stress Analysis

As discussed above, the choice between total and effective stress parameters is governed by the drainage conditions which occur within the sliding mass and along its boundaries. Drainage is dependent upon soil permeability, boundary conditions, and time. For most practical problems the total stress analysis is used for short term stability problems while effective stress analysis is used to analyze long term stability issues with the assumption that any excess pore pressures have had to time to dissipate. Thus undrained strength parameters ($S_u$) are used for total stress analysis assuming that the project area consists primarily of cohesive soil. For effective stress analyses, $\phi$' and c' along with the pore pressure, u, are required to evaluate slope stability.

### 5.5.2.1 Total Stress Analysis

Where drainage cannot occur during shear, use the undrained shear strength parameter, $S_{u,}$ see Section 5.3.2 for more in-depth discussion of the selection of $S_u$. Field vane shear and cone penetration tests may be used to determine undrained soil properties. Examples where a total stress analysis is applicable include:

1. Analysis of cut slopes of normally consolidated or slightly preconsolidated clays. In this case little dissipation of pore water pressure occurs prior to critical stability conditions. Static loading of saturated clay with low OCR (OCR < 4) will be most critical under short term undrained loading conditions.

2. Analysis of embankments on a soft clay stratum. This is a special case as differences in the stress-strain characteristics of the embankment and the foundation may lead to progressive failure.

3. Rapid drawdown of water level providing insufficient time for drainage. Use the undrained strength corresponding to the overburden condition within the soil structure prior to drawdown. Rapid drawdown removes the stabilizing effects of external water pressures. Undrained shear strengths are assumed to apply for all but the most highly permeable and free-draining materials (permeability coefficient, $k > 10^{-3}$ cm/s).

4. End-of-construction condition for fills constructed using cohesive soils. Use the undrained strength of samples compacted to field density and at a water content representative of the embankment.

### 5.5.2.2 Effective Stress Analysis

The effective shear strength parameters (c', $\phi$') should be used for the following cases:

1. Long-term stability of clay fills. Use steady state seepage pressures where applicable.

2. Short-term or end-of-construction condition for fills built of free draining sand and gravel. Friction angle is usually approximated using empirical correlations. Static loading of clean sand will generally be drained (i.e., CD).

3. Rapid drawdown condition of slopes in pervious, relatively incompressible, coarse-grained soils. Use pore pressures corresponding to new lower water level with steady state flow.

4. Long-term stability of cuts in saturated clays. Use steady state seepage pressures where applicable.

5. Cases of partial dissipation of pore pressure in the field. Here, porewater pressures must be measured by piezometers or estimated from consolidation data.

## 5.6 Slope Stability Modeling

Once the conditions for which strength parameters will be used have been established, an appropriate method for evaluating them can be implemented. In practice, limiting equilibrium methods are used in the analysis of slope stability. It is considered that failure is occurring along an assumed or a known failure surface. The shear strength required to maintain a condition of limiting equilibrium is compared with the available shear strength of the soil, giving the average factor of safety along the failure surface. The problem is considered in two dimensions, conditions of plane strain being assumed. It has been shown that a two-dimensional analysis gives a conservative result for a failure on a three-dimensional (dish-shaped) surface. The most common types of slope failures are illustrated in Figure 13.

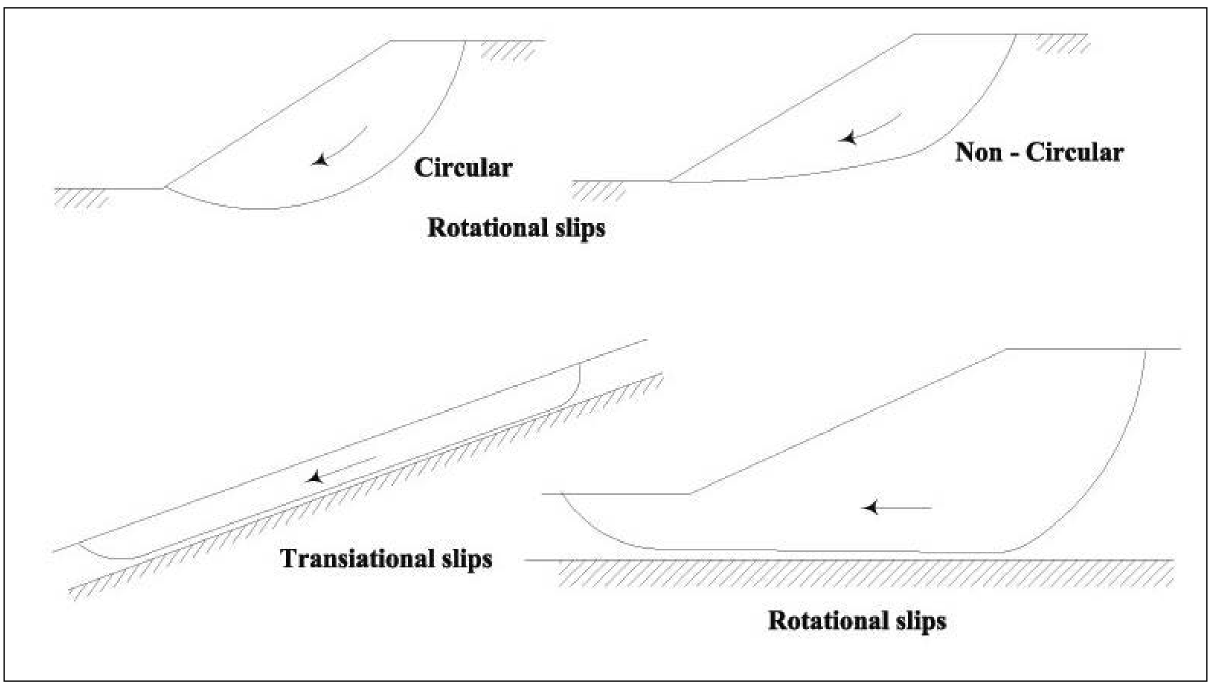



**Figure 13: Types of Slope Failures**

In *rotational* slips the shape of the failure surface may be a circular arc or a non-circular curve. In general, circular slips are associated with homogeneous soil conditions and non-circular slips with non-homogeneous conditions. *Translational* and *compound* slips occur where the geometry of the failure surface is influenced by the presence of an adjacent and denser stratum. Translational slips tend to occur where the adjacent stratum is at a relatively shallow depth below the surface of the slope: the failure surface tends to be plane and roughly parallel to the slope. Compound slips usually occurs where the denser stratum is at greater depth, the failure surface consisting of curved and plane sections.

### 5.6.1 Factor of Safety for Slope Stability Analyses

The factor of safety (FOS) for slopes should be selected based on the supporting structure type, impact of slope failure, uncertainty of soil parameters and temporary or permanent, conditions such as rapid drawdown, seismic etc. For overall stability analysis of walls and structural foundations, the factor of safety selected for design shall be consistent with the AASHTO LRFD Bridge Design Specifications. For slopes adjacent to but not directly supporting structures, a minimum factor of safety of 1.3 shall be used. For foundations on slopes that support structures such as bridges and retaining walls, a minimum factor of safety of 1.50 shall be used. Exceptions to this could include minor walls that have a minimal impact on the stability of the existing slope, in which the 1.30 minimum factor of safety may be used.

For general slope stability analysis of permanent fills and landslide repairs that do not support structures, a minimum safety factor of 1.3 shall be used. A minimum factor of safety of 1.5 shall be used of cut slopes. For temporary or staged construction a minimum factor of safety of 1.2 may be used. For extreme events such as a rapid drawdown or earthquake event the minimum factor of safety is equal to 1.1. Larger safety factors should be used if there is significant uncertainty in the analysis input parameters.

### 5.6.2 Software Programs

The Agency has selected the SLIDE software program developed by RocScience as its preferred slope stability software program. SLIDE's slope stability analysis software can be used to perform 2D Limit Equilibrium analyses with finite element groundwater seepage analysis, rapid drawdown, sensitivity and probabilistic analysis and support design capabilities. All types of soil and rock slopes, embankments, earth dams and retaining walls can be analyzed. SLIDE also has CAD capabilities which allow the user to create and edit complex models.

SLIDE includes a finite element groundwater seepage analysis for steady state or transient conditions. Flows, pressures and gradients are calculated based on user defined hydraulic boundary conditions. Seepage analysis is fully integrated with the slope stability analysis or can be used as a standalone module. It has probabilistic analysis capabilities; statistical distributions may be assigned to almost any input parameters, including material properties, support properties, loads, and water table location. The probability of failure/reliability index is calculated, and provides an objective measure of the risk of failure associated with a slope design. Sensitivity analysis allows the user to determine the effect of individual variables on the safety factor.

The Agency also has past experience with using SLIDE exclusively for over 10 years. There are other computer programs that the Agency uses such as ReSSA that perform both internal and external stability calculations, these programs may be used to complement the global stability analysis conducted using SLIDE. The Agency's Consultants are required to use SLIDE so that their analysis may be reviewed, verified and/or modified as necessary by Agency personnel.

### 5.6.3 Acceptable Methods for Slope Stability Analyses

#### 5.6.3.1 Computer Analysis

The methods of Morgenstern and Price, Spencer, and Janbu's generalized procedure of slices will yield reasonable estimates of the factor of safety for failure surfaces of any shape. However, because of the difficulty associated with selecting an appropriate force function for use with the Morgenstern and Price method, and the frequent numerical instability problems associated with Janbu's generalized procedure, those methods may not be suitable for general engineering practice. Spencer’s Method uses an iterative procedure to satisfy both force and equilibrium conditions for all selected slices. **As a result, it is recommended that Spencer's method be used for analyses of failure surfaces of any shape.** In addition, it is recommended that the Taylor and Bishop modified methods be used for the analysis of circular failure surfaces. If a stability analysis has been performed using a method other than the Spencer, Taylor, or Bishop methods, it is recommended that the factors of safety for critical surfaces be checked using one of these three methods.

#### 5.6.3.2 Simplified Design Charts

For very simplified cases, design charts are available to perform a preliminary assessment of slope stability. Examples of simplified design charts are provided in NAVFAC DM-7. These charts are for a c-ϕ soil, and apply only to relatively uniform soil conditions within and below the cut slope. They do not apply to fills over relatively soft ground, as well as to cuts in primarily cohesive soils. Since these charts are for a c-ϕ soil, a small cohesion will be needed to be assumed to perform the assessment.

### 5.6.4 Search for Critical Failure Surfaces

It is essential to perform a thorough search for the critical slip surface to ensure that the minimum factor of safety is calculated for the slope. The searching method needs to be varied depending on the geologic conditions in the slope. The search for failure surfaces should consider both circular and non-circular failures. Regardless of the identified failure surface, a sufficient number of failure surfaces should be generated so that a range of reasonable failure paths is considered. For a simple slope failure a minimum number of 5000 failure surfaces is recommended for the initial review. However, there is no exact value that can be used as each slide area has a different geometry and additional modifications may be necessary depending upon the slope geometry.

Care should be exercised to include obvious failure initiation points such as the toe of the slope or points where the slope angle changes significantly. Careful consideration should be given to specifying the range and spacing such that obvious initiation and exit points are expressly checked by the search.

[Note: SLIDE can be utilized to search for a critical circular or noncircular failure surface using a grid. Multiple analyses using various grid sizes must be conducted to ensure that all local minimums are found.]

### 5.6.4.1 Infinite Slope Condition

The infinite slope condition as presented in most text books is also referred to as a surficial slide. For this type of failure condition the failure mechanism is anticipated to be relatively shallow and parallel to the slope face, with or without seepage affects. For infinite slopes, consisting of <u>cohesionless soils</u>, that are above the water table or that are fully submerged, the factor of safety for slope stability is determined as follows:

$$FS = \frac{\mathrm{Tan}\,\emptyset}{\mathrm{Tan}\,\beta}$$

Where,

| | | |
|---|---|---|
| $\phi$ | = | the angle of internal friction for the soil |
| $\beta$ | = | the slope angle relative to the horizontal |

For infinite slopes that have seepage at the slope face, the factor of safety for slope stability is determined as follows:

$$FS = \frac{\gamma_b}{\gamma_s}\frac{\mathrm{Tan}\,\emptyset}{\mathrm{Tan}\,\beta}$$

Where,

| | | |
|---|---|---|
| $\gamma_b$ | = | the buoyant unit weight of the soil |
| $\gamma_s$ | = | the saturated unit weight of the soil |

Considering that the buoyant unit weight is roughly one-half of the saturated unit weight, seepage on the slope face can reduce the factor of safety by a factor of two. This is a condition which should be avoided through some type of drainage; otherwise much flatter slopes will be needed to achieve an adequate factor of safety. When using the infinite slope method, if the FS is near or below 1.0 to 1.15, severe erosion or shallow slumping is likely. Vegetation on the slope can help to reduce this problem, as the vegetation roots add cohesion to the surficial soil, improving stability.

[Note: Conducting an infinite slope analysis does not preclude the need to check for deeper slope failure mechanisms.] .

Natural slopes and manufactured fill slopes can be subject to shallow surficial failure referred to as soil slumps or soil slips during periods of intense rainfall or excessive irrigation. These failures are typically less than about 4 feet in depth and have small thickness to length ratios. These failures are typically analyzed using the infinite slope model suggested by Campbell (1975). The infinite slope model, depicted in Figure 14, assumes an infinitely long failure surface parallel to the ground surface with a perched groundwater table parallel to and coincident with the ground surface and is applicable to both <u>cohesionless and cohesive soils</u>.

The equation for factor of safety based on that model is:

$$F = \frac{c' + (\gamma - m\gamma_w)hcos^2\beta tan\phi'}{\gamma h \sin\beta \cos\beta}$$

Where,

| | | |
|---|---|---|
| $F$ | = | the factor of safety |
| $c'$ | = | the cohesion intercept |
| $h$ | = | the vertical depth of the slip surface |
| $\beta$ | = | the slope angle |
| $\gamma$ | = | the saturated unit weight (density) of the soil |
| $\gamma_w$ | = | the unit weight of water |
| $m$ | = | the fraction of h such that mh is the vertical height of the groundwater table above the slip surface |
| $\phi$’ | = | the angle of shearing resistance |

It should be noted that the shear strength parameters applicable for use in this equation must be determined at very low normal stress (100 to 300 pounds per square foot). Direct shear tests performed at those low normal stresses can be unreliable. Therefore, it is recommended that tests be performed at relatively low normal stresses such as 400, 800, and 1,500 pounds per square foot and that a curved failure envelope passing through or nearly through the origin be fitted to the test results. The shear strength parameters used in the analysis should be represented by the tangent to the curved envelope at the effective normal stress being analyzed.

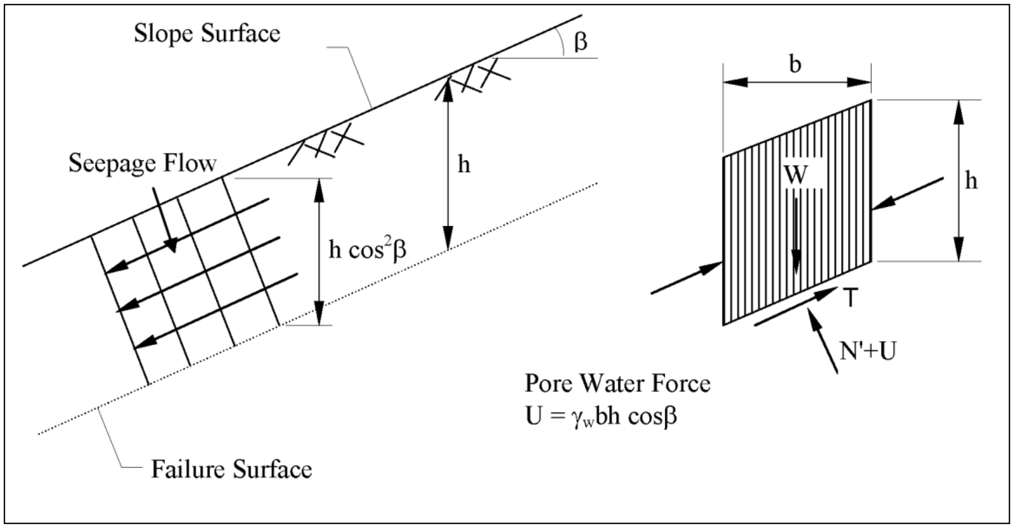



**Figure 14: Infinite Slope Condition**

Skempton and DeLory (1957) concluded there is "rather strong evidence suggesting that, on a geological time scale, stiff-fissured clays in natural slopes behave as if c' = 0 psf” even though their laboratory shear test data indicated an average cohesion of about 250 psf. Therefore, the Engineer should be cautious when using cohesion in the infinite slope formula to determine factor of safety. The infinite slope analysis method discussed above when combined with properly determined shear strength parameters represents an analysis that accurately represents the worst case conditions for this type of failure. A factor of safety of 1.3 shall be applied to analyses based on shear strength parameters determined from a failure envelope that passes through the origin.

All slopes steeper than a 1V:2H gradient shall be evaluated for stability. Slopes that are flatter than 1V:2H should be analyzed based on engineering judgment and local experience with geometry, surroundings, and subsurface conditions.

#### 5.6.4.2 Circular (or Rotational) Failure Surfaces

Circular failures generally occur in slopes composed of homogenous material. Often these slope failures are due to a combination of a sudden or gradual loss of strength, negative groundwater effects and changes in the geometric profile such as steepening of an existing slope or embankment.

#### 5.6.4.3 Non-Circular & Translational (Sliding Block) Failure Surfaces

Translational (block) or noncircular failure surfaces are generally more appropriate for modeling thin weak layers or suspected planes of weakness. If there is a disparately strong unit either below or above a thin weak unit, the modeled failure plane should lie within the suspected weak unit so that the most critical failure surface is modeled as accurately as possible. Circular searches for these types of conditions do not generally model the most critical failure surface.

If non-circular failure surfaces are to be used, geologic judgment and kinematics need to be considered. For example, if Spencer's method is used to generate a failure surface that has a nearly right-angle bend, then the calculated factor of safety may be too high because the program has produced a failure surface with a kinematically unreasonable geometry. That problem can be detected by checking for very high base-of-slice normal-stresses and shear resistances in narrow slices. Those high stresses and resistances result from the concentration of high side forces at the right angle bend, which creates high base-of-slice normal-forces and unreasonably high shear resistance.

Spencer's analysis can yield factors of safety that are significantly higher than those produced by a simplified Janbu analysis when kinematically unreasonable surfaces are specified (dip-slope analyses with passive toe wedges can create that problem). The problem can often be resolved by searching for similar, but kinematically more reasonable surfaces, in nearly the same area. When using SLIDE to generate a large number of non-circular randomly shaped surfaces, the engineer should carefully evaluate the results for convergence; sound engineering judgment often results in identifying more critical failure surfaces.

#### 5.6.5 Surcharge Conditions

Loads are applied to slopes as a result of both construction activities and operational needs. They can be permanent such as a structure or utility or temporary as in construction equipment or the passage of a vehicle or train. For the purposes of highway embankment analysis, a live load surcharge of 280 psf is used over the entire roadway surface. For railroads, a live load surcharge of 1882 psf, corresponding to a Cooper E80 loading is applied over a tie width of 8.5 feet, see Appendix B. Other construction loads should be evaluated on a case by case basis.

## 6.0 Remediation Strategies

Remediation strategies included herein can be used to resolve slope instabilities during design or existing slope stability issues.

### 6.1 Overview

In general, guidance outlined in Chapter 17 "Stabilization of Soil Slopes" of the TRB Special Report 247 entitled "*Slides: Investigation and Mitigation*" should be followed. Table 17-1 of this TRB report outlines the general approaches to the design of stable remediated slopes for these three categories or strategies:

- Avoid the problem area
- Reduce the forces tending to cause movement
- Increase the forces resisting movement

Several feasible options or combination of alternatives (typically including berm, shear key, flattened slope, excavation/replacement, etc.) should be considered. Other methods (retaining walls, slope reinforcement, lightweight fills, various drainage options, etc.) may also be technically and economically feasible. However, it is important to first correctly diagnose the failure mechanism so as to determine the applicable mitigation strategies.

### 6.2 Non-structural Mitigation Strategies

Historically the utilization of a stone counterberm and/or stone key in combination with the installation of an underdrain system has been the most frequently used mitigation strategy. This approach has proven to be an economically feasible and incorporates a combination of strategies outlined in Section 6.1.

#### 6.2.1 Avoidance of the Problem Area

Consider relocation or realignment of the proposed or existing roadway. Complete or partial removal of unstable materials should also be considered. In some instances, spanning the unstable area with a structure (bridge) supported on driven piles or drilled shafts may be feasible.

#### 6.2.2 "Do Nothing" Approach

This approach may be the least expensive, but a decision to employ this alternative should be based on an adequate understanding of the risks, remediation alternatives and costs, and potential consequences. This approach does not increase the level of stability; instead this approach may increase future maintenance activities and costs. The “do nothing” approach can be used where a slope failure does not appear to be imminent, an impending threat lies outside the ROW or where financial resources are not available for remediation and the risk to the traveling public can be effectively managed.

### 6.2.3 Balanced Approach

As the term balanced approach implies, this alternative’s primary goal is to achieve a balance between conducting maintenance activities and implementing cost effective and viable construction solutions. Typically the balanced approach is implemented because the desired safety factor cannot be attained within the allotted budget or schedule. A reduced factor of safety that deviates from the guidance given in this manual shall be discussed and documented in writing by the Geotechnical Engineering Manager. It can be either a short or long term approach depending upon the infrastructure requirements and the Agency’s priorities.

Immediate maintenance activities are those activities which have an overall improvement on slope stability, such as ditching, existing drainage system repair/rehabilitation, and the implementation of safety measures such as raising guard rail and leveling. Typically, these solutions are low cost remediation plans that do not have ROW or environmental conflicts nor do they require large scale plan development. These solutions may offer a limited improvement on slope stability.

The Agency, through its utilization of the “balanced approach”, recognizes that the factor of safety is below the desired level and so assumes certain risks. The Agency should perform a risk management assessment to identify and quantify the risks prior to implementation. The Agency also commits itself to employing acceptable risk management strategies to manage the associated risks. Risks are assigned and accepted based on safety and budget considerations and the effect of the impact on the use of the present infrastructure.

These projects typically include some form of instrumentation monitoring to continually assess the site condition and safety. For example monitoring wells and inclinometers are installed and monitored remotely. The results of this instrumentation monitoring are provided to the Engineer by the Geotechnical Engineering Section Instrumentation Technician so that the Engineer can monitor the slope’s movement and verify that the slope does not represent an immediate safety concern to the traveling public.

### 6.2.4 Slope Flattening

An effective mitigation measure, if ROW and cost allow, is to decrease, or “flatten” the existing slope. The horizontal distance from the toe of the slope to the slope crest is increased thus decreasing the slope angle and increasing the factor of safety. This alternative has the most benefit for over-steepened slopes. This alternative may be combined with other measures such as roadway realignment and the lowering of the roadway grade or additional drainage installation or maintenance.

### 6.2.5 Stone Counterberm and Key

A counterberm is a large mass of material placed adjacent to an existing slope that resists or prevents the rotation or translation of the slope due to excessive driving forces. The use of a stone counterberm is designed to prevent slope movement by providing sufficient dead weight or restraint near the toe of an unstable mass. The berm must be stable against rotation and sliding at or below its base. Both stone fill and granular borrow can be used for counterberm material. If the counterberm is to be placed adjacent to a river, Type IV stone should be used and keyed into the existing ground surface as described below.

Typically, when a slide occurs adjacent to a river it is because the river eroded the toe of the slope (resisting forces were removed), which allowed a rotational failure to occur. If it is determined that a counterberm is not needed for slope stability reasons and the slope only requires protection against future erosion, a Type IV stone key is placed at the toe of the existing slope to a depth of 4 to 6 feet below the ground surface to resist movement and to protect the toe of the slope from future erosion. The Type IV stone fill typically transitions to a 2 foot Type II stone blanket at the ordinary high water elevation. The stone blanket prevents surficial sloughing of the slope. In addition, a slope stability analysis should be conducted to verify the design geometry for each project.

Stone keys are also effective tool at mitigating sliding block failures. Keys are constructed by excavating into competent material in the area below the shear zone or slide plane and replacing it with rock to prevent further sliding.

See Appendix D and E for typical details.

### 6.2.6 Dewatering: Drainage/Groundwater Lowering

Mitigating the presence of surface water and ground water through the implementation of various drainage strategies is the most widely used and generally the most successful slope stabilization method utilized. The removal of surface water flowing into tension cracks or ponding on the slope surface will prevent the saturation and erosion of the upper soil layers. Positive drainage will reduce the pore pressures within the soil matrix resulting in an increase in the soil shear strength.

#### 6.2.6.1 Darcy’s Law

Darcy's law provides a means of calculating seepage flow rates and velocities in saturated soils. There are numerous practical applications of Darcy's law in the analysis of groundwater flow and design of subsurface drainage. It is commonly used to determine the capacity of underdrains and pavement drainage systems.

Darcy's law relates flow through porous media linearly to a proportionality constant, k, and the hydraulic gradient, i. Darcy's law is expressed in the following form:

$$Q = k\,i\,A$$

Where,

| | | |
|---|---|---|
| $Q$ | = | discharge through an area [volume/time] |
| $K$ | = | coefficient of permeability (length/time) |
| $i$ | = | hydraulic gradient (ratio of change in hydraulic head and linear distance of fluid flow) (dimensionless) |
| $A$ | = | area through which flow occurs (length$^2$). |

Darcy's law is valid within the range of steady state, laminar flow. This holds for flow in most naturally occurring deposits and man-made fills. In highly permeable granular materials where turbulent flow may exist, the validity of Darcy's law is questionable should be used with caution. Experience has shown Darcy's law to be valid for soils finer than coarse sand and gravel deposits with permeability up to 3,000 ft/day.

#### 6.2.6.2 Determination of Coefficient of Permeability

The coefficient of permeability, *k* in Darcy's law, is defined as the flow rate through a unit area with a unit hydraulic gradient. It indicates the capability of a material to carry water. Both soil and fluid properties affect the coefficient of permeability. Permeability is a function of soil particle size, soil void ratio, mineral composition, soil fabric, and degree of saturation. The coefficient of permeability is also a function of the fluid density and viscosity.

It is always preferred to determine permeability by direct methods in the laboratory or field. These methods include:

- Laboratory Constant Head Tests,
- Falling Head Tests, and
- Field Pump Tests

Tests to determine the coefficient of permeability for fine grained soils can take a considerable time to perform; therefore permeability is sometimes determined indirectly from triaxial compression test results or from consolidation tests. Procedures for the above mentioned testing methods can be found in soil mechanics texts or laboratory manuals. Although field or laboratory determinations of permeability are ideal, they can pose both great expense and difficulty. In practice it is often necessary to estimate the permeability soil or well filter material with empirical equations or charts that relate permeability to soil gradation.

The relationship between soil grain size and permeability can be used to estimate the permeability. Permeability of granular soils has been found to be proportional to grain size by Hazen's Formula:

$$k = C\,D_{10}^2$$

Where;

| | | |
|---|---|---|
| $k$ | = | coefficient of permeability (in/sec) |
| $C$ | = | proportionality constant ($C$ = 1 for coarse sands and gravel); |
| $D_{10}$ | = | effective grain size in inches (the particle diameter for which 10 percent of the soil mass passes in a sieve analysis) |

It should be noted that the coefficient of permeability varies over many orders of magnitude depending on the soil properties. In natural deposits and some compacted soils permeability may be much greater in one direction than in the other. The coefficient of permeability for a soil is a very difficult value to determine and results obtained from these methods are approximations which should be used with discretion.

#### 6.2.6.3 Underdrains

The location and depth of underdrains depends largely on the function intended and local geology. Multiple underdrain installations are often constructed in a herringbone pattern. Such installations are well suited for collecting large quantities of groundwater, such as springs under roadbeds, and for stabilizing fill foundation areas.

Most of the components of underdrains are also integral components of other subsurface drains. Each component of a subsurface drainage system serves a particular function in ensuring that the drain performs as intended. Design of any subsurface drain should ensure a system that is cost effective, constructible, compatible with the surrounding soils, and that will provide adequate drainage throughout its design life. The following are the basic components of a subsurface drainage system:

- Filter/separator layer;
- Conducting drainage layer;
- Collector pipe;
- Outlet; and
- Appurtenances.

It is important for the engineer to understand the function and interaction of the basic components of subsurface drains. Site specific conditions must be considered and each component appropriately designed to ensure that the installed drain will perform as intended. A discussion of the function, interaction, and design criteria of each component is included in the following sections.

It is current practice to attempt to lower the water table with the use of horizontal underdrains. Underdrains have routinely been placed anywhere from 6 to 20 feet in depth and their primary purpose is to intercept ground water flowing down the slope, see Figure 15. Sometimes multiple underdrains can be used on one project if the slope consists of weak soils with saturated soil conditions.

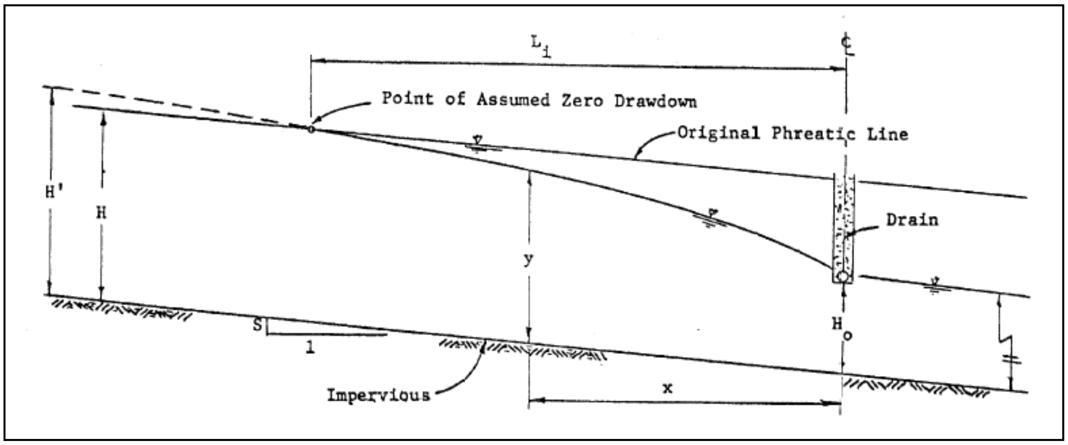



**Figure 15: Flow Toward a Single Underdrain**

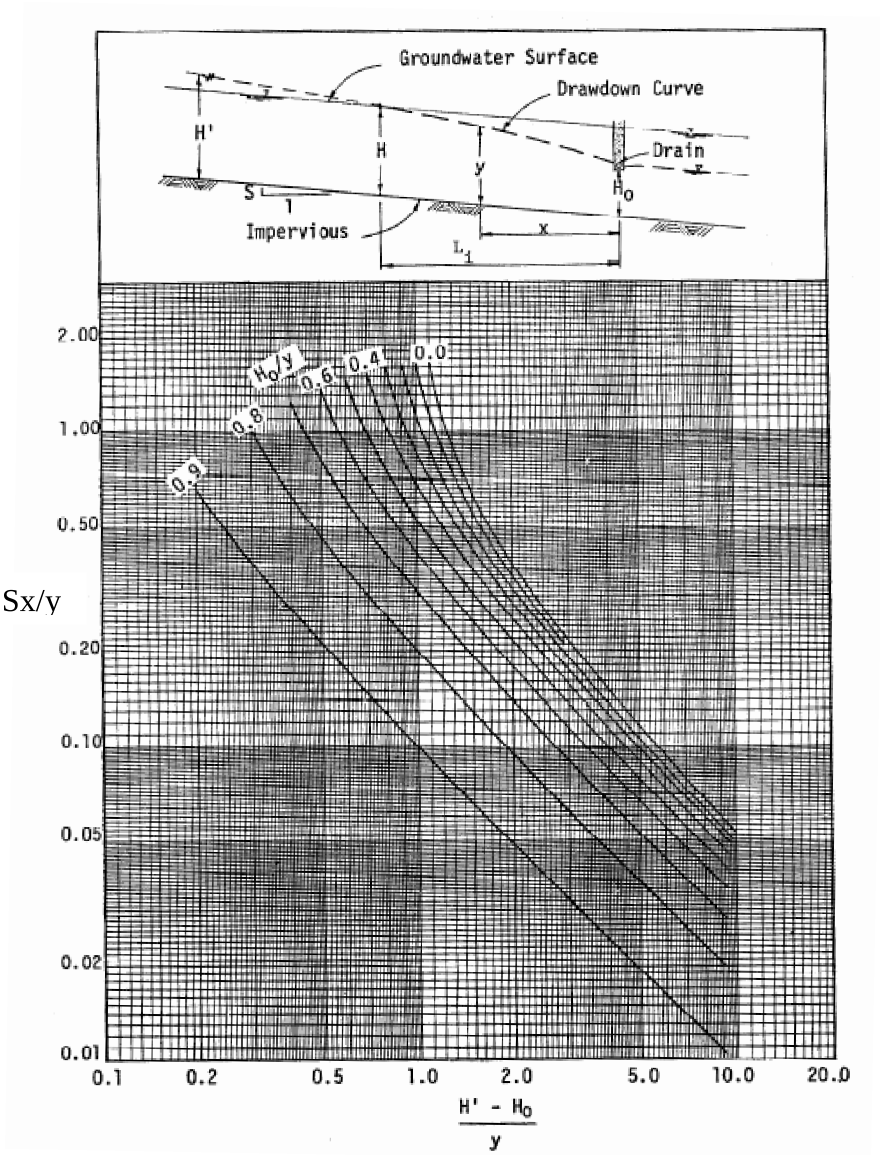
Deep underdrains, or interceptor drains, can be used to lower groundwater levels in slopes and intercept seepage before it can reach the slope face. Interceptor drains are most effective when deep enough to intercept an impervious layer below the surface. Although interceptor drains as deep as 30 feet have been constructed, construction techniques and worker safety should be considered before recommending an underdrain. Often other drainage methods will need to be considered when subsurface drainage is required at greater depths. If continued movement of the slope is possible, perforated pipe in an underdrain is likely to rupture and fail. This may warrant using an aggregate drain without a collector pipe also referred to as a french drain.

The profile of the lowered water table can be estimated using Figure 16 while the quantity of the flow towards a single underdrain can be estimated by means of a flow net analysis. The first step will be to determine the "radius of influence" or drawdown influence distance, which can be estimated, for practical purposes, by means of the following expression:

$$L_i = 3.8\,(H - H_o)$$

Where,

| | | |
|---|---|---|
| $L_i$ | = | the influence distance (feet) |
| $(H - Ho)$ | = | the amount of drawdown (feet) |




Source: FHWA Highway Subdrainage Design Manual

**Figure 16: Chart for Determining Drawdown Curves for Interceptor Drains**

### 6.2.6.4 Horizontal Drains

Horizontal drains are relatively inexpensive and can be effective in lowering groundwater levels and relieving stresses on slopes, side hill fills and behind retaining structures. Their principle use is in slope stabilization applications. A horizontal drain is a perforated or slotted pipe advanced into a slope with a special auger typically orientated at 5 to 10 degrees above horizontal, see Figure 17. The last 10 ft of pipe is non-perforated to assure that water flows out. Filter material or filter fabric should be used if clogging is expected; this can greatly extend the life of the drain but is extremely difficult to install. These drains are commonly installed in fan-shaped arrays of several pipes emanating from a common point.

Construction of horizontal drains can often be complicated depending on the drilling capabilities and techniques used. Soil conditions and moisture can affect stability of borings. Horizontal and vertical controls are essential to ensure that the drains are installed as intended. Regular maintenance and inspection of horizontal drain installations is critical to ensure effectiveness. Horizontal drains can clog from precipitation of metals, piping of fine particles and root penetration. Clogged drains can sometimes be cleaned with high pressure water systems. Drains installed in unstable soil slopes which continue to move after installation can fail. Steady discharge from the drain may cause dense vegetation to grow at the outlet which can conceal and plug the outlet if regular maintenance is not performed.

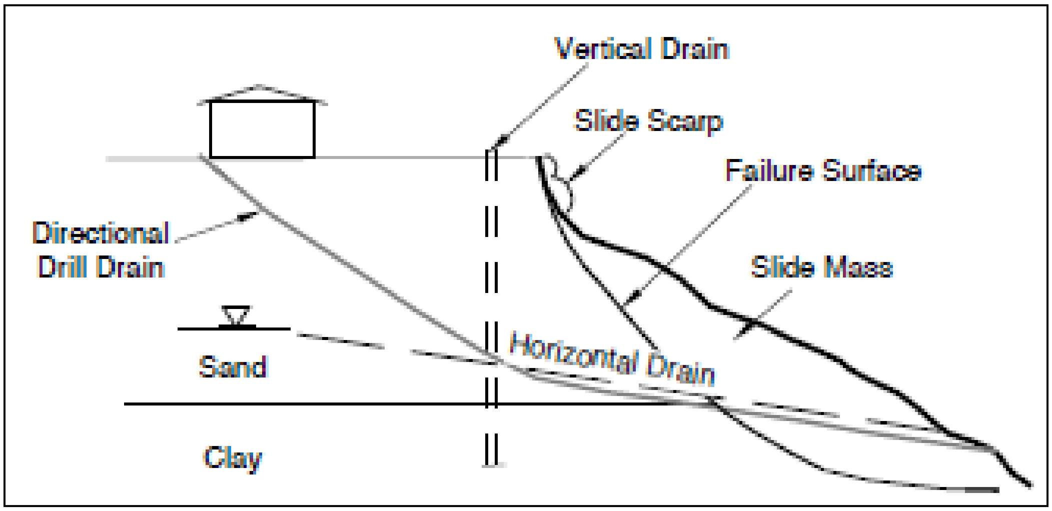



**Figure 17: Horizontal and Vertical Drainage Elements**

#### 6.2.6.5 Vertical Drains

Although gravity drainage is possible in certain circumstances, vertical drains are typically installed in combination with pneumatic pumps, especially when the stability of the slope requires a significantly lowering of the groundwater table.

Vertical drains are used to lower the groundwater level in a landslide or marginally stable slope where the depth to groundwater is too deep for dewatering using horizontal drains or underdrains, see Figure 18. The main advantage of vertical drains (or wells) is that they can be installed at virtually any depth. Limitations include relatively high cost and the ability to intercept a sufficient amount of the permeable water-bearing zones to effectively lower the groundwater level. A thorough understanding of the subsurface soil and groundwater conditions is essential in planning a dewatering system using drilled drains. The Engineer should consult with a hydrogeologist to explore the subsurface conditions, evaluate groundwater flow, and perform slope stability studies to develop an optimum drain configuration.

The hydrogeologist should design the most appropriate drain spacing, well diameter, and well screen size. Pumping tests or other aquifer tests are commonly required to evaluate the effectiveness of proposed drilled drains. If drilled drains are selected as an element in improving the stability of a slope, groundwater monitoring wells should also be installed and monitored before and after drain construction to verify that the drains are achieving the degree of lowering in the groundwater levels desired. These groundwater monitoring wells can also be used to monitor the effectiveness of the system over time.

If vertical drains are suitable, the site access limitations, the subsurface conditions, and construction costs typically dictate which system is feasible for each particular site.

### 6.2.7 Lightweight Fill

The use of lightweight backfill materials can be very effective in reducing the gravitational driving forces tending to cause instability. Therefore, it is usually considered as an option under a roadway and near the top of a failing slope. Lightweight materials include expanded clay or shale, expanded polystyrene (EPS) or shredded and chipped tires. When EPS (unit weight as low as 2 pcf) is specified to replace material with a unit weight of 125 to 150 pcf, the factor of safety can be dramatically improved. The mitigation method of substituting lightweight fills for standard embankment material has the largest positive effect on small scale slopes where the replacement of the lightweight fill significantly lowers the driving forces.

## 6.3 Structural Mitigation Strategies

Sometimes the use of a counterberm or lightweight fill material is not possible because of site limitations or cost. For instance the cost of encroaching on adjacent private property can be undesirable; therefore the construction of a retaining wall may be a feasible alternative.

There are externally and internally stabilized systems that can be used to mitigate slope instabilities. Externally stabilized systems rely on external structural walls or fills against which the forces are mobilized, such as gravity, braced, or tied-back walls. Internally stabilized slopes usually consist of reinforcement or soil nailing/doweling. Stabilization systems can be used at different points in the slope, depending on the type of failure and site characteristics. Sometimes a retaining wall can be put at the toe of slope to prevent or minimize toe erosion by river scour. Retaining walls can also be used to decrease the angle of the slope, but often have to be used in combination with an anchor system to resist the destabilizing forces. Some basic types of externally stabilized wall systems are concrete cantilever, timber crib, sheet pile, and steel bin walls. Internally stabilized wall systems include reinforced soil walls and walls with in-situ reinforcement (soil nail walls, soil doweling, reticulated micro-piles).

### 6.3.1 Reinforced Soil Slopes (RSS)

Reinforced soil slopes are feasible alternatives for situations where a steep slope lacks stability (typically alongside a highway) and additional ROW cannot be obtained to flatten the slope. Stability is achieved through the use of geosynthetic reinforcing strips (grid or fabric). Often the face of these slopes is "wrapped" with geotextile fabric to prevent surficial sloughing. More information can be referenced in FHWA's GEC 11: Design of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes.

### 6.3.2 Ground Anchors

A ground anchor is a system used to transfer tensile loads to the ground (either soil or rock), which includes the pre-stressing steel, anchorage, corrosion protection, and grout. The anchor systems increase the resisting forces by applying external restraint to a moving soil mass. The most common use of ground anchors for the Agency is in combination with a support wall, such as a soldier pile wall. This remediation approach is usually considered a costly one, but sometimes necessary with project right-of-way limitations and large driving forces. PTI's "Recommendations for Prestressed Rock and Soil Anchors" should be consulted during the design and development of project specifications. Refer to FHWA's Engineering Circular on Ground Anchors and Anchored Systems for more information.

### 6.3.3 Soil Nail Walls

Soil nails are steel bars that are grouted into predrilled boreholes. The nails form reinforced-soil structures capable of stopping the movement of unstable slopes or of supporting temporary excavations. Soil nails differ from tieback systems because the nails are passive elements that are not post-tensioned and they are generally spaced closer than tiebacks. A high water table may present construction difficulties for soil nailing because the hole will most likely not stay open without aid. Refer to FHWA's Engineering Circular on Soil Nail Walls for more information.

### 6.3.4 Soil Nails & Wire Mesh Stabilization

This remediation combines the use of soil nails with a high strength steel wire mesh facing. The soil nails are installed to increase the internal strength of the soils in the failure zone, therefore increasing the local stability of the slope. The mesh is then attached to the nails, and as the nail head plate assemblies are torqued, the mesh is pulled into the soil placing the mesh in tension and constraining the near-surface soils in the slope. Both the plate and the mesh that are used within the system are key factors when looking at the three types of possible failure in the soil nails; pullout (force required to pull the length of the nail out of the slope), tensile failure (maximum axial capacity of the soil nail), and stripping (slope failure occurs, but nail remains embedded in slope). This system has been successfully implemented for translational failure mechanisms where bedrock has been relatively shallow.

A typical soil nail force diagram, which exhibits all three failure modes, is shown below in Figure 18. In this example, the plate (mesh) capacity is less than the tensile capacity, and therefore "stripping" is a possible failure mode. If the plate capacity is greater than or equal to the tensile capacity, then stripping cannot occur, and the soil nail force diagram will be determined only by the Tensile and Pullout failure modes.

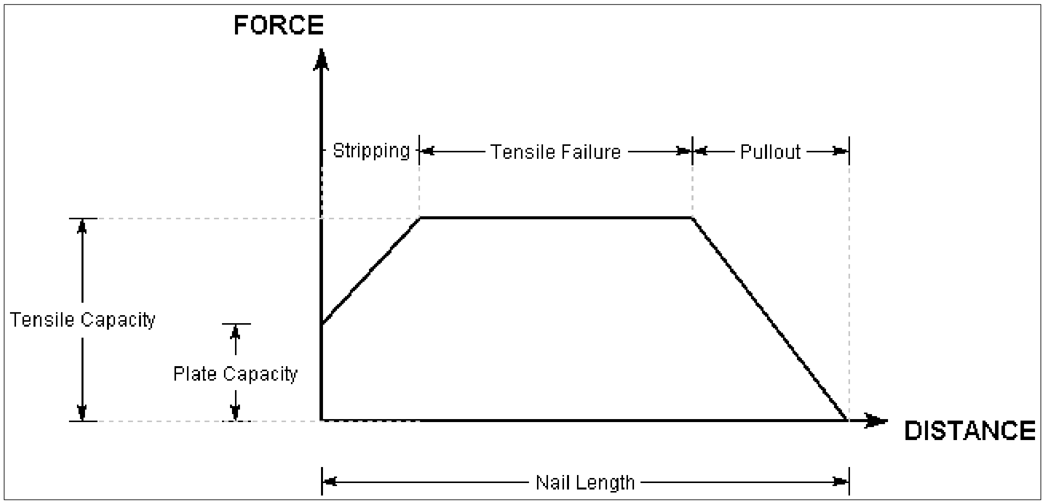



**Figure 18: Soil Nail Force Diagram**

A future VTrans MREI will discuss the use of soil nails and wire mesh for soil slope stabilization strategies in more detail and will provide additional design guidance on plate capacity, material options and properties, plan detailing, testing requirements and torque values. Additional information can be found in the technical reference manual entitled “Tecco Slope Stabilization System and Ruvolum Dimensioning Method”.

### 6.3.5 Aggregate Piers

Aggregate piers are structural stone columns that are typically installed at the toe of slopes. They can be installed below the subgrade to act as a shear key to provide sliding resistance. The aggregate piers improve stability by providing significant increases in the composite shear resistance due to their high angle of internal friction of the aggregate. The piers also act as vertical drains reducing the possible excess pore water pressures in the slope. Aggregate piers should be considered when soft, loose or very low strength soil is present in the slope. The design of the piers is typically performed by either a proprietary company or specialty contractor.

## 6.4 Driven Piles

Piles used for slope stabilization purposes are installed through the critical slip surface of the slope to provide (shear) reinforcement to resist shearing forces that develop along the planar failure surface. Piles are typically spaced at 2 to 4 times their diameter to ensure that the moving soil above the failure plane does not laterally flow around or between the piles. Other limit states that need to be checked include failure of the stable soil into which the piles are embedded due to excessive lateral earth pressure against the piles from the moving soil, axial pullout of the piles from the stable soil, axial pullover of the soil above the failure surface and structural pile failure.

The slope stability program SLIDE requires an input parameter called ‘pile shear strength’ which applies a resisting force at the location of the pile and depth of the critical sliding surface. This shear resistance can be found by performing a pile-soil response analysis in LPile. By applying lateral soil mass movements at various sliding depths along the pile, both the shear resistance and moment along the pile can be found. An appropriate shear resistance can be determined which corresponds to an acceptable amount of lateral movement and bending stress in the pile. Since the contribution for shear resistance of the piles is a function of ground movement, the amount of tolerable movement needs to be carefully considered in the design. Limiting soil movement to 3 to 5 inches is recommended for slopes not supporting a structure.

A minimum embedment depth of 15 feet below the failure plane into medium dense to dense soil is recommended for driven piles. If bedrock is shallower than 15 feet below the failure surface, micropiles should be considered as an alternative to driven piles. Further guidelines on the use of piles to stabilize slopes will be made available in a future MREI.

### 6.5 Mitigation Selection Process

Once an "existing conditions" design model is developed as described in Section 4.0, remediation alternatives can be designed to satisfy the specified factor of safety. Alternatives shall be considered and chosen based on the following discussion. Once a satisfactory alternative is found, the alternative must be applied to key cross-sections to ensure applicability and constructability of the entire slope.

The slope geometry can differ greatly throughout the slope which makes developing several cross sections with the chosen alternatives very important. It shall be the judgment of the Engineer based on slope geometry as well as design and construction considerations to determine the actual number of sections the remediation solution must be applied to in order to ensure uniformity and stability between the project limits.

Several elements such as factor of safety, cost and constructability should be considered during the selection of the preferred mitigation alternative. The cost estimates for each of the alternatives should be analyzed and reviewed. The selection process should be documented and placed in the project folder so that the process can be followed at a later date to determine how and why the chosen alternative was selected.

### 6.6 Cost Estimates

Cost estimates for mitigation alternatives shall be developed by the Engineer at the request of the project manager. Often times a cost estimate is needed for each alternative to aid the project manager in choosing the most feasible and cost effective alternative. Typically project managers from VTrans Operations Bureau will rely on the Engineer to develop the cost estimate whereas a VTrans Project Delivery project manager often performs the cost analyses as part of the overall design.

The cost estimate should include VTrans unit price estimates that are averaged and based on recent bid prices for different materials. Estimates should at a minimum include figures for all geotechnical components. Any components left out, such as traffic control, acquisition of land and reconstruction of any roadway items must be detailed in the section of the geotechnical report discussing associated costs. A cost-benefit analysis may be necessary for large budget solutions and/or high risk projects.

## 6.7 VTrans' Preferred Alternative

Several factors should be considered during the selection of the preferred alternative. The cost estimates for each of the alternatives should be analyzed and reviewed. The selection process for the preferred alternative should be documented so that the process can be followed at a later date to determine how and why the chosen alternative was selected. This process should be clearly documented and placed in the project folder.

VTrans typically examines the feasibility of the installation of a stone berm at the toe of the slope, in combination with the construction of an underdrain system on the uphill side of the slope adjacent to the roadway. Due to the high level of success in the past, ease of construction, and cost feasibility, this alternative has been VTrans' preferred mitigation alternative and should be considered for any slope remediation. See Section 8.1 for a remediation typical including material details.

## 6.8 Construction Monitoring & Instrumentation

Recommendations pertaining to construction monitoring and instrumentation should be considered based upon what alternatives are recommended in the geotechnical report described in Section 7.0. Typically, inclinometers and/or monitoring wells will have been installed on site to provide data for use during the project's subsurface investigation and design phases. The instrumentation installed during the subsurface investigation should be evaluated to determine if it remains viable for use to monitor construction activities. If not, additional instrumentation may need to be installed. Instrumentation plans and requirements should be documented in the geotechnical report, so that design details and requirements can be transferred into the contract documents.

### 6.8.1 Construction Instrumentation

Inclinometers should be considered when the project is either a very high risk project or the remediation includes ground anchors or soil nails. Inclinometers measure movement during the construction phase to ensure stability of the slope and safety of the construction personnel. Monitoring is necessary for the construction of a soil nail wall to observe wall movement as top-down construction occurs. Any recommendations pertaining to construction instrumentation and monitoring should be provided in the geotechnical report.

If the effectiveness of remediation alternative depends on the use of underdrains then monitoring wells should be installed during construction to monitor the water level and ensure long term performance.

Results of stability analysis during staged construction can be uncertain and therefore multiple instrumentation types are recommended. Piezometers should be used to monitor the build-up of excess pore pressures; results should be compared to calculated values to verify the rate of fill placement. Inclinometers and settlement platforms should be used to measure horizontal movements in the foundation soils beneath the toe of slope and settlements under embankments. The observed data can then be used to determine if movement is due to settlement or an impending instability.

### 6.8.2 Post Construction Monitoring

Post construction monitoring of inclinometers and monitoring wells may be required to measure slope movement, ground water fluctuation, or mitigation performance after construction.

#### 6.8.2.1 Frequency of Readings

The frequency of readings depends on the level of impact associated with failure and the interval of time that has passed since construction of the project. Manual readings are obtained on a daily or weekly basis during construction. Thereafter, the Engineer typically requests readings every one to three months for about a year. After a year, the frequency can be continued at 3 months, changed to a longer interval or performed at the Engineer's request. If a slope is very sensitive to saturated ground conditions, the Engineer may request more frequent readings during the spring and fall months, and less frequent readings over the winter and summer months.

#### 6.8.2.2 Distribution of Results

The results of post construction monitoring should be provided by the Engineer to the Geotechnical Engineer who will observe and track the overall performance of the instrumentation. Construction monitoring results should be provided to the Resident Engineer and the Project Manager as well as the Geotechnical Engineering Manager. The Resident Engineer will disseminate the information to the Contractor unless prior authorization has been given to the Engineer or Geotechnical Engineering Manager to communicate directly with the Contractor. The Geotechnical Engineering Manager will determine when to provide updates to other Agency personnel.

The Engineer shall compile all instrumentation monitoring results in an electronic format and placed in both the electronic and paper project folders. The Engineer shall review the instrumentation reports ensuring that the reports are accurate, legible and easy to comprehend.

## 6.9 Design and/or Constructability Review

All designs shall be peer reviewed within the Geotechnical Engineering Section. The review shall be performed by a competent Engineer designated by the Geotechnical Engineering Manager with a minimum of four years of experience in conducting slope stability analyses. The reviewer shall, at a minimum, review the development of soil stratum and design parameters, methodology used, design assumptions, computer models for the existing conditions and design alternative(s), as well as the geotechnical report. The reviewer shall initial and date each page of material reviewed. All reports shall go through the Geotechnical Engineering Manager prior to being forwarding to the Project Manager or Town.

The Geotechnical Engineering Manager may require an external peer review if the project is considered a high risk project. A few of the factors that could warrant an external review are:

- Large monetary cost for remediation
- High public safety concern (Interstate or state highway)
- Design complexity
- Limited experience with chosen mitigation alternative(s)
- Constructability concerns

External reviews may be completed for Agency or Consultant designed projects by Agency personnel, consultants, or contractor group representation. A consultant designed project may require an independent consultant's review based on any of the factors above. An Agency designed project may require a constructability review by the Agency's Construction Section before the design goes too far into the plan development stages. On large projects, or projects that will require the services of a specialty contractor, it is a good idea to have a constructability review performed by a contracting entity or trade organization such as the International Association of Foundation Drilling (ADSC).

## 7.0 Geotechnical Report

Unless otherwise determined, a geotechnical report shall be prepared for each project. The purpose of the geotechnical report is to present the subsurface data collected in a clear and concise manner, to provide data evaluation and to provide design recommendations for use by highway designers, structural engineers, and maintenance personnel. The geotechnical report will serve as the permanent record for the basis of the geotechnical design prepared for a specific project, with its use spanning the design, construction and post-construction project phases.

While each project will be unique to its site conditions and design constraints, the following itemized list should be used as a guide for the preparation of a geotechnical report for slope stabilization projects. An example of a VTrans geotechnical report provided for an Agency slope stabilization project is included in Appendix C.

### 7.1 General Project Information

1. Description of the project, including location, site map, scope, and any design assumptions or constraints.

2. History of the problem including possible triggering events or mechanisms, (i.e. flooding, prolonged periods of rain, excavations at toe, loading at top of slope, etc).

3. Description of the issue(s) and failure mechanism(s).

4. Description of existing roadways and structures, including possible ROW limitations.

5. Summary of information provided to the engineer (plans, cross-sections, alignments, hydraulics report, etc).
6. Pictures of pertinent site conditions.

## 7.2 Geology and Existing Geotechnical Information

1. Description of significant geologic, hydrogeologic and topographic site features.

2. Description of any observed geotechnical related issues (slope instability or rockfall history, observed settlement, etc.).

3. Description of any significant historical data (existing boring data, maintenance history, subsurface drainage installations, rockfall data, etc.).

## 7.3 Subsurface Investigation(s)

1. Description of the past and present field and scoping investigations performed. Include in the description the scope and purpose of the investigation(s), the methods used, when and why performed and any instrumentation installed. Use language from most recent reports with heading entitled “Field Investigation”.

2. Description of the laboratory testing program. Include the tests performed, the purpose of the testing, and where the information is summarized. Use language from most recent reports with the heading entitled “Field and Laboratory Testing”.

3. Provide a summary of groundwater observations.

## 7.4 Analysis/Design

1. Description of stratification of in-situ materials. A graphic soil profile at the critical section may be required depending on the complexity of the geology and project. The profile can be enclosed within the report as an attachment.

2. Summary of soil properties and groundwater levels recommended for design based on the investigations.

3. Identify the failure mechanism and provide supporting data and documentation.

4. Discussion of the selection of the critical cross sections.

5. Identify the computer software used for analysis.

6. Description of the design alternatives reviewed.

7. Discussion of the AASHTO recommended factor of safety, safety factor selected for design and the alternatives considered, analyzed and designed that meet or exceed this factor of safety. Additional discussion shall be included if factors of safety are below the recommended value.

### 7.5 Recommendations and Conclusions

1. Outline the feasible mitigation alternatives and discuss the advantages, disadvantages, and risks associated with each option.

2. Provide a detailed description of recommended mitigation alternative(s). Including limits of implementation, elevations and offsets of materials or inclusions, grades and dimensions of cut or fill areas, size of pipes, etc.

3. Include diagram or visual schematic of recommended alternative(s) with corresponding factor of safety.

4. Provide summary of cost estimates; determine if the cost estimate for all design alternatives considered are presented or only the preferred alternative.

5. Specify materials according to the latest VTrans Standard Specifications for Construction.

6. Include any project specific construction considerations or recommendations such as:
    a. Temporary excavation, including maximum slopes and suitable types of temporary support
    b. Right-of-way limits
    c. Site access
    d. Utility constraints
    e. Limits of excavation
    f. Types and location of any instrumentation used for monitoring groundwater, settlement or slope movements
    g. Testing requirements for solutions involving soil nails or dowels, ground anchors or any other geotechnical inclusions.
    h. Recommendations for any special construction method not addressed by the Agency's Standard Specifications.

7. Include language specifying that proper construction details need to be developed prior to construction.

### 7.6 Attachment / Appendix Information

As a minimum, geotechnical reports should include items 1-3 below, while items 4– 6 should be included in the report as necessary.

1. Boring logs
2. Site plan depicting boring locations

3. Critical soil profile/cross-sections
4. Design details and drawings
5. Detailed cost estimate (if deemed necessary)
6. Special provisions

E-mail may be used for the distribution of geotechnical reports and for providing recommendations in certain circumstances. E-mails may also be used to transmit review of construction submittals or to transmit preliminary geotechnical recommendations. In both cases, a hard copy of the geotechnical report should be placed in the project file.

## 8.0 Design Details & Drawings

The conceptualized solutions may require the development of drawings such as cross sections, design details, and typical sections. When developed these drawings should be included as attachments to the geotechnical report. The Project Manager should convey the chosen design alternative, after which the Engineer can then begin to develop material specifications and project specific cross sections and design details that support the project's recommended design alternative.

### 8.1 Preferred Slope Detail

Section 6.0 outlines the Agency's preferred alternative that is usually considered first in slide investigations. This typically includes the installation of a stone key at the toe of slope, sometimes a stone blanket extending up the slope to the top, and a vertical interceptor underdrain on the opposite side of the road. Depending on the type of failure and geometry characteristics of the slope, the stone key is typically embedded 4 to 6 feet into the ground. A typical counterberm detail (without a key) is provided in Appendix D. This detail may need to be modified to provide adequate scour protection for ice and strong stream forces for slopes adjacent to waterways. However, the key components to any remediation strategy should address, at a minimum, the following items:

- Type of stone fill for counterberm / stone key
- Type of stone fill for stone blanket
- Type of filter layer between the in-situ soil and stone blanket
- Depth of horizontal underdrain
- Specify underdrain materials (aggregate type, geotextile type, size of pipe)
- Dimensions for stone counterberm / stone key / stone blanket
- Offset and elevation of counterberm location
- Offset and elevation of underdrain location

### 8.2 Surficial Slide Slope Detail

When a slide is seen more as a surficial slide than a deep-seated slide, often times no geotechnical investigation is needed. The solution is to simply replace the failed slope materials with material that is better suited and will remain in place to prevent further sloughing or erosion. These projects are often ones that are identified by maintenance personnel and require a site visit with limited design effort. To aid in the explanation of what should be the remediation for the surficial slide, the slope typical developed for Section 8.2 included in Appendix E can be applied here. This typical section is to be used as a general guideline of what materials should be used and where and not as an exact design for every surficial slide that occurs.

## 9.0 Plan & Specification Review

The following items should be considered and addressed during the review of the plans and specifications for all slope stabilization projects:

1. Review the FHWA Checklist (http://www.fhwa.dot.gov/engineering/geotech/pubs/reviewguide/checklist.pdf) , beginning on page 35, under Preliminary Plans and Specifications

2. Review the geotechnical report to ensure compliance or agreement with page 22 of the FHWA checklist.

3. Ensure that the design guidance and geotechnical recommendations outlined in the geotechnical report have been correctly transferred to the plan set. Ideally this would happen prior to the Final Plan review stage; possibly during plan development or the Preliminary Plan stage.

4. A consultant-based design may require further examination and review depending upon whether or not the design was reviewed in-house.

5. Review the design details and project notes to ensure that specific instructions are included to successfully construct the project as detailed.

6. Review the project's specifications (Special, Supplemental and General Special Provisions) to ensure that the latest specification was used as a template and that specific project information is conveyed. Review specifications for certification, sampling and testing requirements for common construction materials. Specialized materials may need to reference an atypical test method.

7. Verify the proper location of any instrumentation. Ensure that the contract plans and specifications adequately address the specified instrumentation.

8. Ensure that any comments from a constructability review (see Section 6.9) are properly and thoroughly addressed.

## 10.0 Construction and Post Construction Monitoring & Inspection

### 10.1 Instrumentation

Inclinometers and monitoring wells, see Section 3.4.5., may also be installed post construction to monitor future movements and groundwater levels in order to assess post-construction performance and to determine the necessity of any future maintenance items.

Other instruments may include piezometers, settlement platforms, weather stations and data collectors. All instrumentation types have equipment variations that possess upgrade capabilities that support remote sensing options allowing the Agency to "call-in" and obtain data over telephone lines, via cell phones, or monitor results over web based applications.

### 10.2 Construction Monitoring & Inspection

Construction monitoring and inspection should include the following steps as a minimum:

1. Review the plans and geotechnical report to develop an understanding of the instrumentation objectives.

2. Verify that the instrumentation proposed by the Contractor complies with the project specifications.

3. Observe the installation of the instrumentation to verify that the instrumentation is installed per manufacturer's recommendations or current practice. The observer should be someone with adequate firsthand experience and equipment knowledge. Ensure that the instrumentation installation conforms to the project plans and specifications.

4. Document any physical changes to the plans and submit these changes to the Resident Engineer for inclusion into the as-built plan set.

5. Some projects may require the Contractor, or their Consultant, to be responsible for specialized instrumentation. In these special cases it may be necessary to obtain "training" from the Contractor or instrumentation supplier for any unfamiliar, new or different instrumentation.

6. Ensure that instrumentation monitoring is conducted at the frequencies and durations specified in the plans or at the direction of the Geotechnical Engineering Manager. Typically monitoring is performed on a daily (and sometimes more frequently) basis during construction.

7. Verify the operational status of the instrumentation on a weekly basis during construction.

8. Notify the Resident Engineer of any malfunctioning or damaged equipment and coordinate its replacement with the Contractor. Identify the circumstances surrounding why the equipment was damaged so that appropriate accommodations can be considered on future projects.

9. Develop a Field Instrumentation Report that details the instrumentation installation activities, including any modifications or deviations from the manufacturer's recommendations. This report should provide a location description for each piece of

equipment. The report should provide current results and data sets and convey the operational status of each piece of equipment.

10. Instrumentation data should be stored in the appropriate project folder in the Z:\drive.

### 10.3 Post Construction Monitoring & Inspection

Typically Contractor's are required to hire geotechnical engineering consultants to monitor instrumentation during construction, once construction is completed the instrumentation and the equipment used to monitor the installations are turned over to the Agency. At this juncture it is imperative to verify that the equipment being delivered to the Agency was what was required by and purchased in conformance with the contract provisions. The equipment should be calibrated (if necessary) and in good operational condition.

The post construction inspection duration and frequency will be as requested by the Geotechnical Engineering Manager. Typically the frequency is less than the construction monitoring frequency and the duration is one year or less depending upon the results. If the results are less than desirable, or anticipated, then the duration and testing frequency will likely be increased.

Results of any post construction monitoring should be provided to the Engineer within one week from the test date. The Engineer shall distribute the test results after consulting with the Geotechnical Engineering Manager. Instrumentation data should be stored in the appropriate project folder the Z:/drive.

## 11.0 References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# APPENDIX A

## Site Condition Survey Form

VERMONT AGENCY OF TRANSPORTATION

# FIELD RECONNAISSANCE REPORT

PROJECT NO.: ______________________ COUNTY ______________________ STA NO. ______________

REPORTED BY: ________________________________________ DATE ____________________

**1. STAKING OF LINE**
- ❑ WELL STAKED
- ❑ POORLY STAKED (WE CAN WORK)
- ❑ POORLY STAKED (WE MUST REPLACE)
- ❑ REQUEST DIVISION TO RESTAKE

**2. BENCH MARKS**
IN PLACE: ❑ YES ❑ NO
DISTANCE FROM BRIDGE - METERS ______________

**3. PROPERTY OWNERS**
GRANTED PERMISSION: ❑ YES ❑ NO
REMARKS ON BACK

**4. UTILITIES**
WILL DRILLERS ENCOUNTER UNDERGROUND UTILITIES?
❑ YES ❑ NO
AT WHICH HOLES? ______________
WHAT TYPE? ______________
WHO TO SEE FOR DEFINITE LOCATION ______________

**5. GEOLOGIC FORMATION**

**6. SURFACE SOILS**
❑ SAND ❑ CLAY ❑ SANDY CLAY ❑ SILT ❑ MUCK
❑ OTHER

**7. GENERAL SITE DESCRIPTION**
❑ LEVEL ❑ ROLLING ❑ HILLSIDE ❑ VALLEY
❑ SWAMP ❑ GULLIED
GROUND COVER
❑ CLEARER ❑ FARMED ❑ BUILDINGS
❑ HEAVY WOODS ❑ LIGHT WOODS
❑ OTHER
REMARKS ON BACK

**8. BRIDGE SITE**
REPLACING ______________
WIDENING ______________
RELOCATION ______________
RIG TYPE ______________

- ❑ TRUCK MOUNTED SKID RIG
- ❑ SKID RIG
- ❑ ROCK CORING RIG
- ❑ WASH BORING EQUIPMENT
- ❑ WATER WAGON
- ❑ PUMP
- ❑ HOSE - METERS ______________

**8. BRIDGE SITE - CONTINUED**
CUT SECTION - METERS ______________
FILL SECTION - METERS ______________
IF STREAM CROSSING:
WILL BARGE BE NECESSARY: ______________
EASILY PLACED IN WATER ______________
CAN CABLE BE STRETCHED ACROSS STREAM ❑ YES ❑ NO
HOW LONG? ______________
CURRENT: ❑ SWIFT ❑ MODERATE ❑ SLOW
IF PRESENT BRIDGE NEARBY:
TYPE OF FOUNDATION ______________
ANY PROBLEMS EVIDENT IN OLD BRIDGE (DESCRIBE ON BACK)
IS WATER NEARBY FOR WET DRILLING - METERS ________

**9. GROUND WATER TABLE**
CLOSE TO SURFACE - METERS ______________
NEARBY WELLS - DEPTH - METERS ______________
INTERMEDIATE DEPTH - METERS ______________

**10. ROCK**
BOULDERS OVER AREA? ❑ YES ❑ NO
DEFINITE OUTCROP? ❑ YES ❑ NO
(SHOW SKETCH ON BACK) WHAT KIND? ______________

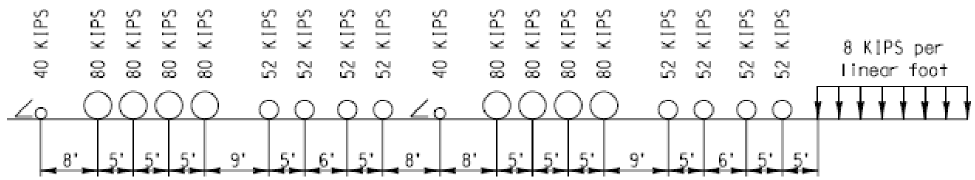
**11. SPECIAL EQUIPMENT NECESSARY**

**12. REMARKS ON ACCESS**
DESCRIBE ANY PROBLEMS ON ACCESS

**13. DEBRIS AND SANITARY DUMPS**
STATIONS ______________
REMARKS ______________

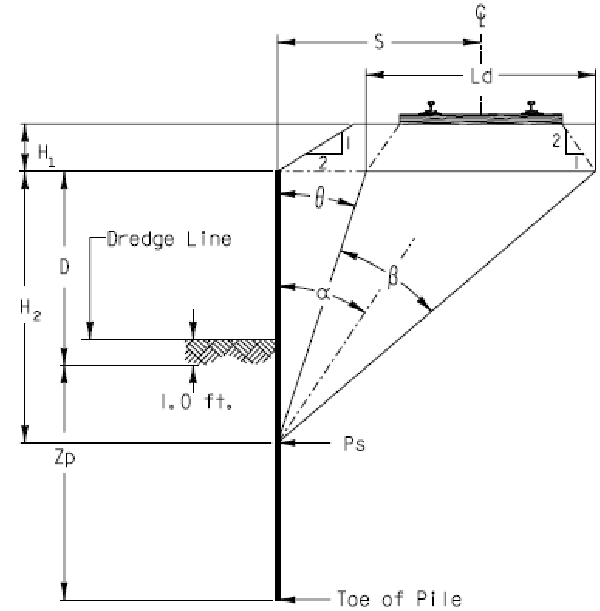
# APPENDIX B

## Cooper E-80 Loading




## COOPER E80 LOAD

SCALE: (NOT TO SCALE)




## PLAN

SCALE: (NOT TO SCALE)

Vertical pressure q shall be based on a distribution width Ld.
Ld is the length of tie plus $H_1$.
$H_1$ is the height from the bottom of tie to the top of shoring.
$H_2$ is the depth of point being evaluated with the Boussinesq equation.
S is a distance perpendicular from centerline of track to the face of shoring.
D is from top of shoring to one foot below dredge line.
Zp is the minimum embedment depth.
Length of tie is 9 feet
q is the intensity of strip load due to E80 Railroad live load and shall be calculated as follows:

For $H_1 = 0$ Ld = length of tie; therefore, $q = \frac{80,000 \text{ lb}}{(5 \text{ feet})(9 \text{ feet})} = 1,778$ psf

For $H_1 > 0$ Ld = length of tie + $H_1$; therefore, $q = \frac{80,000 \text{ lb}}{(5 \text{ feet})(Ld)}$

CASE 1: Lateral live load pressure Ps, due to E80 loading for track **parallel** to shoring system is calculated using the Boussinesq Strip Load Equation.

$$Ps = \frac{2q}{\pi}(\beta + \sin\beta \sin^2\alpha - \sin\beta \cos^2\alpha)$$

The above equation can be simplified into the following equivalent form:

$$Ps = \frac{2q}{\pi}[\beta - \sin\beta \cos(2\alpha)]$$

$\alpha$ and $\beta$ are angles measured in radians, $\alpha = \theta + \frac{\beta}{2}$

CASE 2: Live load pressure due to E80 loading for track at a **right angle** to the shoring system can be calcuated using the following equation.

$Ps = K_A q$

where $K_A = \tan^2(45 - \frac{\phi}{2})$

$\phi$ is the angle of internal friction in degrees

| REVISIONS | | |
|---|---|---|
| DATE | LTR. | DESCRIPTION |
| / | | |
| / | | |
| / | | |
| / | | |
| / | | |
| / | | |

DESIGN BY: PGP | DRAWN BY: JFS | CHECKED BY: AA

APPROVED:

K.H. Jennison

BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN

George J Meyer 9-1-04

UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN

**BRIDGE SHORING STANDARDS**

**LIVE LOAD PRESSURE DUE TO COOPER E80**

FILE OWNER: UPRR | DATE:

PLAN NO.: 710001 | SHEET: 1 OF 1

PLOTTED: $$$$DATE$$$$ $TIME

# APPENDIX C

## VTrans Geotechnical Report Example

**To:** Doug Newton, District 6 Project Manager

CEE CAA

**From:** Callie E. Ewald, Geotechnical Engineer via Chad A. Allen for Christopher C. Benda, P.E., Soils and Foundations Engineer

**Date:** June 30, 2009

**Subject:** Hyde Park VT Rt. 15 Slide Remediation

## 1.0 INTRODUCTION

In the spring of 2008 a slide occurred on the north side of Route 15 in Hyde Park, Vermont, approximately 0.4 miles east of the intersection of VT 100 and VT 15. The scarp at the top of the slope is located in the back yard of local resident Bill Hale. Three borings were performed to gather pertinent subsurface information to aid in developing a remediation for the slope. Contained herein are the results of field sampling and testing, laboratory analyses of soil samples, and subsequent slope stability analyses.

## 2.0 FIELD INVESTIGATION

The field investigation was conducted from June 9, 2009 through June 11, 2009. The boring location site plan and the CADD generated boring logs are attached to this report. The site plan illustrates the locations of the borings as taken in the field and as listed in Table 2.1.

| Boring | Station | Offset (ft) | Ground Elevation (ft) |
|---|---|---|---|
| B - 101 | 0 + 96.0 | -110.6 | 695.3 |
| B - 102 | 1 + 54.7 | -114.2 | 695.3 |
| B - 103 | 1 + 26.0 | - 29.5 | 657.2 |

*Table 2.1: Boring Locations*

During the boring operations, split spoon samples and standard penetration tests (SPT) were taken on all 3 borings. SPT testing was performed continuously to 25 feet and then at 5 foot intervals to specified depths for B-101 and B-102, and continuously to 10 feet followed by 5 foot intervals to presumed ledge in B-103. Soil samples were visually classified in the field and SPT blow counts were recorded on the boring logs. Soil samples were preserved and returned to the Materials and Research Laboratory for testing and further evaluation. Upon completion of the laboratory testing, the field boring logs were revised to reflect the results of the laboratory classification results. The attached boring logs display the types of soils and strata encountered and include the laboratory test results, SPT data and any pertinent observations made by the boring crew.

## 3.0 FIELD AND LABORATORY TESTING

The standard penetration resistance of the in-situ soil is determined by the number of blows required to drive a 2 inch OD split barrel sampler into the soil with a 140 pound hammer dropped from a height of 30 inches, in accordance with procedures specified in AASHTO T206. During the standard penetration test (SPT), the sampler is driven for a total length of 2 feet, while counting the blows for each 6 inch increment. The SPT N-value, which is defined as the sum of the number of blows required to drive the sampler through the second and third increments, is commonly used with established correlations to estimate a number of soil parameters, particularly the shear strength and density of cohesionless soils.

## 4.0 SOIL PROFILE

Review of laboratory data and boring logs revealed the following information pertaining to the soil strata:

A groundwater reading was observed at a depth of thirty six feet below the ground surface in B-101. For modeling purposes, it was assumed that the water table was higher at the time of the failure last spring, approximately 2 to 5 feet below ground surface.

The top twenty five feet of soil on the slope is very loose to loose silt underlain by a fifteen foot thick layer of very loose silt and soft clay. Below the silt and clay layer is a denser layer of gravelly silty sand.

It is our understanding that a 2 foot layer of gravel was laid on the entire slope following original construction to aid in slope stability. This 2 foot thick layer was assumed as a sandy gravel for modeling purposes. A cross section of the soil profile for this site can be viewed in Figure 5.1.

## 5.0 RESULTS

### Stability Analysis

A computer model was developed using the software program Slide, version 5.035, developed by Rocscience. The program considers rotational and translational failure mechanisms. A cross-section of the slope was chosen based on the available subsurface information and by identifying the portion of the slide area with the highest potential for failure. The section analyzed was at Station 1+25. Soil properties were back calculated based on a factor of safety of 1.0, using the cross sections, the boring information, and a very conservative water table elevation. A table of these soil properties established is found below in Table 5.1. This information provides a model of existing conditions to evaluate the location of the critical slip surface.

| ID | Soil Description | Saturated Unit Weight (pcf) | Friction Angle (degrees) | Cohesion (psf) [1] |
|---|---|---|---|---|
| 1 | Very Loose Silt | 100 | 29 | --- |
| 2 | Loose Silt | 105 | 31 | --- |
| 3 | Soft Silty Clay | 100 | 26 | 270 |
| 4 | Very Loose Clayey Silt | 105 | 27 | 190 |
| 5 | Loose Sandy Silt | 115 | 30 | --- |
| 6 | Dense Gravelly Silty Sand* | 120 | 32 | --- |
| 7 | Dense Sandy Silt | 120 | 32 | --- |

*Table 5.1: Material properties from Slide model*

* Assumed 2 ft thick layer of material on slope face

There are a few physical characteristics to note that were discovered during the site visit. At the very top of the slope exists an 8 foot tall hedge spanning the top of the slide area. The hedge dropped 5 feet and is currently located just inside the scarp. Large trees also existed on the slope when the failure first happened in the spring of 2008. Since then, the trees have been removed to decrease the mass and moment applied to the upper slope, thereby lowering the driving forces in the slope. Since the removal of the trees and the settling of the hedge, little to no movement has occurred. Figure 5.1 illustrates the existing conditions at failure based upon the assumed type of failure. The red arrows toward the top of the slope are modeling the loads applied for the hedge and several large trees.

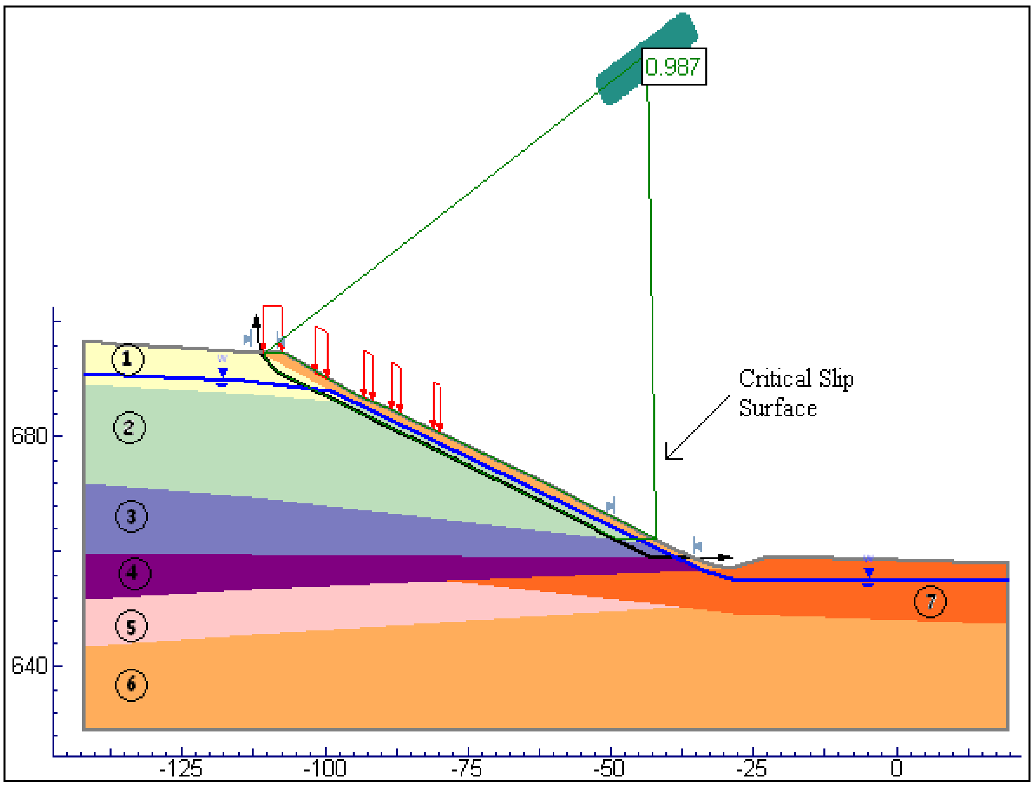



*Figure 5.1 Slide model of initial conditions*

1. Lindeburg, *Civil Engineering Reference Manual for the PE Exam*, 8th ed.

Visual inspection of the site as well as the factors previously discussed lead to the possibility of the slope failing in a translational manner, rather than a more common rotational failure. A translational failure occurs when a mass of soil is sliding down the hill, shown in Figure 5.1 as the green critical slip surface. After modeling the slope with the assumed existing grade prior to failure and additional vegetative loads, it was confirmed as a translational failure. The slip surface exits the slope in the same location as was visually observed in the field, as a layer of soil sliding over another.

AASHTO (American Association of State Highway Transportation Officials) recommends that a minimum factor of safety of 1.3 be achieved for non-critical structures. In order to achieve this factor of safety, a stone key remediation was chosen to stabilize the slope. Figure 5.2 illustrates the slope with a stone key and stone fill covering the face of the slope, yielding a factor of safety greater than 1.3.

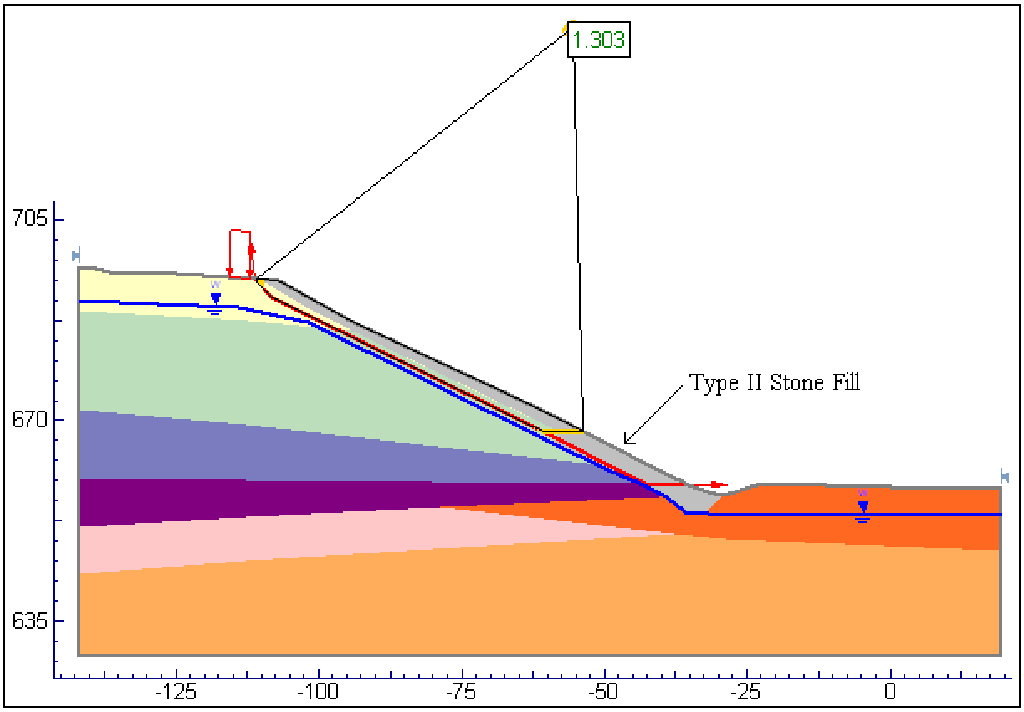



*Figure 5.2 Location of the stone key and stone fill along the slope face*

## 6.0 CONCLUSION

In order to achieve the AASHTO recommended minimum factor of safety for this slope we recommend that a stone key be placed at the toe of the slope as well as a stone blanket on the face of the slope. The stone berm is designed to key into the denser material to help stabilize the face, as well as increase the overall stability of the slope. The stone fill should meet the specifications of *Type II Stone Fill*, as listed in VTrans' 2006 Standard Specifications for Construction. It should be constructed based on the typical section detail attached. We recommend constructing this remediation along the total length of the visual scarp, approximately station 0+60 to 2+20; however this length may be adjusted in the field based on visual inspection of the unstable area.

To prevent water from collecting at the toe of the slope, the bottom of the stone key should taper to the existing ditch grade at a location away from the remediated area. Since the grade of the slope is slightly downhill to the east on VT 15, we recommend gradually daylighting the stone key to the existing ditch elevation at a point east of Station 2+20. This will stop the water from pooling or seeping into the subbase of the pavement, carrying it away from the slope.

We recommend a geotextile, VTrans Pay Item 649.31, meeting the requirements outlined in VTrans spec 720.04A, to be placed under the stone fill, against the native soil. This will help prevent the very silty soils from intermixing with the stone fill.

If the hedge is going to remain at the top of the slope, we also recommend it should be replanted at least 5 feet away from the top of the slope, back toward the property owner's house. This added weight of the hedge was a likely contributor to the failure in the first place; therefore it is recommended it be either removed or relocated to guarantee future stability.

The typical section attached is based on a cross section cut from the middle of the slope, Station 1+25. We understand the entire length of slide differs in elevations and slope, yet using this section as a guideline should ensure appropriate construction for a stable slope.

If any further analysis is needed or you would like to discuss this report, please contact us at (802) 828-2561.

Computer generated boring logs are attached and can be accessed at M:\Projects\009x508\MaterialsResearch CADD design folder.

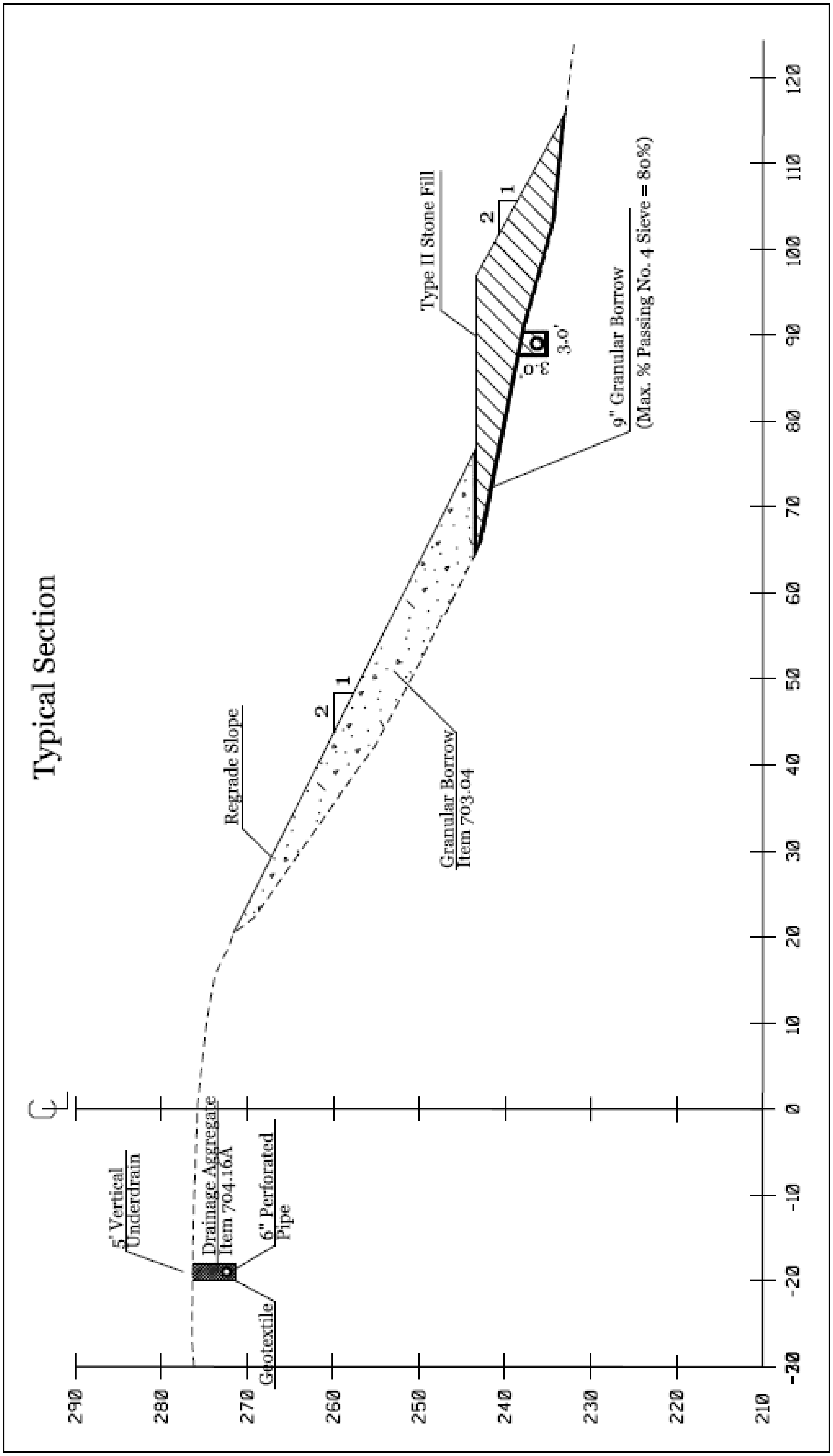
Enclosures: Plan View of Slide Area/Boring Layout
Boring and core logs – 3 pages
Typical Section Detail

c: Electronic Read File/WEA
Project File/CCB
CEE

G:\Soils and Foundations\Projects\Hyde Park Vt. 15 Slide\Reports\HydePark VT15 Slope Report_Revised.doc

# APPENDIX D

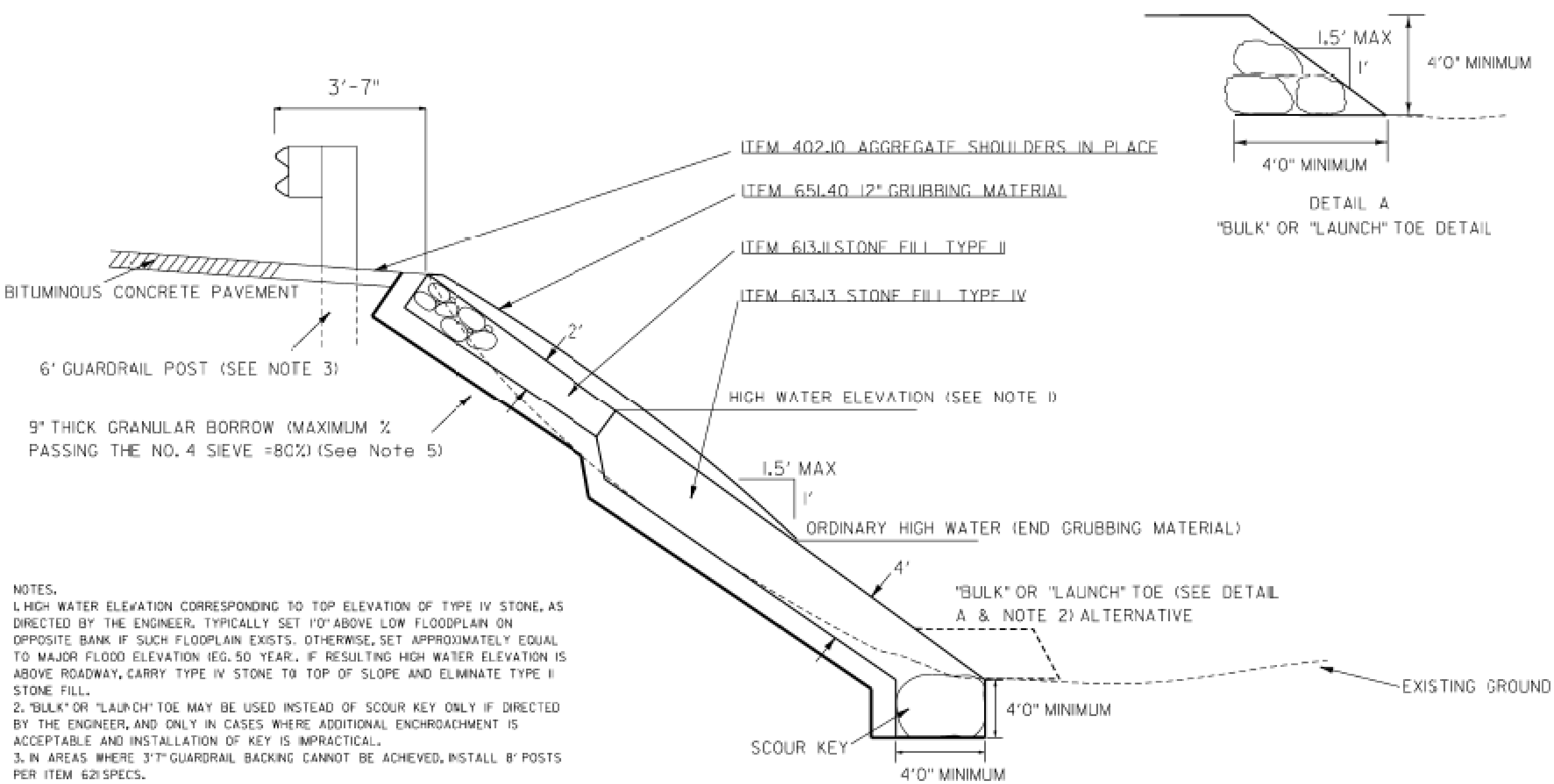
## Counterberm Typical Section


Typical Section
Regrade Slope
Granular Borrow
Item 703.04
Type II Stone Fill
9" Granular Borrow
(Max. % Passing No. 4 Sieve = 80%)
3.0'
3.0'
2
1
2
1
5' Vertical
Underdrain
Drainage Aggregate
Item 704.16A
6" Perforated
Pipe
Geotextile
290
280
270
260
250
240
230
220
210
-30
-20
-10
0
10
20
30
40
50
60
70
80
90
100
110
120

# APPENDIX E

## Details for Surficial Slope Stabilization

# STONE FILL SLOPE PROTECTION

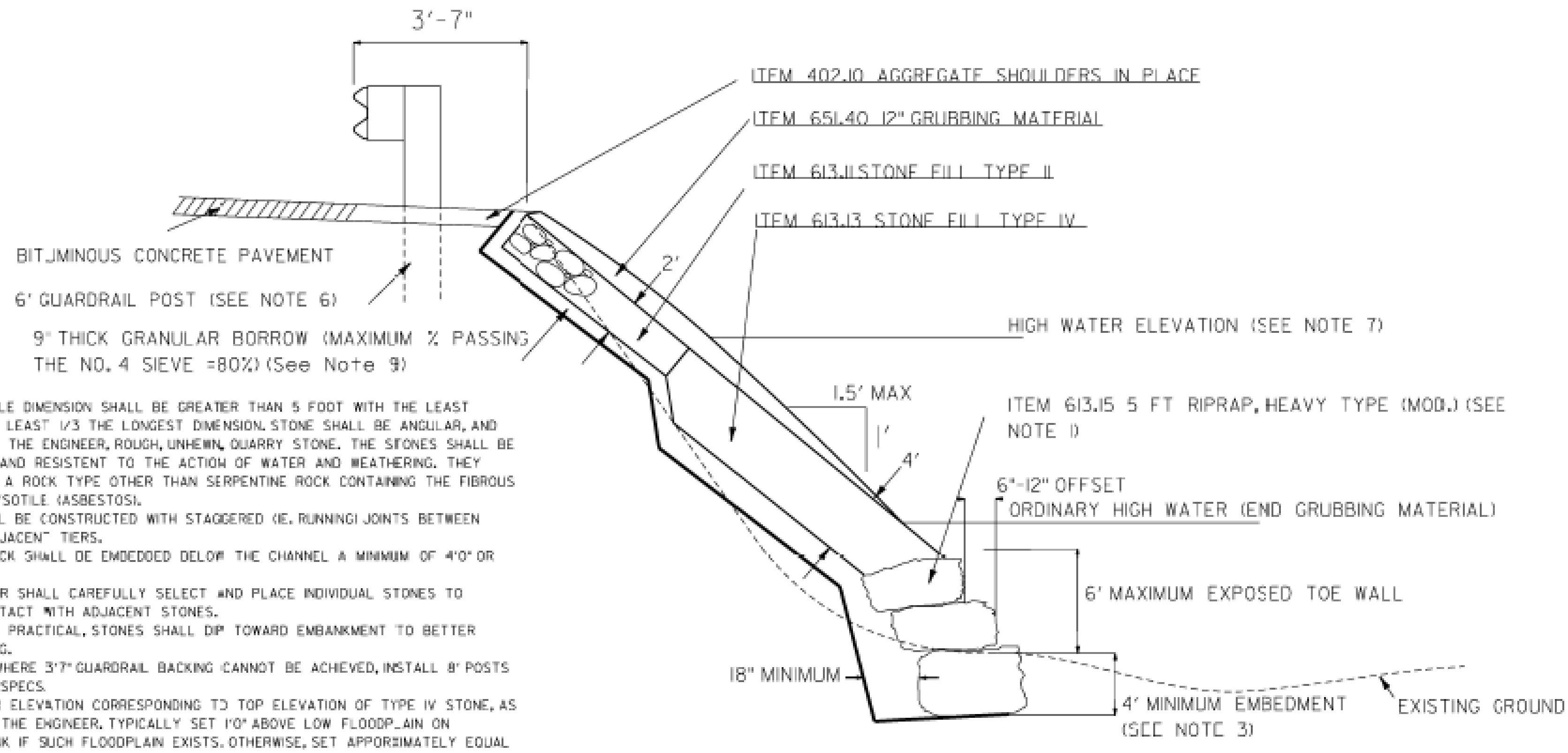



NOTES.

1. HIGH WATER ELEVATION CORRESPONDING TO TOP ELEVATION OF TYPE IV STONE, AS DIRECTED BY THE ENGINEER. TYPICALLY SET 1'0" ABOVE LOW FLOODPLAIN ON OPPOSITE BANK IF SUCH FLOODPLAIN EXISTS. OTHERWISE, SET APPROXIMATELY EQUAL TO MAJOR FLOOD ELEVATION (EG. 50 YEAR). IF RESULTING HIGH WATER ELEVATION IS ABOVE ROADWAY, CARRY TYPE IV STONE TO TOP OF SLOPE AND ELIMINATE TYPE II STONE FILL.
2. "BULK" OR "LAUNCH" TOE MAY BE USED INSTEAD OF SCOUR KEY ONLY IF DIRECTED BY THE ENGINEER, AND ONLY IN CASES WHERE ADDITIONAL ENCROACHMENT IS ACCEPTABLE AND INSTALLATION OF KEY IS IMPRACTICAL.
3. IN AREAS WHERE 3'7" GUARDRAIL BACKING CANNOT BE ACHIEVED, INSTALL 8' POSTS PER ITEM 621 SPECS.
4. THE STONE SLOPE SHALL BE NO STEEPER THAN 1.5H TO 1.0V UNLESS DIRECTED BY THE ENGINEER.
5. GEOTEXTILE ITEM 649.31 CAN BE USED INSTEAD OF GRANULAR BORROW AT THE TOP 5 FT OF SLOPE. GEOTEXTILE MAY BE SUBSTITUTED FOR GRANULAR BORROW FOR THE TOTAL LENGTH OF SLOPE WHEN THE MAXIMUM HEIGHT OF SLOPE IS LESS THAN 15 FEET.

| | |
|---|---|
| PROJECT NAME: RIPRAP SLOPE PROTECTION | |
| PROJECT NUMBER: | |
| FILE NAME: TYPICALSECTIONS.DGN | PLOT DATE: ####DATE#### |
| PROJECT LEADER: CHRIS BUMP | DRAWN BY: ANDREW BERNIER |
| DESIGNED BY: MATT MURAWSKI | CHECKED BY: ------ |
| ------ ------ | SHEET 2 OF 3 |

# STONE FILL SLOPE PROTECTION WITH STONE TOE WALL




NOTES.

1. THE PRINCIPLE DIMENSION SHALL BE GREATER THAN 5 FOOT WITH THE LEAST DIMENSION AT LEAST 1/3 THE LONGEST DIMENSION. STONE SHALL BE ANGULAR, AND APPROVED BY THE ENGINEER, ROUGH, UNHEWN, QUARRY STONE. THE STONES SHALL BE HARD, SOUND AND RESISTENT TO THE ACTION OF WATER AND WEATHERING. THEY SHALL BE OF A ROCK TYPE OTHER THAN SERPENTINE ROCK CONTAINING THE FIBROUS VARIETY CHRYSOTILE (ASBESTOS).
2. WALL SHALL BE CONSTRUCTED WITH STAGGERED (IE. RUNNING) JOINTS BETWEEN ROCKS ON ADJACENT TIERS.
3. FOOTER ROCK SHALL BE EMBEDDED BELOW THE CHANNEL A MINIMUM OF 4'0" OR ON BEDROCK.
4. CONTRACTOR SHALL CAREFULLY SELECT AND PLACE INDIVIDUAL STONES TO MAXIMIZE CONTACT WITH ADJACENT STONES.
5. TO EXTENT PRACTICAL, STONES SHALL DIP TOWARD EMBANKMENT TO BETTER RESIST SLIDING.
6. IN AREAS WHERE 3'7" GUARDRAIL BACKING CANNOT BE ACHIEVED, INSTALL 8' POSTS PER ITEM 621 SPECS.
7. HIGH WATER ELEVATION CORRESPONDING TO TOP ELEVATION OF TYPE IV STONE, AS DIRECTED BY THE ENGINEER. TYPICALLY SET 1'0" ABOVE LOW FLOODPLAIN ON OPPOSITE BANK IF SUCH FLOODPLAIN EXISTS. OTHERWISE, SET APPROXIMATELY EQUAL TO MAJOR FLOOD ELEVATION (EG. 50 YEAR). IF RESULTING HIGH WATER ELEVATION IS ABOVE ROADWAY, CARRY TYPE IV STONE TO TOP OF SLOPE AND ELIMINATE TYPE II STONE FILL.
8. THE STONE SLOPE SHALL BE NO STEEPER THAN 1.5H TO 1.0V UNLESS DIRECTED BY THE ENGINEER.
9. GEOTEXTILE ITEM 649.31 CAN BE USED INSTEAD OF GRANULAR BORROW AT THE TOP 5 FEET OF SLOPE. GEOTEXTILE MAY BE USED FOR THE TOTAL LENGTH OF SLOPE WHEN THE MAXIMUM HEIGHT OF THE SLOPE IS LESS THAN 15 FEET.

| | |
|---|---|
| PROJECT NAME: RIPRAP SLOPE PROTECTION WITH STONE TOE WALL | |
| PROJECT NUMBER: | |
| FILE NAME: TYPICAL SECTIONS.DGN | PLOT DATE: ****DATE**** |
| PROJECT LEADER: CHRIS GUMP | DRAWN BY: ANDREW BERNIER |
| DESIGNED BY: MATT MURAWSKI | CHECKED BY: ------ |
| ------ ------ | SHEET 3 OF 3 |

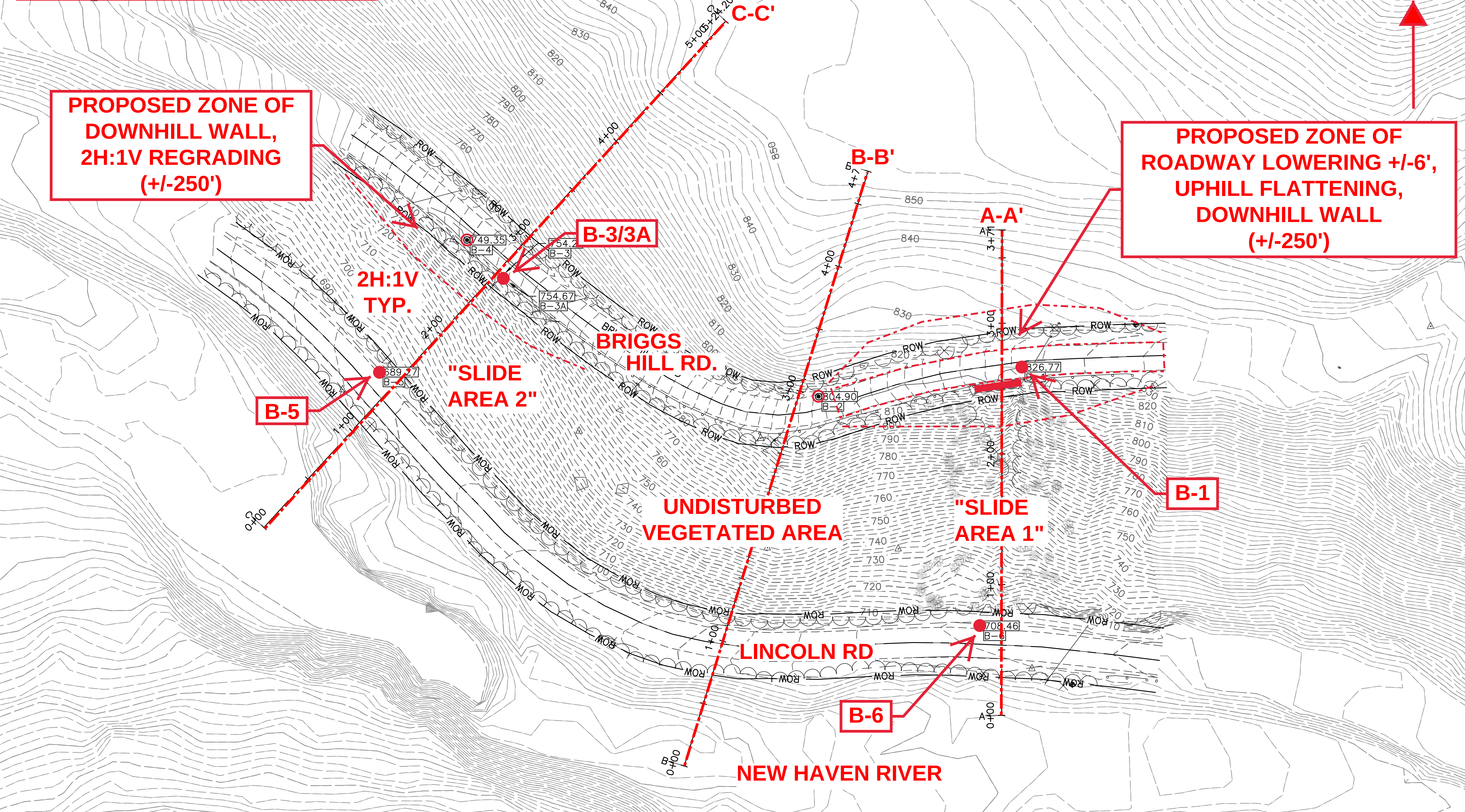
FEMA SUMMARY

734189-DR4720VT

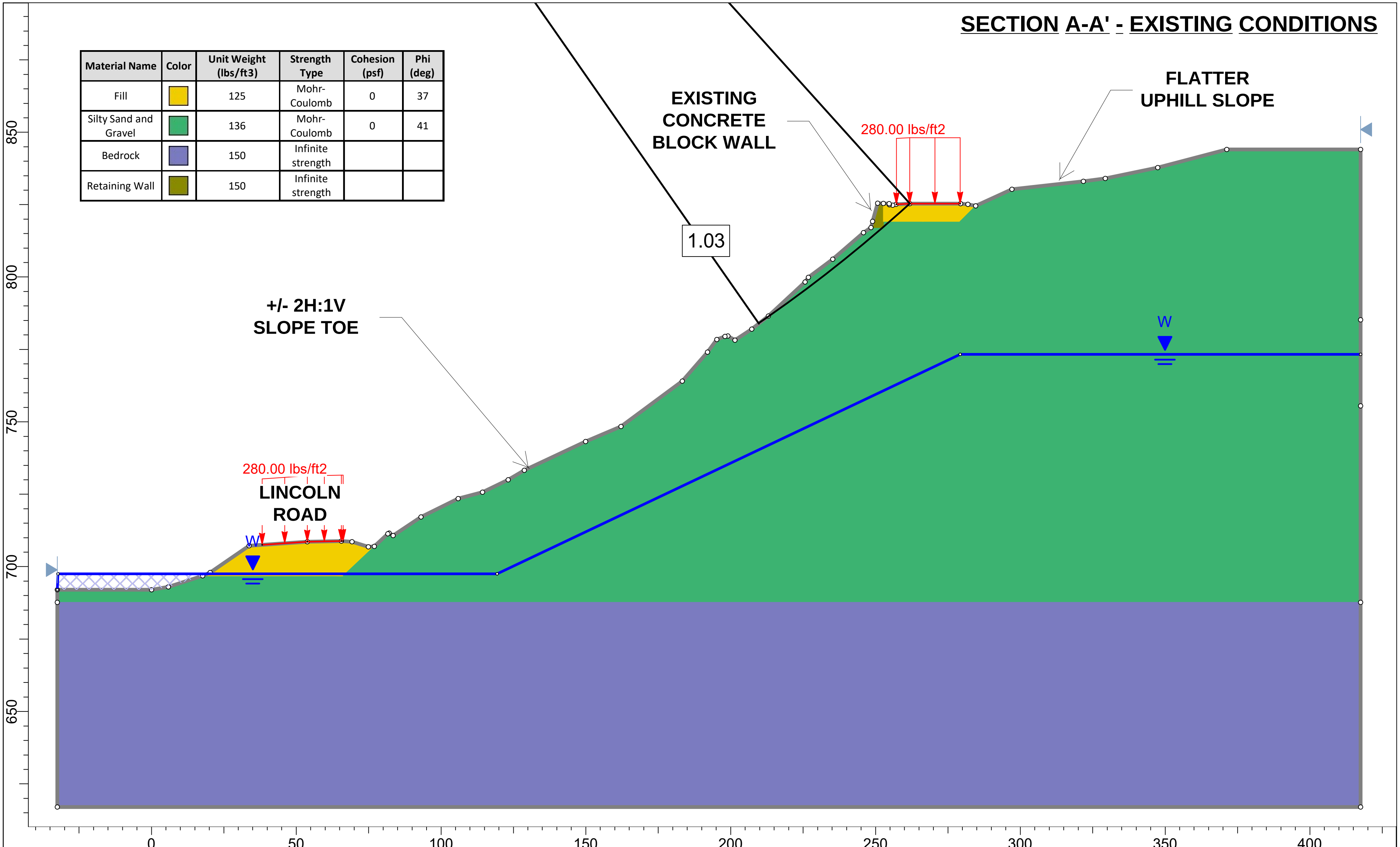
Bristol – Briggs Hill Road

# Slope Stability Modeling

FIGURE 1 - OVERALL SITE PLAN




NOTE: TOPOGRAPHIC BASE PLAN PROVIDED BY D&K.

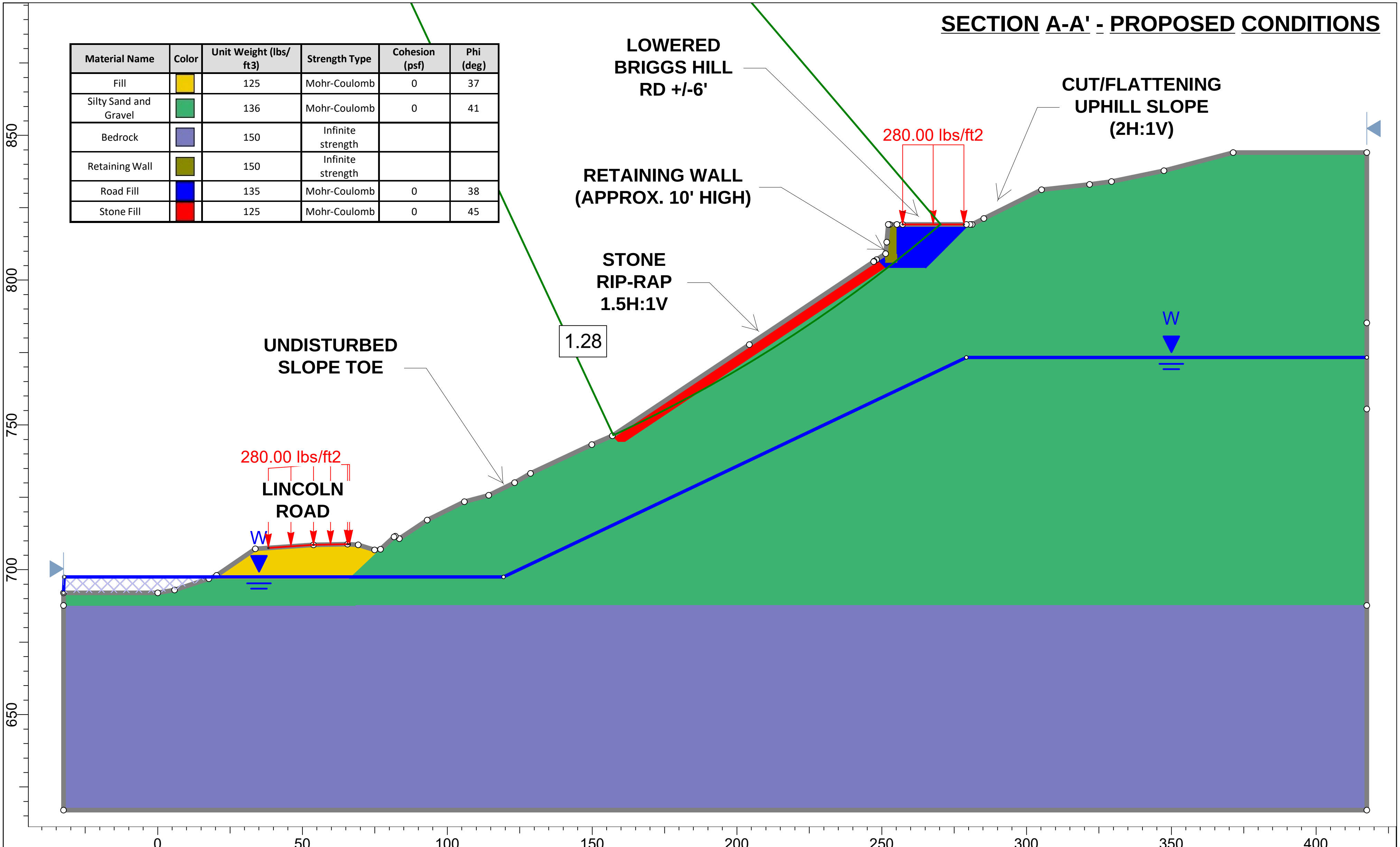



| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) |
|---|---|---|---|---|---|
| Fill | | 125 | Mohr-Coulomb | 0 | 37 |
| Silty Sand and Gravel | | 136 | Mohr-Coulomb | 0 | 41 |
| Bedrock | | 150 | Infinite strength | | |
| Retaining Wall | | 150 | Infinite strength | | |

rocscience

SLIDEINTERPRET 9.023

| | | | |
|---|---|---|---|
| Project | BRIGGS HILL SLOPE INSTABILITY - BRISTOL, VT<br>GEODESIGN FILE NO. 804-56 | | |
| Group | Group 1 | Scenario | Master Scenario |
| Drawn By | | Company | |
| Date | 1/17/2024, 4:02:35 PM | File Name | 240117 SECTION A-A'.slmd |




| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) |
|---|---|---|---|---|---|
| Fill | | 125 | Mohr-Coulomb | 0 | 37 |
| Silty Sand and Gravel | | 136 | Mohr-Coulomb | 0 | 41 |
| Bedrock | | 150 | Infinite strength | | |
| Retaining Wall | | 150 | Infinite strength | | |
| Road Fill | | 135 | Mohr-Coulomb | 0 | 38 |
| Stone Fill | | 125 | Mohr-Coulomb | 0 | 45 |

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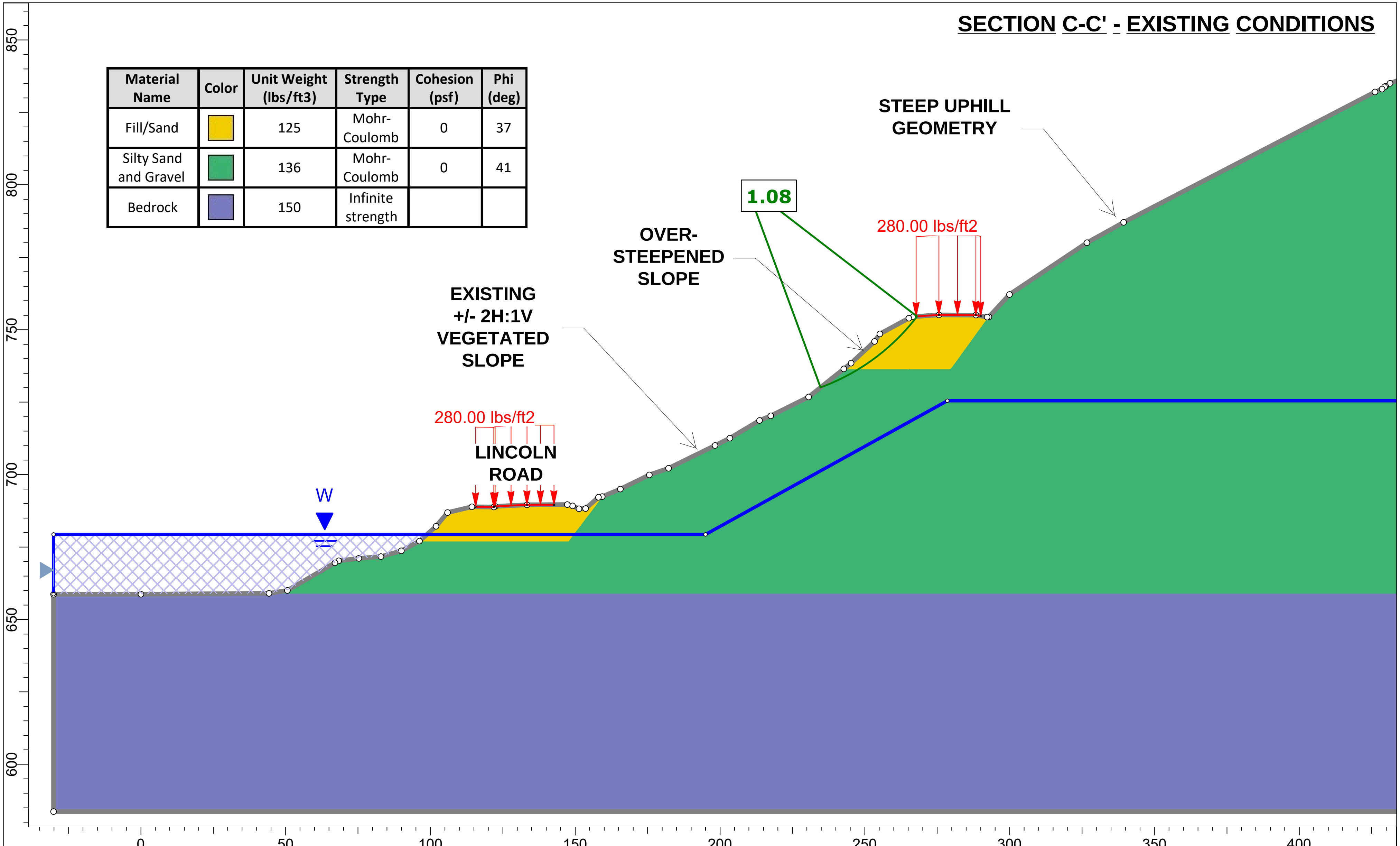
SLIDEINTERPRET 9.023

| | |
|---|---|
| *Project* | BRIGGS HILL SLOPE INSTABILITY - BRISTOL, VT<br>GEODESIGN FILE NO. 804-56 |
| *Group* Group 1 | *Scenario* Master Scenario |
| *Drawn By* | *Company* |
| *Date* 1/17/2024, 4:02:35 PM | *File Name* 240202 SECTION A-A' LOWERED ROAD WITH CREST WALL.slmd |




| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) |
|---|---|---|---|---|---|
| Fill/Sand | | 125 | Mohr-Coulomb | 0 | 37 |
| Silty Sand and Gravel | | 136 | Mohr-Coulomb | 0 | 41 |
| Bedrock | | 150 | Infinite strength | | |

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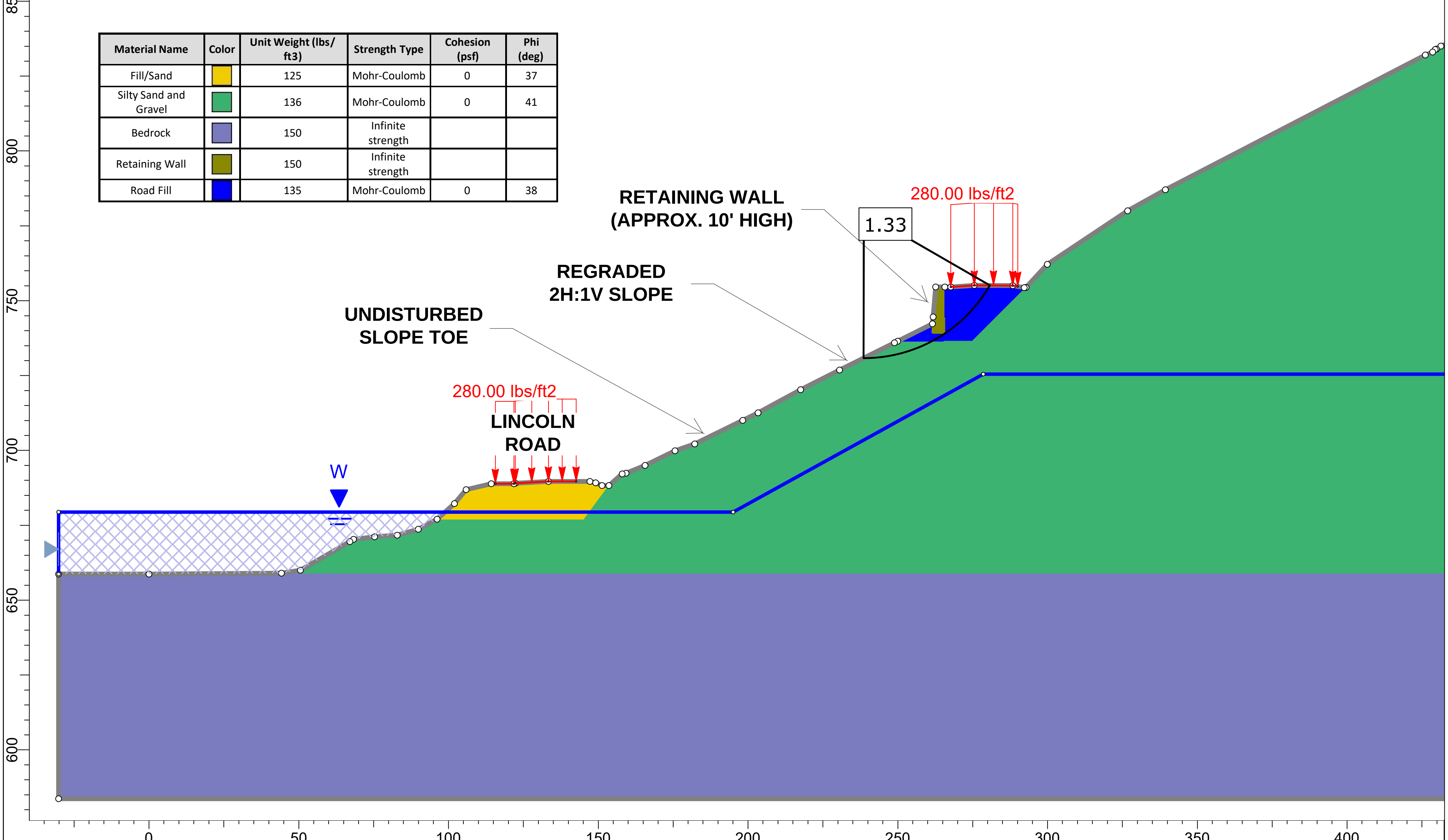
SLIDEINTERPRET 9.023

| | | | |
|---|---|---|---|
| *Project* | BRIGGS HILL SLOPE INSTABILITY - BRISTOL, VT<br>GEODESIGN FILE NO. 804-56 | | |
| *Group* | Group 1 | *Scenario* | Master Scenario |
| *Drawn By* | | *Company* | |
| *Date* | 1/17/2024, 4:02:35 PM | *File Name* | 240117 SECTION C-C'.slmd |

# SECTION C-C' - PROPOSED CONDITIONS




| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) |
|---|---|---|---|---|---|
| Fill/Sand | | 125 | Mohr-Coulomb | 0 | 37 |
| Silty Sand and Gravel | | 136 | Mohr-Coulomb | 0 | 41 |
| Bedrock | | 150 | Infinite strength | | |
| Retaining Wall | | 150 | Infinite strength | | |
| Road Fill | | 135 | Mohr-Coulomb | 0 | 38 |

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SLIDEINTERPRET 9.023

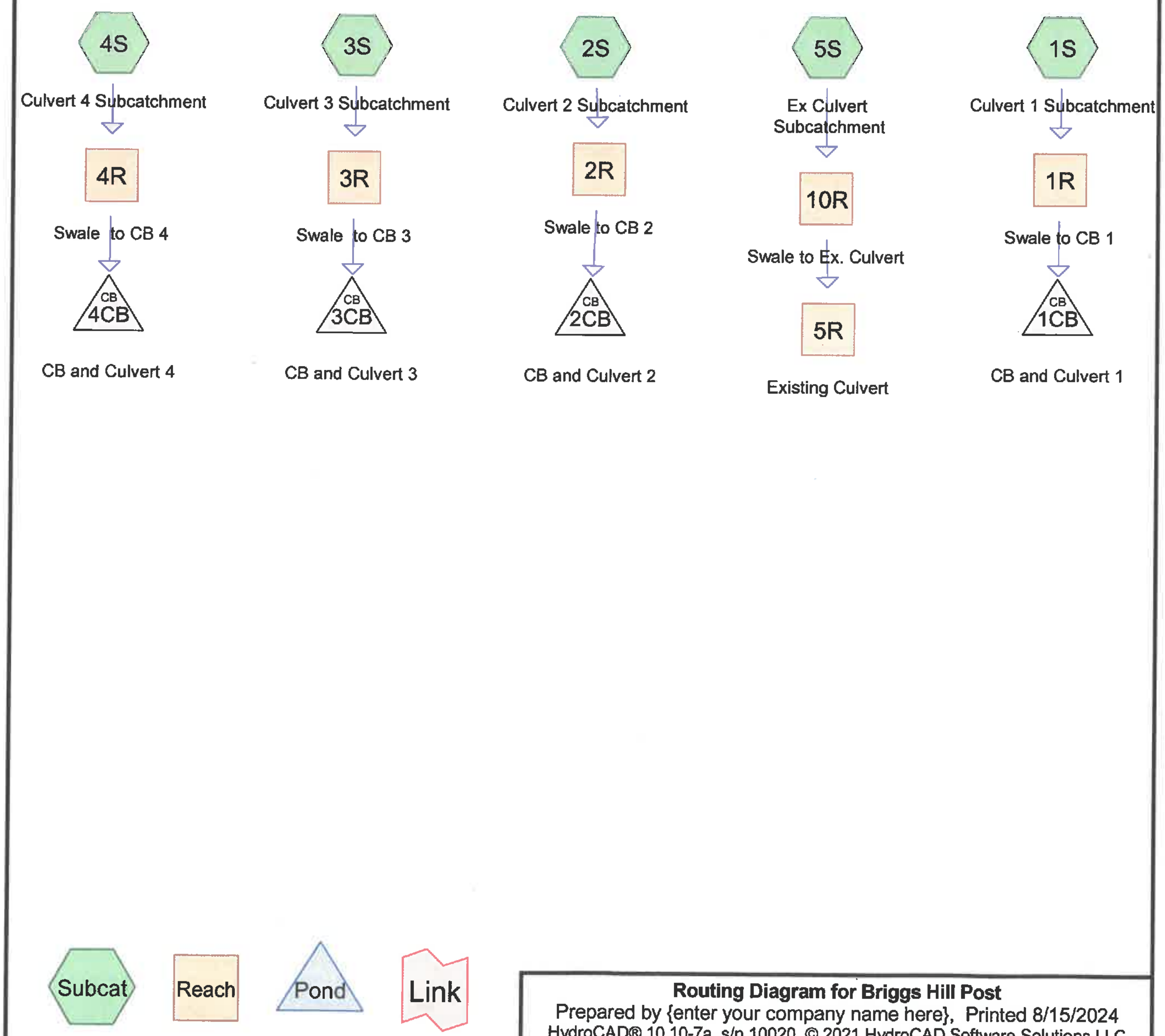
| | |
|---|---|
| *Project* | BRIGGS HILL SLOPE INSTABILITY - BRISTOL, VT<br>GEODESIGN FILE NO. 804-56 |
| *Group* Group 1 | *Scenario* Master Scenario |
| *Drawn By* | *Company* |
| *Date* 1/17/2024, 4:02:35 PM | *File Name* 240202 SECTION C-C' CREST WALL AND 2_1 SLOPE.slmd |

FEMA SUMMARY

734189-DR4720VT

Bristol – Briggs Hill Road

# HydroCAD Modeling for Culverts Sizing




**Routing Diagram for Briggs Hill Post**
Prepared by {enter your company name here}, Printed 8/15/2024
HydroCAD® 10.10-7a s/n 10020 © 2021 HydroCAD Software Solutions LLC

## Project Notes

Rainfall events imported from "Gunstock Parking Lot - Pre.hcp"
Rainfall events imported from "Gunstock Parking Lot - Predevelopment.hcp"
Rainfall events imported from "NRCS-Rain.txt" for 9007 VT Addison

## Rainfall Events Listing (selected events)

| Event# | Event Name | Storm Type | Curve | Mode | Duration (hours) | B/B | Depth (inches) | AMC |
|---|---|---|---|---|---|---|---|---|
| 1 | 50-Year | NRCC 24-hr | A | Default | 24.00 | 1 | 4.64 | 2 |

## Area Listing (all nodes)

| Area (acres) | CN | Description (subcatchment-numbers) |
|---|---|---|
| 0.422 | 84 | 50-75% Grass cover, Fair, HSG D (2S, 3S, 4S, 5S) |
| 0.216 | 98 | Paved parking, HSG A (1S, 2S, 3S, 4S, 5S) |
| 3.676 | 79 | Woods, Fair, HSG D (1S, 2S, 3S, 4S, 5S) |
| **4.314** | **80** | **TOTAL AREA** |

## Soil Listing (all nodes)

| Area (acres) | Soil Group | Subcatchment Numbers |
|---|---|---|
| 0.216 | HSG A | 1S, 2S, 3S, 4S, 5S |
| 0.000 | HSG B | |
| 0.000 | HSG C | |
| 4.098 | HSG D | 1S, 2S, 3S, 4S, 5S |
| 0.000 | Other | |
| **4.314** | | **TOTAL AREA** |

## Ground Covers (all nodes)

| HSG-A (acres) | HSG-B (acres) | HSG-C (acres) | HSG-D (acres) | Other (acres) | Total (acres) | Ground Cover | Subcatchment Numbers |
|---|---|---|---|---|---|---|---|
| 0.000 | 0.000 | 0.000 | 0.422 | 0.000 | 0.422 | 50-75% Grass cover, Fair | 2S, 3S, 4S, 5S |
| 0.216 | 0.000 | 0.000 | 0.000 | 0.000 | 0.216 | Paved parking | 1S, 2S, 3S, 4S, 5S |
| 0.000 | 0.000 | 0.000 | 3.676 | 0.000 | 3.676 | Woods, Fair | 1S, 2S, 3S, 4S, 5S |
| **0.216** | **0.000** | **0.000** | **4.098** | **0.000** | **4.314** | **TOTAL AREA** | |

## Pipe Listing (all nodes)

| Line# | Node Number | In-Invert (feet) | Out-Invert (feet) | Length (feet) | Slope (ft/ft) | n | Width (inches) | Diam/Height (inches) | Inside-Fill (inches) |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 5R | 825.03 | 820.50 | 46.4 | 0.0976 | 0.013 | 0.0 | 18.0 | 0.0 |
| 2 | 1CB | 820.34 | 819.32 | 51.0 | 0.0200 | 0.013 | 0.0 | 18.0 | 0.0 |
| 3 | 2CB | 801.11 | 800.64 | 47.0 | 0.0100 | 0.013 | 0.0 | 18.0 | 0.0 |
| 4 | 3CB | 752.40 | 750.72 | 42.0 | 0.0400 | 0.013 | 0.0 | 18.0 | 0.0 |
| 5 | 4CB | 732.85 | 731.02 | 42.0 | 0.0436 | 0.013 | 0.0 | 18.0 | 0.0 |

Time span=0.00-42.00 hrs, dt=0.01 hrs, 4201 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

**Subcatchment 1S: Culvert 1 Subcatchment** Runoff Area=31,145 sf 11.03% Impervious Runoff Depth=2.67"
Flow Length=391' Slope=0.2637 '/' Tc=2.8 min CN=81 Runoff=3.34 cfs 0.159 af

**Subcatchment 2S: Culvert 2 Subcatchment** Runoff Area=50,183 sf 2.96% Impervious Runoff Depth=2.58"
Flow Length=673' Slope=0.1896 '/' Tc=5.3 min CN=80 Runoff=4.79 cfs 0.248 af

**Subcatchment 3S: Culvert 3 Subcatchment** Runoff Area=33,815 sf 1.97% Impervious Runoff Depth=2.58"
Flow Length=296' Slope=0.2412 '/' Tc=2.4 min CN=80 Runoff=3.54 cfs 0.167 af

**Subcatchment 4S: Culvert 4 Subcatchment** Runoff Area=30,766 sf 3.89% Impervious Runoff Depth=2.58"
Flow Length=369' Slope=0.3034 '/' Tc=2.6 min CN=80 Runoff=3.22 cfs 0.152 af

**Subcatchment 5S: Ex Culvert Subcatchment** Runoff Area=41,998 sf 6.24% Impervious Runoff Depth=2.67"
Flow Length=544' Slope=0.1944 '/' Tc=4.3 min CN=81 Runoff=4.30 cfs 0.214 af

**Reach 1R: Swale to CB 1** Avg. Flow Depth=0.45' Max Vel=4.39 fps Inflow=3.34 cfs 0.159 af
n=0.041 L=201.3' S=0.0777 '/' Capacity=79.43 cfs Outflow=3.29 cfs 0.159 af

**Reach 2R: Swale to CB 2** Avg. Flow Depth=0.52' Max Vel=5.17 fps Inflow=4.79 cfs 0.248 af
n=0.041 L=121.5' S=0.0926 '/' Capacity=86.71 cfs Outflow=4.76 cfs 0.248 af

**Reach 3R: Swale to CB 3** Avg. Flow Depth=0.37' Max Vel=6.10 fps Inflow=3.54 cfs 0.167 af
n=0.041 L=258.8' S=0.1827 '/' Capacity=121.81 cfs Outflow=3.51 cfs 0.167 af

**Reach 4R: Swale to CB 4** Avg. Flow Depth=0.37' Max Vel=5.57 fps Inflow=3.22 cfs 0.152 af
n=0.041 L=124.3' S=0.1532 '/' Capacity=111.53 cfs Outflow=3.18 cfs 0.152 af

**Reach 5R: Existing Culvert** Avg. Flow Depth=0.37' Max Vel=12.81 fps Inflow=4.27 cfs 0.214 af
18.0" Round Pipe n=0.013 L=46.4' S=0.0976 '/' Capacity=32.82 cfs Outflow=4.27 cfs 0.214 af

**Reach 10R: Swale to Ex. Culvert** Avg. Flow Depth=0.49' Max Vel=5.03 fps Inflow=4.30 cfs 0.214 af
n=0.041 L=80.0' S=0.0929 '/' Capacity=86.84 cfs Outflow=4.27 cfs 0.214 af

**Pond 1CB: CB and Culvert 1** Peak Elev=821.33' Inflow=3.29 cfs 0.159 af
18.0" Round Culvert n=0.013 L=51.0' S=0.0200 '/' Outflow=3.29 cfs 0.159 af

**Pond 2CB: CB and Culvert 2** Peak Elev=802.37' Inflow=4.76 cfs 0.248 af
18.0" Round Culvert n=0.013 L=47.0' S=0.0100 '/' Outflow=4.76 cfs 0.248 af

**Pond 3CB: CB and Culvert 3** Peak Elev=753.43' Inflow=3.51 cfs 0.167 af
18.0" Round Culvert n=0.013 L=42.0' S=0.0400 '/' Outflow=3.51 cfs 0.167 af

**Pond 4CB: CB and Culvert 4** Peak Elev=733.82' Inflow=3.18 cfs 0.152 af
18.0" Round Culvert n=0.013 L=42.0' S=0.0436 '/' Outflow=3.18 cfs 0.152 af

**Total Runoff Area = 4.314 ac Runoff Volume = 0.940 af Average Runoff Depth = 2.62"**
**95.00% Pervious = 4.098 ac 5.00% Impervious = 0.216 ac**

## Summary for Subcatchment 1S: Culvert 1 Subcatchment

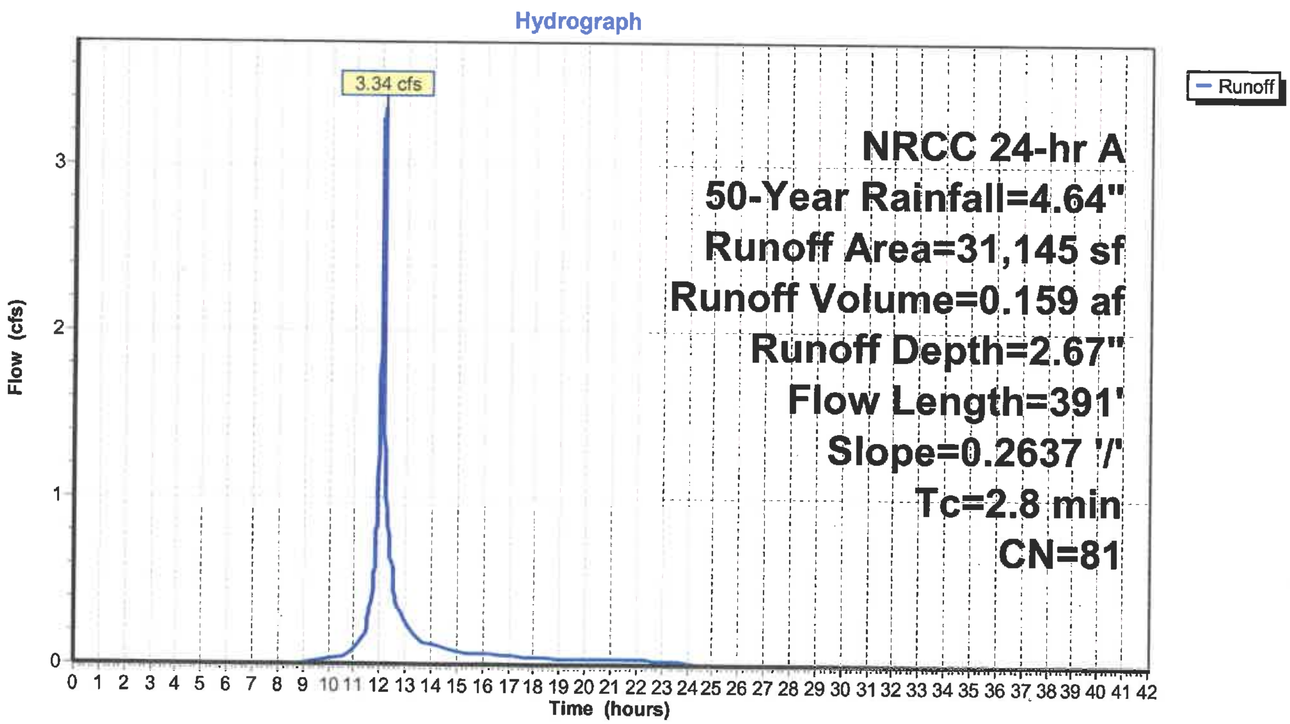
Runoff = 3.34 cfs @ 12.11 hrs, Volume= 0.159 af, Depth= 2.67"
Routed to Reach 1R : Swale to CB 1

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-42.00 hrs, dt= 0.01 hrs
NRCC 24-hr A 50-Year Rainfall=4.64"

| Area (sf) | CN | Description |
|---|---|---|
| 3,435 | 98 | Paved parking, HSG A |
| 27,710 | 79 | Woods, Fair, HSG D |
| 31,145 | 81 | Weighted Average |
| 27,710 | | 88.97% Pervious Area |
| 3,435 | | 11.03% Impervious Area |

| Tc (min) | Length (feet) | Slope (ft/ft) | Velocity (ft/sec) | Capacity (cfs) | Description |
|---|---|---|---|---|---|
| 2.8 | 391 | 0.2637 | 2.30 | | **Lag/CN Method,** |

### Subcatchment 1S: Culvert 1 Subcatchment



## Summary for Subcatchment 2S: Culvert 2 Subcatchment

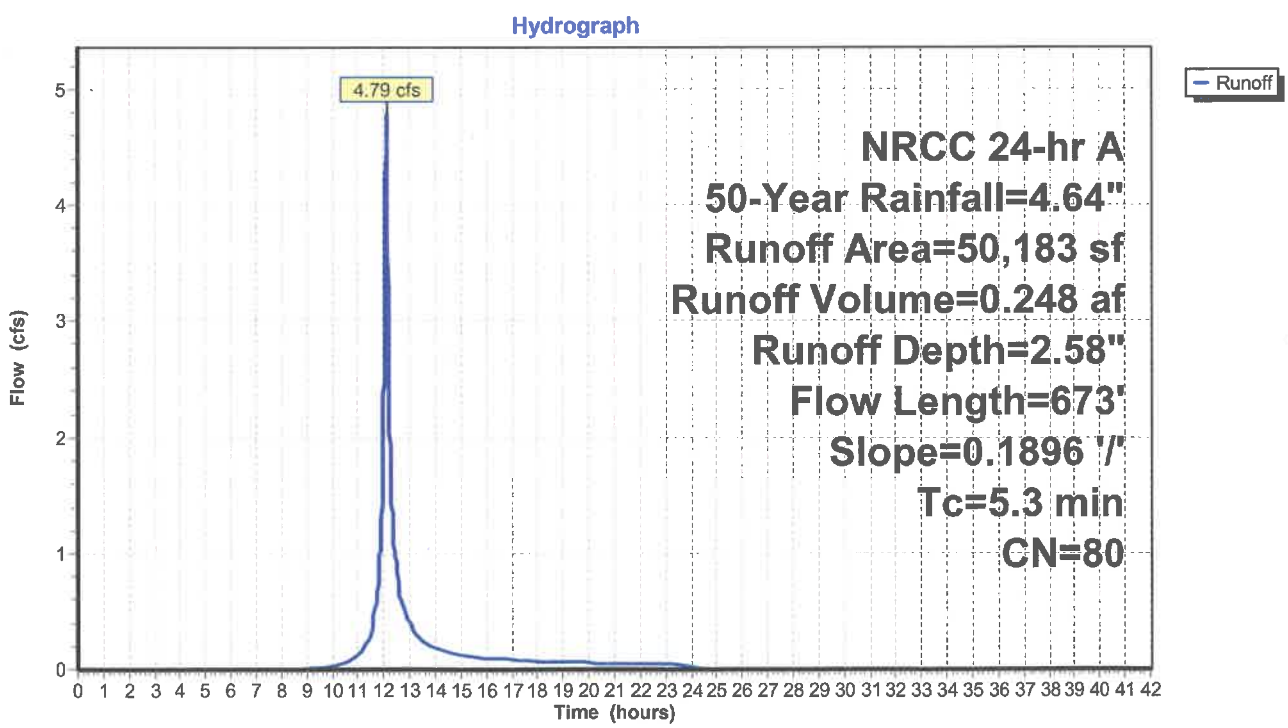
Runoff = 4.79 cfs @ 12.13 hrs, Volume= 0.248 af, Depth= 2.58"
Routed to Reach 2R : Swale to CB 2

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-42.00 hrs, dt= 0.01 hrs
NRCC 24-hr A 50-Year Rainfall=4.64"

| Area (sf) | CN | Description |
|---|---|---|
| 1,485 | 98 | Paved parking, HSG A |
| 40,548 | 79 | Woods, Fair, HSG D |
| 8,150 | 84 | 50-75% Grass cover, Fair, HSG D |
| 50,183 | 80 | Weighted Average |
| 48,698 | | 97.04% Pervious Area |
| 1,485 | | 2.96% Impervious Area |

| Tc (min) | Length (feet) | Slope (ft/ft) | Velocity (ft/sec) | Capacity (cfs) | Description |
|---|---|---|---|---|---|
| 5.3 | 673 | 0.1896 | 2.11 | | **Lag/CN Method,** |

### Subcatchment 2S: Culvert 2 Subcatchment



## Summary for Subcatchment 3S: Culvert 3 Subcatchment

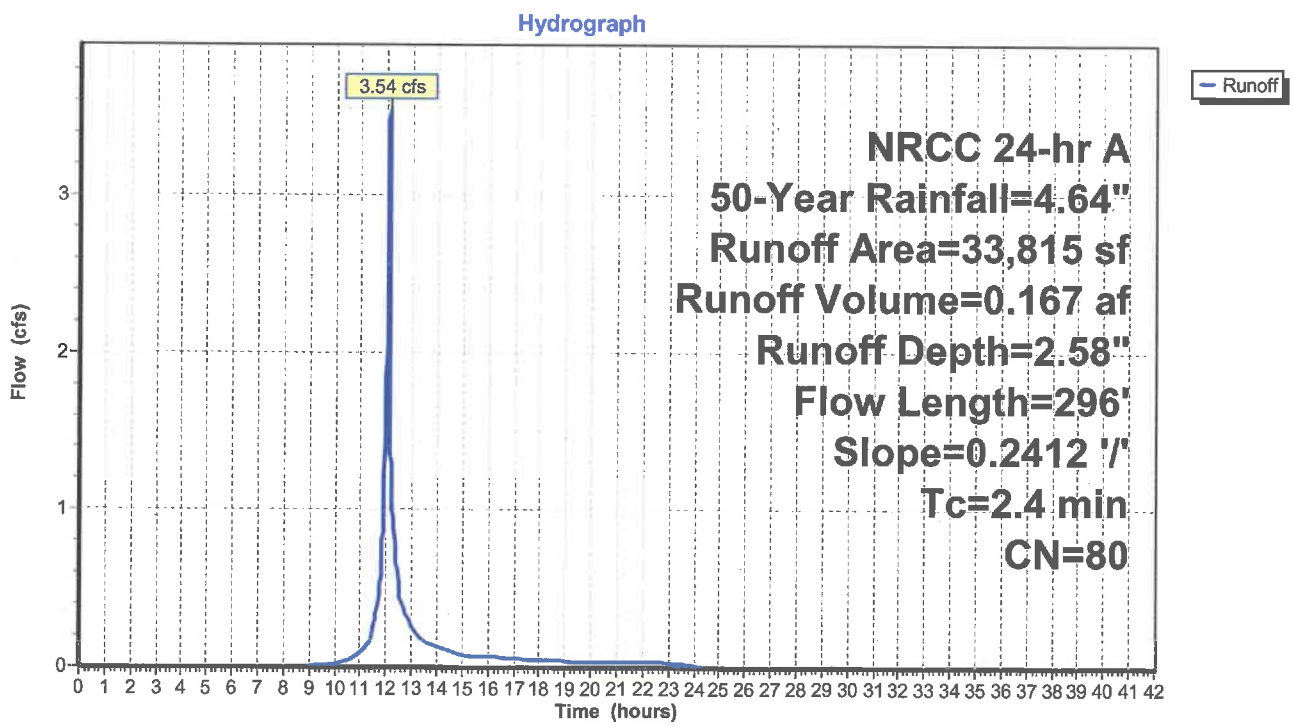
Runoff = 3.54 cfs @ 12.10 hrs, Volume= 0.167 af, Depth= 2.58"
Routed to Reach 3R : Swale to CB 3

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-42.00 hrs, dt= 0.01 hrs
NRCC 24-hr A 50-Year Rainfall=4.64"

| Area (sf) | CN | Description |
|---|---|---|
| 667 | 98 | Paved parking, HSG A |
| 32,144 | 79 | Woods, Fair, HSG D |
| 1,004 | 84 | 50-75% Grass cover, Fair, HSG D |
| 33,815 | 80 | Weighted Average |
| 33,148 | | 98.03% Pervious Area |
| 667 | | 1.97% Impervious Area |

| Tc (min) | Length (feet) | Slope (ft/ft) | Velocity (ft/sec) | Capacity (cfs) | Description |
|---|---|---|---|---|---|
| 2.4 | 296 | 0.2412 | 2.02 | | **Lag/CN Method,** |

### Subcatchment 3S: Culvert 3 Subcatchment



## Summary for Subcatchment 4S: Culvert 4 Subcatchment

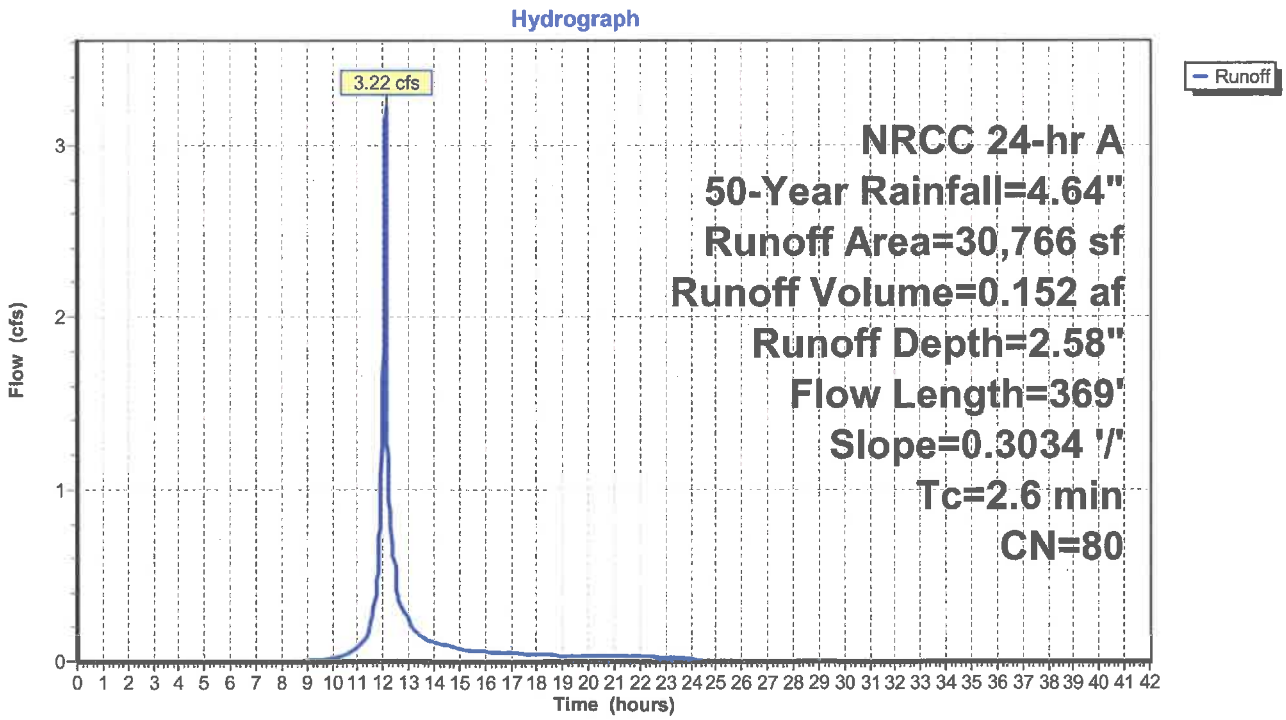
Runoff = 3.22 cfs @ 12.11 hrs, Volume= 0.152 af, Depth= 2.58"
Routed to Reach 4R : Swale to CB 4

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-42.00 hrs, dt= 0.01 hrs
NRCC 24-hr A 50-Year Rainfall=4.64"

| Area (sf) | CN | Description |
|---|---|---|
| 1,198 | 98 | Paved parking, HSG A |
| 28,595 | 79 | Woods, Fair, HSG D |
| 973 | 84 | 50-75% Grass cover, Fair, HSG D |
| 30,766 | 80 | Weighted Average |
| 29,568 | | 96.11% Pervious Area |
| 1,198 | | 3.89% Impervious Area |

| Tc (min) | Length (feet) | Slope (ft/ft) | Velocity (ft/sec) | Capacity (cfs) | Description |
|---|---|---|---|---|---|
| 2.6 | 369 | 0.3034 | 2.37 | | **Lag/CN Method,** |

### Subcatchment 4S: Culvert 4 Subcatchment



## Summary for Subcatchment 5S: Ex Culvert Subcatchment

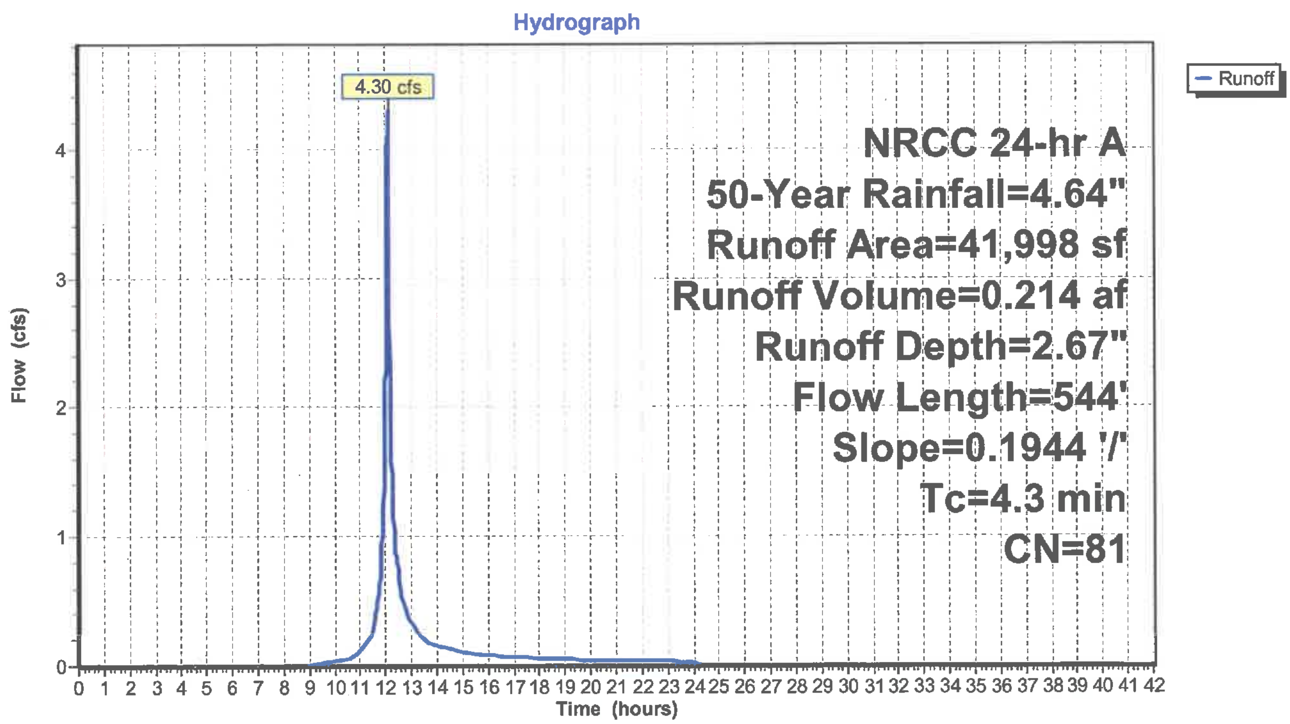
Runoff = 4.30 cfs @ 12.12 hrs, Volume= 0.214 af, Depth= 2.67"
Routed to Reach 10R : Swale to Ex. Culvert

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-42.00 hrs, dt= 0.01 hrs
NRCC 24-hr A 50-Year Rainfall=4.64"

| Area (sf) | CN | Description |
|---|---|---|
| 2,619 | 98 | Paved parking, HSG A |
| 31,112 | 79 | Woods, Fair, HSG D |
| 8,267 | 84 | 50-75% Grass cover, Fair, HSG D |
| 41,998 | 81 | Weighted Average |
| 39,379 | | 93.76% Pervious Area |
| 2,619 | | 6.24% Impervious Area |

| Tc (min) | Length (feet) | Slope (ft/ft) | Velocity (ft/sec) | Capacity (cfs) | Description |
|---|---|---|---|---|---|
| 4.3 | 544 | 0.1944 | 2.11 | | **Lag/CN Method,** |

### Subcatchment 5S: Ex Culvert Subcatchment



## Summary for Reach 1R: Swale to CB 1

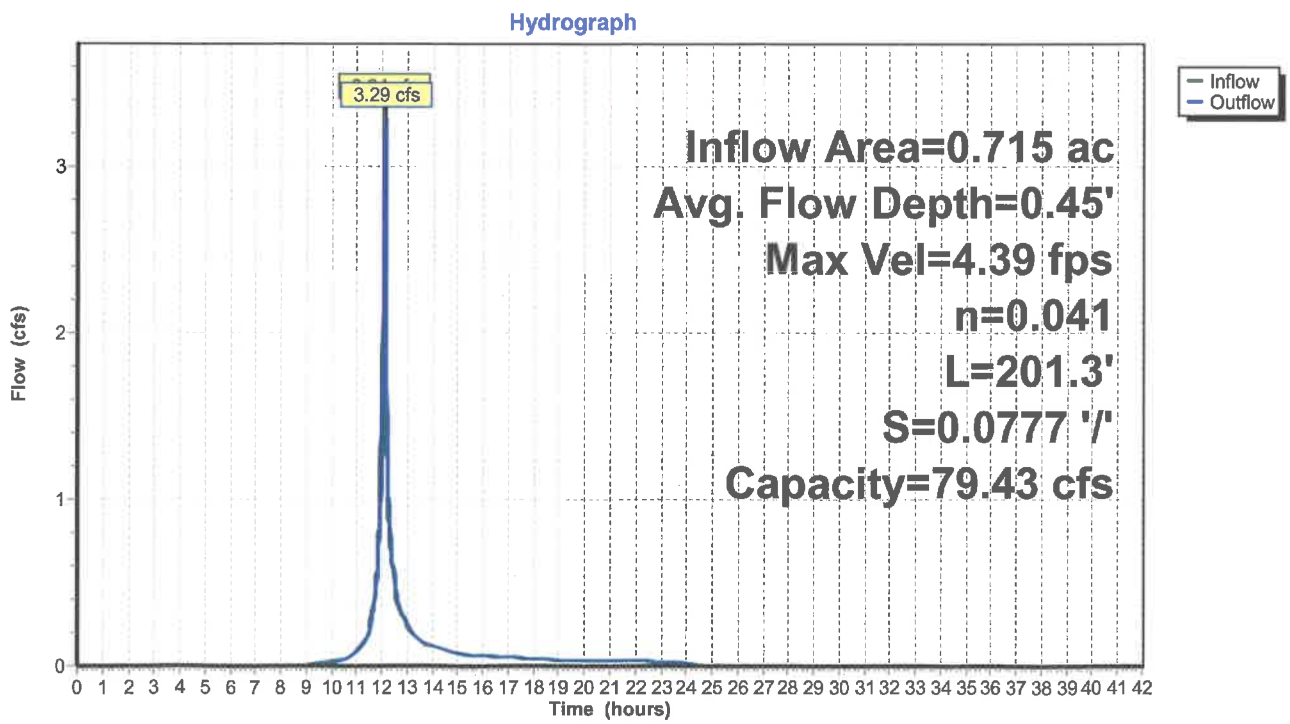
Inflow Area = 0.715 ac, 11.03% Impervious, Inflow Depth = 2.67" for 50-Year event
Inflow = 3.34 cfs @ 12.11 hrs, Volume= 0.159 af
Outflow = 3.29 cfs @ 12.12 hrs, Volume= 0.159 af, Atten= 2%, Lag= 1.1 min
Routed to Pond 1CB : CB and Culvert 1

Routing by Stor-Ind+Trans method, Time Span= 0.00-42.00 hrs, dt= 0.01 hrs
Max. Velocity= 4.39 fps, Min. Travel Time= 0.8 min
Avg. Velocity = 1.25 fps, Avg. Travel Time= 2.7 min

Peak Storage= 151 cf @ 12.11 hrs
Average Depth at Peak Storage= 0.45' , Surface Width= 2.35'
Bank-Full Depth= 2.00' Flow Area= 8.0 sf, Capacity= 79.43 cfs

1.00' x 2.00' deep channel, n= 0.041 Riprap, 2-inch
Side Slope Z-value= 1.5 '/' Top Width= 7.00'
Length= 201.3' Slope= 0.0777 '/'
Inlet Invert= 844.94', Outlet Invert= 829.30'

### Reach 1R: Swale to CB 1



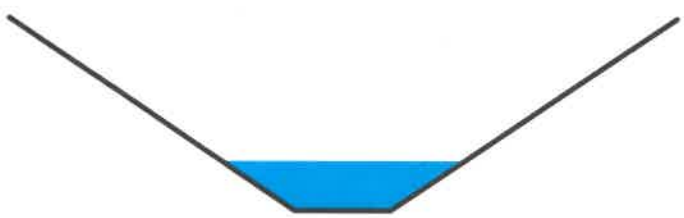
## Summary for Reach 2R: Swale to CB 2

Inflow Area = 1.152 ac, 2.96% Impervious, Inflow Depth = 2.58" for 50-Year event
Inflow = 4.79 cfs @ 12.13 hrs, Volume= 0.248 af
Outflow = 4.76 cfs @ 12.14 hrs, Volume= 0.248 af, Atten= 1%, Lag= 0.6 min
Routed to Pond 2CB : CB and Culvert 2

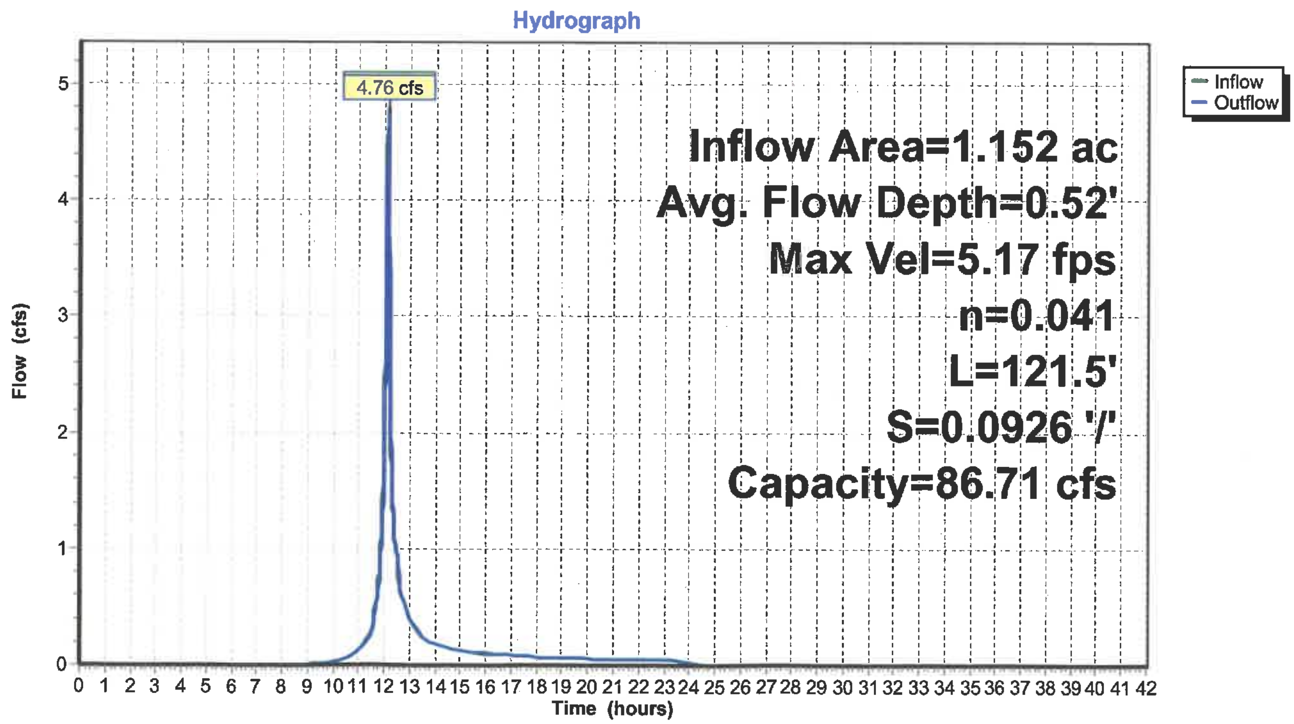
Routing by Stor-Ind+Trans method, Time Span= 0.00-42.00 hrs, dt= 0.01 hrs
Max. Velocity= 5.17 fps, Min. Travel Time= 0.4 min
Avg. Velocity = 1.55 fps, Avg. Travel Time= 1.3 min

Peak Storage= 112 cf @ 12.13 hrs
Average Depth at Peak Storage= 0.52' , Surface Width= 2.56'
Bank-Full Depth= 2.00' Flow Area= 8.0 sf, Capacity= 86.71 cfs

1.00' x 2.00' deep channel, n= 0.041 Riprap, 2-inch
Side Slope Z-value= 1.5 '/' Top Width= 7.00'
Length= 121.5' Slope= 0.0926 '/'
Inlet Invert= 820.85', Outlet Invert= 809.60'



**Reach 2R: Swale to CB 2**



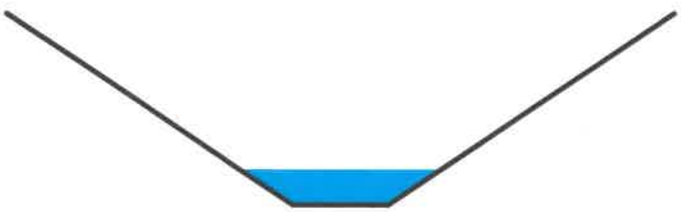
## Summary for Reach 3R: Swale to CB 3

Inflow Area = 0.776 ac, 1.97% Impervious, Inflow Depth = 2.58" for 50-Year event
Inflow = 3.54 cfs @ 12.10 hrs, Volume= 0.167 af
Outflow = 3.51 cfs @ 12.12 hrs, Volume= 0.167 af, Atten= 1%, Lag= 0.8 min
Routed to Pond 3CB : CB and Culvert 3

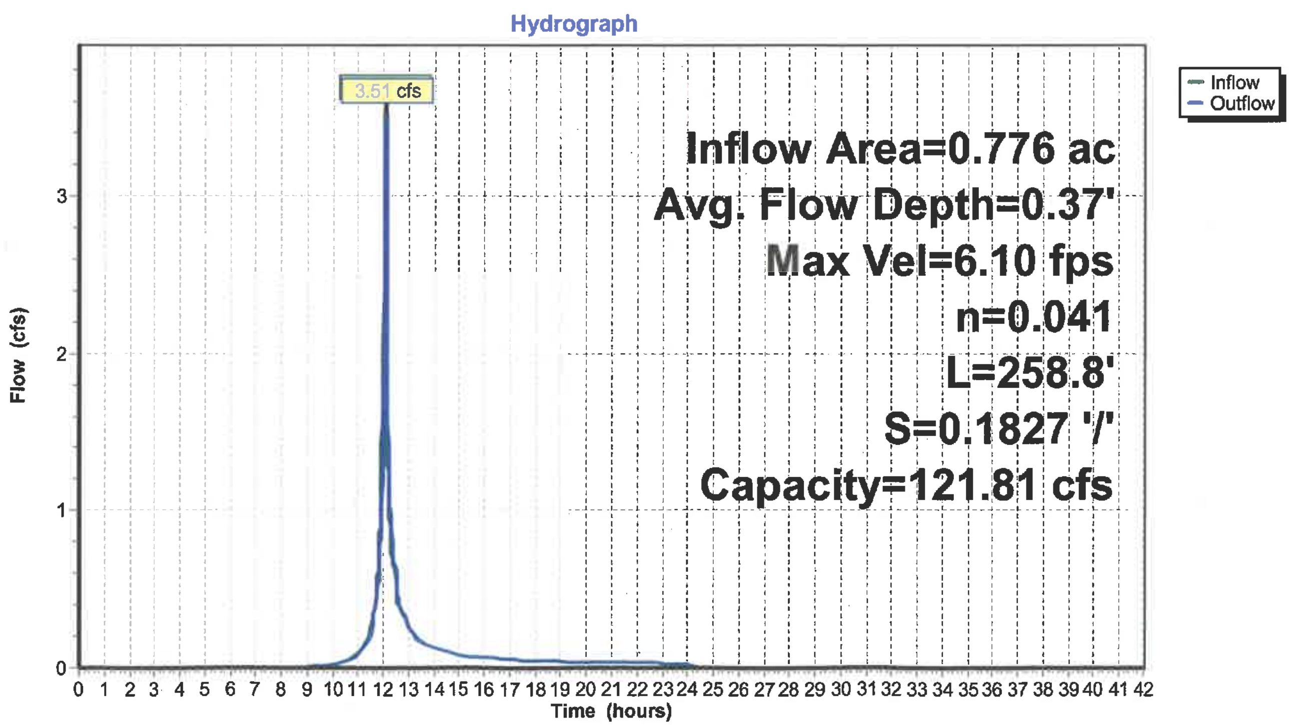
Routing by Stor-Ind+Trans method, Time Span= 0.00-42.00 hrs, dt= 0.01 hrs
Max. Velocity= 6.10 fps, Min. Travel Time= 0.7 min
Avg. Velocity = 1.71 fps, Avg. Travel Time= 2.5 min

Peak Storage= 149 cf @ 12.11 hrs
Average Depth at Peak Storage= 0.37' , Surface Width= 2.11'
Bank-Full Depth= 2.00' Flow Area= 8.0 sf, Capacity= 121.81 cfs

1.00' x 2.00' deep channel, n= 0.041 Riprap, 2-inch
Side Slope Z-value= 1.5 '/' Top Width= 7.00'
Length= 258.8' Slope= 0.1827 '/'
Inlet Invert= 809.69', Outlet Invert= 762.40'



### Reach 3R: Swale to CB 3



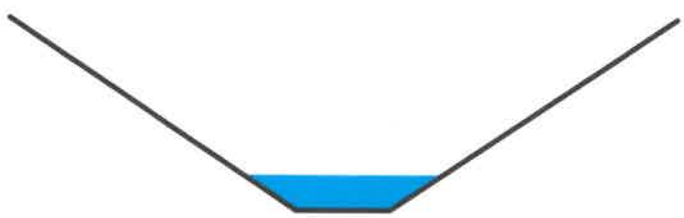
## Summary for Reach 4R: Swale to CB 4

Inflow Area = 0.706 ac, 3.89% Impervious, Inflow Depth = 2.58" for 50-Year event
Inflow = 3.22 cfs @ 12.11 hrs, Volume= 0.152 af
Outflow = 3.18 cfs @ 12.11 hrs, Volume= 0.152 af, Atten= 1%, Lag= 0.4 min
Routed to Pond 4CB : CB and Culvert 4

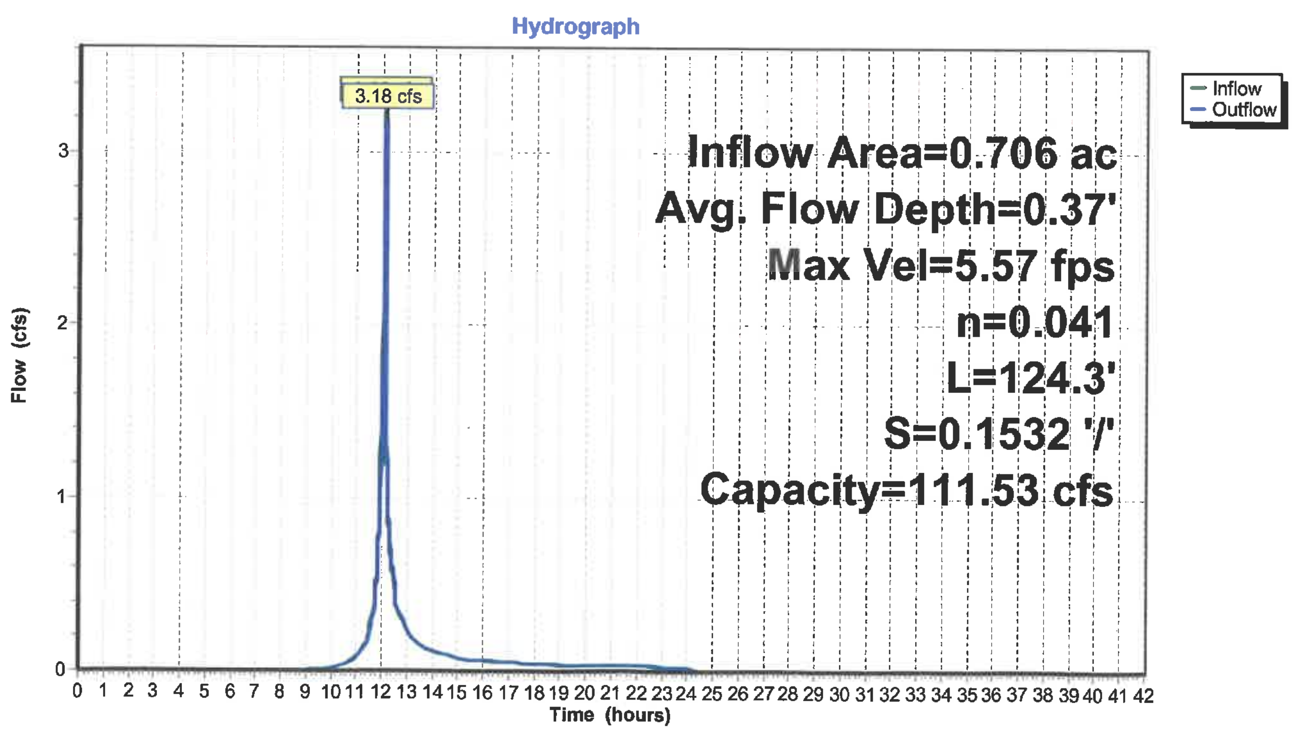
Routing by Stor-Ind+Trans method, Time Span= 0.00-42.00 hrs, dt= 0.01 hrs
Max. Velocity= 5.57 fps, Min. Travel Time= 0.4 min
Avg. Velocity = 1.57 fps, Avg. Travel Time= 1.3 min

Peak Storage= 71 cf @ 12.11 hrs
Average Depth at Peak Storage= 0.37' , Surface Width= 2.11'
Bank-Full Depth= 2.00' Flow Area= 8.0 sf, Capacity= 111.53 cfs

1.00' x 2.00' deep channel, n= 0.041 Riprap, 2-inch
Side Slope Z-value= 1.5 '/' Top Width= 7.00'
Length= 124.3' Slope= 0.1532 '/'
Inlet Invert= 762.00', Outlet Invert= 742.96'



### Reach 4R: Swale to CB 4



## Summary for Reach 5R: Existing Culvert

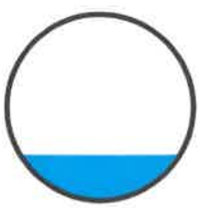
[52] Hint: Inlet/Outlet conditions not evaluated
[62] Hint: Exceeded Reach 10R OUTLET depth by 3.34' @ 24.14 hrs

Inflow Area = 0.964 ac, 6.24% Impervious, Inflow Depth = 2.67" for 50-Year event
Inflow = 4.27 cfs @ 12.12 hrs, Volume= 0.214 af
Outflow = 4.27 cfs @ 12.13 hrs, Volume= 0.214 af, Atten= 0%, Lag= 0.1 min

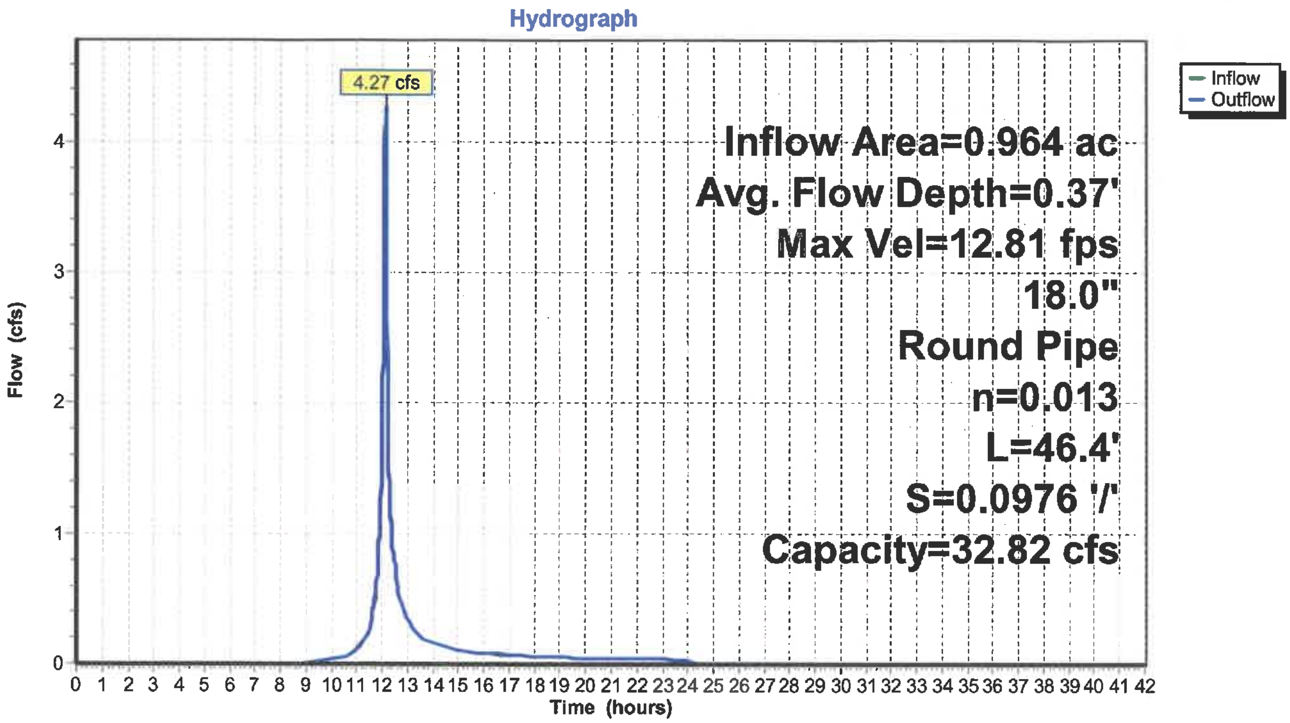
Routing by Stor-Ind+Trans method, Time Span= 0.00-42.00 hrs, dt= 0.01 hrs
Max. Velocity= 12.81 fps, Min. Travel Time= 0.1 min
Avg. Velocity = 3.92 fps, Avg. Travel Time= 0.2 min

Peak Storage= 15 cf @ 12.13 hrs
Average Depth at Peak Storage= 0.37' , Surface Width= 1.29'
Bank-Full Depth= 1.50' Flow Area= 1.8 sf, Capacity= 32.82 cfs

18.0" Round Pipe
n= 0.013 Corrugated PE, smooth interior
Length= 46.4' Slope= 0.0976 '/'
Inlet Invert= 825.03', Outlet Invert= 820.50'



## Reach 5R: Existing Culvert



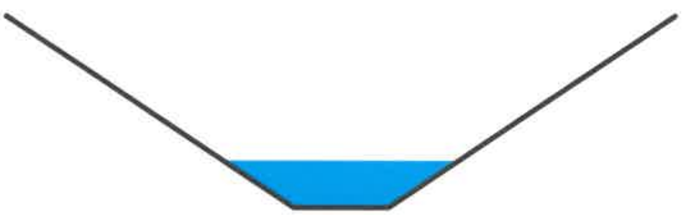
## Summary for Reach 10R: Swale to Ex. Culvert

Inflow Area = 0.964 ac, 6.24% Impervious, Inflow Depth = 2.67" for 50-Year event
Inflow = 4.30 cfs @ 12.12 hrs, Volume= 0.214 af
Outflow = 4.27 cfs @ 12.12 hrs, Volume= 0.214 af, Atten= 1%, Lag= 0.4 min
Routed to Reach 5R : Existing Culvert

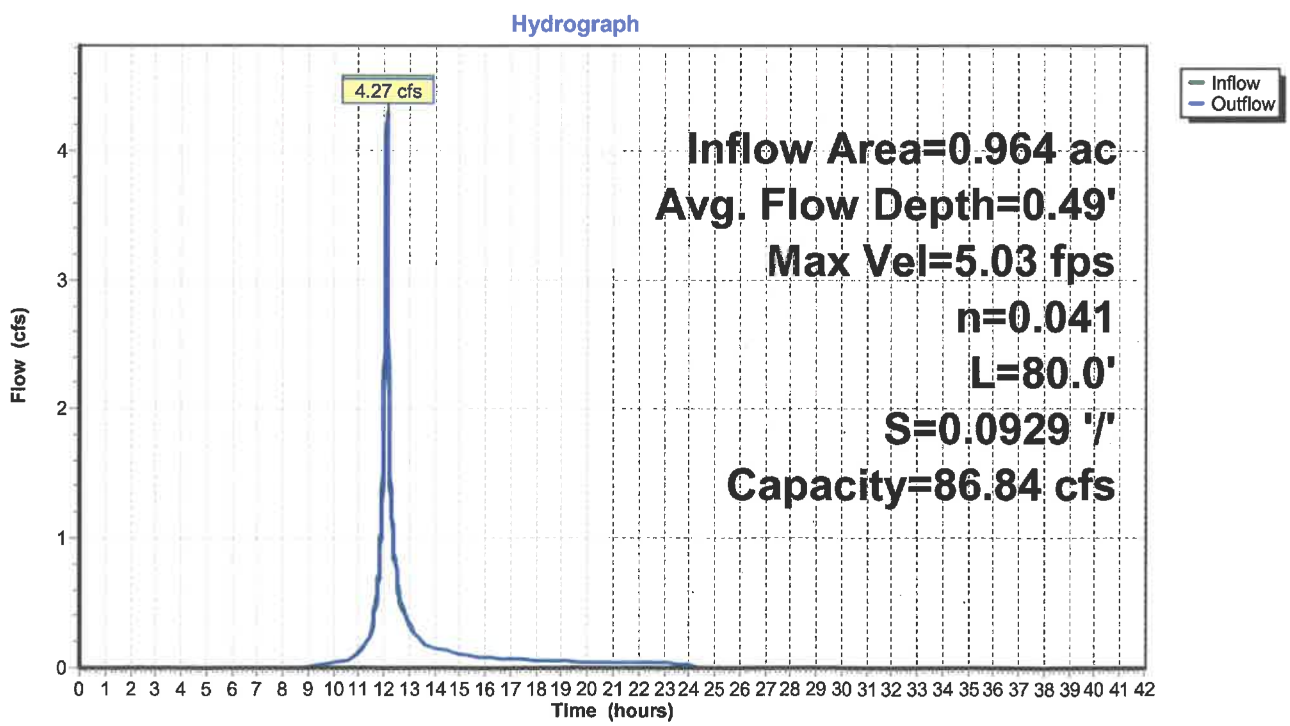
Routing by Stor-Ind+Trans method, Time Span= 0.00-42.00 hrs, dt= 0.01 hrs
Max. Velocity= 5.03 fps, Min. Travel Time= 0.3 min
Avg. Velocity = 1.47 fps, Avg. Travel Time= 0.9 min

Peak Storage= 68 cf @ 12.12 hrs
Average Depth at Peak Storage= 0.49' , Surface Width= 2.47'
Bank-Full Depth= 2.00' Flow Area= 8.0 sf, Capacity= 86.84 cfs

1.00' x 2.00' deep channel, n= 0.041 Riprap, 2-inch
Side Slope Z-value= 1.5 '/' Top Width= 7.00'
Length= 80.0' Slope= 0.0929 '/'
Inlet Invert= 829.13', Outlet Invert= 821.70'



### Reach 10R: Swale to Ex. Culvert



## Summary for Pond 1CB: CB and Culvert 1

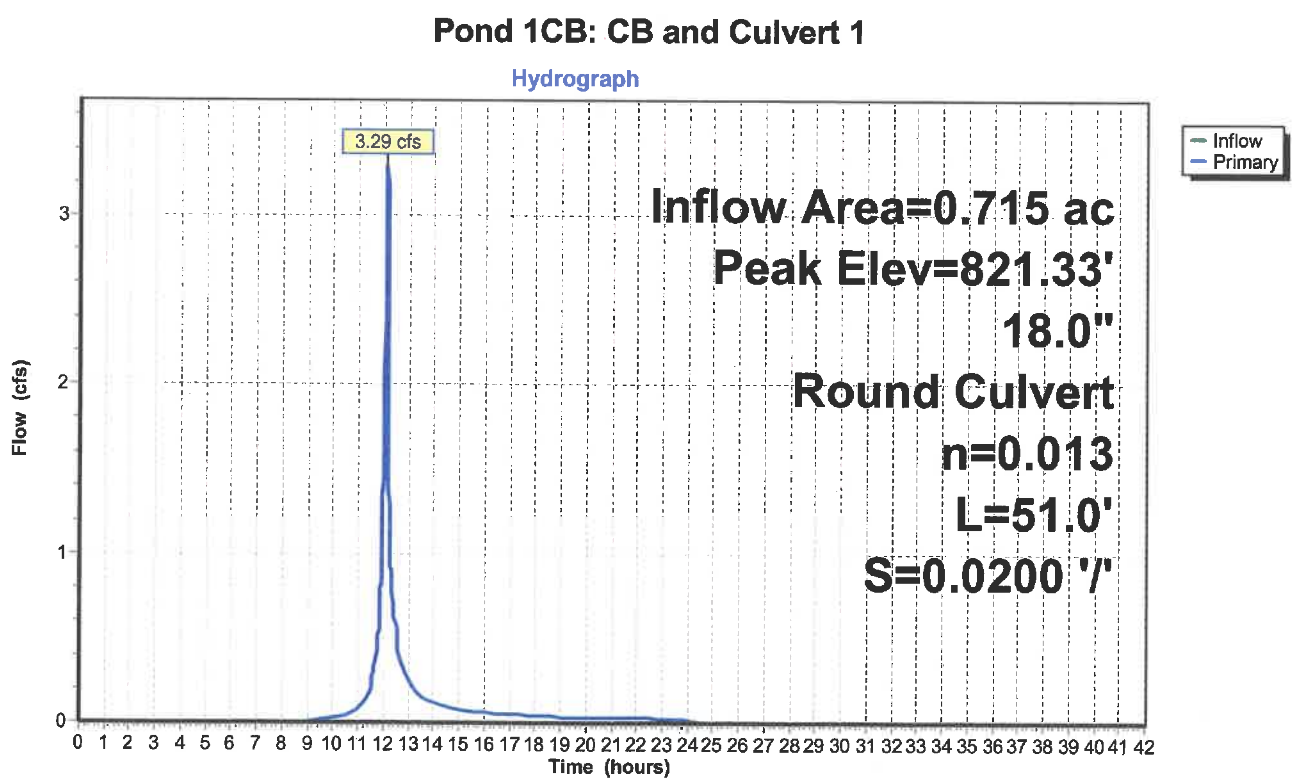
[57] Hint: Peaked at 821.33' (Flood elevation advised)

| | | | | |
|---|---|---|---|---|
| Inflow Area = | 0.715 ac, | 11.03% Impervious, | Inflow Depth = 2.67" | for 50-Year event |
| Inflow = | 3.29 cfs @ | 12.12 hrs, | Volume= | 0.159 af |
| Outflow = | 3.29 cfs @ | 12.12 hrs, | Volume= | 0.159 af, Atten= 0%, Lag= 0.0 min |
| Primary = | 3.29 cfs @ | 12.12 hrs, | Volume= | 0.159 af |

Routing by Stor-Ind method, Time Span= 0.00-42.00 hrs, dt= 0.01 hrs
Peak Elev= 821.33' @ 12.12 hrs

| Device | Routing | Invert | Outlet Devices |
|---|---|---|---|
| #1 | Primary | 820.34' | **18.0" Round Culvert**<br>L= 51.0' CPP, projecting, no headwall, Ke= 0.900<br>Inlet / Outlet Invert= 820.34' / 819.32' S= 0.0200 '/' Cc= 0.900<br>n= 0.013 Corrugated PE, smooth interior, Flow Area= 1.77 sf |

**Primary OutFlow** Max=3.28 cfs @ 12.12 hrs HW=821.32' (Free Discharge)
└─**1=Culvert** (Inlet Controls 3.28 cfs @ 2.67 fps)



## Summary for Pond 2CB: CB and Culvert 2

[57] Hint: Peaked at 802.37' (Flood elevation advised)

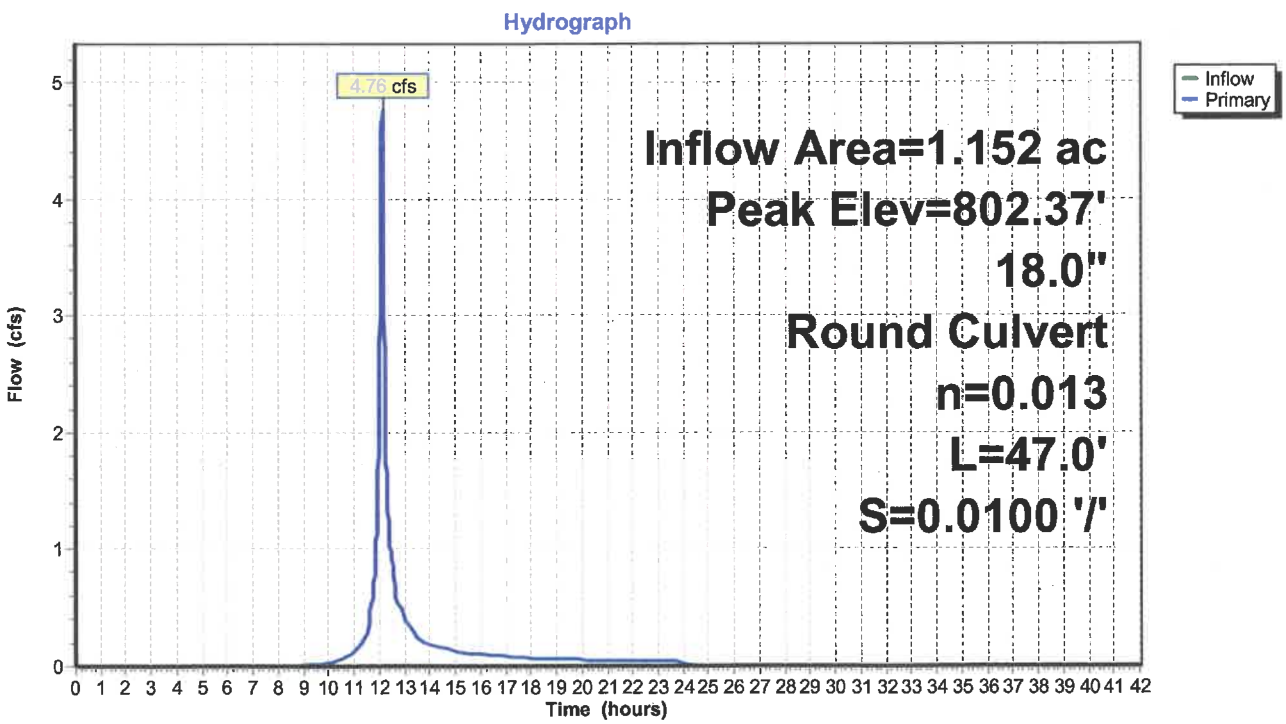
| | | | | | |
|---|---|---|---|---|---|
| Inflow Area = | 1.152 ac, | 2.96% Impervious, | Inflow Depth = 2.58" | for 50-Year event | |
| Inflow = | 4.76 cfs @ | 12.14 hrs, | Volume= | 0.248 af | |
| Outflow = | 4.76 cfs @ | 12.14 hrs, | Volume= | 0.248 af, | Atten= 0%, Lag= 0.0 min |
| Primary = | 4.76 cfs @ | 12.14 hrs, | Volume= | 0.248 af | |

Routing by Stor-Ind method, Time Span= 0.00-42.00 hrs, dt= 0.01 hrs
Peak Elev= 802.37' @ 12.14 hrs

| Device | Routing | Invert | Outlet Devices |
|---|---|---|---|
| #1 | Primary | 801.11' | **18.0" Round Culvert**<br>L= 47.0' CPP, projecting, no headwall, Ke= 0.900<br>Inlet / Outlet Invert= 801.11' / 800.64' S= 0.0100 '/' Cc= 0.900<br>n= 0.013 Corrugated PE, smooth interior, Flow Area= 1.77 sf |

**Primary OutFlow** Max=4.75 cfs @ 12.14 hrs HW=802.36' (Free Discharge)
↑**1=Culvert** (Inlet Controls 4.75 cfs @ 3.01 fps)

### Pond 2CB: CB and Culvert 2



## Summary for Pond 3CB: CB and Culvert 3

[57] Hint: Peaked at 753.43' (Flood elevation advised)

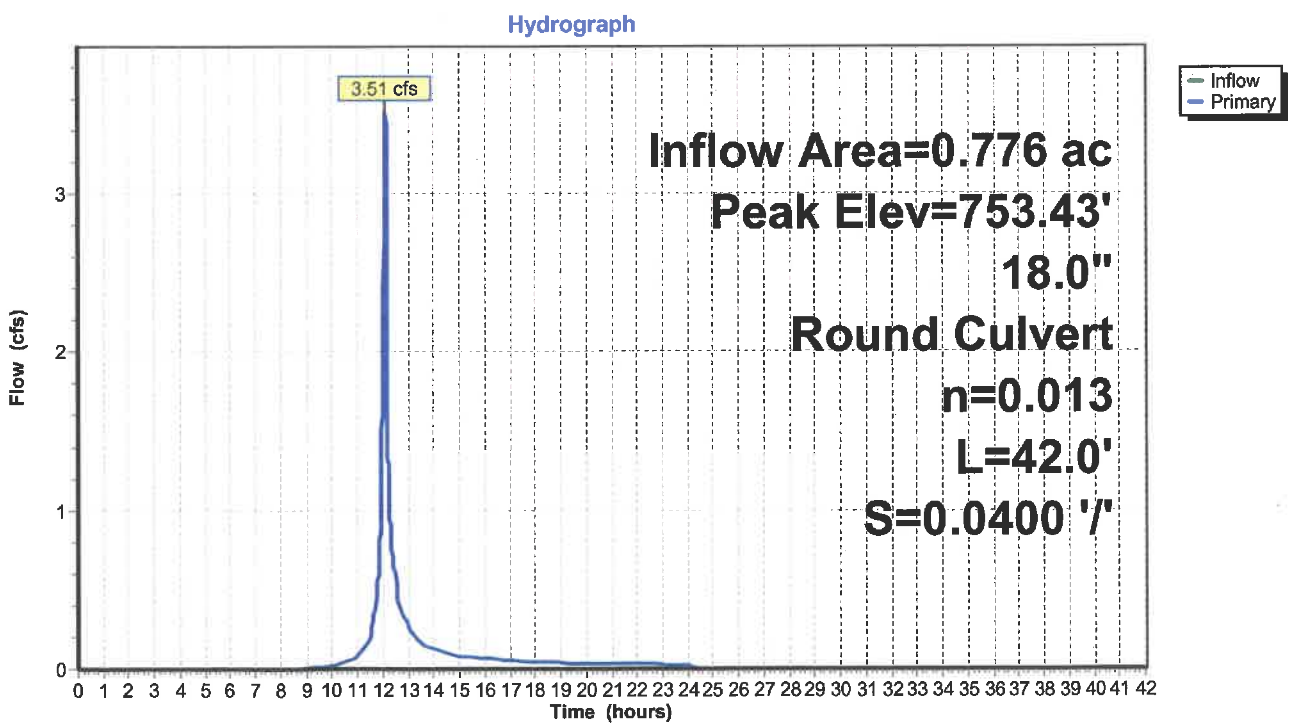
Inflow Area = 0.776 ac, 1.97% Impervious, Inflow Depth = 2.58" for 50-Year event
Inflow = 3.51 cfs @ 12.12 hrs, Volume= 0.167 af
Outflow = 3.51 cfs @ 12.12 hrs, Volume= 0.167 af, Atten= 0%, Lag= 0.0 min
Primary = 3.51 cfs @ 12.12 hrs, Volume= 0.167 af

Routing by Stor-Ind method, Time Span= 0.00-42.00 hrs, dt= 0.01 hrs
Peak Elev= 753.43' @ 12.12 hrs

| Device | Routing | Invert | Outlet Devices |
|---|---|---|---|
| #1 | Primary | 752.40' | **18.0" Round Culvert**<br>L= 42.0' CPP, projecting, no headwall, Ke= 0.900<br>Inlet / Outlet Invert= 752.40' / 750.72' S= 0.0400 '/' Cc= 0.900<br>n= 0.013 Corrugated PE, smooth interior, Flow Area= 1.77 sf |

**Primary OutFlow** Max=3.51 cfs @ 12.12 hrs HW=753.43' (Free Discharge)
└**1=Culvert** (Inlet Controls 3.51 cfs @ 2.72 fps)

### Pond 3CB: CB and Culvert 3



## Summary for Pond 4CB: CB and Culvert 4

[57] Hint: Peaked at 733.82' (Flood elevation advised)

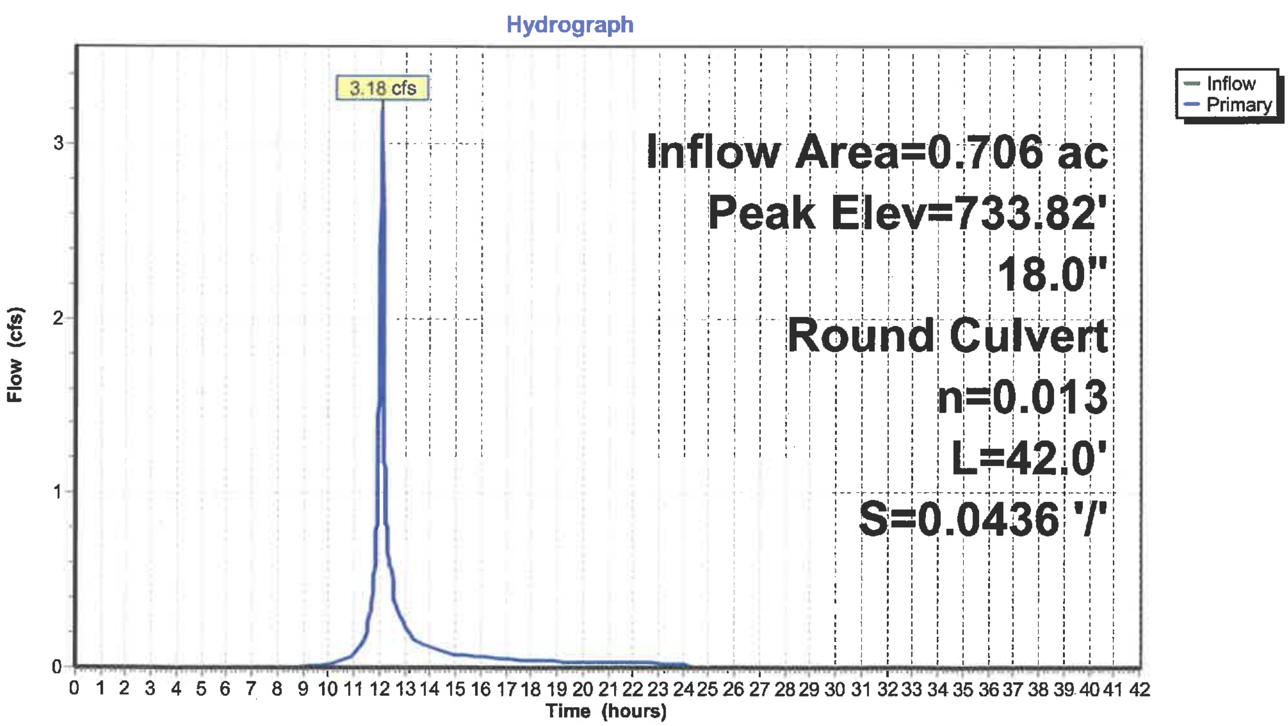
| | | | | |
|---|---|---|---|---|
| Inflow Area = | 0.706 ac, | 3.89% Impervious, Inflow Depth = 2.58" | for 50-Year event | |
| Inflow = | 3.18 cfs @ | 12.11 hrs, Volume= | 0.152 af | |
| Outflow = | 3.18 cfs @ | 12.11 hrs, Volume= | 0.152 af, | Atten= 0%, Lag= 0.0 min |
| Primary = | 3.18 cfs @ | 12.11 hrs, Volume= | 0.152 af | |

Routing by Stor-Ind method, Time Span= 0.00-42.00 hrs, dt= 0.01 hrs
Peak Elev= 733.82' @ 12.11 hrs

| Device | Routing | Invert | Outlet Devices |
|---|---|---|---|
| #1 | Primary | 732.85' | **18.0" Round Culvert**<br>L= 42.0' CPP, projecting, no headwall, Ke= 0.900<br>Inlet / Outlet Invert= 732.85' / 731.02' S= 0.0436 '/' Cc= 0.900<br>n= 0.013 Corrugated PE, smooth interior, Flow Area= 1.77 sf |

**Primary OutFlow** Max=3.18 cfs @ 12.11 hrs HW=733.82' (Free Discharge)
└**1=Culvert** (Inlet Controls 3.18 cfs @ 2.64 fps)

### Pond 4CB: CB and Culvert 4



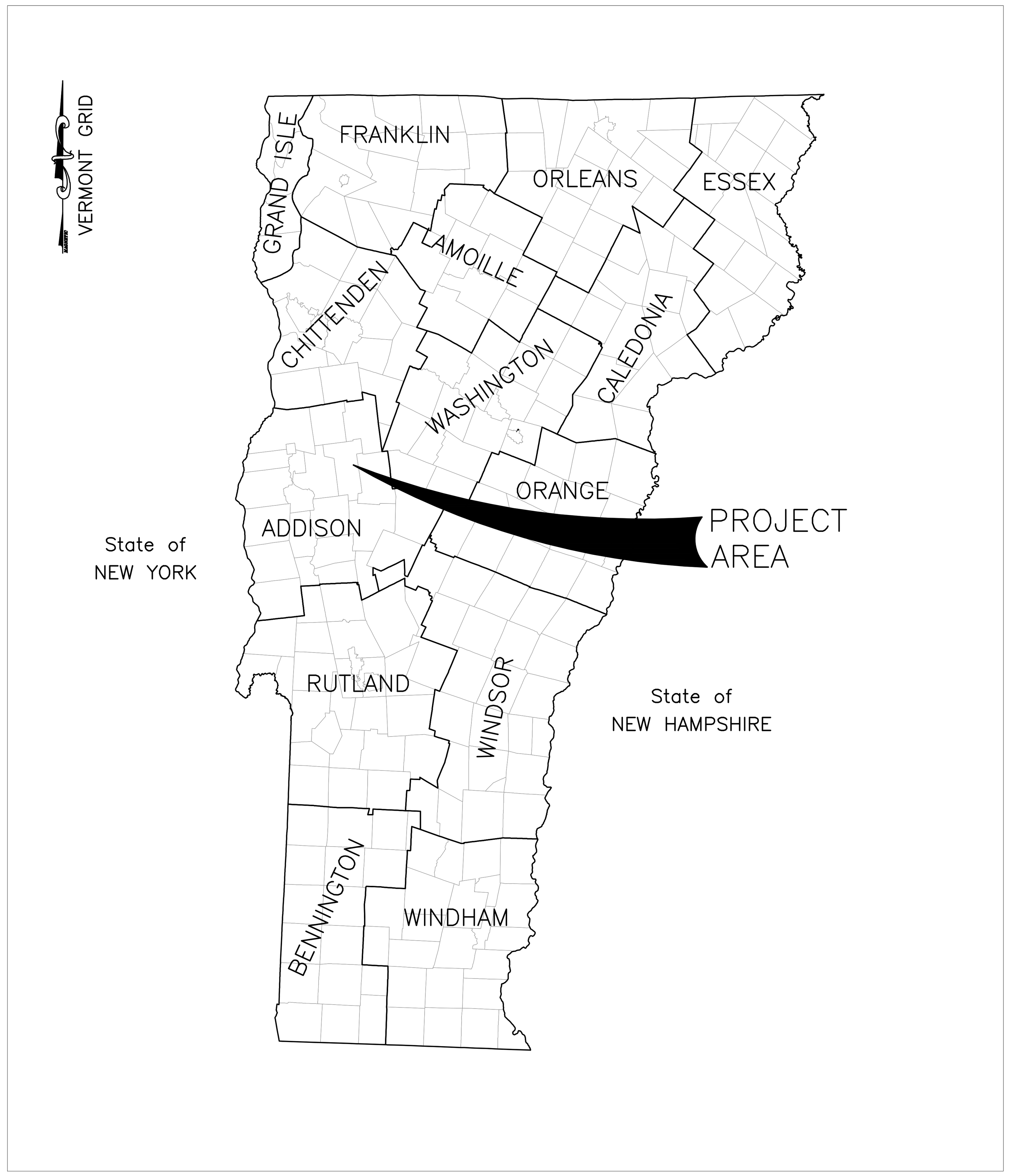
FEMA SUMMARY

734189-DR4720VT

Bristol – Briggs Hill Road

# Proposed Design Plans

# TOWN OF BRISTOL
# BRIGGS HILL RD SLOPE STABILIZATION




LOCATION MAP
NOT TO SCALE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RANDOLPH, VT 05060
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GEO DESIGN

85 GRANITE SHED LANE
UNIT 1
MONTPELIER, VT 05602
802.674.2033
geocompanies.com


JULY 2024
PROGRESS SET
NOT FOR CONSTRUCTION


PROJECT LOCATION MAP
APPROX. SCALE: 1" = 1000' ±

## LIST OF DRAWINGS



| REVISION NO. | DATE | DESCRIPTION |
|---|---|---|
| | | |

PROJECT NO. 229369

## GENERAL NOTES

1. THIS PROJECT CONSISTS OF STABILIZING SLOPES ALONG THE DOWNHILL SIDE OF BRIGGS HILL ROAD THAT HAVE SHOWN SIGNS OF INSTABILITY AND EROSION. REPAIRS ARE NECESSARY TO STABILIZE SPECIFIC PORTIONS OF THE SLOPE, SUPPORT THE ROADWAY, AND PREVENT FURTHER SCOUR/EROSION ALONG THE SLOPE AND AT THE CREST.

2. SLOPE REPAIRS GENERALLY CONSIST OF CONSTRUCTING ONE OR A COMBINATION OF THE FOLLOWING:
   - REGRADED AND VEGETATED SOIL SLOPES NO STEEPER THAN 2H:1V.
   - STONE ARMORED (RIP-RAP) SLOPES NO STEEPER THAN 1.5H:1V.
   - RETAINING WALL (MODULAR BLOCK) NO TALLER THAN 10 FEET HIGH (EXPOSED FACE).
   - MAINTAINING UNDISTURBED AND INHERENTLY STABLE AREAS (E.G., VEGETATED WITH TREES).

1. GEOTECHNICAL DESIGN AND CONSTRUCTION SPECIFICATIONS FOR SLOPE REPAIRS AT THIS SITE ARE BY GEODESIGN, INC. AND SITE CIVIL ENGINEERING IS BY DUBOIS & KING.

2. THE METHODS AND MATERIALS USED FOR REPAIRS AND CONSTRUCTION SHALL CONFORM TO THE VERMONT AGENCY OF TRANSPORTATION (VAOT) 2024 STANDARD SPECIFICATIONS FOR CONSTRUCTION AND THE NOTES SHOWN ON THESE PLANS. IN THE CASE OF CONFLICT, THE MORE STRINGENT SPECIFICATION SHALL APPLY AS DETERMINED BY THE ENGINEER.

3. REFER TO PLANS, DETAILS, AND CONSTRUCTION NOTES FOR FURTHER INFORMATION.

## STONE SLOPE REPAIR MATERIALS

1. SLOPE REPAIR MATERIALS SHALL COME FROM APPROVED SOURCES AND ARE DEFINED FURTHER BELOW. REFER TO THE PLANS AND DETAILS FOR MATERIAL PLACEMENT LOCATIONS.

2. STONE FILL SLOPE PROTECTION SHALL CONSIST OF VAOT 706.04B TYPE II.

3. GRANULAR FILTER SHALL CONSIST OF VAOT 704.07 GRAVEL FILTER FOR SLOPE STABILIZATION, OR VAOT 704.01A FINE AGGREGATE FOR CONCRETE.

4. GRANULAR FILL SHALL CONSIST OF VAOT 704.08A OR APPROVED EQUIVALENT.

5. REFER TO THE MODULAR BLOCK WALL NOTES FOR MATERIALS RELATED TO THE RETAINING WALLS.

## STONE SLOPE CONSTRUCTION NOTES

1. THE AREA WITHIN THE SLOPE REPAIR WORK LIMITS SHALL BE CLEARED AND GRUBBED. ALL SUBGRADE SOILS SHALL BE OBSERVED BY THE GEOTECHNICAL ENGINEER. TOPSOIL AND MATERIALS DEEMED UNSUITABLE BY THE GEOTECHNICAL ENGINEER (SUCH AS LOOSE, FROZEN, DISTURBED SOILS, TRASH, DEBRIS VEGETATION, TREE STUMPS, OVERSATURATED, OR OTHERWISE UNSTABLE SOILS) SHALL BE REMOVED AND A SMOOTH, FIRM, AND STABLE SUBGRADE SHALL BE PREPARED ON UNDISTURBED SOILS.

2. SUBGRADE SOILS (I.E., EXCAVATED NATURAL SOILS SERVING AS A BASE FOR SLOPE REPAIR MATERIALS) SHALL BE PROTECTED DURING CONSTRUCTION IN ACCORDANCE WITH THE FOLLOWING PROCEDURES:

2.1 EXCAVATION SHALL BE PERFORMED IN A MANNER TO LIMIT DISTURBANCE AND LOOSENING OF THE SUBGRADE.

2.2 FINAL PREPARED SUBGRADE SHALL BE SMOOTH AND FREE OF UNSUITABLE SOIL.

2.3 PRIOR TO EXCAVATING SUBGRADES POTENTIAL SOURCES OF SURFACE WATER SHALL BE DIRECTED AWAY FROM THE EXCAVATIONS.

2.4 GRANULAR FILTER MATERIAL SHALL BE PLACED OVER EXPOSED SUBGRADES IMMEDIATELY UPON ACHIEVEING A FIRM, STABLE SURFACE. THE GRANULAR FILTER LAYER PLACEMENT SHOULD BE PERFORMED SIMULATANEOUSLY AS THE EXCAVATION PROGRESSES.

2.5 THE SUBGRADE WILL BE VEFIRIFED IN THE FIELD BY THE GEOTECHNICAL ENGINEER DURING CONSTRUCTION. THE CONTRACTOR IS ALERTED TO THE PRESENCE OF DENSE, GLACIAL SOILS CONTAINING COBBLES AND BOULDERS ANTICIPATED ON THIS PROJECT.

3. SHALLOW GROUNDWATER AND DEWATERING MEASURES ARE NOT ANTICIPATED BASED ON THE SOIL BORINGS. HOWEVER, THE CONTRACTOR SHOULD BE PREPARED TO DIVERT SURFACE WATER AWAY FROM EXCAVATIONS TO ALLOW FOR PREPARING SUBGRADES IN DRY CONDITIONS. SUMPS OR OTHER LOCALIZED WATER COLLECTION LOCATIONS MAY BE REQUIRED FOR PUMPING WATER.

4. SCHEDULE AND COORDINATE EXCAVATION, BACKFILL, AND STONE PLACEMENT AROUND THE WEATHER FORECAST TO PREVENT OPEN EXCAVATIONS AND ACCUMULATION OF STORMWATER RUNOFF. CONTACT GEODESIGN WITH AT LEAST 48 HOURS NOTICE TO SCHEDULE SUBGRADE PREPARATION AND OBSERVATIONS. UNSUITABLE MATERIAL OR WEAK/SOFT/UNSTABLE AREAS IDENTIFIED DURING THE SUBGRADE PREPARATION SHALL BE EXCAVATED AND REPLACED WITH GRANULAR FILL OR OTHERWISE APPROVED MATERIAL BY THE GEOTECHNICAL ENGINEER, AND COMPACTED TO 95% MODIFIED PROCTOR DENSTIY.

5. SLOPE REPAIR MATERIALS SHALL BE PLACED STARTING AT THE TOE AND WORKED PROGRESSIVELY UPWARD TOWARD THE CREST OF THE SLOPE AT BRIGGS HILL ROAD. THE MATERIAL SHOULD BE TRANSPORTED AND PLACED USING METHODS THAT AVOIDS SEGREGATION. END DUMPING MATERIAL FROM THE CREST OF THE SLOPE, DUMPING BY WAY OF CHUTES, OR SPREADING WITH A BULLDOZER IS NOT ALLOWED. TEMPORARY ROCK FILLS THAT ARE MAY BE NEEDED TO CREATE ACCESS AND NECESSARY TO CORRECTLY PLACE STONE FILL REPAIR MATERIALS IS PART OF THE WORK AND SHALL NOT BE THE BASIS FOR ADDITIONAL CHARGE.

6. STONE FILL LAYERS SHALL BE PLACED TO THE REQUIRED THICKNESS SHOWN ON THE PLANS, IN LIFTS RESULTING IN A WELL GRADED HOMOGENEOUS MASS WITH A RELATIVELY LOW VOID RATIO AND TIGHTLY INTEGRATED WITH PRIOR LIFTS. PLACEMENT OF STONE IN LIFTS REQUIRES A MINIMUM OF 2 OVERLAPPING PASSES BY TRACKED EQUIPMENT, SUCH AS THE EXCAVATOR OR BULLDOZER. PLACEMENT TECHNIQUES RESULTING IN CLUSTERS OF EITHER SMALL OR LARGE STONES FOR FINAL SLOPE ARMORING WILL NOT BE ALLOWED. STONE FILL SHALL NOT BE PLACED ON FROZEN MATERIAL.

7. THE GRANULAR FILTER LAYER SHALL BE PLACED WITH A MINIMUM 12" SEPARATION BETWEEN PREPARED/ACCEPTED SUBGRADE AND IN PLACE STONE FILL. THE GRANULAR FILTER MATERIAL SHALL NOT BE FROZEN. GRANULAR FILTER AND PLACEMENT SHALL CONSIST OF:

7.1 6" TO 12" OF VAOT 706.04A STONE FILL TYPE I OR 704.07 GRAVEL FILTER FOR SLOPE STABILIZATION IF SUBGRADES ARE SILTY SAND OR GRAVELLY SAND (AS DETERMINED BY THE GEOTECHNICAL ENGINEER).

7.2 THE GEOTECHNICAL ENGINEER WILL INFORM THE CONTRACTOR WHICH GRANULAR FILTER TYPE IS REQUIRED FOLLOWING A REVIEW OF EXPOSED SUBGRADE CONDITIONS.

7.3 COMPACTION OF GRANULAR FILTER SHALL BE PERFORMED USING OVERLAPPING AND REPEATED USE OF DOWN PRESSURE FROM THE EXCAVATOR BUCKET.

7.4 PLACEMENT OF STONE FILL SHALL NOT OCCUR UNTIL FULL COMPACTED THICKNESS OF GRANULAR FILTER HAS BEEN INSTALLED.

8. WHERE APPLICABLE, INSTALL GRUBBING MATERIAL ABOVE THE FINAL STONE FILL OR REGRADED SURFACE AS INDICATED ON THE PLANS. GRUBBING MATERIAL SHALL EXTEND OVER THE SLOPE REPAIR LIMITS WHERE THE SOIL SLOPE HAS BEEN DISTURBED DURING CONSTRUCTION. THE PURPOSE OF THE GRUBBING MATERIAL IS TO ESTABLISH NATIVE PLANT GROWTH THAT CAN ROOT BETWEEN AND BELOW THE STONE FILL. ON STEEPER AREAS (I.E., STONE FILL SLOPES APPROACHING 1.5H:1V) LIVE STAKING OF NATIVE PLANTS AND SHRUBS MAY BE REQUIRED AT THE DIRECTION OF THE ENGINEER.

9. CONTRACTOR SHALL PROVIDE AS-BUILT RECORD DRAWINGS (CAD AND .PDF) OF THE PROJECT AFTER COMPLETION. THE RECORD DRAWING SHALL INCLUDE THE LIMITS, ELEVATIONS, AND THICKNESSES OF NEW STONE FILL AND 1-FOOT CONTOURS.

10. REMOVE ALL CLEARED, STRIPPED, AND GRUBBED MATERIAL FROM THE SITE. BURNING OF MATERIAL ON THE SITE IS NOT PERMITTED. EXCAVATED SOILS SHALL BE PERMANENTLY REMOVED FROM THE PREMISES AND LEGALLY DISPOSED OF.

## MODULAR BLOCK WALL

### GENERAL

1. THIS PROJECT HAS BEEN DESIGN USING THE REDI-ROCK ™ MODULAR BLOCK WALL SYSTEM. NO OTHER BLOCK WALL SYSTEM SHALL BE USED OR SUBSTITUTED WITHOUT PRIOR AUTHORIZATION FROM THE GEOTECHNICAL ENGINEER.

2. IT IS THE RESPONSIBILITY OF THE INSTALLER TO REVIEW THE NOTES AND DETAILS ON ALL SHEETS OF THIS PLAN SET. IF CONDITIONS ARE DIFFERENT THAN THOSE STATED IN THESE DRAWINGS AND DETAILS, THE CONTRACTOR MUST CONTACT THE GEOTECHNICAL ENGINEER PRIOR TO PROCEEDING WITH THE CONSTRUCTION OF THE WALL TO ALLOW FOR REVISIONS BASED ON ACTUAL/CHANGED CONDITIONS.

3. FINAL WALL ALIGNMENT SHALL BE AS INDICATED ON THE PLANS AND DETAILS.

4. PROVIDE LATERAL DRAINAGE SWALES TO DIRECT WATER FLOWS AROUND THE ENDS OF THE WALL AND AWAY FROM THE WALL DURING CONSTRUCTION. PERMANENT SWALES SHALL BE PITCHES TO THE WALL ENDS TO PROMOTE DRAINAGE OF SURFACE WATER UNOFF. REFER TO THE DRAINAGE DETAILS SHOWN ON THE SECTIONS.

5. THE ENDS OF THE RETAINING WALL SHALL BE BLENDED INTO THE PROPOSED/EXISTING GRADE IN A MANNER SATISFACTORY TO THE OWNER AND GEOTECHNICAL ENGINEER.

### BLOCK MATERIALS

1. ALL BLOCKS SHALL BE MANUFACTURED BY A LICENSE REDI-ROCK ™ MANUFACTURER.

2. BLOCKS SHALL MEET THE MINIMUM REDI-ROCK™ SPECIFICATIONS OF 4000 PSI WITH AN AIR CONTENT OF 4%-8%.

3. THE REDI-ROCK™ BLOCK UNITS MAY UTILIZE EITHER THE SPLIT LIMESTONE OF COBBLESTONE FACE CONFIGURATION AS CHOSEN BY THE OWNER OR OWNER'S REPRESENTATIVE.

### SITE PREPARATION

1. PLAN AND COORDINATE WORK WITH THE STONE SLOPE CONSTRUCTION NOTES FOR THIS PROJECT.

2. STRIP ALL VEGETATION, ORGANIC SOILS, AND UNSUITABLE FILL SOILS AS DIRECTED BY THE GEOTECHNICAL ENGINEER FROM THE WALL ALIGNMENT AREA. DO NOT OVER EXCAVATE UNLESS DIRECTED TO DO SO.

3. REMOVE ALL CONCRETE BLOCKS AND RELATED BACKFILL ASSOCIATED WITH THE EXISTING WALL IN THE VICINITY OF SECTION A-A'.

4. BENCH CUT ALL EXCAVATED SLOPES (OR AS INDICATED ON THE PLANS) AND FOLLOW GENERAL SLOPE SUBGRADE PREPARATION PROCEDURES SPECIFIED FOR THE STONE SLOPE AREAS. DO NOT DISTURB AREAS OTHER THAN THOSE DESIGNATED WITH THE WORK LIMITS SHOWN ON THE PLANS.

5. COORDINATE WITH THE GEOTECHNICAL ENGINEER TO REVIEW THE FOUNDATION SOILS ONCE EXPOSED.

6. THE LEVELING PAD SHALL CONSIST OF DENSE GRADED ¾" CRUSHED STONE, 12" THICK AND EXTENDING AT LEAST 12" TO EITHER SIDE OF THE BASE BLOCK.

7. THE MINIMUM EMBEDMENT OF THE WALL BEOW FINISH GRADE SHALL BE AS INDICATED ON THE WALL FACE DRAWING.

8. FOLLOW APPLICABLE PROVISIONS OF THE MANUFACTURER'S INSTALLATION INSTRUCTIONS AND WRITTEN SPECIFICATIONS, ESPECIALLY WITH REGARDS TO LEVELING OF BLOCKS AND BASE.

### BACKFILL AND COMPACTION

1. BACKFILL AND COMPACT GRANULAR FILL MATERIAL BEHIND THE WALL AS THE WALL IS INSTALLED IN LIFTS NO GREATER THAN 12" LOOSE LIFT THICKNESS.

2. COMPACTION TESTS SHALL BE TAKEN AS THE WALL IS INSTALLED AND BACKFILL IS PLACED. THE MINIMUM NUMBER OF TESTS SHALL BE DETERMINED BY THE GEOTECHNICAL ENGINEER.

3. COMPACTION SHALL BE TO A MINIMUM OF 95% MODIFIED PROCTOR DENSITY.

4. RECOMMENDED COMPACTION EQUIPMENT WITHIN 15 FEET OF THE BACK OF THE WALL IS AS FOLLOWS:

A) 0-4 FEET = HAND TAMP OR VIBRATORY PLATE COMPACTOR (WALK-BEHIND)

B) 5-15 FEET = NOTHING LARGER THAN A TWO-DRUM WALK-BEHIND VIBRATORY ROLLER (LARGER ROLLERS CAN BE USED STATICALLY, PROVIDED LIFT SIZE DOES NOT COMPROMISE ACHIEVEMENT OF NECESSARY COMPACTION RESULTS).

## LEGEND

| | |
|---|---|
| | BENCHMARK |
| 500 | EXISTING CONTOURS |
| | INDIVIDUAL TREE |
| UGE UGE | UNDERGROUND ELECTRIC |
| ℙ | PROPERTY LINE |
| ROW | RIGHT OF WAY |
| | TREE LINE |
| | UTILITY POLE |
| 1148 | PROPOSED 1' CONTOUR LINE |
| 1150 | PROPOSED 5' CONTOUR LINE |
| OHW OHW | OVERHEAD WIRE LINE |
| SB 114 | SOIL BORING |
| | ROADWAY GUARDRAIL |


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PROFESSIONAL SEAL

**NOT ISSUED FOR CONSTRUCTION**

| NO. | DATE | DESCRIPTION | BY | CK'D |
|---|---|---|---|---|
| | | | | |

367 BRIGGS HILL ROAD BRISTOL, VT 05443

BRIGGS HILL ROAD SLOPE FAULIRE AND ROADWAY REPAIR

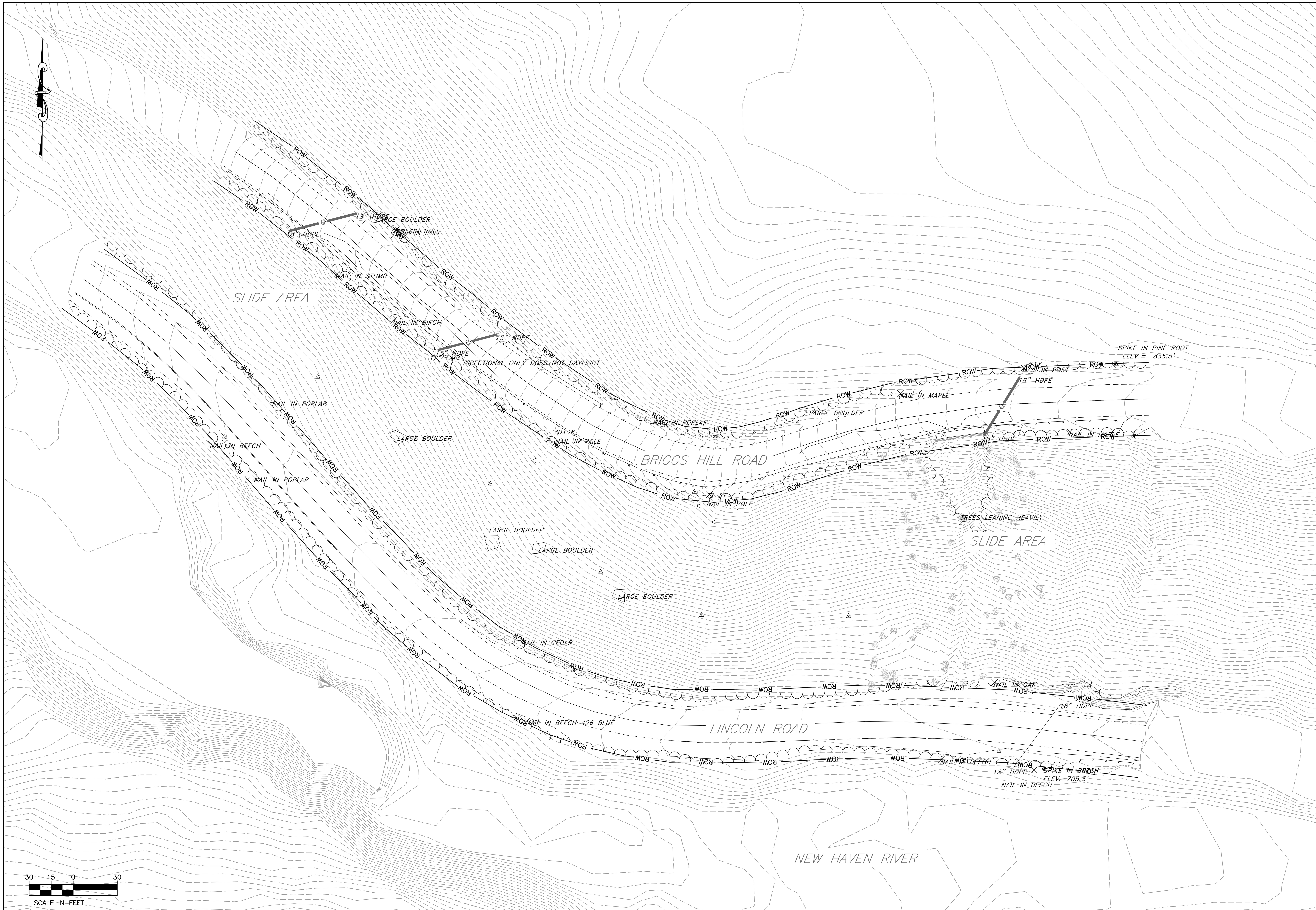
SHEET TITLE

GENERAL NOTES AND LEGEND

| | |
|---|---|
| DRAWN BY ZDB | DATE JULY 2024 |
| CHECKED BY KLL | D&K PROJECT # 229369 |
| PROJ. ENG. JBA | D&K ARCHIVE # |

SHEET NUMBER

G-1

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367 BRIGGS HILL ROAD BRISTOL, VT 05443

BRIGGS HILL ROAD SLOPE FAULIRE AND ROADWAY REPAIR

SHEET TITLE

EXISTING CONDITIONS

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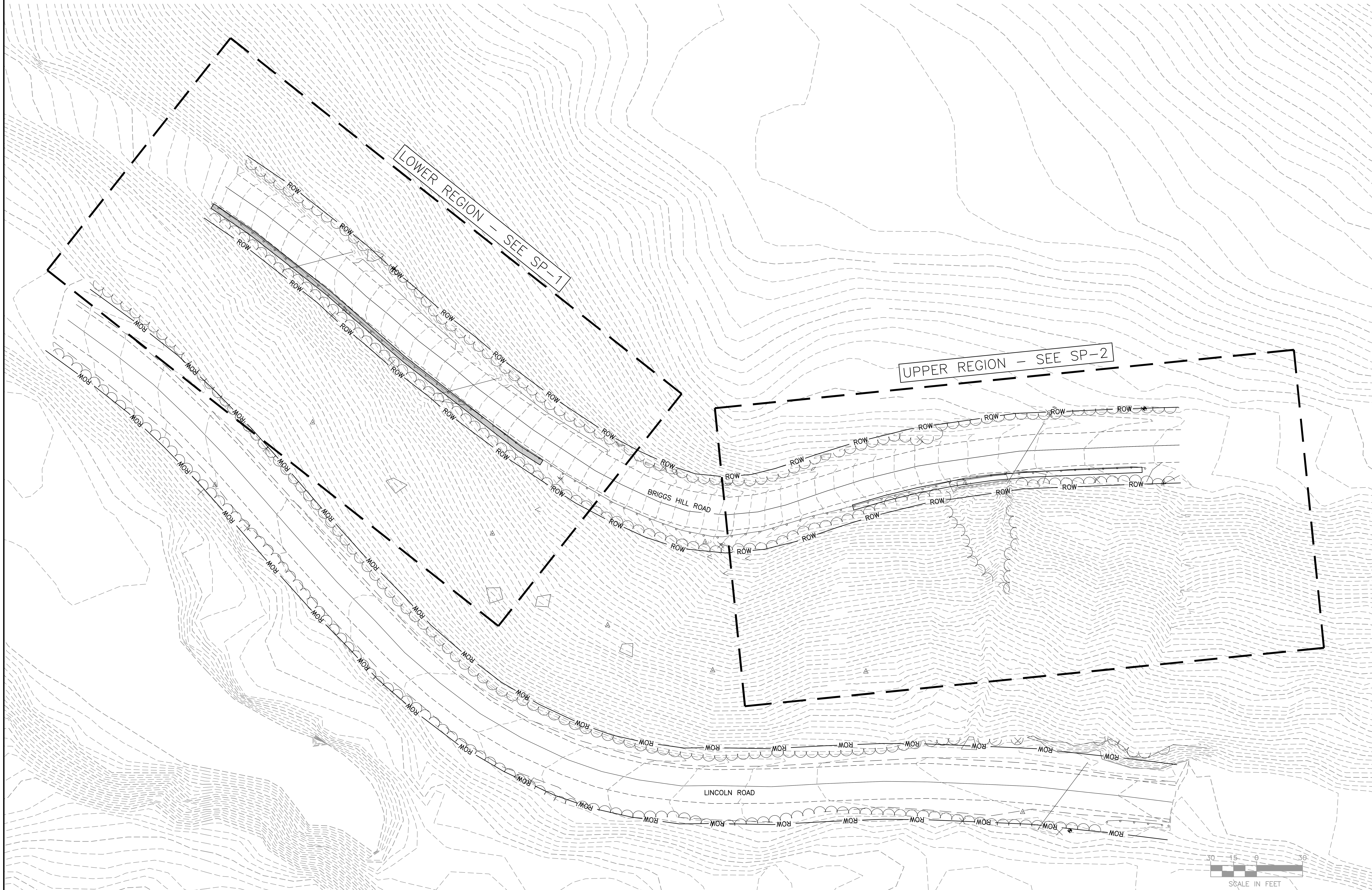
EX-1

SHEET 3 OF 29





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BRIGGS HILL ROAD SLOPE FAULIRE AND ROADWAY REPAIR

SHEET TITLE

OVERALL SITE PLAN

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SP



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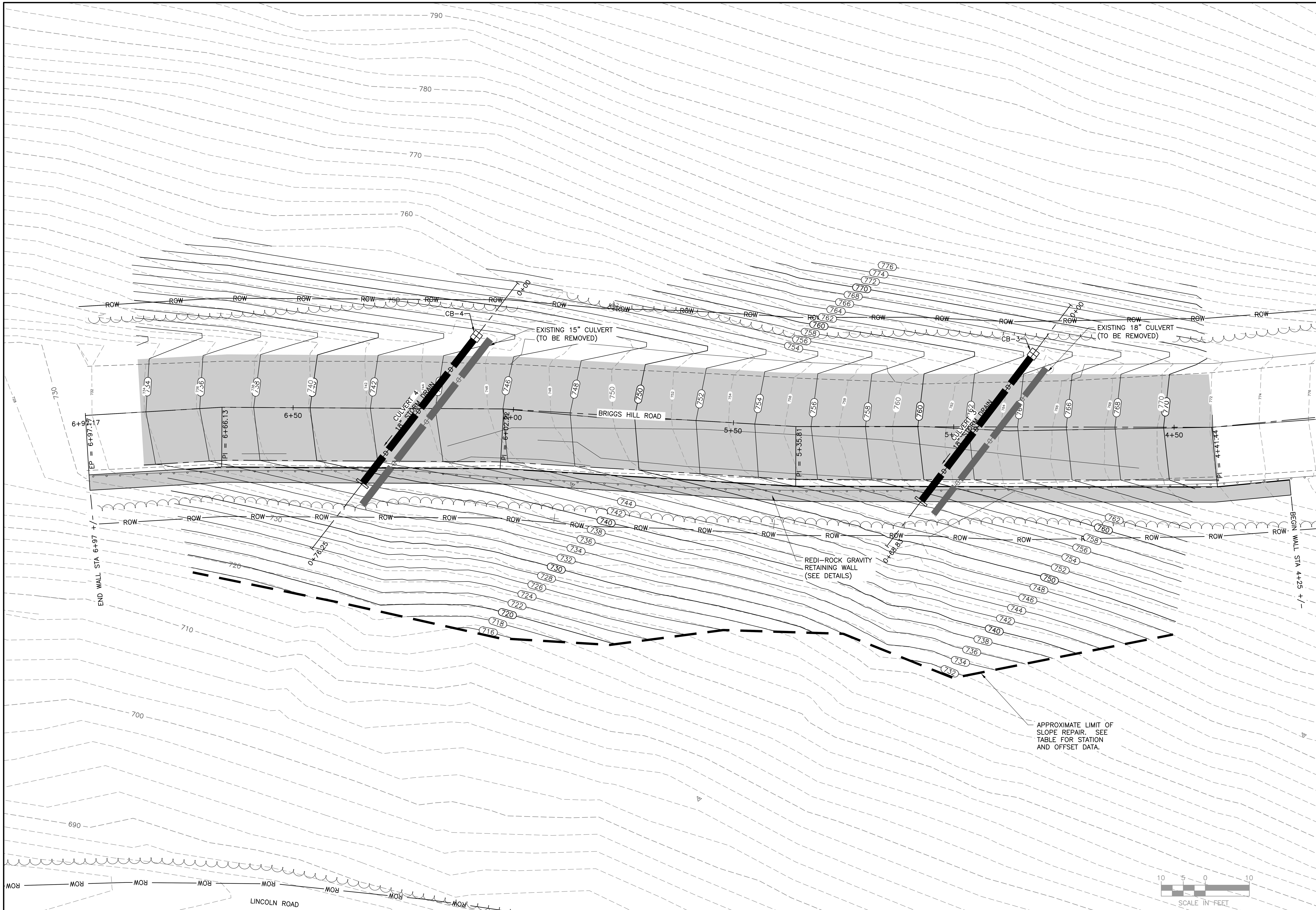


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367 BRIGGS HILL ROAD BRISTOL, VT 05443

BRIGGS HILL ROAD SLOPE FAULIRE AND ROADWAY REPAIR

SHEET TITLE

ROADWAY AND SITE PLAN (LOWER REGION)

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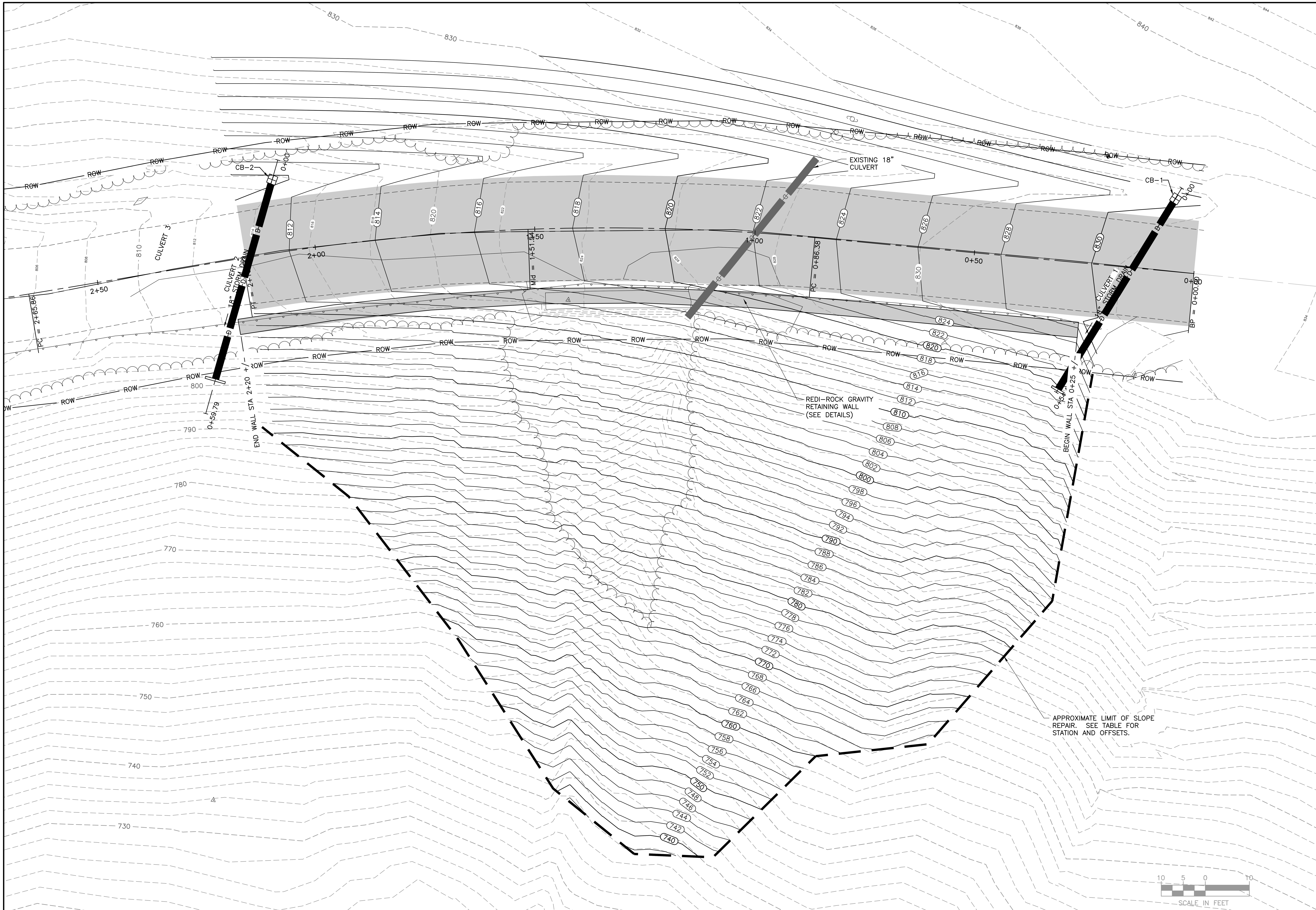


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SHEET TITLE

ROADWAY AND SITE PLAN (UPPER REGION)

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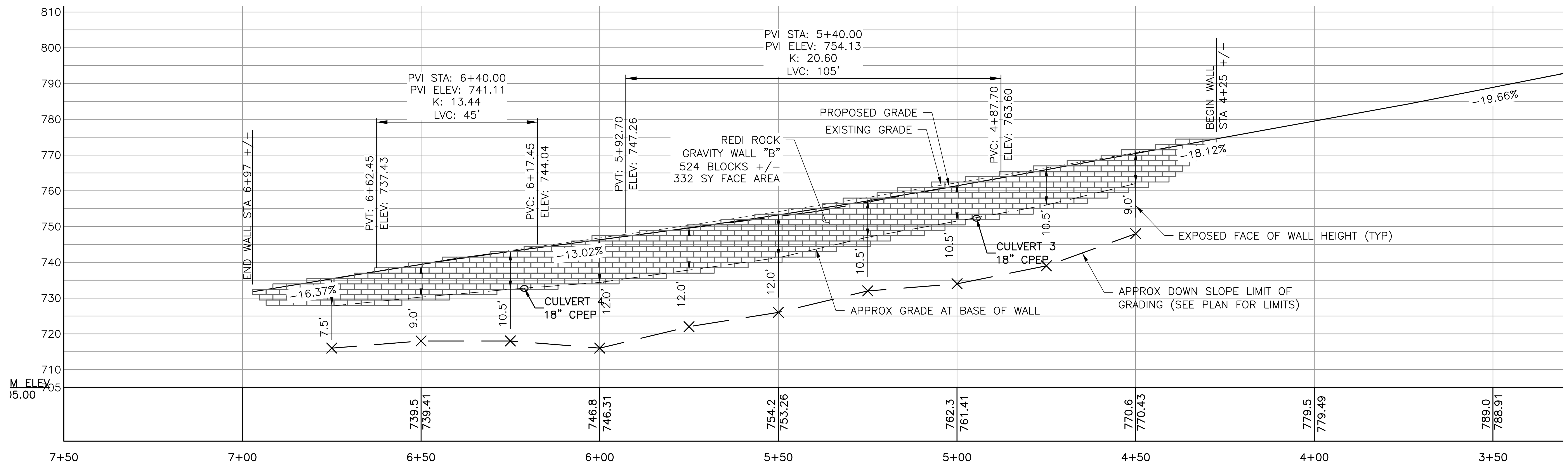
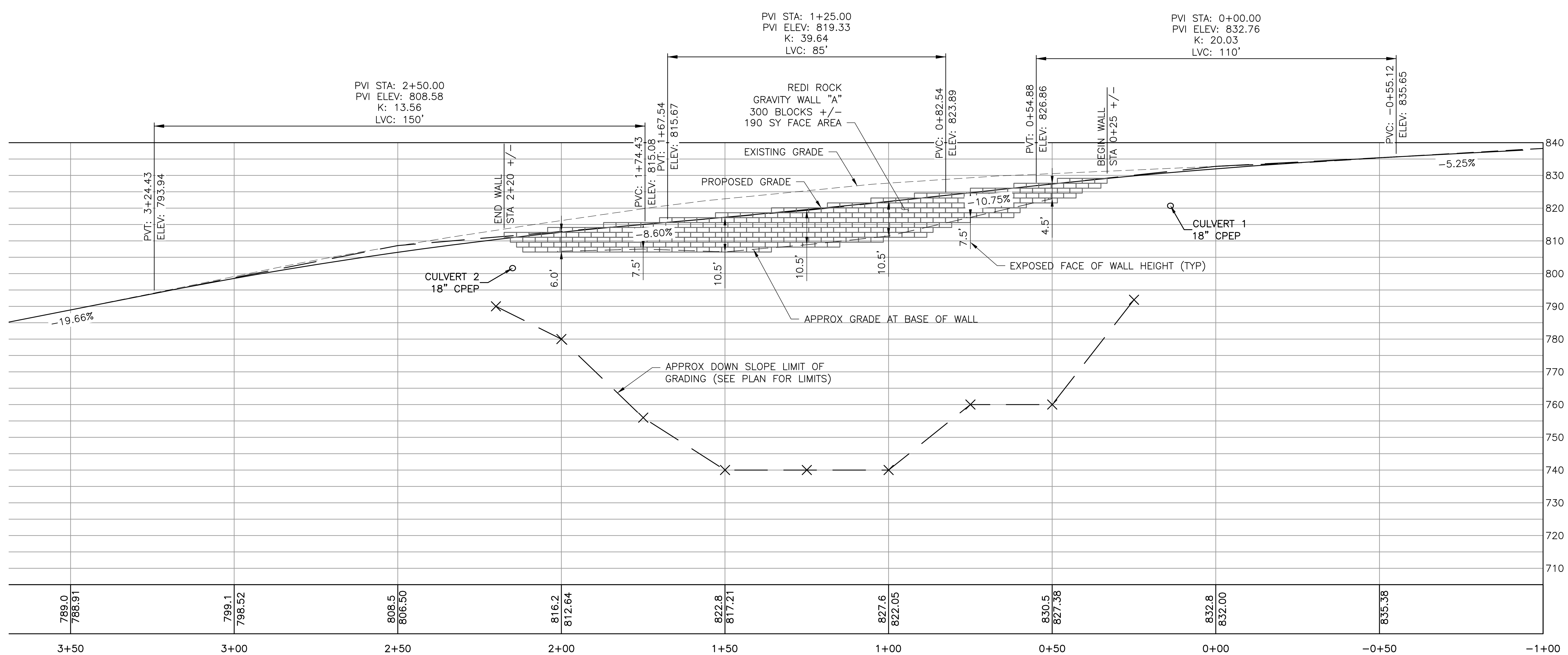
SP-2




ROADWAY PROFILE
(LOWER REGION)

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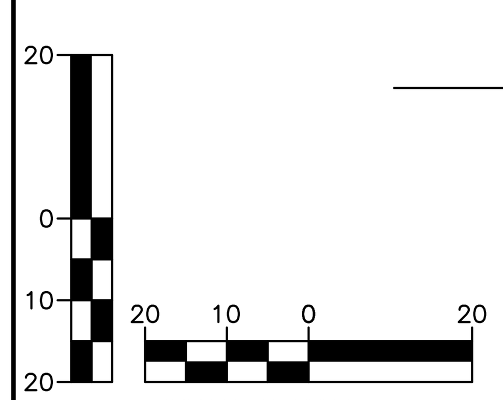
(UPPER REGION)

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SHEET TITLE

ROADWAY PROFILES SHOWING GRAVITY WALL ELEVATION VIEWS

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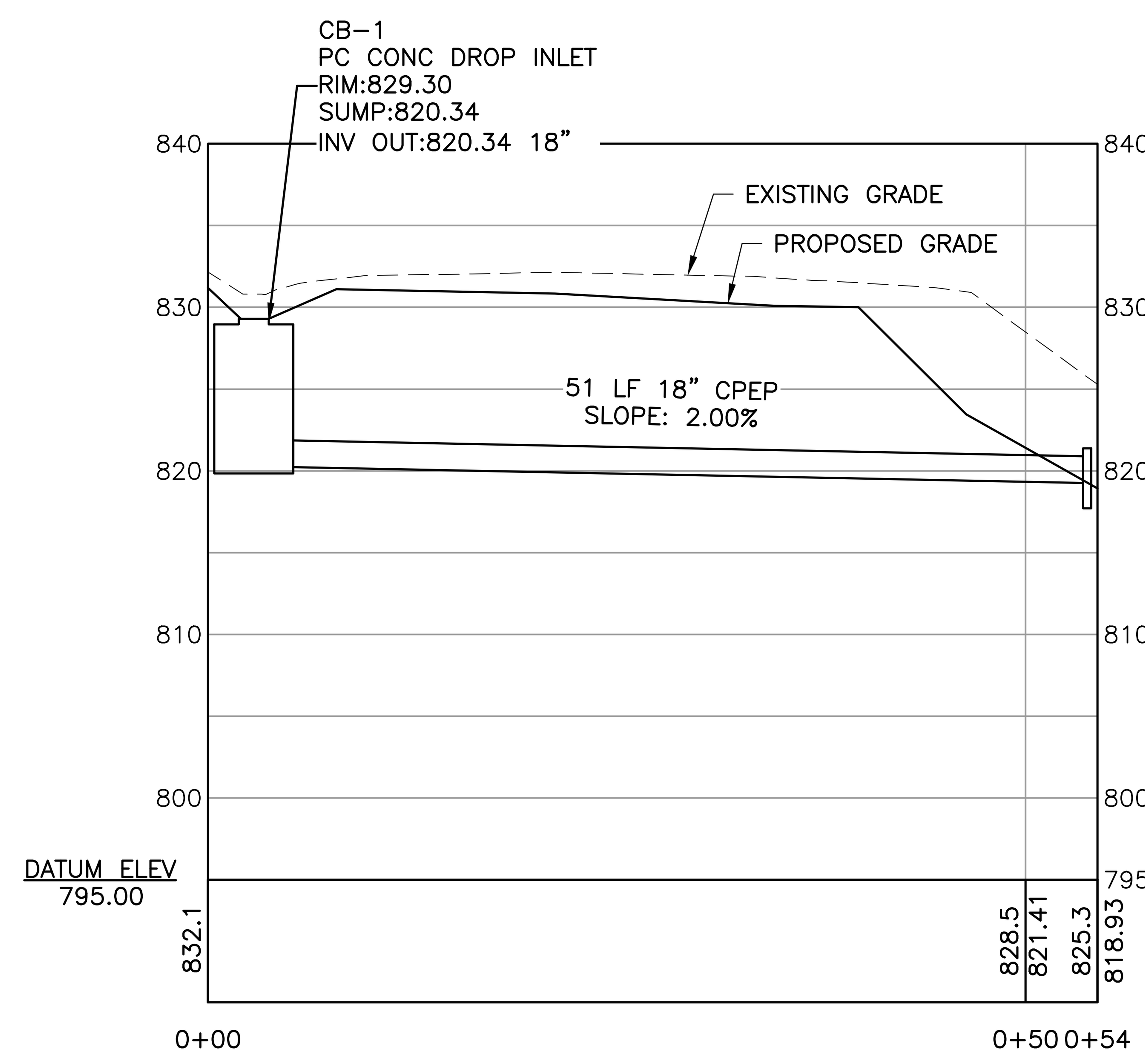
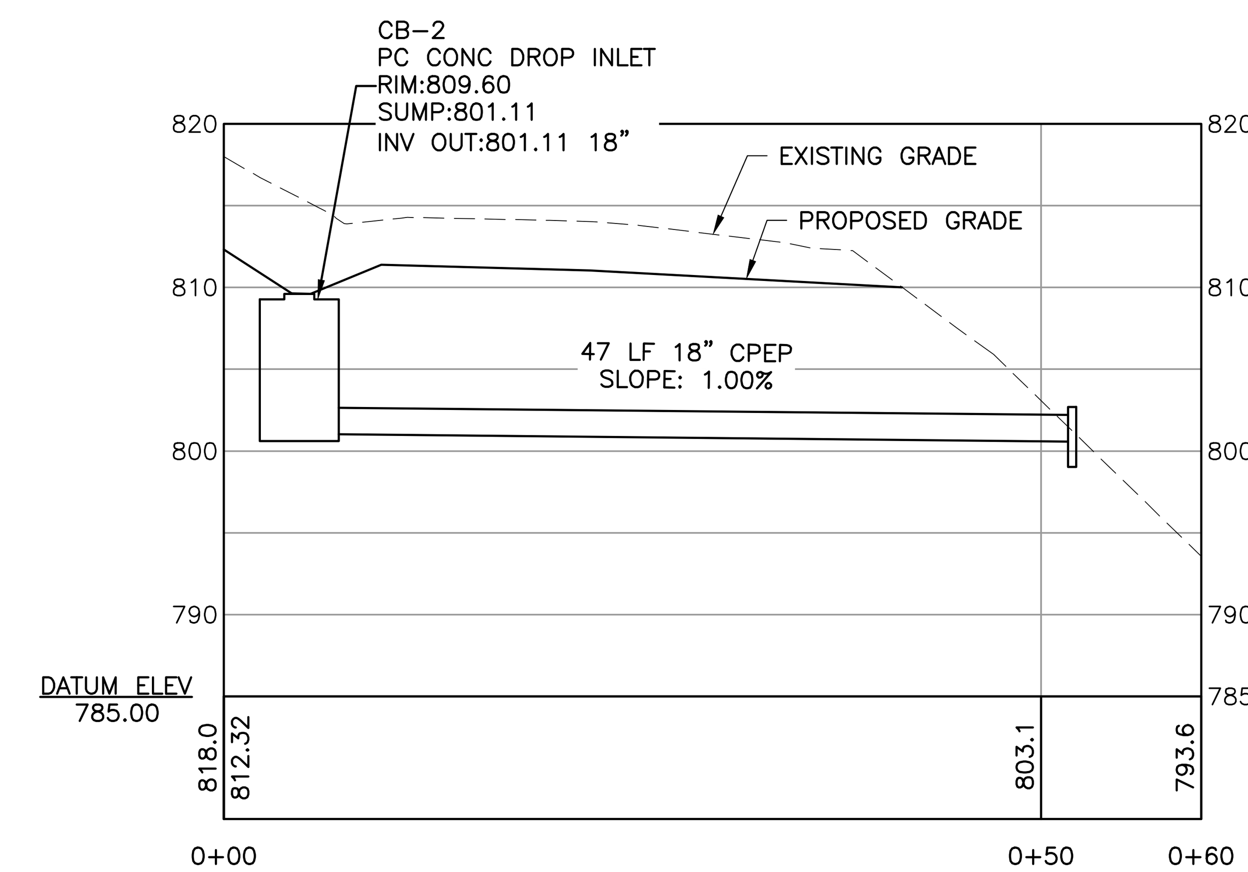



CULVERT 1 PROFILE

SCALE: 1"=10'




CULVERT 2 PROFILE

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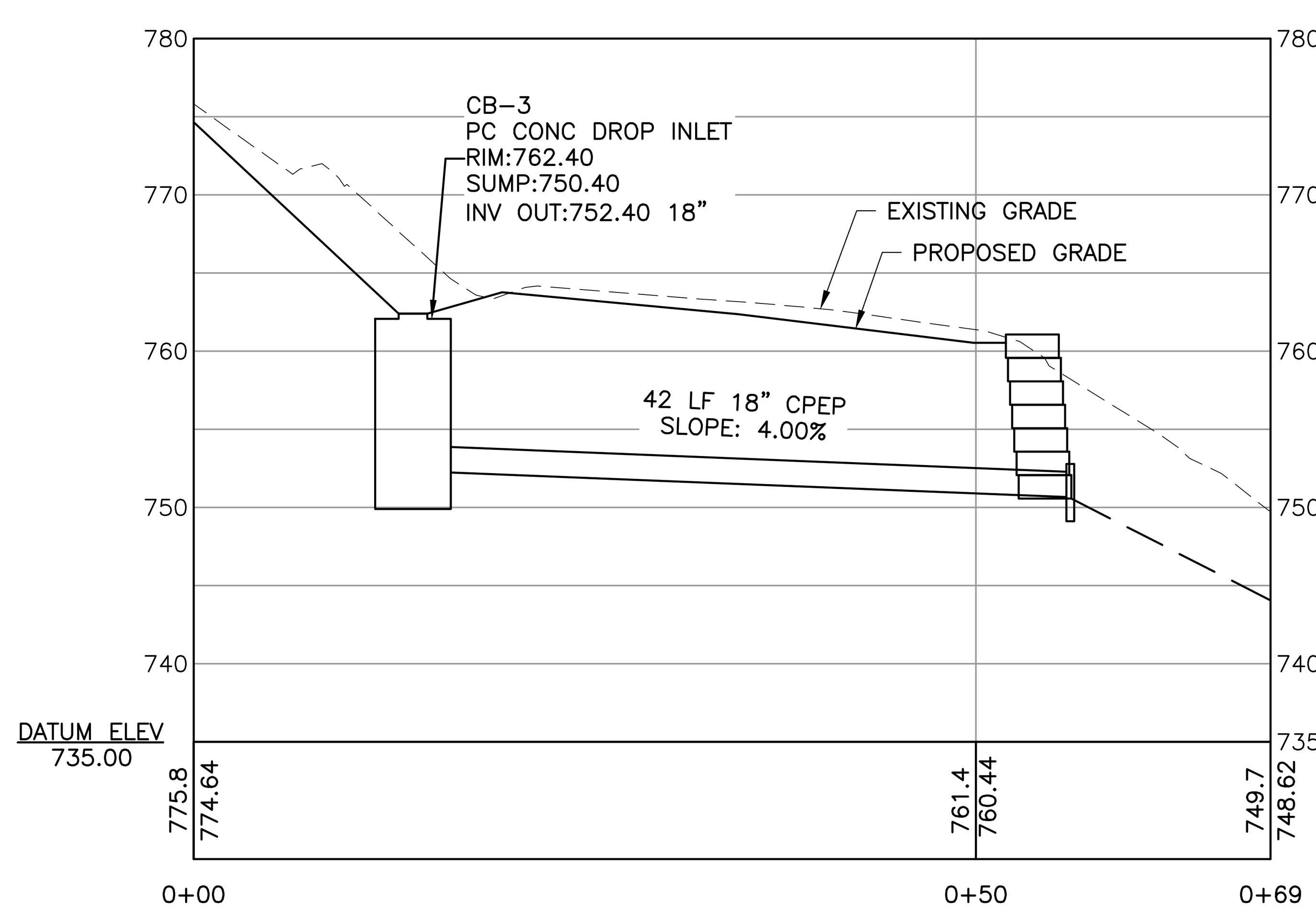


CULVERT 3 PROFILE

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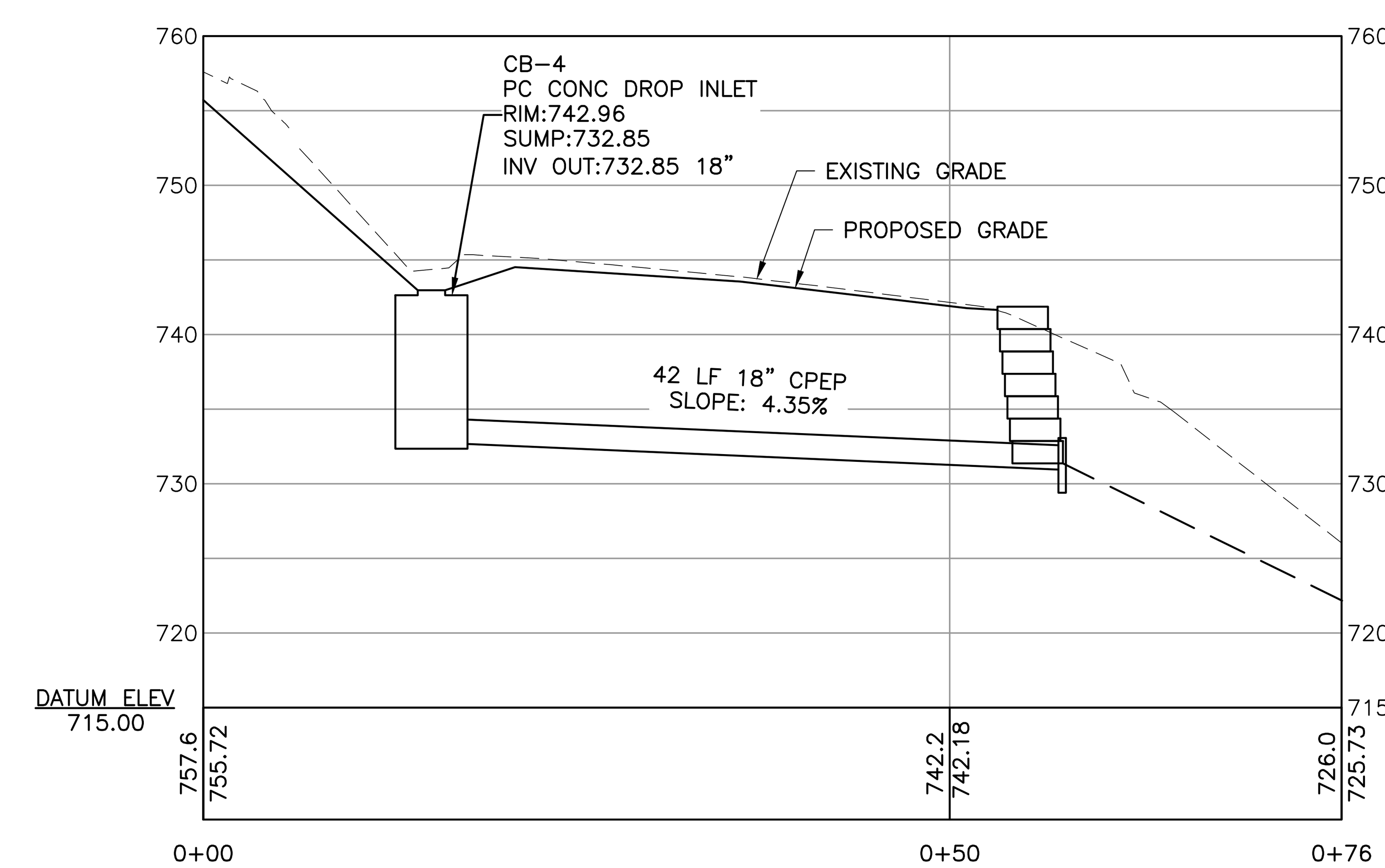


CULVERT 4 PROFILE

SCALE: 1"=10'

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CULVERT PROFILES

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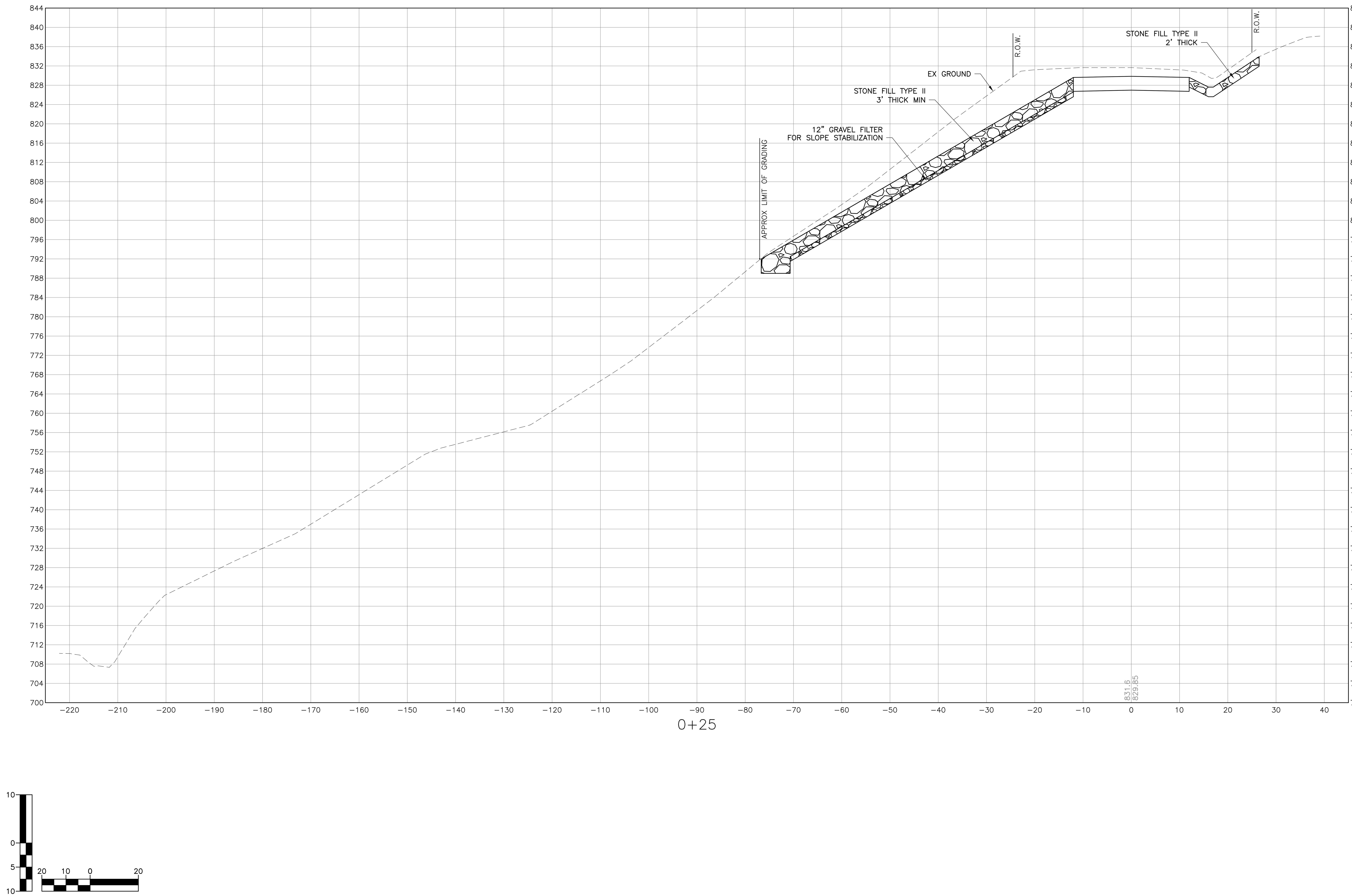



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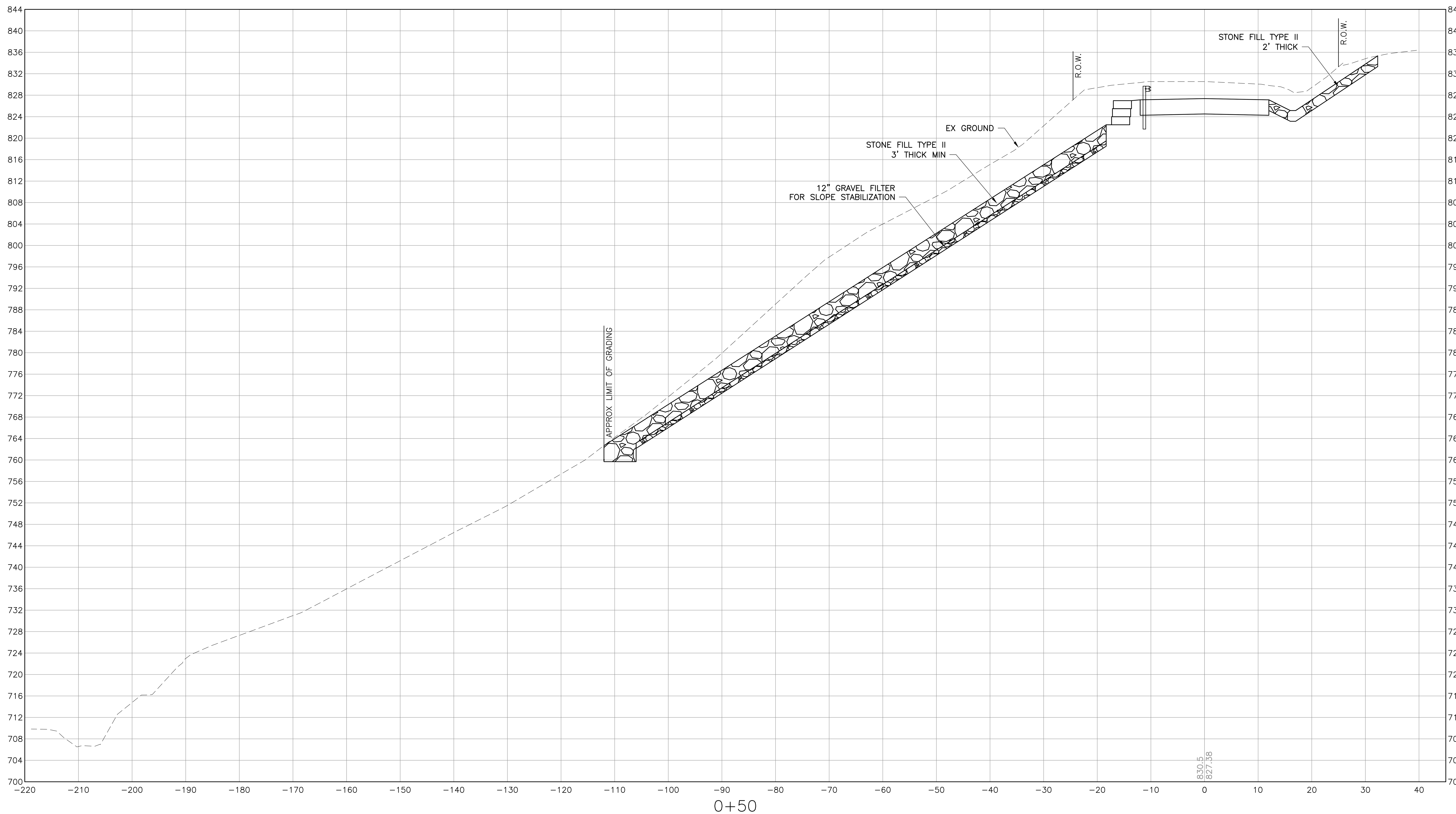


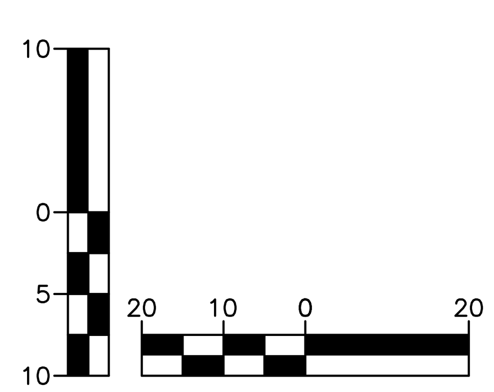




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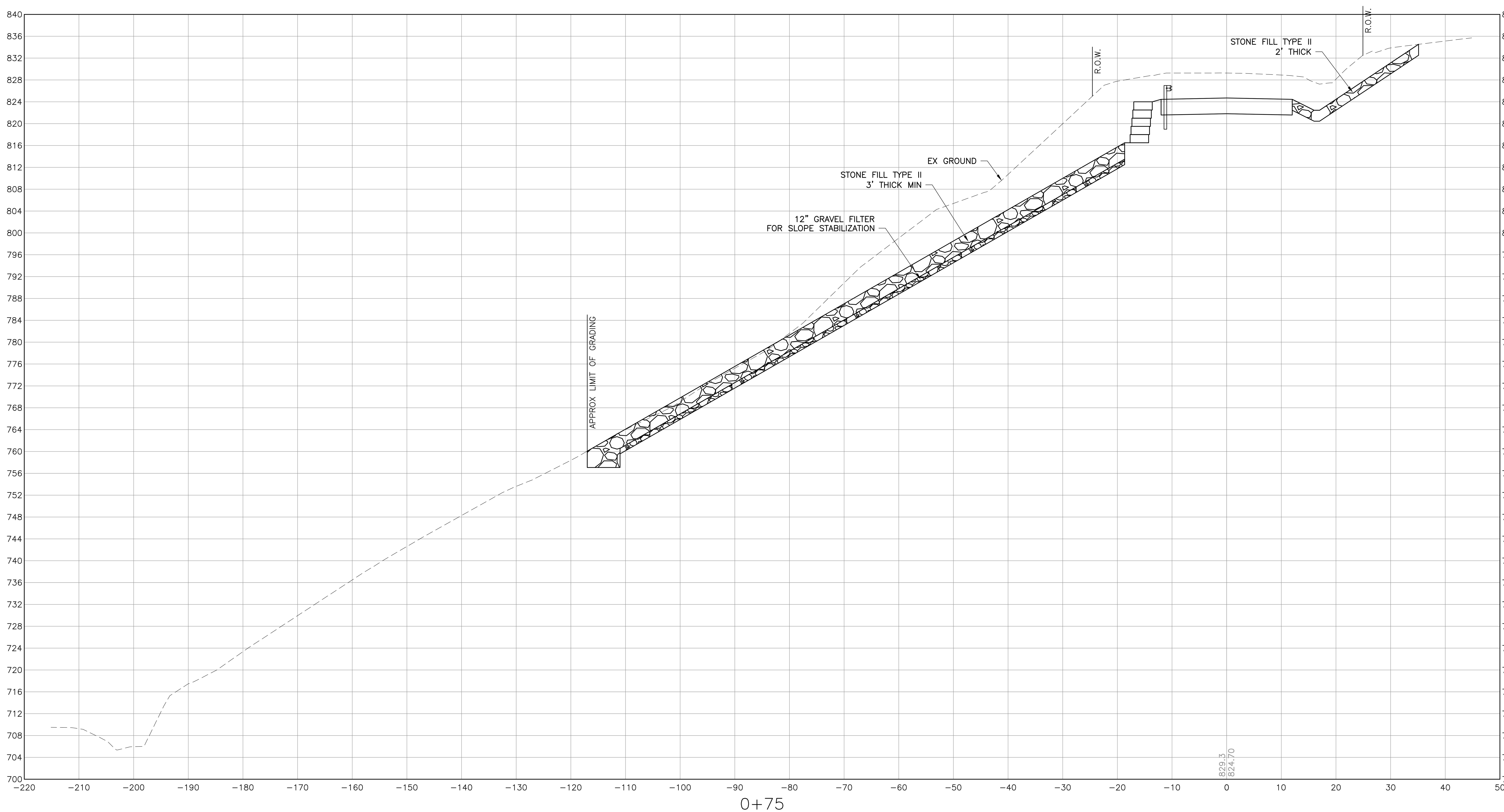


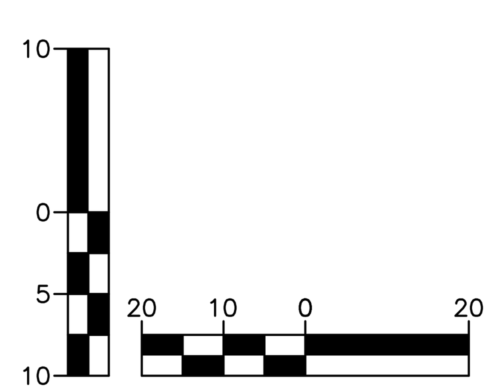


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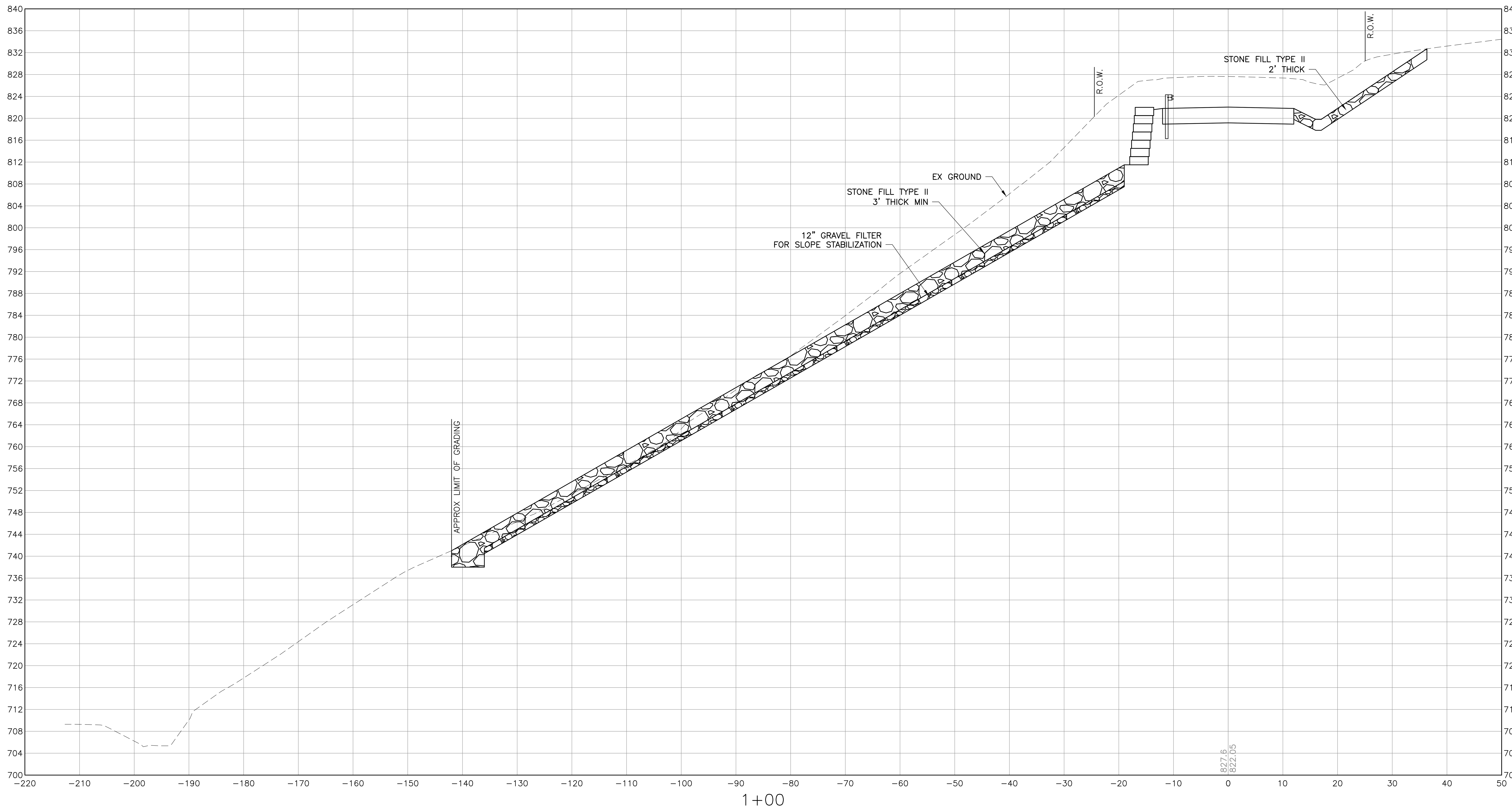


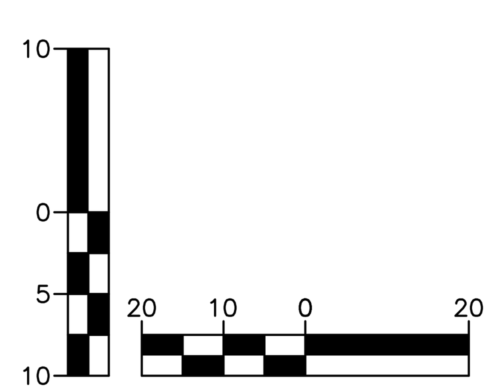



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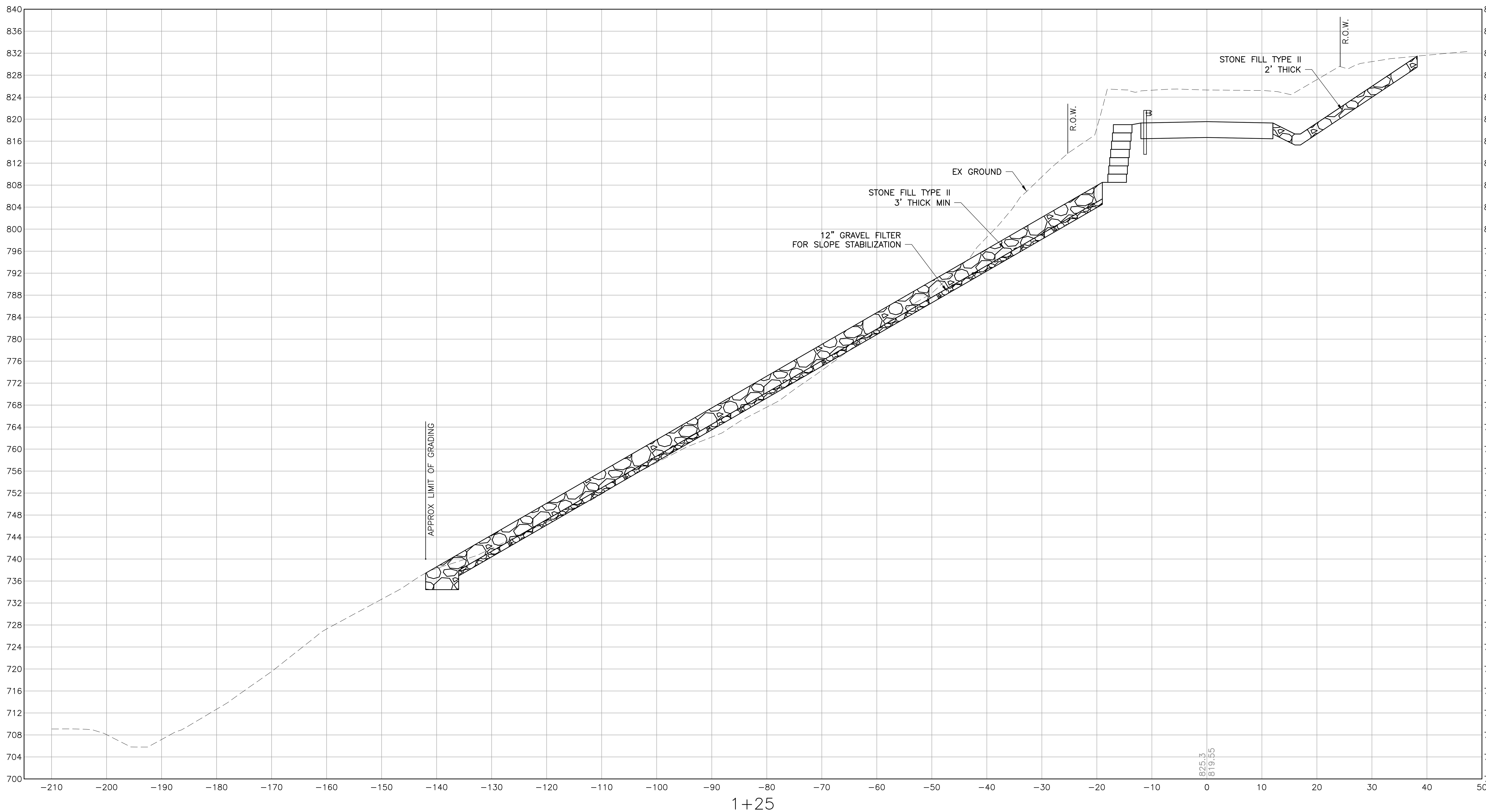


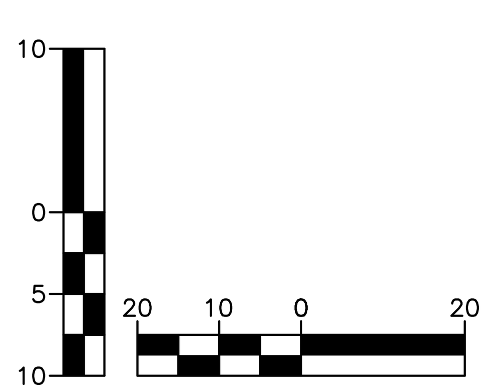


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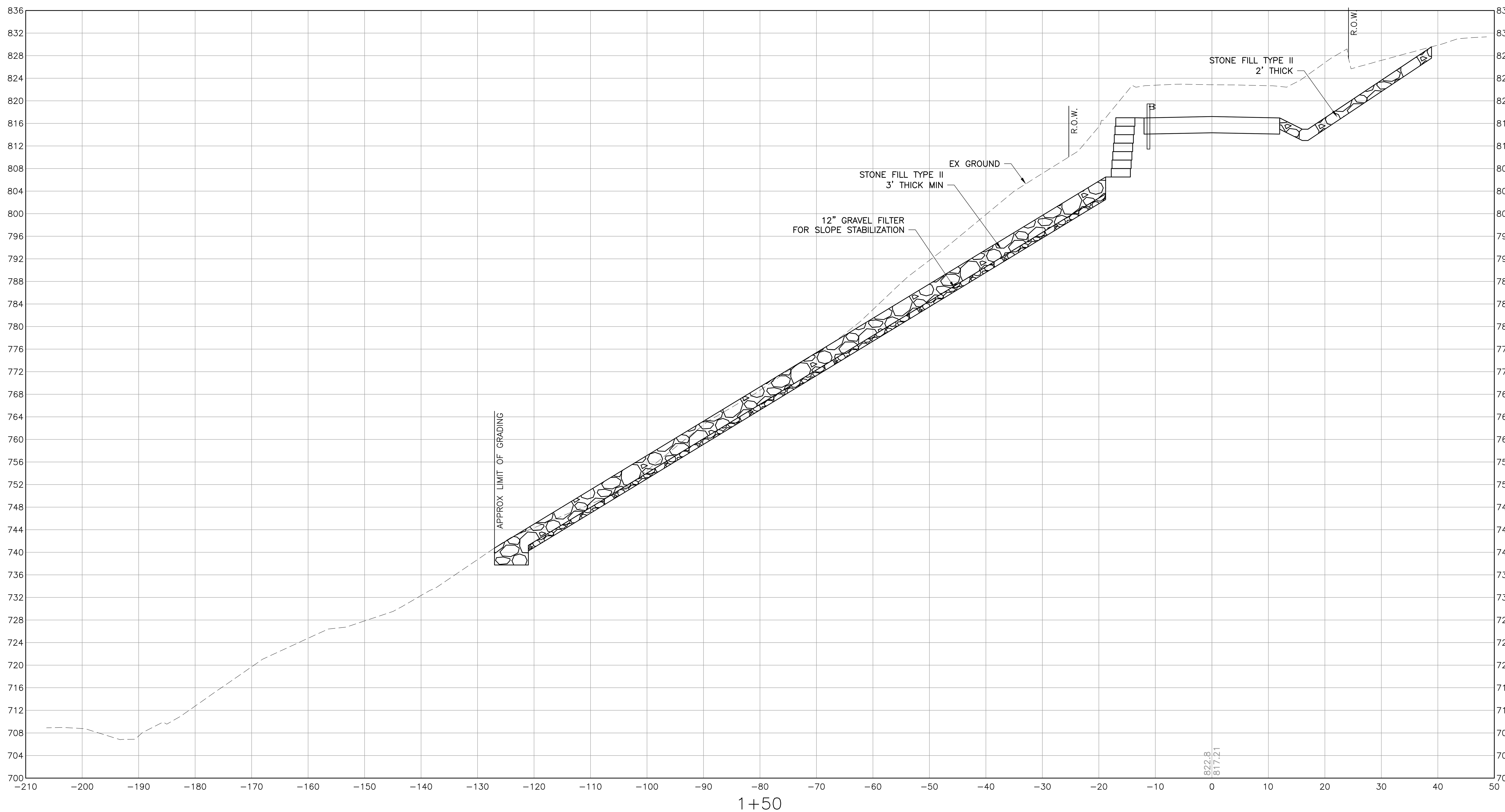


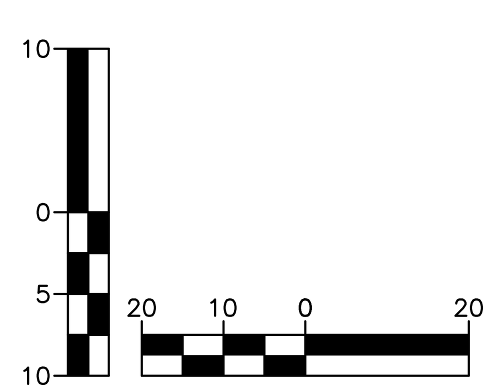


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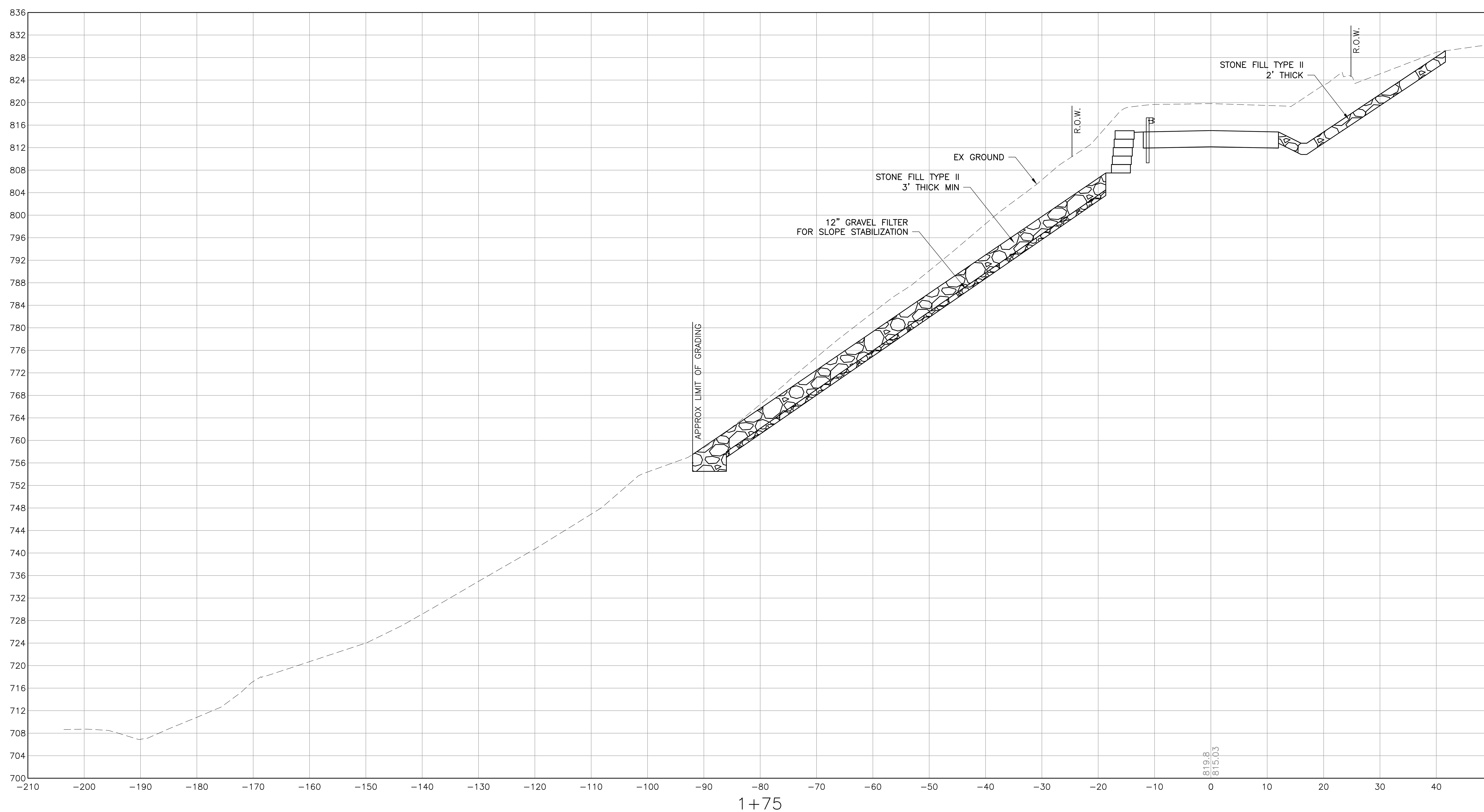


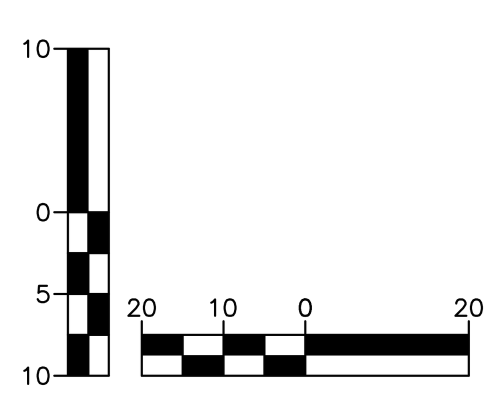


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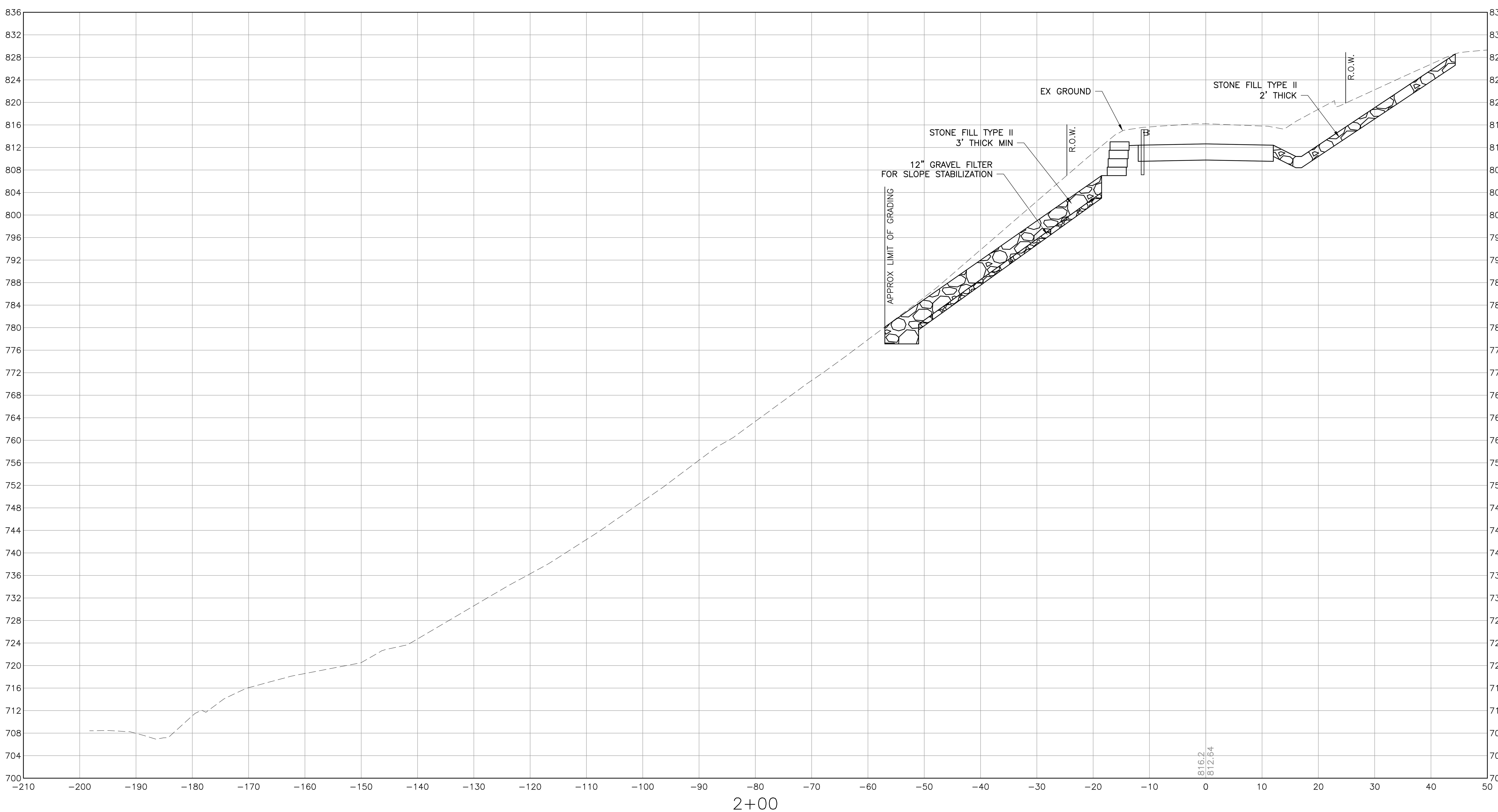
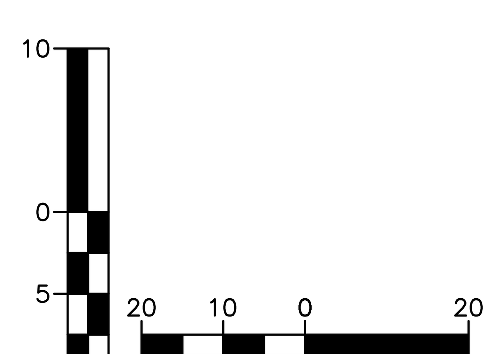
SHEET 15 OF 29





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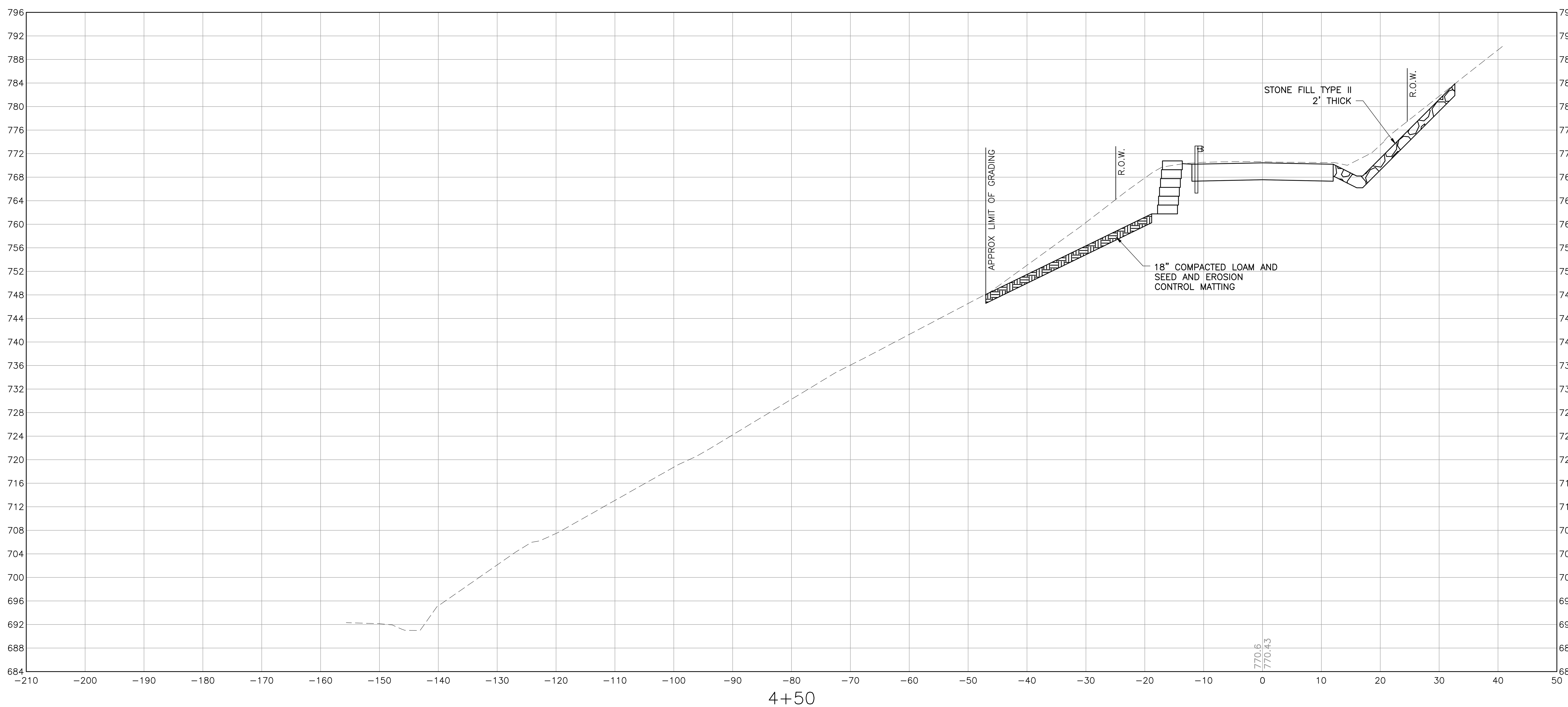


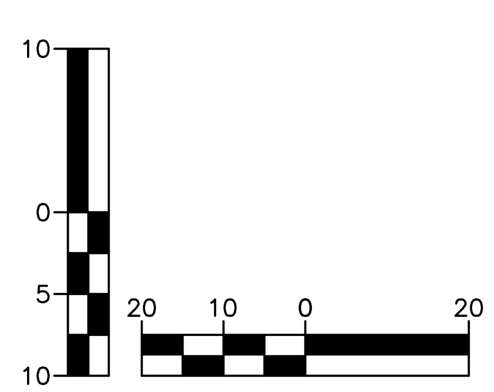



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SHEET 17 OF 29

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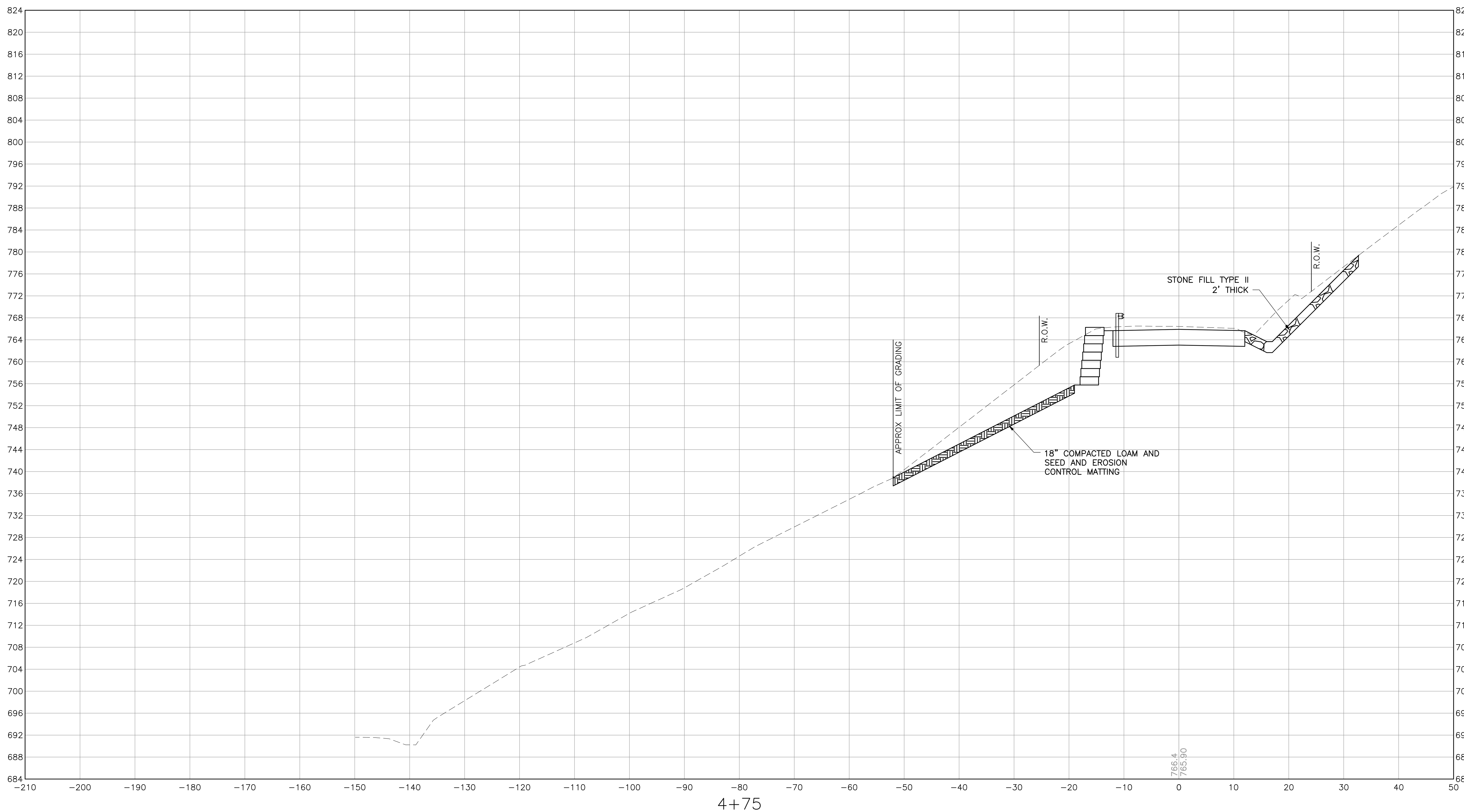


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|---|---|---|---|---|

367 BRIGGS HILL ROAD BRISTOL, VT 05443

BRIGGS HILL ROAD SLOPE FAULIRE AND ROADWAY REPAIR

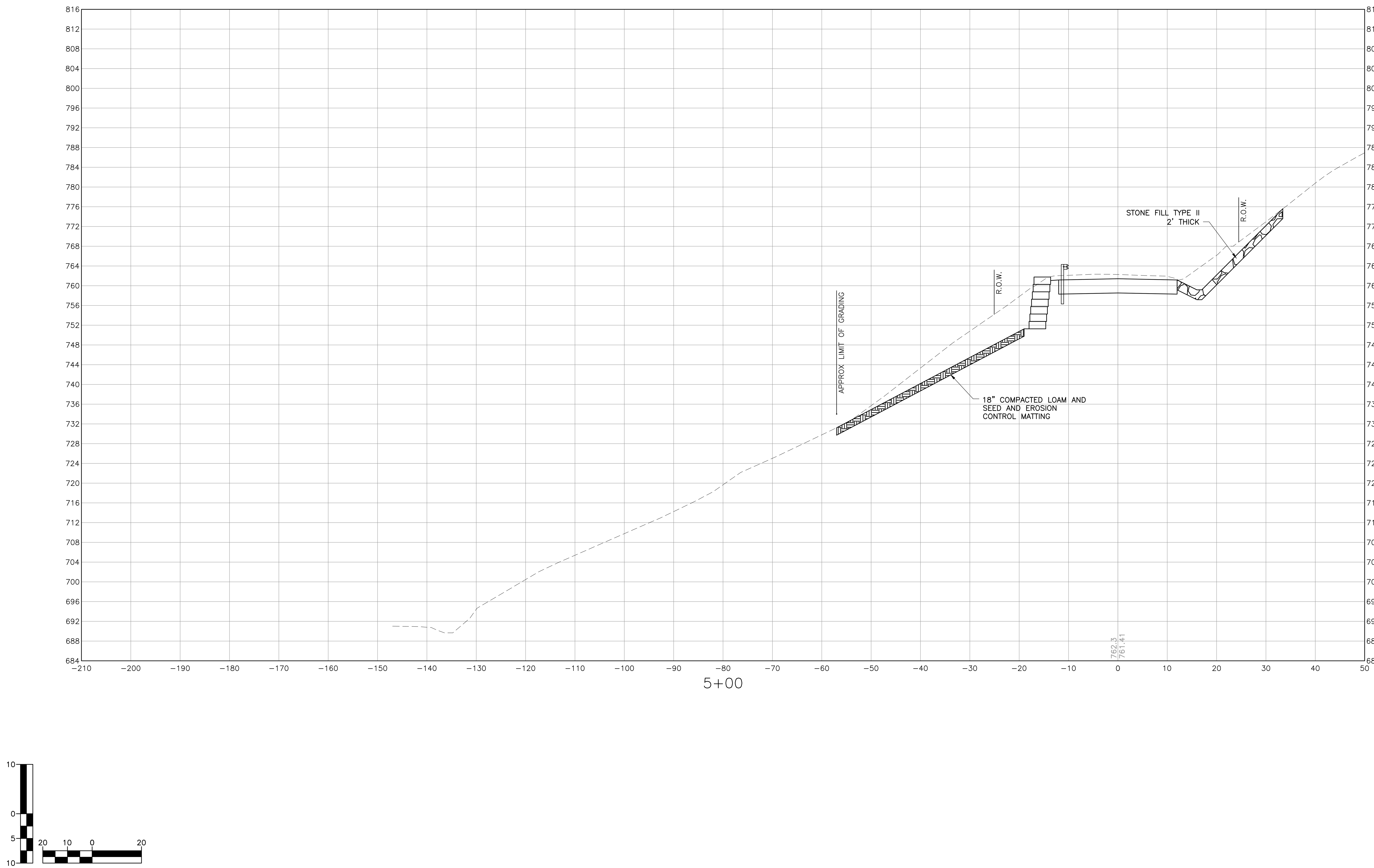
SHEET TITLE

CROSS SECTIONS

| | |
|---|---|
| DRAWN BY KLL | DATE JULY 2024 |
| CHECKED BY | D&K PROJECT # 229369 |
| PROJ. ENG. JBA | D&K ARCHIVE # |

SHEET NUMBER

XS-10





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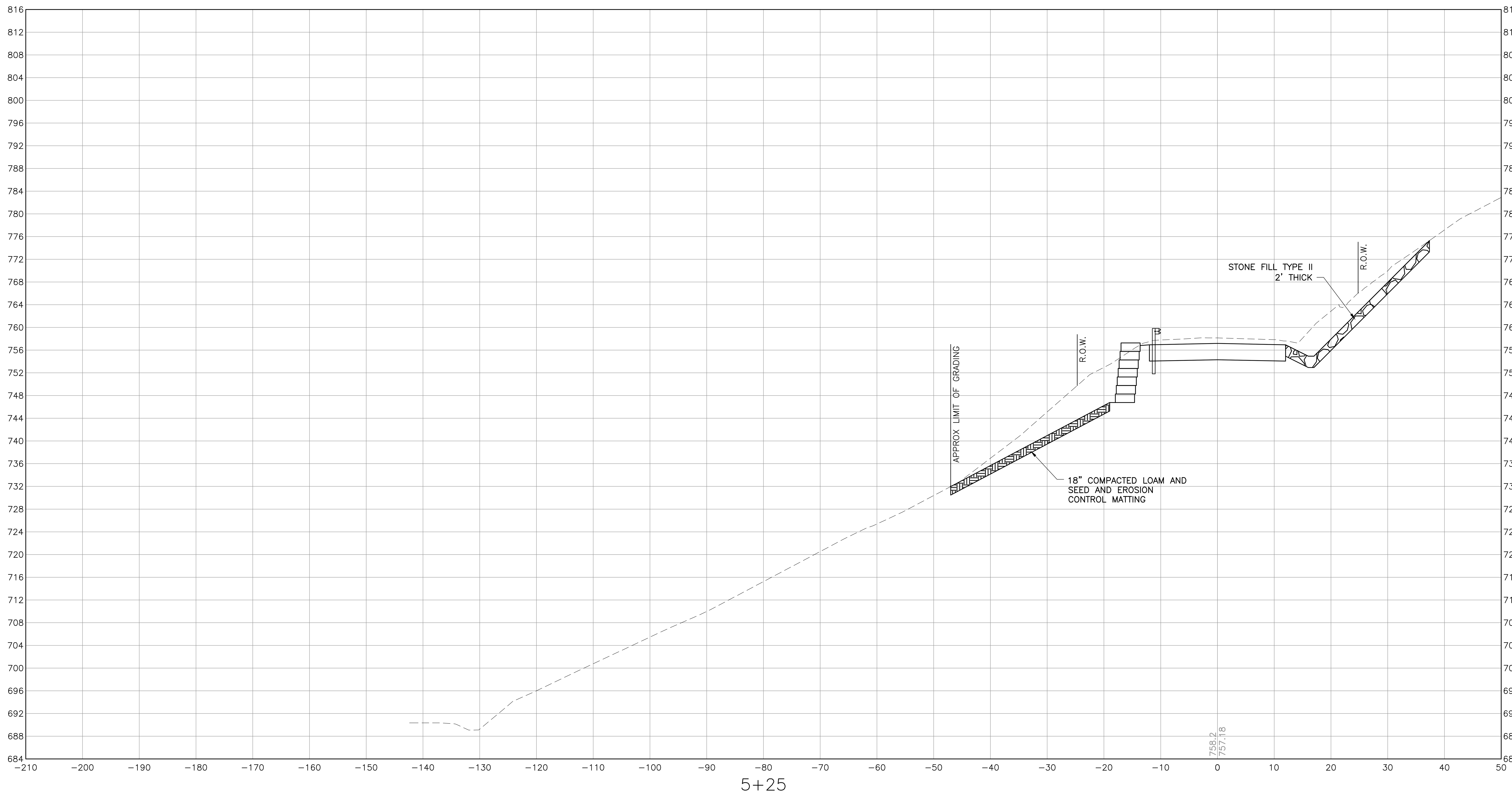
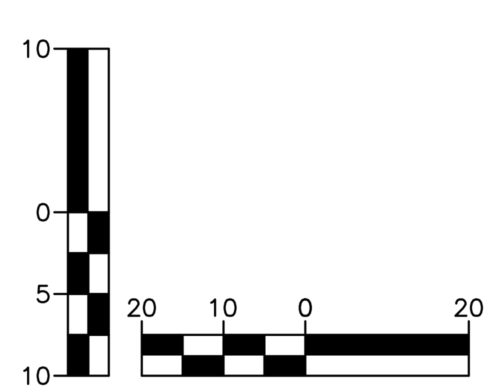
SHEET NUMBER

# XS-11

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SHEET TITLE

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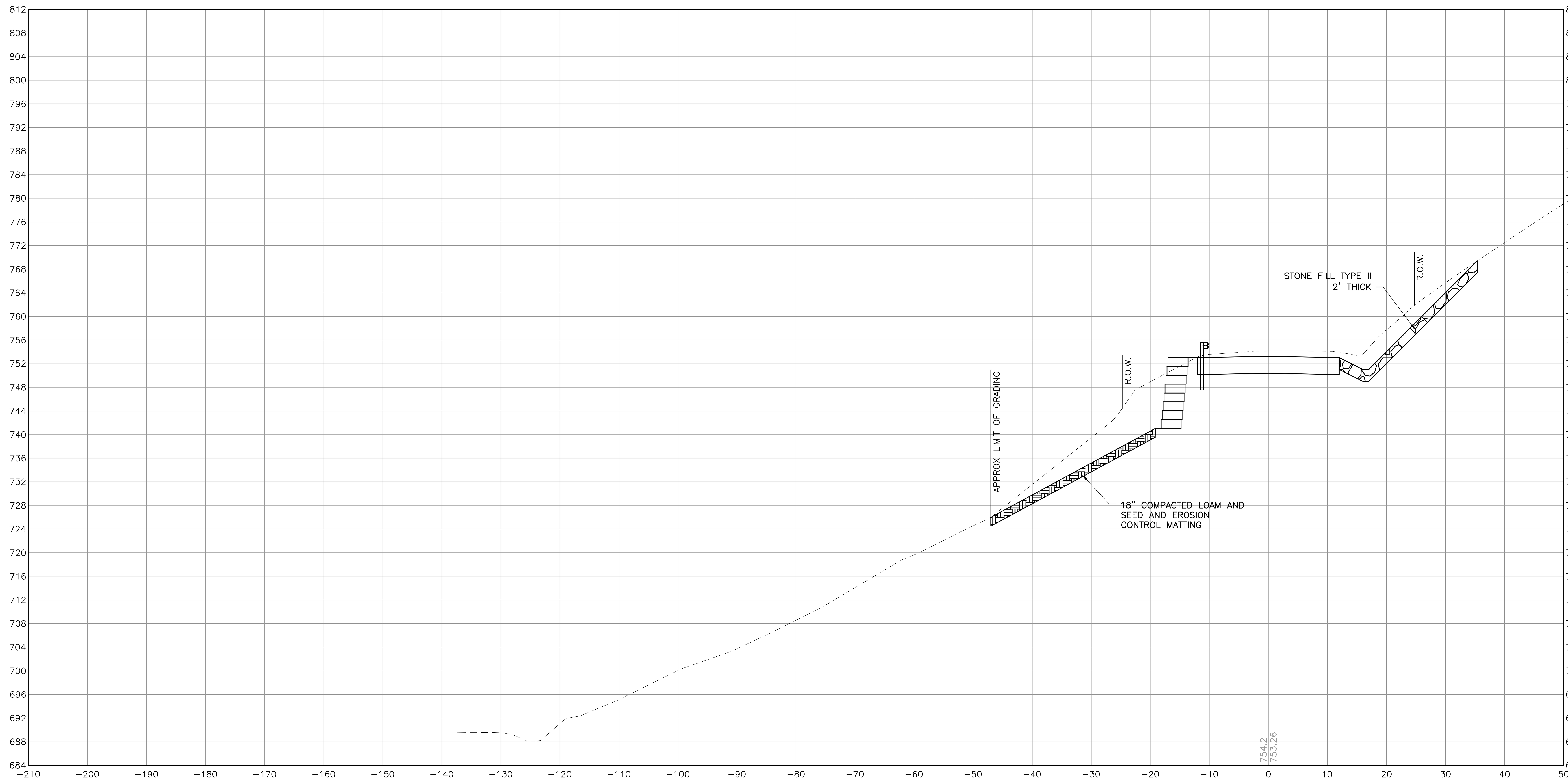
# XS-12




5+50

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| JBA | |

SHEET NUMBER

# XS-13



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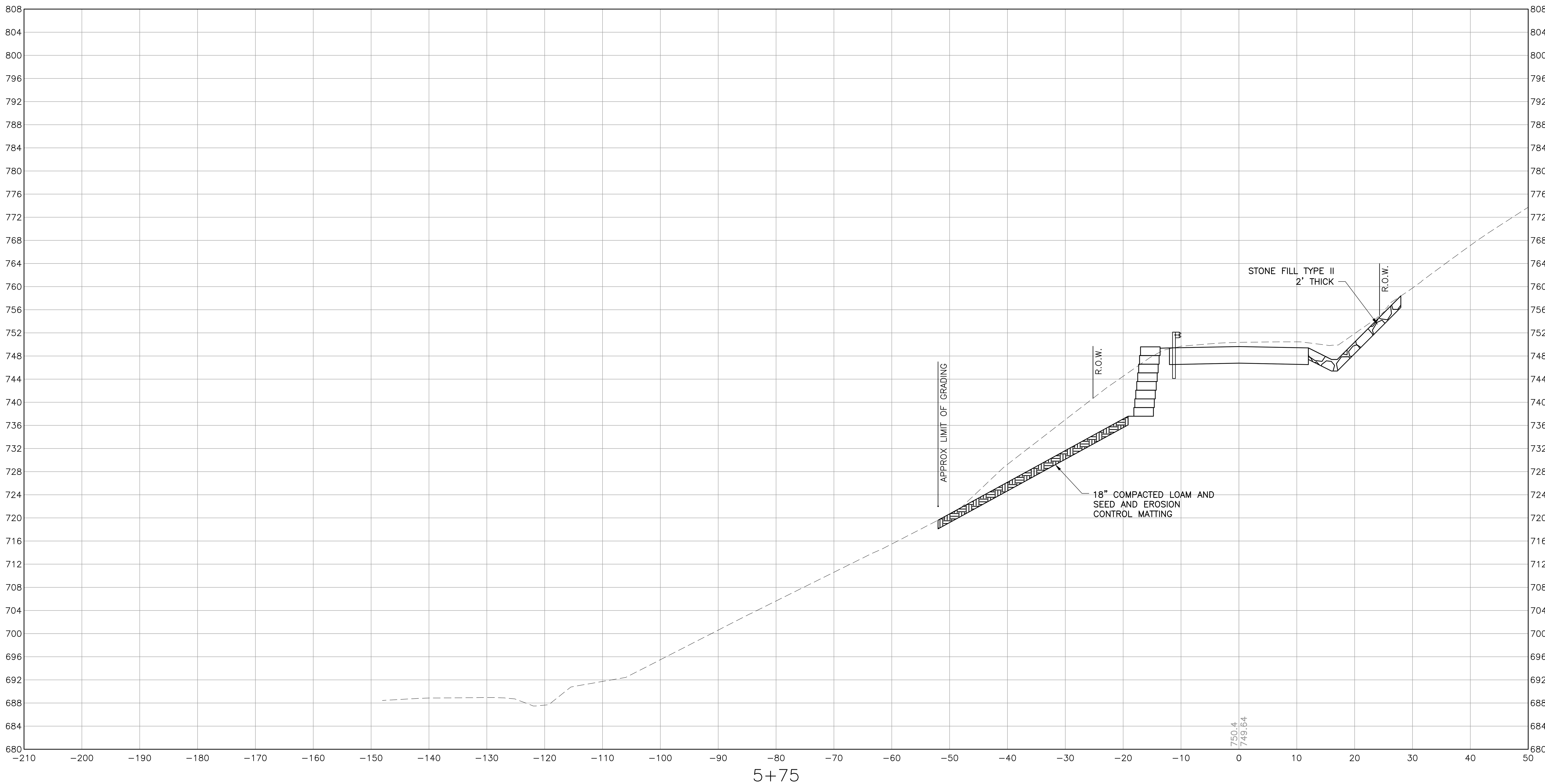


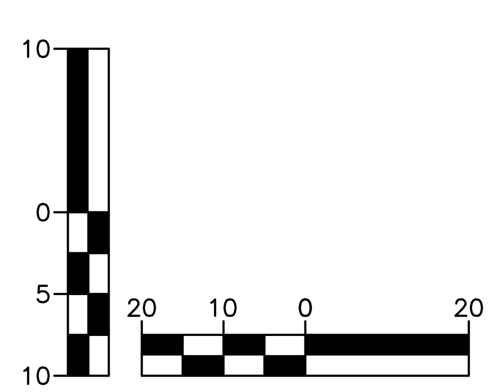


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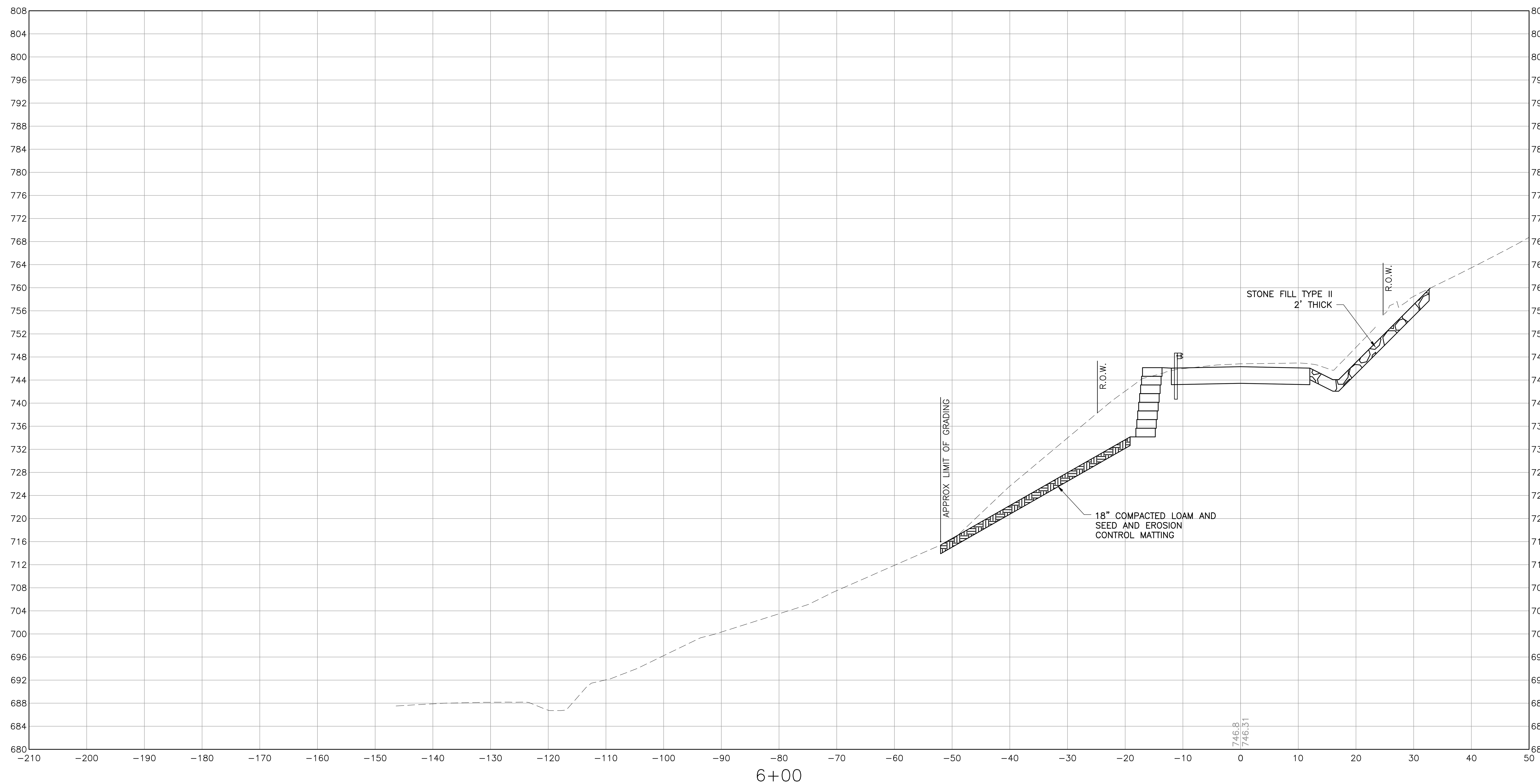


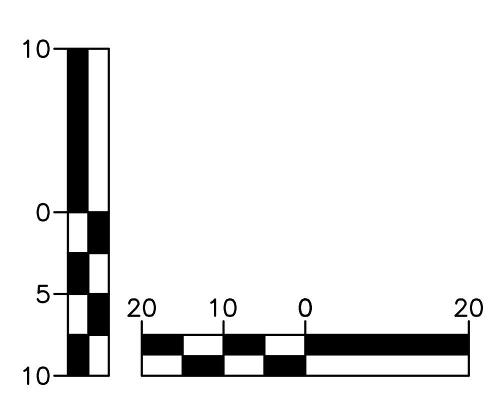


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SHEET TITLE

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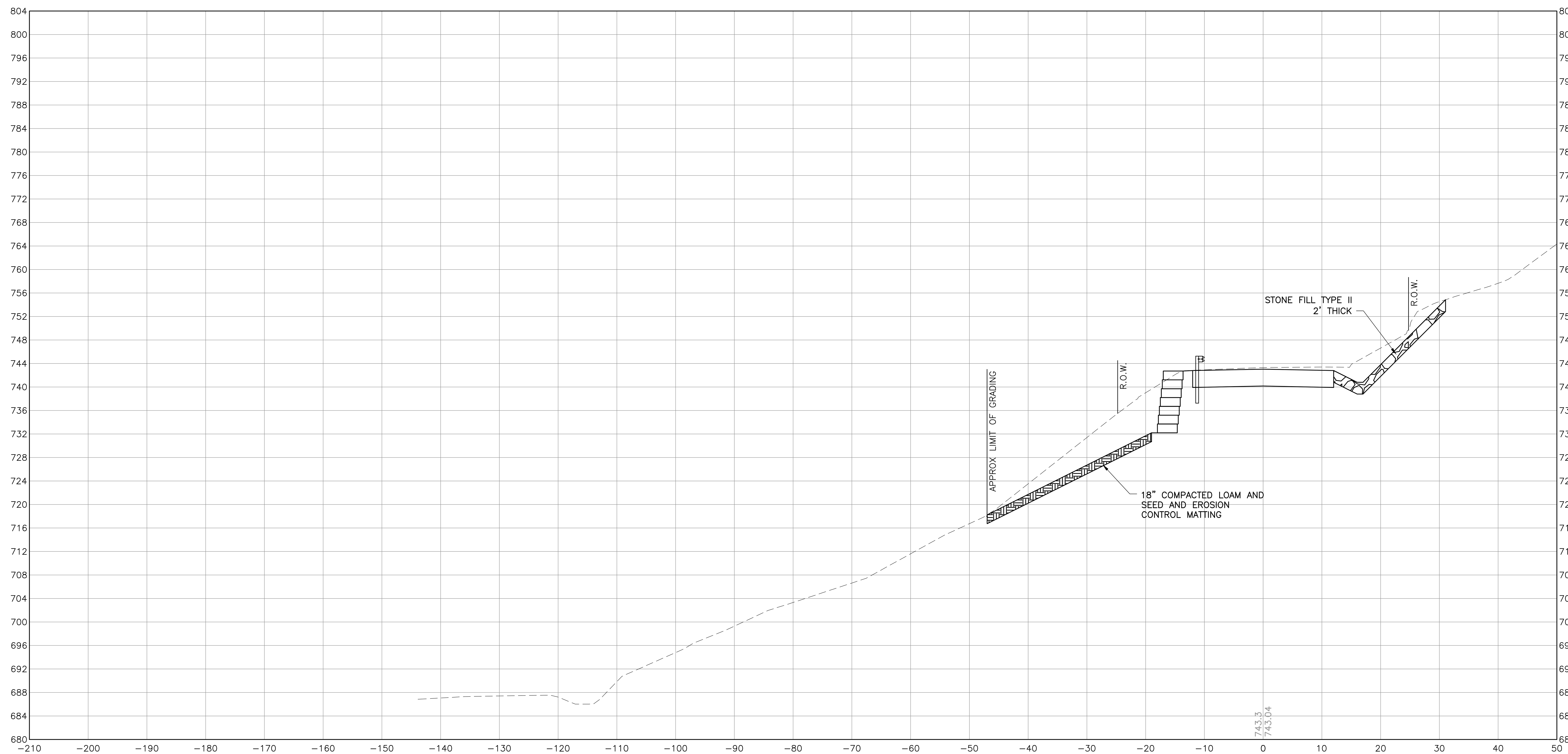
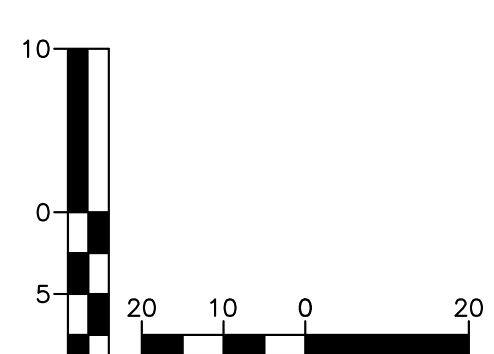
SHEET NUMBER

# XS-15




6+25

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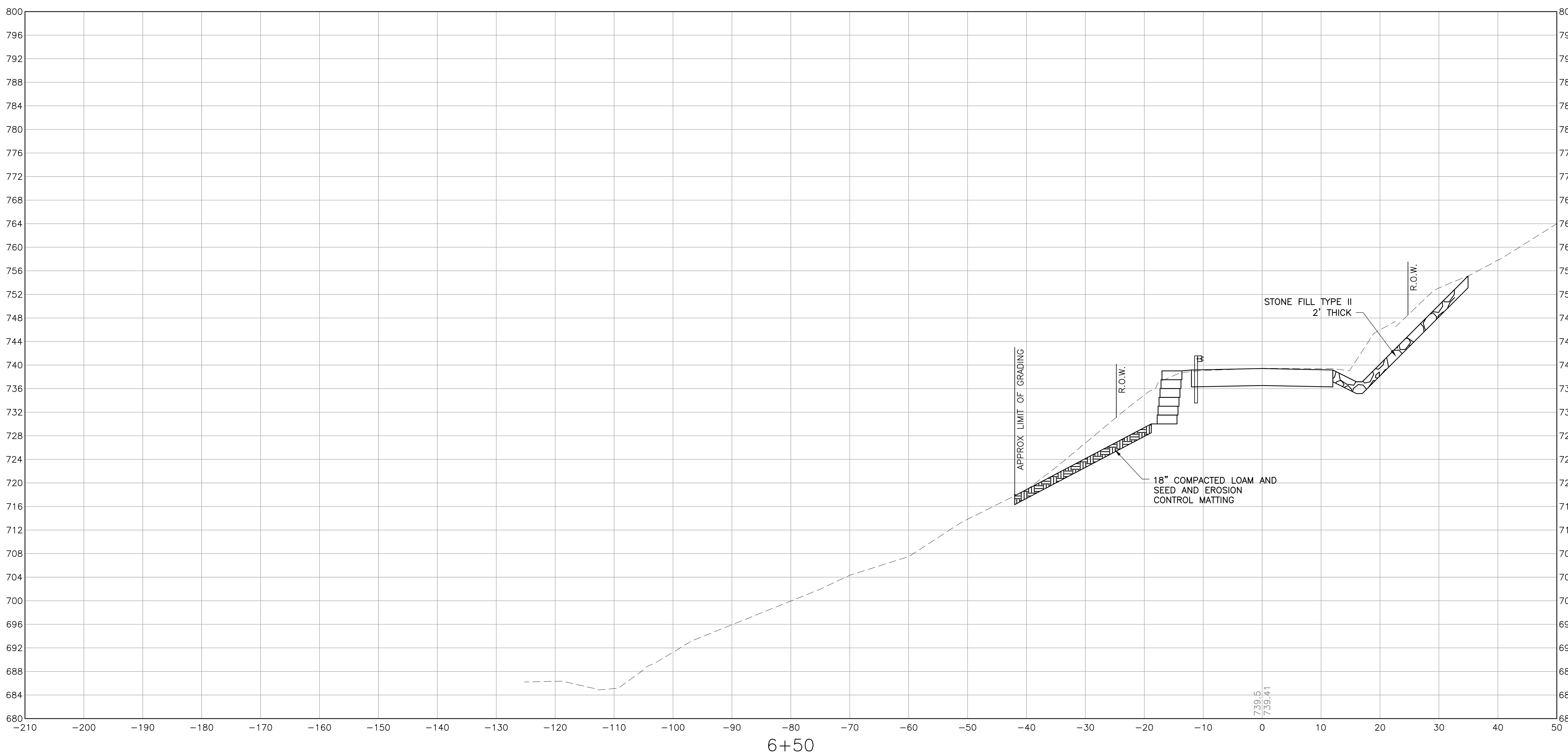
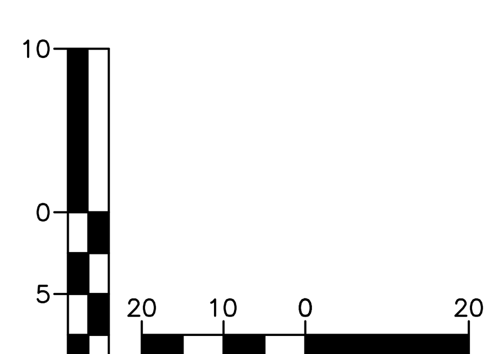
XS-16



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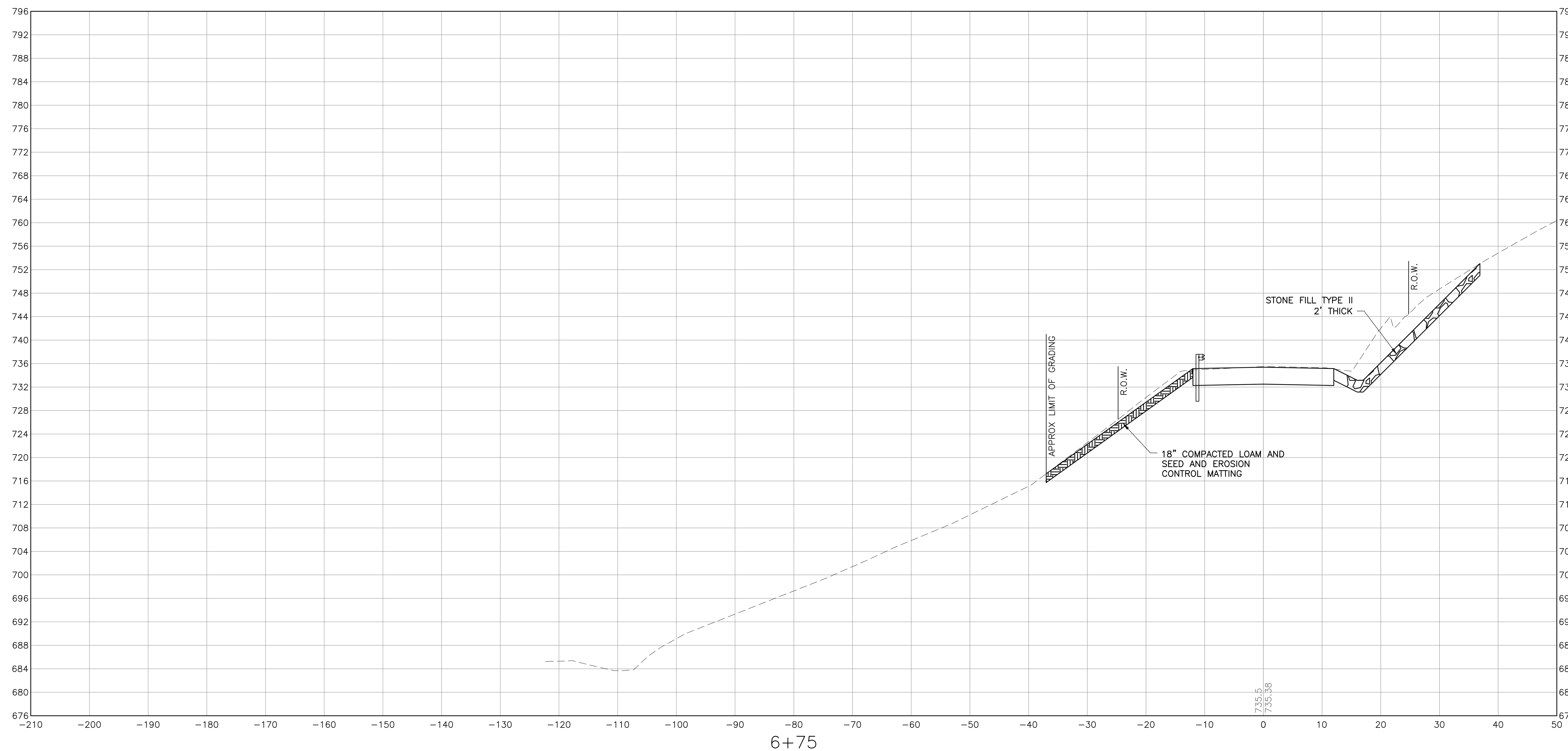


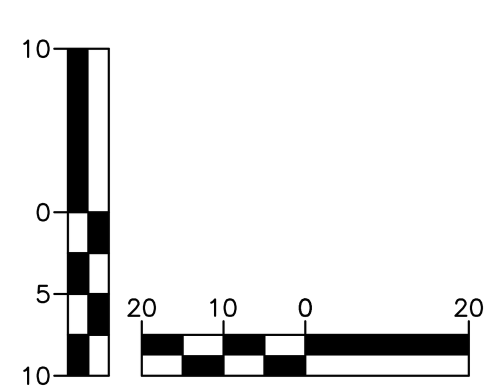



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SHEET TITLE

CROSS SECTIONS

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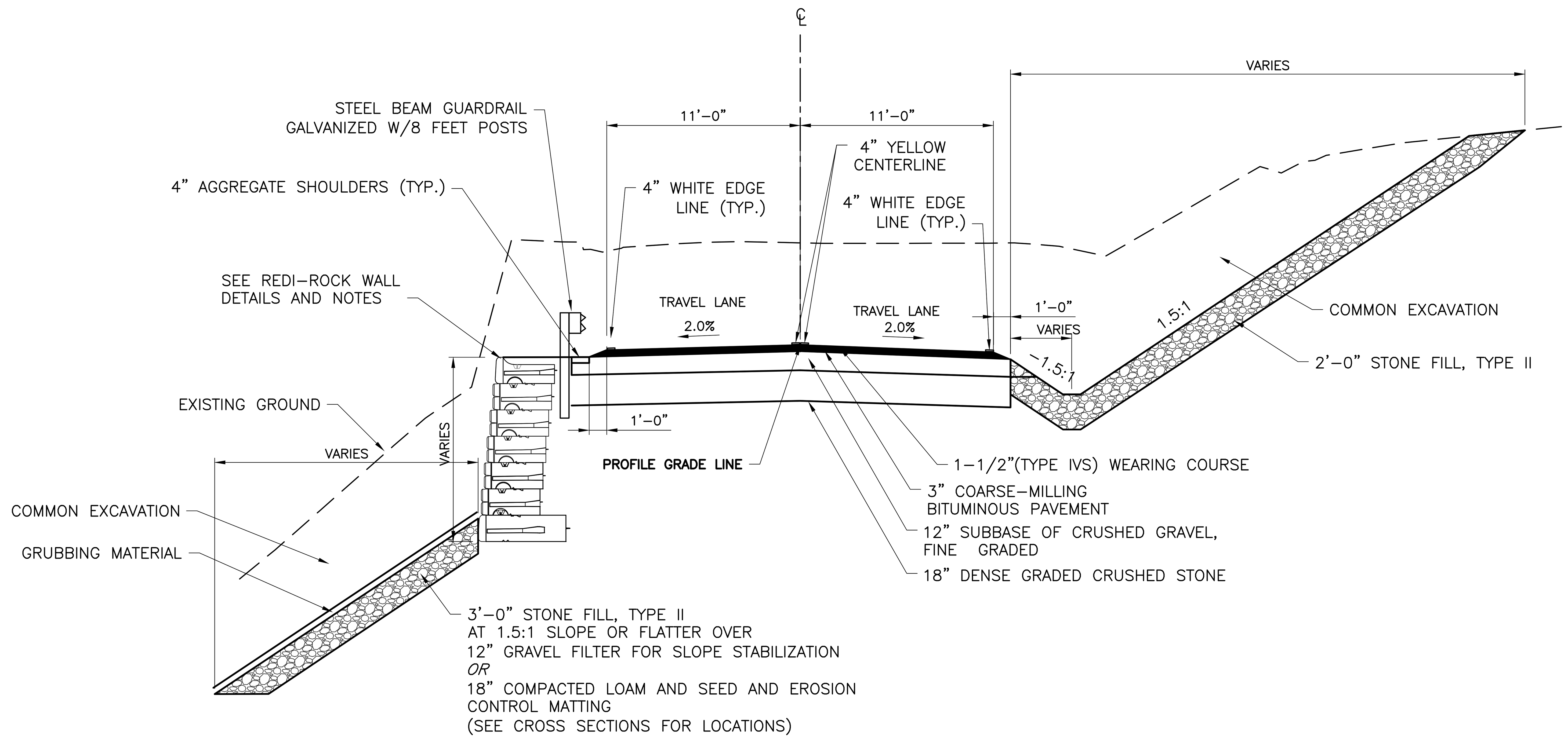
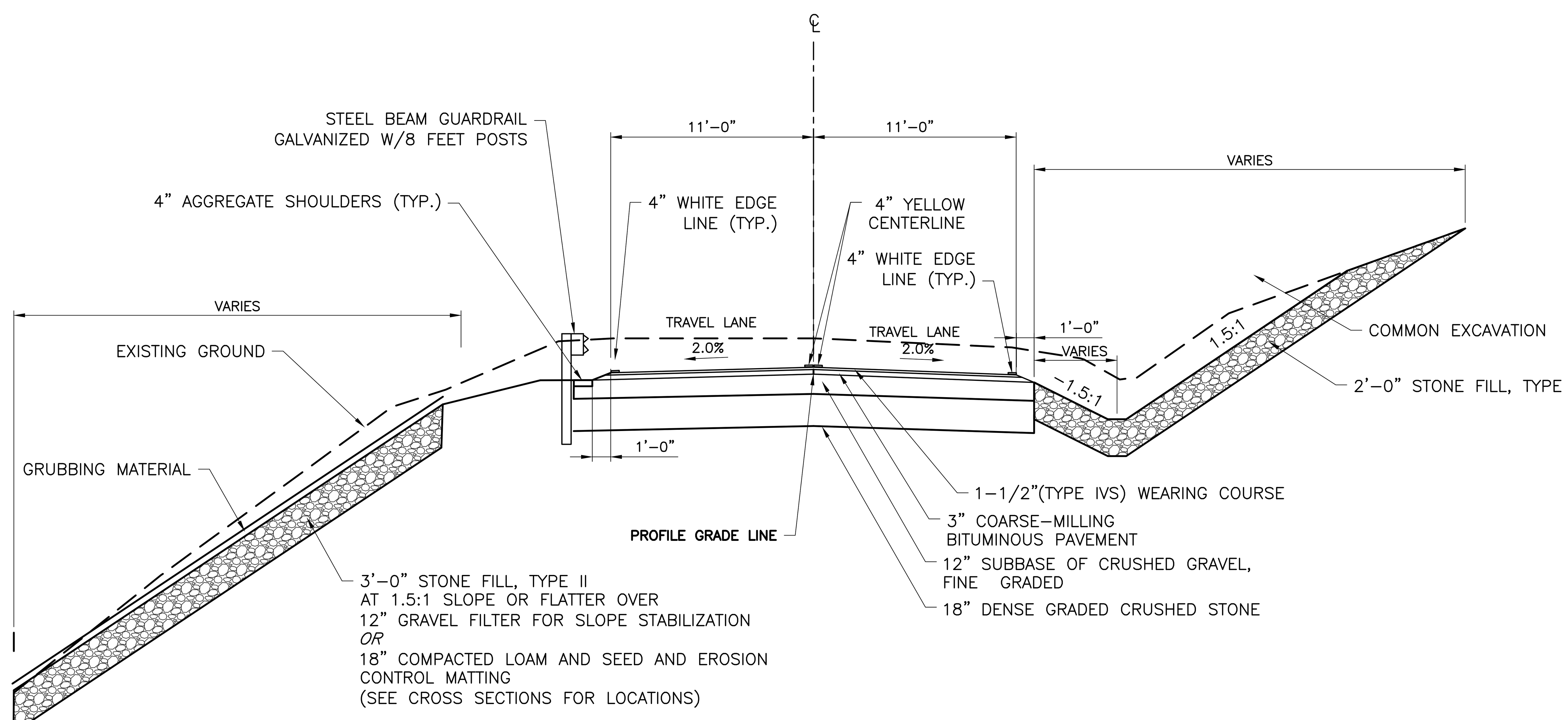
SHEET NUMBER

# XS-18




NORMAL CROWN SECTION WITH RET. WALL




NORMAL CROWN SECTION – NO RET. WALL

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367 BRIGGS HILL ROAD BRISTOL, VT 05443

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TYPICAL SECTIONS

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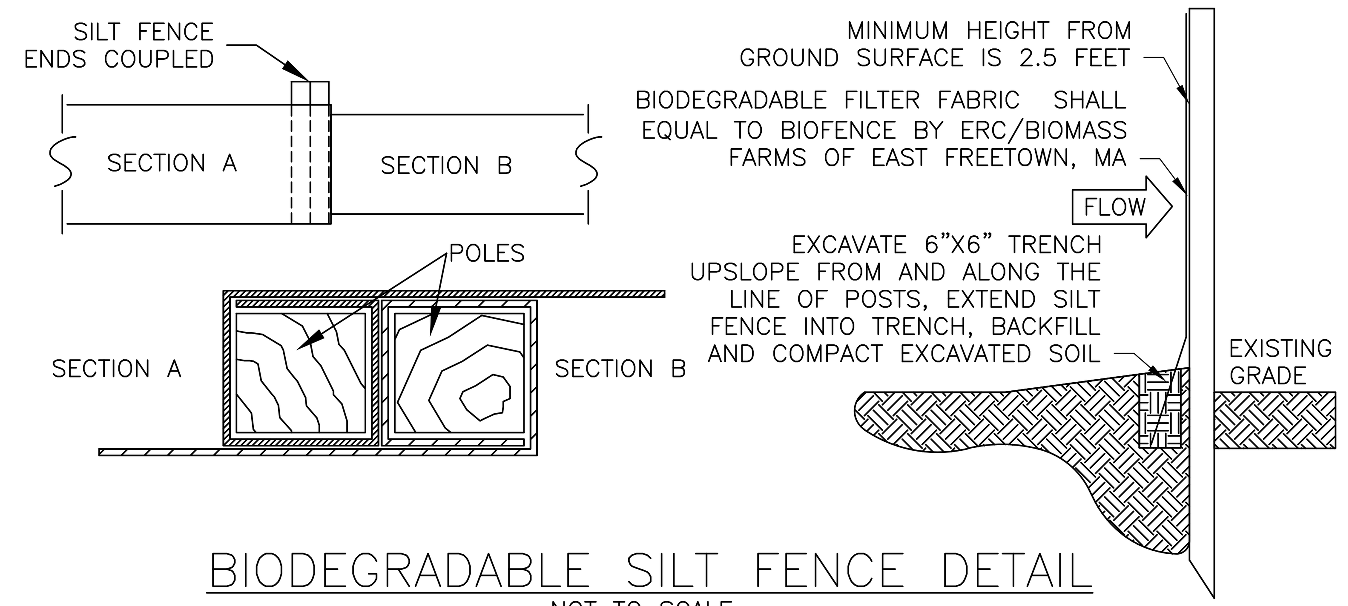
SHEET NUMBER

TS-1




## BIODEGRADABLE SILT FENCE DETAIL

NOT TO SCALE

NOTES:

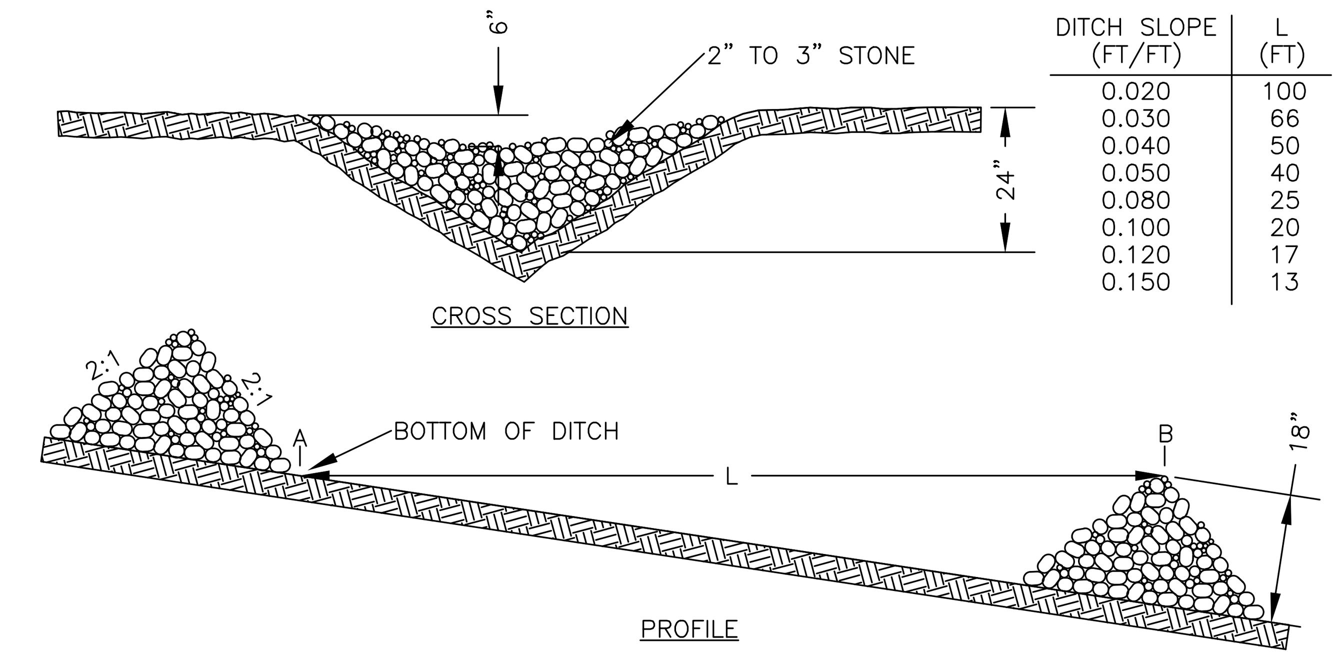
1. CONTRACTOR SHALL FOLLOW MANUFACTURER'S INSTALLATION INSTRUCTIONS, ALONG WITH BELOW FENCE–END JOINING INSTALLATION INSTRUCTIONS.
2. FENCE AND ALL COMPONENTS SHALL BE BIODEGRADABLE, AND SHALL BE FUNCTIONAL FOR A MINIMUM OF 1.5 TO 2 YEARS.
3. WHEN TWO SECTIONS OF SILT FENCE ADJOIN EACH OTHER AS SHOWN JOINING FENCE SECTIONS.

FENCE–END JOINING INSTALLATION INSTRUCTIONS

4. PLACE THE END POST OF ONE FENCE INSIDE THE END POST OF THE OTHER FENCE.
5. ROTATE BOTH POSTS AT LEAST 180 DEGREES IN A CLOCKWISE DIRECTION TO CREATE A TIGHT SEAL WITH THE FABRIC MATERIAL.
6. DRIVE BOTH POSTS 18 INCHES INTO THE GROUND AND BURY THE FLAP IN THE TRENCH.

MAINTENANCE:

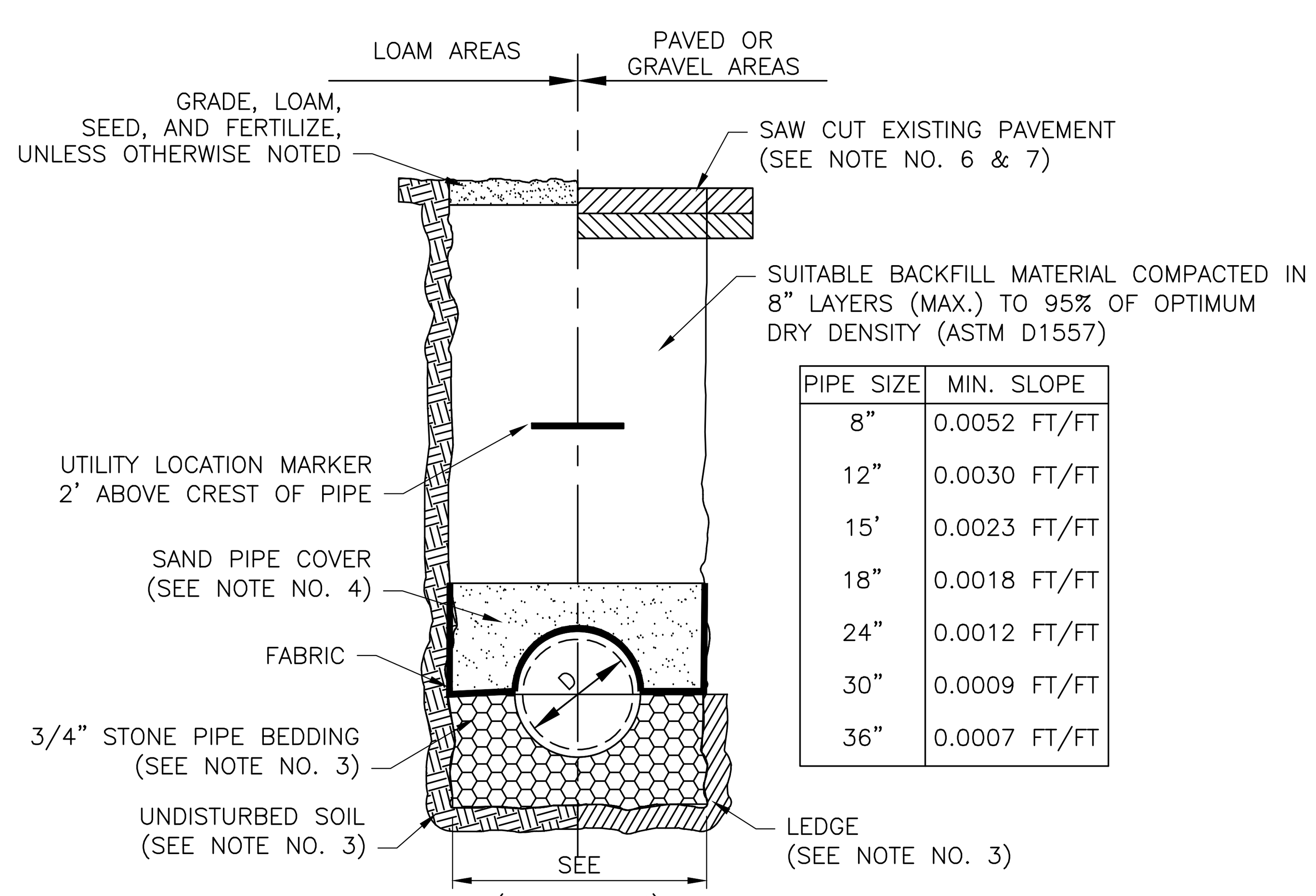
7. SILT FENCES SHALL BE INSPECTED IMMEDIATELY AFTER EACH RAINFALL AND AT LEAST DAILY DURING PROLONGED RAINFALL EVENTS. REPAIRS THAT ARE NEEDED SHALL BE MADE IMMEDIATELY.
8. IF THE FABRIC OF THE SILT FENCE SHOULD DECOMPOSE OR BECOME INEFFECTIVE DURING THE EXPECTED LIFE OF THE FENCE, THE FABRIC SHALL BE REPLACED PROMPTLY.
9. SEDIMENT DEPOSITS SHOULD BE INSPECTED AFTER EVERY STORM EVENT. THE DEPOSITS SHOULD BE REMOVED WHEN DEPOSITS HAS REACHED APPROXIMATELY ONE–HALF THE HEIGHT OF THE BARRIER.
10. SEDIMENT DEPOSITS THAT ARE REMOVED SHALL BE GRADED TO CONFORM WITH THE EXISTING TOPOGRAPHY AND VEGETATED.




| DITCH SLOPE (FT/FT) | L (FT) |
|---|---|
| 0.020 | 100 |
| 0.030 | 66 |
| 0.040 | 50 |
| 0.050 | 40 |
| 0.080 | 25 |
| 0.100 | 20 |
| 0.120 | 17 |
| 0.150 | 13 |

## STONE CHECK DAM DETAIL

NOT TO SCALE




| PIPE SIZE | MIN. SLOPE |
|---|---|
| 8" | 0.0052 FT/FT |
| 12" | 0.0030 FT/FT |
| 15' | 0.0023 FT/FT |
| 18" | 0.0018 FT/FT |
| 24" | 0.0012 FT/FT |
| 30" | 0.0009 FT/FT |
| 36" | 0.0007 FT/FT |

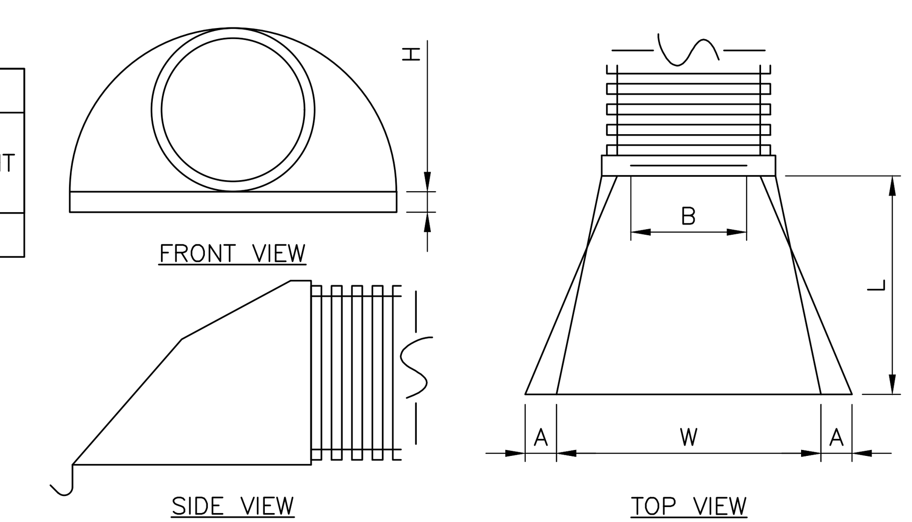
NOTES:

1. ALL EXCAVATION MUST MEET OSHA STANDARDS.
2. STANDARD TRENCH WIDTH SHALL BE D+2' OR 3'; WHICHEVER IS GREATER.
3. BEDDING MATERIAL SHALL BE FULL WIDTH OF TRENCH. BEDDING MATERIAL SHALL BE 6" BELOW PIPE (IN EARTH) OR 12" BELOW PIPE (IN LEDGE) UP 12" OVER OF PIPE.
4. SAND COVER/CUSHION MATERIAL SHALL BE FULL WIDTH OF TRENCH. SAND MATERIAL SHALL BE FROM SPRING LINE UP TO 12" (MINIMUM) ABOVE CREST OF PIPE.
5. STORMDRAIN LAYOUT PLAN PIPE SLOPES SHALL GOVERN OVER MINIMUM PIPE SLOPE SCHEDULE.
6. PAVEMENT REPAIR SHALL HAVE EMULSIFIED TACK BETWEEN ASPHALT LAYERS AND ON ALL JOINTS AND SHALL CONFORM TO STREET OPENING REGULATIONS (WITHIN RIGHT–OF–WAY LIMITS), PAVEMENT SECTION DETAIL (OUTSIDE OF RIGHT–OF–WAY LIMITS), AND FOLLOW NOTE NO. 7.
7. DURING PAVEMENT REPAIR, THE CONTRACTOR SHALL CUT–BACK SUBBASE MATERIAL AN ADDITIONAL 12–INCHES ON EITHER SIDE OF THE TRENCH AND PAVEMENT COLD PLANE MILL AREA TO A DEPTH OF 1–3/4" AN ADDITIONAL 24–INCHES ON EITHER SIDE OF SUBBASE MATERIAL CUT–BACK.

## STORM DRAIN PIPE TRENCH DETAIL

NOT TO SCALE

| FLARED END SECTION SCHEDULE | | | | | |
|---|---|---|---|---|---|
| PIPE DIA. (D) | A | THREADED ROAD (MAX.) (B) | END LENGTH (L) | END WIDTH (W) | END HEIGHT (H) |
| 15" | 6.5" | 10" | 25" | 29" | 6.5" |

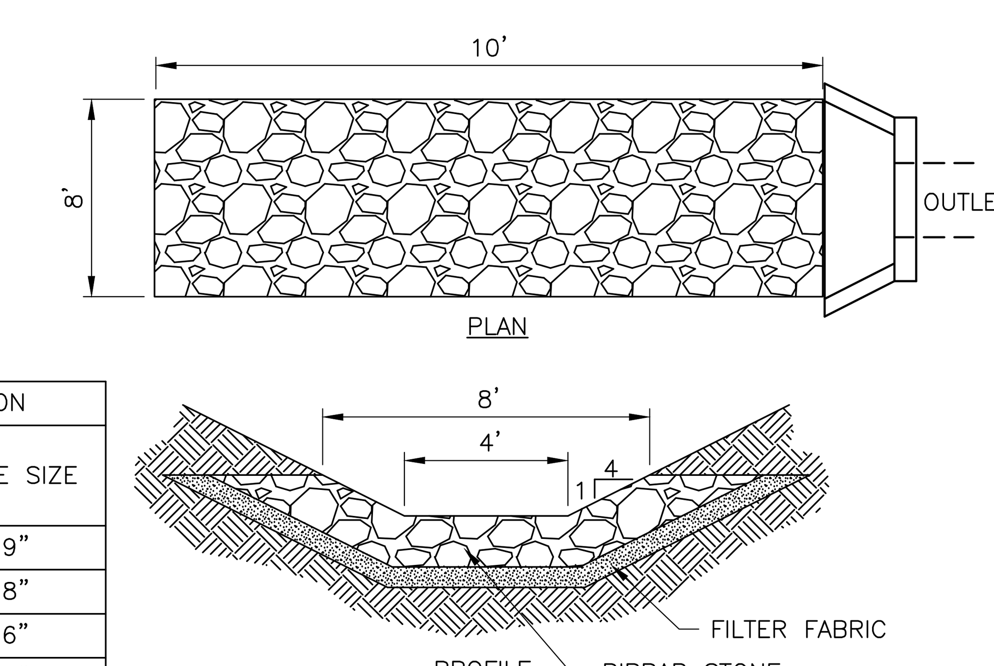



NOTES:

1. THREADED ROD WITH WING NUTS PROVIDED FOR END SECTIONS 12' THROUGH 24". END SECTIONS 30" AND 36" SHALL BE WELDED TO PIPE PER MANUFACTURER'S RECOMMENDATION

## FLARED END SECTION DETAIL

NOT TO SCALE




| STONE GRADATION | |
|---|---|
| PERCENTAGE BY WEIGHT PASSING | SIEVE SIZE |
| 100 | 9" |
| 85 | 8" |
| 50 | 6" |
| 15 | 2" |

## STONE LINED OUTLET PROTECTION

NOT TO SCALE

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BRIGGS HILL ROAD SLOPE FAULIRE AND ROADWAY REPAIR

SHEET TITLE

DETAILS

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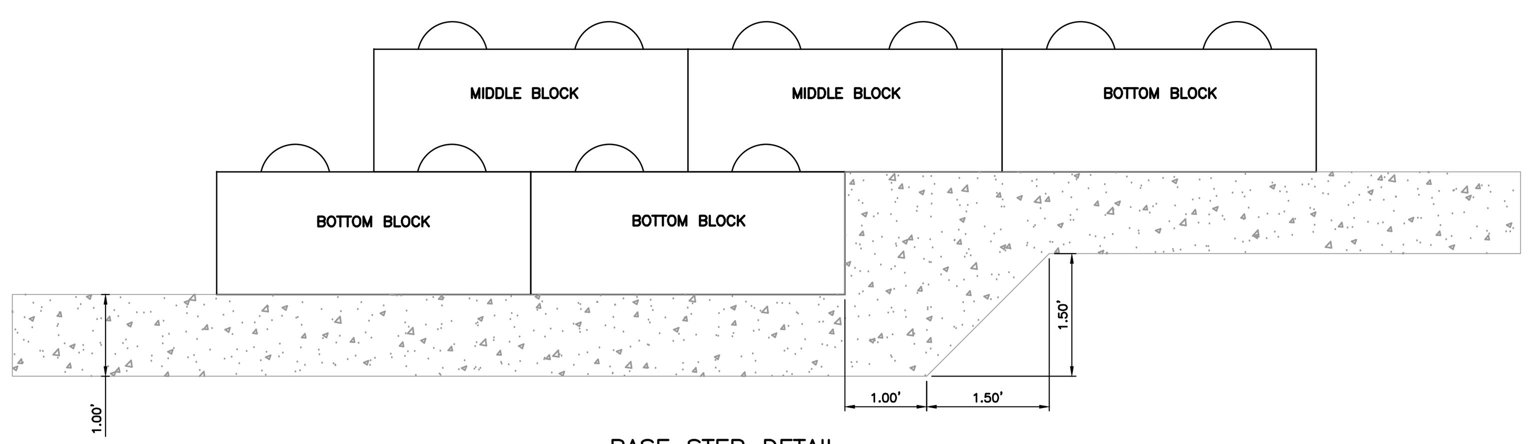
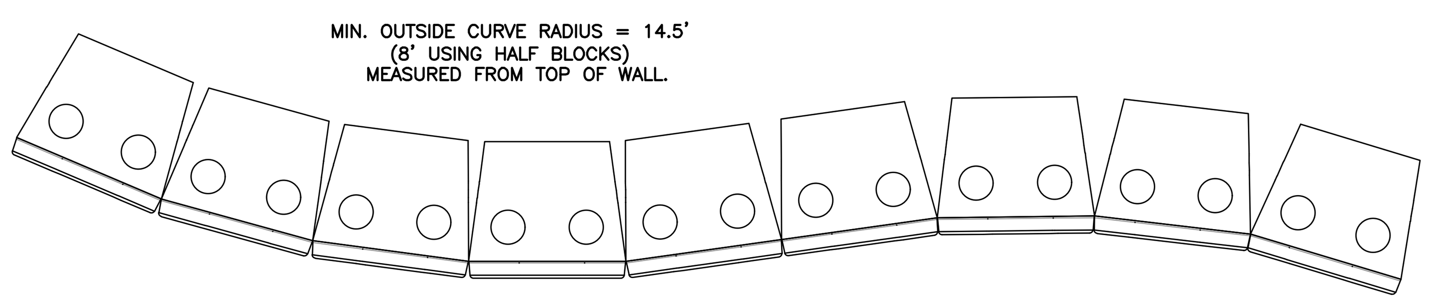
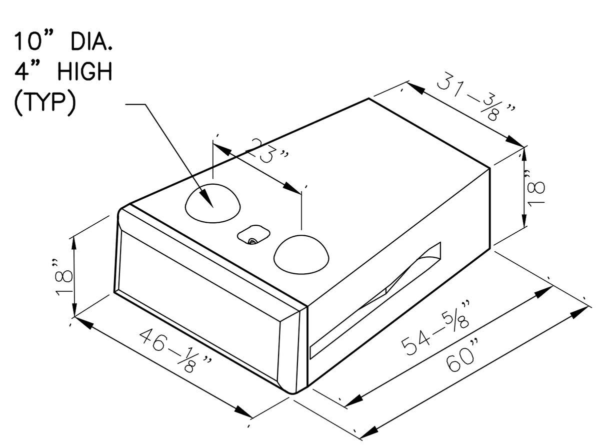


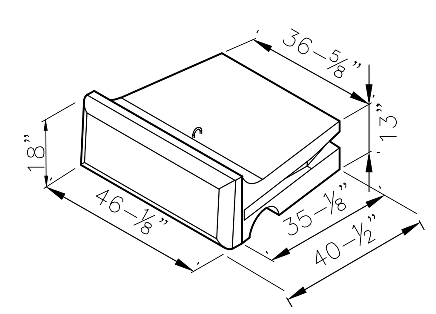

BASE STEP DETAIL
(NOT TO SCALE)



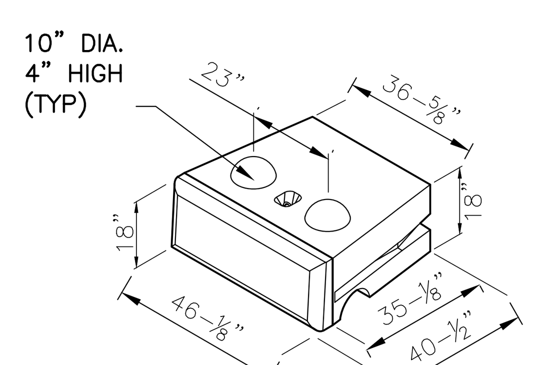

CURVE DETAIL
TYPICAL CURVES
(NOT TO SCALE)



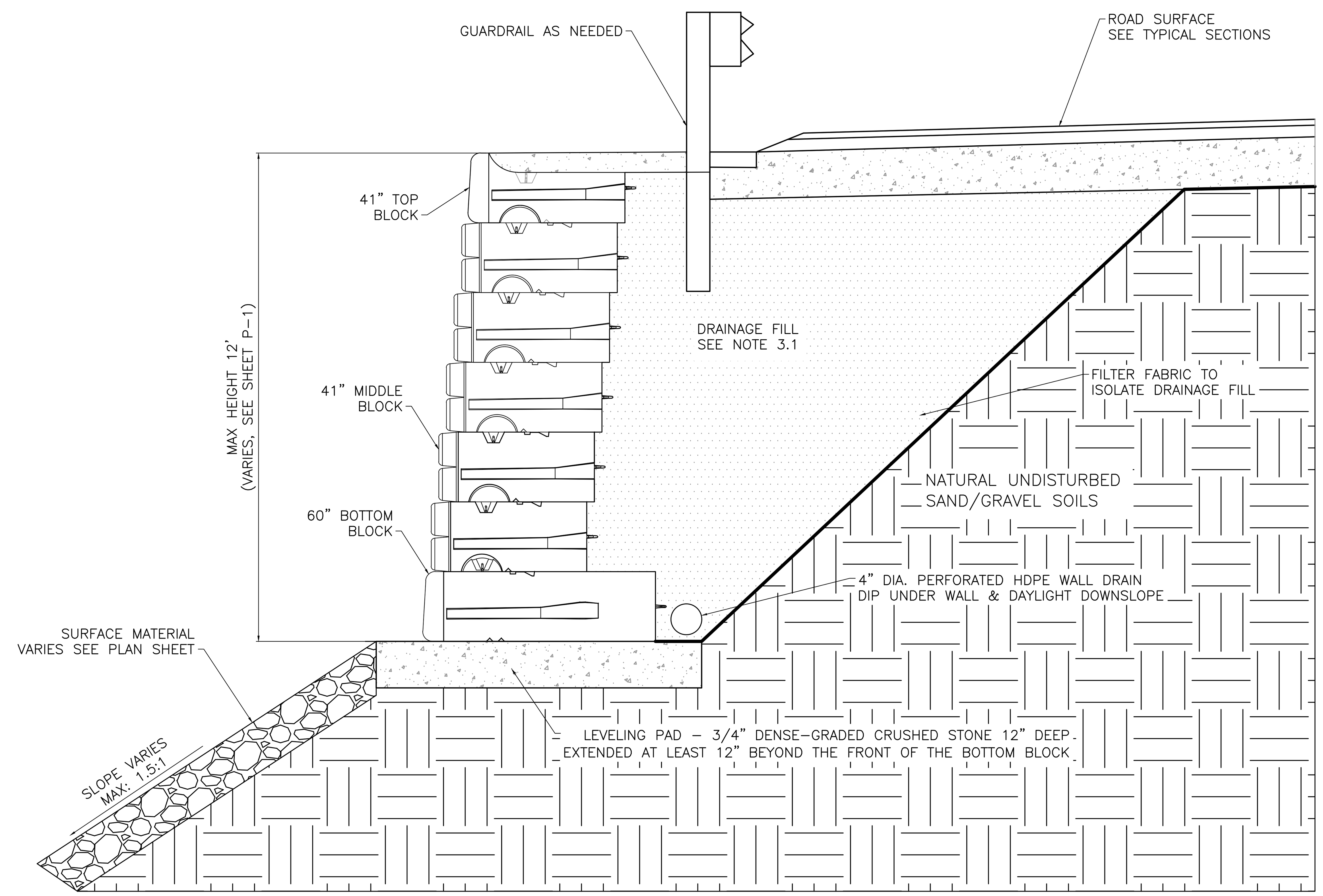

60" BOTTOM BLOCK




41" TOP BLOCK




41" MIDDLE BLOCK




TYPICAL SECTION – GRAVITY WALL
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367 BRIGGS HILL ROAD BRISTOL, VT 05443

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D-2

FEMA SUMMARY

734189-DR4720VT

Bristol – Briggs Hill Road

# OPCC for In-Kind Repair

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- ☐ Bedford, NH 03110 (603) 637-1043
- ☑ S. Burlington, VT 05403 (802) 878-7661
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| SHEET NO. | | OF | |
| CALCULATED BY: | AAT | DATE: | 21-Aug-24 |
| CHECKED BY: | CDL | DATE: | 21-Aug-24 |
| SCALE: | | | |

## Bristol, VT : BRIGGS HILL SLOPE REPAIR AND ROAD RESTORATION
## ENGINEER'S OPINION OF PROBABLE CONSTRUCTION COST

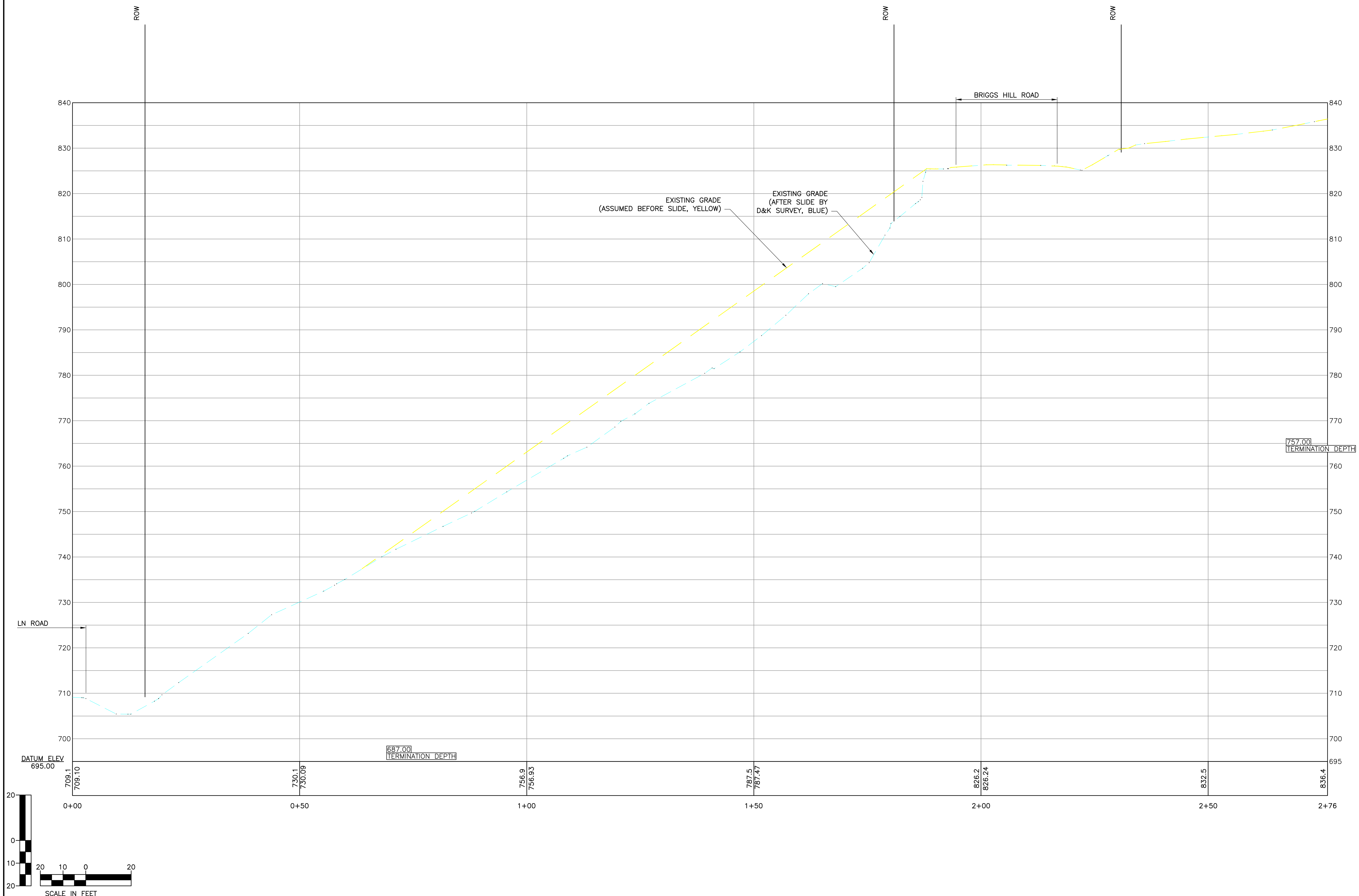
| ITEM NO. | DESCRIPTION | UNIT | QUANT. | UNIT PRICE | AMOUNT |
|---|---|---|---|---|---|
| 201.1000 | CLEARING AND GRUBBING, INCLUDING INDIVIDUAL TREES AND STUMPS | LS | 1 | $60,000.00 | $60,000.00 |
| 203.1500 | COMMON EXCAVATION | CY | 400 | $30.00 | $12,000.00 |
| 203.3000 | EARTH BORROW | CY | 980 | $30.00 | $29,400.00 |
| 406.0230 | BITUMINOUS CONCRETE PAVEMENT, TYPE IIS, QA TIER III | T | 120 | $200.00 | $24,000.00 |
| 406.0430 | BITUMINOUS CONCRETE PAVEMENT, TYPE IVS, QA TIER III | T | 60.0 | $200.00 | $12,000.00 |
| 621.0100 | REMOVAL OF GUARDRAIL | LF | 200 | $20.00 | $4,000.00 |
| 621.1080 | STEEL BEAM GUARDRAIL WITH 8 FOOT POSTS | LF | 200 | $35.00 | $7,000.00 |
| 630.1500 | FLAGGERS | HR | 400 | $55.00 | $22,000.00 |
| 635.1100 | MOBILIZATION/DEMOBILIZATION | LS | 1 | $26,283.50 | $26,283.50 |
| 641.1100 | TRAFFIC CONTROL, ALL-INCLUSIVE | LS | 1 | $10,000.00 | $10,000.00 |
| 651.1500 | TURF ESTABLISHMENT, GENERAL SEED | LB | 200 | $3.00 | $600.00 |
| 651.4018 | GRUBBING MATERIAL, 18 INCHES | SY | 2200 | $25.00 | $55,000.00 |
| 653.0100 | EPSC PLAN | LS | 1 | $3,000.00 | $3,000.00 |
| 653.0200 | MONITORING EPSC PLAN | HR | 40 | $50.00 | $2,000.00 |
| 653.0300 | MAINTENANCE OF EPSC PLAN (N.A.B.I.) | LU | 1 | $7,000.00 | $7,000.00 |
| 653.1000 | HAY MULCH | TON | 1 | $1,120.00 | $1,120.00 |
| 653.2001 | ROLLED EROSION CONTROL PRODUCT, TYPE I | SY | 2140 | $3.50 | $7,490.00 |
| 653.4701 | SILT FENCE, TYPE I | LF | 600 | $6.00 | $3,600.00 |
| 653.5000 | BARRIER FENCE | LF | 750 | $3.50 | $2,625.00 |
| **PARTICIPATING SUBTOTAL** | | | | | **$289,118.50** |

## Construction:

| | |
|---|---|
| **Estimated Total Base Bid** | **$289,118.50** |
| **20% CONTINGENCY** | **$58,000.00** |
| **Total Estimated Cost** | **$347,118.50** |

Note:

In providing opinions of probable construction cost, the Client understands that D&K has no control over the cost or availability of labor, equipment or materials, or over market conditions or the Contractor's method of pricing, and that our Opinion of Probable Construction Costs are made on the basis of our professional judgment and experience. D&K makes no warranty, expressed or implied, that the bids or the negotiated cost of the Work will not vary from the Opinion of Probable Construction Cost provided herein.




I:\2\229369 Bristol Slope Stabilization Briggs Hill Road\Opinion of Probable Cost\229369 QTY UPPER SLIDE slope mitigation volumes.DWG 8/19/2024 9:15 AM

DuBois & King inc.

ENGINEERING • PLANNING •
MANAGEMENT • DEVELOPMENT
28 NORTH MAIN ST.
RANDOLPH, VT 05060
TEL: (802) 728-3376
FAX: (802) 783-7101
www.dubois-king.com
SO. BURLINGTON, VT
SPRINGFIELD, VT
BRANDON, VT
BEDFORD, NH LACONIA, NH



PROFESSIONAL SEAL

**NOT ISSUED FOR CONSTRUCTION**

| NO. | DATE | DESCRIPTION | BY | CK'D |
|---|---|---|---|---|

367 BRIGGS HILL ROAD BRISTOL, VT 05443

BRIGGS HILL ROAD SLOPE FAULIRE AND ROADWAY REPAIR

SHEET TITLE

UPPER SLIDE SECTION

| DRAWN BY | DATE JULY 2024 |
|---|---|
| CHECKED BY | D&K PROJECT # 229369 |
| PROJ. ENG. JBA | D&K ARCHIVE # |

SHEET NUMBER

# XS

FEMA SUMMARY

734189-DR4720VT

Bristol – Briggs Hill Road

# OPCC for In-Kind Repair with Codes and Standards Plus Mitigation

DuBois & King inc.
☐ Randolph, VT 05060 (802) 728-3376
☐ Bedford, NH 03110 (603) 637-1043
☑ S. Burlington, VT 05403 (802) 878-7661
☐ Laconia, NH 03246 (603) 524-1166
Engineering • Planning • Development • Management

PROJECT: 229369 Briggs Hill Rd
SHEET NO.: OF:
CALCULATED BY: AAT DATE: 22-Jul-24
CHECKED BY: CDL DATE: 22-Jul-24
SCALE:

# Bristol, VT : BRIGGS HILL SLOPE REPAIR AND ROAD RESTORATION
## ENGINEER'S OPINION OF PROBABLE CONSTRUCTION COST

| ITEM NO. | DESCRIPTION | UNIT | QUANT. | UNIT PRICE | AMOUNT |
|---|---|---|---|---|---|
| 201.1000 | CLEARING AND GRUBBING, INCLUDING INDIVIDUAL TREES AND STUMPS | LS | 1 | $70,000.00 | $70,000.00 |
| 203.1500 | COMMON EXCAVATION | CY | 10200 | $30.00 | $306,000.00 |
| 203.1500 | COMMON EXCAVATION (WALL ONLY) | CY | 1200 | $30.00 | $36,000.00 |
| 203.3000 | EARTH BORROW | CY | 600 | $30.00 | $18,000.00 |
| 203.3500 | GRAVEL FILTER FOR SLOPE STABILIZATION | CY | 630 | $30.00 | $18,900.00 |
| 204.2000 | TRENCH EXCAVATION OF EARTH | CY | 620 | $35.00 | $21,700.00 |
| 204.3000 | GRANULAR BACKFILL FOR STRUCTURES | CY | 125 | $70.00 | $8,750.00 |
| 225.0400 | RETAINING WALL, PRECAST CONCRETE | LS | 1 | $704,700.00 | $704,700.00 |
| 301.2600 | SUBBASE OF CRUSHED GRAVEL, FINE GRADED | CY | 700 | $80.00 | $56,000.00 |
| 301.3500 | SUBBASE OF DENSE GRADED CRUSHED STONE | CY | 950 | $50.00 | $47,500.00 |
| 406.0230 | BITUMINOUS CONCRETE PAVEMENT, TYPE IIS, QA TIER III | T | 400 | $200.00 | $80,000.00 |
| 406.0430 | BITUMINOUS CONCRETE PAVEMENT, TYPE IVS, QA TIER III | T | 100.0 | $200.00 | $20,000.00 |
| 601.0915 | 18" CPEP | LF | 200 | $90.00 | $18,000.00 |
| 604.1800 | PRECAST REINFORCED CONCRETE DI WITH CAST IRON GRATE | EA | 4 | $5,800.00 | $23,200.00 |
| 613.0603 | E-STONE FILL, TYPE III | CY | 210 | $85.00 | $17,850.00 |
| 613.1002 | STONE FILL, TYPE II | CY | 3000 | $75.00 | $225,000.00 |
| 621.0100 | REMOVAL OF GUARDRAIL | LF | 705 | $20.00 | $14,100.00 |
| 621.1060 | STEEL BEAM GUARDRAIL | LF | 25 | $35.00 | $875.00 |
| 621.1080 | STEEL BEAM GUARDRAIL WITH 8 FOOT POSTS | LF | 700 | $35.00 | $24,500.00 |
| 621.1520 | ANCHOR FOR STEEL BEAM GUARDRAIL | EA | 2 | $1,300.00 | $2,600.00 |
| 630.1500 | FLAGGERS | HR | 800 | $55.00 | $44,000.00 |
| 635.1100 | MOBILIZATION/DEMOBILIZATION | LS | 1 | $183,269.00 | $183,269.00 |
| 641.1100 | TRAFFIC CONTROL, ALL-INCLUSIVE | LS | 1 | $10,000.00 | $10,000.00 |
| 646.4030 | DURABLE 4 INCH WHITE LINE, EPOXY PAINT | LF | 1000 | $2.00 | $2,000.00 |
| 646.4130 | DURABLE 4 INCH YELLOW LINE, EPOXY PAINT | LF | 1000 | $2.00 | $2,000.00 |
| 651.1500 | TURF ESTABLISHMENT, GENERAL SEED | LB | 150 | $3.00 | $450.00 |
| 651.4012 | GRUBBING MATERIAL, 12 INCHES | SY | 2100 | $15.00 | $31,500.00 |
| 651.4018 | GRUBBING MATERIAL, 18 INCHES | SY | 50 | $25.00 | $1,250.00 |
| 653.0100 | EPSC PLAN | LS | 1 | $3,000.00 | $3,000.00 |
| 653.0200 | MONITORING EPSC PLAN | HR | 40 | $50.00 | $2,000.00 |
| 653.0300 | MAINTENANCE OF EPSC PLAN (N.A.B.I.) | LU | 1 | $7,000.00 | $7,000.00 |
| 653.1000 | HAY MULCH | TON | 2 | $1,120.00 | $2,240.00 |
| 653.2001 | ROLLED EROSION CONTROL PRODUCT, TYPE I | SY | 2100 | $3.50 | $7,350.00 |
| 653.4701 | SILT FENCE, TYPE I | LF | 600 | $6.00 | $3,600.00 |
| 653.5000 | BARRIER FENCE | LF | 750 | $3.50 | $2,625.00 |
| PARTICIPATING SUBTOTAL | | | | | $2,015,959.00 |

## Construction:

| | |
|---|---|
| **Estimated Total Base Bid** | **$2,015,959.00** |
| **20% CONTINGENCY** | **$403,000.00** |
| **Total Estimated Cost** | **$2,418,959.00** |

| | |
|---|---|
| **IN KIND** | **$1,930,009.00** |
| **MITIGATION** | **$85,950.00** |
| **20% Contingency** | **$403,000.00** |
| **Total** | **$2,418,959** |