TRANSMITTAL MEMORANDUM 19-02

To: Division of Structural Design Staff
Design Consultants
KYTC Staff

From: Bart Asher
Director
Division of Structural Design

Date: June 21, 2019

Subject: KYTC Division of Structural Design
Geotechnical Guidance Manual
Chapter 600 Engineering Analysis

With this memorandum the attached Chapter 600 Engineering Analysis and accompanying exhibits from the Kentucky Transportation Cabinet's Division of Structural Design's Geotechnical Guidance Manual replaces and supersedes the referenced section in the most current (publication date June 2005) manual.

BA/MC

GEOTECHNICAL

Section
SLOPE STABILITY

Subject
Slope Stability Analysis

SLOPE STABILITY ANALYSIS

A slope stability analysis yields a safety factor, which is the ratio of available shear strength of the soil to the strength required to maintain equilibrium of the slope. The safety factor obtained from the various methods of analyzing the stability of slopes does not necessarily constitute a reserve of unused strength. Rather, it is a working element of design, where the safety factor is used to allow for uncertainties in modeling the site geometry, characteristics of the soil and construction materials, location of the water table, and construction techniques.

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GEOTECHNICAL

TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch

Section

SLOPE STABILITY

Subject

Strength Parameters

OVERVIEW

The subsurface investigation will permit a determination of whether the foundation soils are relatively uniform and homogeneous (in which case a single set of soil parameters may be used) or if soil layers with varying properties are apparent. In a layered foundation, different soils should be defined to allow the stability model developed to be a reasonably close approximation to field conditions.

COHESIVE SOILS

Strength parameters of cohesive foundation soils should be based on laboratory and/or in-situ test results, along with correlations with published information and knowledge of the area. Total stress parameters shall be based upon unconfined compression tests, unconsolidated-undrained triaxial tests, or consolidated-undrained triaxial tests, as applicable. Effective stress parameters shall be based upon consolidated-undrained triaxial tests.

ESTIMATED PARAMETERS

Tube samples may not be available for testing in some cases due to lack of samples, poor sample recovery, rocky samples, bent tubes, shallow depth to rock, etc. Strength parameters may be estimated in these cases by correlating Standard Penetration Test (SPT) N-values, Cone Penetrometer Test (CPT), and soil classifications with published information. Refer to Naval Facilities Design Manual (NAVFAC) or the FHWA Soils and Foundations Reference Manual for correlations.

Strength parameters of granular foundation soils should be estimated using corrected SPT blow counts and published correlations such as presented in NAVFAC or the FHWA Soils and Foundations Reference Manual.

EMBANKMENTS

Strength tests are typically not performed on proposed embankment materials because it is usually uncertain where these materials will be obtained. It is generally a reasonable assumption that the embankment material will be similar to, but slightly better than, the foundation soils at the site, since their strength should be improved somewhat by the required compaction. NAVFAC provides typical strength properties of compacted materials.

STRENGTH PARAMETERS FOR ROCK EMBANKMENT

The following parameters may be used as a guide for embankments constructed of rock:

ROCK TYPE	SHORT TERM	LONG TERM
Nondurable Shale Class III (SDI = 0-49)	ø = 0º c = 1000-1500 psf	ø' = 18º-22º c' = 200 psf
Nondurable Shale Class II (SDI = 50-79)	ø = 0º c = 1000-1500 psf	ø' = 23º-27º c' = 150 psf
Nondurable Shale Class I (SDI = 80-94)	ø = 0º c = 1000-1500 psf	ø' = 28º-32º c' = 100 psf
Friable Sandstone	ø' = 34º-36º c' = 0	ø' = 34º-36º c' = 0
Durable Shale and Non-Friable Sandstone	ø' = 36º-38º c' = 0	ø' = 36º-38º c' = 0
Limestone from Roadway Excavation	ø' = 38º-40º c' = 0	ø' = 38º-40º c' = 0
Coarse Aggregate*	ø' = 38º-42º c' = 0	ø' = 38º-42º c' = 0

*Coarse Aggregates shall meet the requirements of Section 805 of the *Standard Specifications for Road and Bridge Construction,* current edition.

Assumes non-durable shales will be constructed in accordance with the current edition of Section 206 of the Standard Specifications for Road and Bridge Construction. Special construction method “Embankment of Rock/Shale/Soil Combination” or “Embankments Principally of Non-Durable Shale (SDI less than 95 according to KM 64-513)”.

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GEOTECHNICAL

TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch

Section

SLOPE STABILITY

Subject

Target Safety Factors

GUIDE IN SELECTION

A target safety factor is a function of many intangibles, such as quality and scope of the subsurface investigation, as well as confidence in construction methods. The following may be used as a guide in selecting target safety factors, depending on confidence in available data, etc.

	SHORT TERM	INTERMEDIATE TERM	LONG TERM	RAPID DRAWDOWN
Roadway embankments	1.1 — 1.3	***	1.4 — 1.6	1.0 — 1.2
Bridge approach slopes, walls, and culverts*	1.2 — 1.4	***	1.6 — 1.8	1.0 — 1.2
Cutslopes in soil	1.2 — 1.4	1.2 — 1.4	1.4 — 1.6	***
Landslide corrections	***	***	1.4 — 1.6	1.1 min.

*Bridge approach slopes and retaining walls shall have target safety factors of 1.0 — 1.2 for earthquake design.

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GEOTECHNICAL	Section: SLOPE STABILITY
Transportation Cabinet, Department of Highways, Division of Structural Design, Geotechnical Branch	Subject: Cut Slopes in Soil

WHEN & WHERE ANALYSES ARE REQUIRED

Stability analyses are generally required when the depth of cut in overburden is greater than 10 feet. The analyses shall be made at the location where the overburden soils are deepest. Cuts of lesser depth should be analyzed if unusual conditions are encountered. Also, cut stability analyses may be performed near each end of cuts when problems in the cut-to-fill transitions are expected.

TYPICAL SLOPE CONFIGURATIONS

Cut slope recommendations in overburden and disintegrated rock are usually 2H:1V. However, flatter slopes are occasionally required. In mountainous terrain where overburden depths are shallow (3 - 16 feet), it is often necessary to steepen slopes to 3H:2V or 1H:1V.

SHORT-TERM ANALYSES

Short-term (total stress) analyses may be warranted for cut slopes in cohesive soils and are performed using total stress parameters. When the state of total stress is changed in cohesive soils, excess pore pressures develop due to the low permeability of the cohesive soils. These pore pressures are due to two components: change in total confining stress and change in total shear stress.

The component resulting from the change in total confining stress is likely to be negative in cut slopes. However, the component of excess pore pressure resulting from change in shear stress may be positive and greater in magnitude than the component resulting from change in confining stress. This effect is likely to occur in soft (that is, normally consolidated or lightly over-consolidated) cohesive soils that have a tendency to develop high positive excess pore pressures during shear.

Although the critical condition for cut slopes in cohesive soils is likely to be the intermediate-term or the long-term case, the short-term case may be critical, and the geotechnical engineer should consider performing total stress analyses for cut slopes in soft cohesive soils. The decision to perform such analyses should be made on a case-by-case basis.

For granular soils or granular components of layered systems, the short-term condition is identical to the intermediate-term condition and shall be performed using effective stress parameters.

INTERMEDIATE-TERM ANALYSES

The intermediate-term condition for cut slopes in cohesive soils is the condition after excess pore pressures due to changes in total stress have dissipated. Intermediate-term cut slope stability analyses are based on effective stress strength parameters determined for each different soil type or horizon. Accurate determination of the water table is necessary for a meaningful intermediate-term analysis of a cut slope. The water table for the intermediate-term condition shall be positioned at its maximum anticipated height. Even though the water table may be lowered during the time that excess pore pressures dissipate, the maximum elevation shall be used to be conservative.

LONG-TERM ANALYSES

Long-term analyses model the condition long after excess pore pressures have dissipated and the groundwater table has been lowered due to the presence of the cut. Lowering of the water table over time contributes to increasing the factor of safety. However, swelling of cohesive soils (due to exposure) should be expected, which decreases the factor of safety and usually outweighs the increase due to lowering of the water table. The geotechnical engineer shall account for this condition by reducing the cohesion for long-term analyses to 20 percent (80 percent reduction in cohesion) of the value used in the intermediate-term analysis.

The cohesion for long-term analyses may be taken as zero in areas with highly plastic clays, severe swelling or softening, or large potentials for sloughing failures.

The long-term safety factor is frequently lower than the short-term and intermediate-term safety factors.

PRESENTATION OF ANALYSES RESULTS

Results of the cut slope stability analyses shall be presented in the Geotechnical Engineering Report and shown on a cut stability section (**Exhibit 32**).

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GEOTECHNICAL	Section: SLOPE STABILITY
	Subject: Embankments, Bridge Approach Slopes, & Excess Material Sites

WHEN & WHERE ANALYSES ARE REQUIRED

Embankment stability sections (including both long-term and short-term conditions) shall be prepared for each embankment over 20 feet high. However, analysis for each section need not be performed if embankment height and foundation conditions are similar for several sections. The embankment height for stability analysis is the difference in elevation from the shoulder to toe measured vertically.

Embankments of lesser height shall be analyzed if unusual conditions are encountered, with the approval of the Geotechnical Branch. One cross-section (typically) shall be chosen for stability analysis from each 1,000 feet of embankment. The cross-section analyzed shall be, in most cases, the highest embankment in the area represented.

SHORT-TERM ANALYSES

Short-term analyses model conditions that will exist immediately after completion of embankment construction. When the state of total stress is changed in cohesive soils, excess pore pressures develop due to the low permeability of the cohesive soils. For the case of embankment construction, these pore pressures are likely to be positive; hence the short-term condition is typically the critical condition for embankment stability. This condition should be modeled using total stress parameters for cohesive soils.

In granular soils or granular components of layered systems, excess pore pressures dissipate relatively quickly, and short-term stability analyses shall be performed using effective stress parameters.

LONG-TERM ANALYSES

Long-term analyses model conditions long after the embankment has been constructed and excess pore pressures have dissipated. These analyses shall be performed using effective stress parameters for both cohesive and granular soils.

EXCESS MATERIAL SITES

Excess material sites shall be analyzed using the same procedures and minimum safety factors as are applicable to roadway embankments.

RAPID DRAWDOWN ANALYSES

Rapid drawdown analysis is required at stream and river crossings (wet crossings) unless the embankment is granular or not affected by high water. Rapid drawdown analysis may be required for embankments not immediately at the bridges but influenced by adjacent streams or rivers. This analysis shall be performed using effective stress parameters for both cohesive and granular soils.

STABILITY ANALYSES AT BRIDGES

Stability analyses shall be performed on all bridge approach embankments over 20 feet in height. The spill-through slope under the bridge will be in most cases more critical than the side slope near the abutment. Proposed pile foundations shall not be considered when performing stability analyses.

GRANULAR FILL

Constructing granular fills may be preferable to other options for increasing the stability safety factors. Free-draining granular fills are an effective method of obtaining adequate safety factors for rapid drawdown analysis. Granular embankment shall be considered for bridge approaches when adequate safety factors cannot be obtained with cohesive embankments. Granular embankments shall be recommended when sufficient quantities of durable rock are available from roadway excavation. Flatter slopes may be more economical than processing or transporting granular material long distances.

PRESENTATION OF ANALYSES RESULTS

Results of the embankment slope stability analyses shall be presented in the Geotechnical Engineering Report and shown on an embankment stability section (**Exhibit 33**).

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GEOTECHNICAL

	Section SLOPE STABILITY
	Subject Landslides

FIELD EVALUATION

Information obtained from a field investigation shall be assembled to determine the extent and geometry of landslides. Locations of scarps or breaks, toe bulges, depths of movement in slope inclinometers, and other indications of slope movement should be used to estimate the location and shape of the failure surface. The failure surface should be described as rotational, or translational, to aid the analysis process.

MODELING

The failure surface, along with the water table and the assumed parameters of the various soil layers, should be plotted on the most critical section. The most critical cross-section is usually the section where movement is deepest, and it is often located near the middle of the slide area.

ANALYSES

After the critical section has been determined, the strength parameters of the failed materials shall be determined. All materials within the failure may be considered as homogeneous for purposes of analyses. The strength parameters may be determined by assuming the factor of safety equal to one (FS = 1.0) and "back-calculating" values of c' and ø'. The value of cohesion should be held to (or very near) zero and generally should not exceed 20 psf. The back-calculated values considered to represent the soil strength along the failure surface are used in the analyses of the slide corrections.

RECOMMENDATIONS

Several feasible correction alternatives (typically including berm, shear key, flattened slope, excavation/replacement, etc.) should be considered. Other methods (retaining walls, slope reinforcement, lightweight fill, etc.) may also be technically and economically feasible. Constructability issues of the correction alternatives (such as water table elevation, limits on excavation, floodplains, right-of-way limits, etc.) shall be addressed in the Geotechnical Engineering Report.

PRESENTATION OF ANALYSES RESULTS

Results of the landslide slope stability analyses shall be presented in the Geotechnical Engineering Report and shown on a stability section (**Exhibit 34**).

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GEOTECHNICAL	**Section** SLOPE STABILITY
TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch	**Subject** Controlled Loading

OVERVIEW This subject presents guidelines pertaining to the use of controlled loading to increase slope stability safety factors.

WHEN TO CONSIDER Controlled loading (staged construction) may be used in cases where short-term safety factors are too low but long-term safety factors are adequate. This method allows for some pore pressure dissipation, consolidation, and strength gain in the foundation soils to occur prior to placing the full loading conditions on the foundation. As a result, this method can be cost-effective because it eliminates the need to use other methods of increasing stability (granular replacement, berms, flattened slopes, etc.).

RECOMMENDED PROCEDURE The engineer shall determine the maximum embankment height that can be constructed without allowing the short-term safety factor to fall below 1.2 for structures or 1.1 for roadways. Construction above this elevation shall be subject to controlled loading.

Estimates of the rates of consolidation (as described in **GT-603**, “Settlement”) must be made to allow a determination of an appropriate loading rate. It may be assumed that the strength gain from short-term to long-term is linearly proportional to the percentage of consolidation. However, due to the uncertainty in predicting rates of consolidation, it is recommended that a safety factor of 3.0 be applied to the calculated loading rate to establish the allowable loading rate.

Soils that will consolidate very slowly may require methods such as wick drains to accelerate the consolidation rate used in conjunction with controlled loading (see **GT-603**, “Settlement”).

ANALYSIS DURING CONSTRUCTION Monitoring of the pore pressures and settlement rates using piezometers and settlement platforms is an alternative method of controlling the loading rates. The pore pressures as measured by the piezometers during construction are used in the stability analysis (using effective stress parameters). If the factor of safety is less than 1.2 for structures or 1.1 for roadways, construction is halted until pore pressures dissipate.

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GEOTECHNICAL

TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch

Section

SLOPE STABILITY

Subject

Ground Improvement

OVERVIEW

Ground improvement methods may be used to modify the ground, soil, and rock to allow construction of earthwork, bridges, earth retaining structures, or other facilities.

IMPROVEMENT TECHNIQUES

Ground improvement techniques include but are not limited to:

- Grouting
- Vertical Drains
- Stone Columns
- Lightweight Fill
- Vibro Compaction
- Dynamic Compaction
- Deep Soil Mixing
- Column-Supported Embankments

The engineer shall consider the availability and economics of feasible alternatives in determining the method of modification.

REFERENCES

The following FHWA publication may be used as ground improvement references:

- **FHWA GEC 13 Ground Modification Reference Manual**

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GEOTECHNICAL	Section: SLOPE STABILITY
Transportation Cabinet, Department of Highways, Division of Structural Design, Geotechnical Branch	Subject: Groundwater

HIGH GROUNDWATER TABLE

The presence of a high groundwater table will have an adverse effect upon the stability of slopes. For this reason one of the commonly used methods of increasing slope stability safety factors is to provide some means of lowering the water table.

All computer models used to evaluate the stability of slopes allow for input of the water surface so that the effect of lowering the water table may be more precisely determined.

RAPID DRAWDOWN ANALYSIS

In the design of embankments that might be affected by a high groundwater table due to flooding, and particularly for the analysis of the approach slopes of bridges crossing rivers or streams, a rapid drawdown analysis is required. The analysis is based upon the following assumptions:

- The water level of the surface stream rises through flooding to the elevation of the 100-year high water.
- The flood level remains that high for a sufficient amount of time to saturate the embankment.
- The water then falls so rapidly that no drainage (lowering of the groundwater table within the embankment) can occur.

The possibility of all these occurring is quite remote; as a result, a safety factor of 1.0 for stability during rapid drawdown is considered adequate.

SLOPE DESIGN MODIFICATION

If the safety factor for rapid drawdown is less than 1.0, modification to the embankment is required. Typically, this modification is handled by requiring that the entire embankment—from the toe of slope back to a distance of half the embankment height behind the abutment (maximum

50 feet) and from the original ground surface up to the elevation of the 100-year flood—be constructed with granular embankment (see Standard Drawings **RGX-100** and **RGX-105**).

The granular embankment shall meet the materials requirements of **Section 805** of the current edition of the *Standard Specifications for Road and Bridge Construction*, non-erodible only.

LANDSLIDES

In landslides, where the materials involved are in-situ and have already failed, the method used to lower the water table is to install drains (consisting of a small-diameter, slotted plastic tubing) that allow water to flow out of the slope under the influence of gravity.

Drains are typically installed near the toe of slope (or sometimes at multiple levels) and are drilled horizontally into the slope. Horizontal drains are sometimes used in conjunction with vertical drains drilled at (or near) the top of slope. Toe drains may also be installed at a slight inclination (near horizontal).

GEOTECHNICAL

TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch

Section

BEARING RESISTANCE FOR SHALLOW FOUNDATIONS

Subject

Use of Spread Footings

GENERAL GUIDELINES

Generally, spread footings on soil are not used at stream crossings due to scour considerations at bridges. Spread footings are typically the preferred foundation type in the following instances:

- For bridges—whenever bedrock (usually less than 15 to 20 feet from design roadway grade at the abutments or less than 15 to 20 feet from proposed groundline at the piers) or soils are capable of supporting the design loads

  Note: Spread footings on soil for bridges will only be allowed with prior approval of the Division of Structural Design.

  Note: Generally, spread footings on soil are used only for simple span bridges (at dry crossings) to limit problems with settlement.

- For culverts—whenever bedrock (occurring at shallow depths usually less than 3 feet below flowline) or soils are capable of supporting the design loads

- For walls—whenever bedrock or soils are capable of supporting the design loads

The geotechnical engineer shall provide the structure design engineer an estimate of factored bearing resistance.

MINIMUM EMBEDMENT

The bottom of the spread footings on soil shall be embedded a minimum of 2 feet below the finished ground surface as protection against frost heave.

SHALLOW FOUNDATION DESIGN

Foundations that have widths equal to or greater than the distance from the ground surface to the base of the foundation are considered shallow. Deep foundation analysis methods (such as those for piles) differ from those presented here and are discussed in **GT-605**.

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GEOTECHNICAL

TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch

Section

BEARING RESISTANCE FOR SHALLOW FOUNDATIONS

Subject

Bearing Resistance on Soil

NOMINAL BEARING RESISTANCE

Bearing resistance of soil in strength limit design state is discussed in the current edition of AASHTO LRFD Bridge Design Specifications, Sections 10 and 11. Nominal bearing resistance is an estimate of the unfactored load-carrying capacity of the foundation. Nominal bearing resistance of shallow foundations on soil should be calculated using the method presented in the current edition of AASHTO LRFD Bridge Design Specifications.

Cohesive Soils—The nominal bearing resistance of cohesive soils is typically based upon laboratory testing of samples taken at or near the proposed footing location. Additionally, Cone Pentrometer Test (CPT) can be used.

Granular Soils—The nominal bearing resistance of granular soils is typically based upon estimates of soil strength (friction angle), which, in turn, is based upon grain size, blow counts (N-values) recorded from the Standard Penetration Test, and CPT test data.

FACTORED BEARING RESISTANCE

Factored bearing resistance is determined by multiplying the nominal bearing resistance by the appropriate resistance factor. The resistance factor reduces the nominal bearing resistance based on analysis method and soil type. Refer to the current edition of AASHTO LRFD Bridge Design Specifications for appropriate resistance factors.

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GEOTECHNICAL	Section BEARING RESISTANCE FOR SHALLOW FOUNDATIONS Subject Granular Replacement

APPLICATIONS

Granular replacement of foundation materials may be used in areas where the bearing resistance of the original ground foundation materials is inadequate. When foundation alternatives for bridges are being evaluated and the use of shallow foundations is adversely affected by poor-quality materials, it is almost invariably more economical to switch to deep foundations than it is to modify the soils. However, granular replacement to increase bearing resistance typically is the selected method when poor soils occur beneath retaining walls or culverts. Replacement materials shall consist of granular embankment meeting the requirements in **Section 805** of the current edition of the Standard Specifications for Road and Bridge Construction. The maximum size limit for the granular embankment is 4 inches.

REQUIRED GEOMETRY

The area of granular replacement shall widen, at a minimum, with depth on a 1H:1V slope as shown in **Exhibit 38**. The granular material shall be wrapped with the appropriate geotextile fabric as specified by the Geotechnical Branch.

BEARING RESISTANCE ON GRANULAR REPLACEMENT

The engineer shall check the bearing resistance at two levels if low-strength soils are still present below the granular replacement materials:

- At the base of footing elevation
- At the base of the granular material

PARTIAL REPLACEMENT

Partial replacement may be used even though low-strength soils exist beneath the base of the excavation. The imposed loadings are spread over a larger area as they are transmitted through the granular material; and the greater the depth of granular replacement, the greater the reduction in the required bearing pressures. Also, the bearing resistance of a soil of uniform strength increases with depth.

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GEOTECHNICAL	Section: BEARING RESISTANCE FOR SHALLOW FOUNDATIONS
Transportation Cabinet, Department of Highways, Division of Structural Design, Geotechnical Branch	Subject: Restrictions for Spread Footings on Soil

BRIDGES

In general, spread footings for bridges on soil will be used only at dry crossings due to the possibility that footings used near streams or rivers could be undermined by scour.

Also, in general, single-span bridges are more suited to spread footings on soil versus multi-span bridges due to the potential for differential settlement between substructure units.

Spread footings on soil for bridges shall only be allowed with prior approval of the Division of Structural Design.

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GEOTECHNICAL

Section

BEARING RESISTANCE FOR SHALLOW FOUNDATIONS

Subject

Bearing Resistance of Spread Footings on Rock

EVALUATION TECHNIQUES

The bearing resistance of spread footings on rock may be evaluated based upon:

- Visual inspection of the rock cores by a geologist
- Available bearing resistance correlation and mapping
- Kentucky and/or Standard Rock Quality Designation (RQD)
- Laboratory testing such as slake durability index, jar slake, and unconfined compressive strength

EVALUATING ROCK MASS STRENGTH

The *AASHTO LRFD Bridge Design Specifications*, current edition, specifies using the Rock Mass Rating (RMR) System to evaluate the rock mass strength of spread footings on rock. This method is used because it is the imperfections in the rock mass (such as inclined bedding, joints, and faults) that limit strength and bearing resistance of the rock mass.

According to commentary in Section 10 of the *AASHTO LRFD Bridge Design Specifications*, current edition, other methods for assessing rock mass strength, including in-situ tests or other visual systems that have proven to yield accurate results may be used in lieu of the specified method. FHWA states that an acceptable practice is that footings on rock may be sized at the service limit state using presumptive bearing resistance values. Section 10 of the *AASHTO LRFD Bridge Design Specifications*, current edition and the federal government's *Naval Facilities Design Manual* (NAVFAC DM-7.2) shall be used as a guide in estimating the presumptive bearing resistance of rock at the service limit state.

CHECKING FOOTING SIZES

Typically, footings bearing on bedrock should be sized at the service limit state using the presumptive bearing resistance values. Footing sizes shall be checked at the strength and extreme event limit states using the nominal bearing resistance with appropriate resistance factors. The Geotechnical Branch should be contacted for additional guidance if checking the footing sizes at the strength or extreme limit states are necessary.

SCOUR IMPACTS

Base of footing elevations for spread footings on rock may be controlled by scour, as discussed in **GT-606**, “Scour Considerations.”

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GEOTECHNICAL

Section: SETTLEMENT

Subject: Overview

ANALYSIS

Evaluation of settlement is considered for service limit state design. This subject lists guidelines for performing settlement analyses.

WHEN ANALYSIS IS REQUIRED

Settlement analysis is performed in cases where the settlement magnitudes could be great enough to damage the structure or embankment, or in accordance with AASHTO Guidelines. In general, it is recommended that settlement analyses be performed if the bridge approach embankments are greater than 20 feet in height and the thickness of the compressible foundation soil is greater than 10 feet.

Analyses may be required for smaller approach fills or shallower foundations when the structures are particularly sensitive to settlement or where soils are particularly compressible.

METHODOLOGY

Settlement analyses shall be performed in accordance with the current edition of AASHTO LRFD Bridge Design Specifications.

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GEOTECHNICAL	Section: SETTLEMENT
Transportation Cabinet, Department of Highways, Division of Structural Design, Geotechnical Branch	Subject: Differential Settlement

GENERAL

Differential settlement refers to situations where part of a foundation (or part of a structure) settles more than other parts of the same foundation (structure). Settlements of this type are more likely to damage structures than larger settlements that occur uniformly. A common occurrence of differential settlement in highway construction occurs in lane additions and other widening projects where loading is nonuniform.

Differential settlement can occur even under conditions of uniform loading if there is marked variation in the properties, or depths, of foundation soils. In such cases, it is beneficial to compute and plot the settlement at several positions along a profile or cross-section to evaluate the magnitude and probable effects of differential settlement.

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GEOTECHNICAL	Section: SETTLEMENT
TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch	Subject: Controlled Loading

STAGED CONSTRUCTION

Large settlement magnitudes or differential settlements may be potentially detrimental to a structure or embankment and cause failure. To avoid or reduce the effects of such problems, the designer often recommends that some critical phase of construction not proceed until much of the anticipated settlement has occurred. Estimates of the waiting period are necessary to make such a recommendation.

Because strength gain from short- to long-term conditions can be related to percent of consolidation, settlement rates are used with slope stability to determine the optimum loading rate. Waiting periods are also commonly used to control the driving of piles to eliminate or minimize downdrag or lateral squeeze.

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GEOTECHNICAL	Section: SETTLEMENT
TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch	Subject: Accelerating Consolidation Rates

ACCEPTABLE FOUNDATION MODIFICATIONS

The rate of settlement may be so slow that waiting periods are impractical. The following foundation modifications may be utilized to accelerate consolidation rates:

Ø Wick drain installation

Note: The FHWA's Soils and Foundations Reference Manual presents the recommended design procedure for wick drains. Refer to **Section 711** in the Standard Specifications for Road and Bridge Construction, current edition, and **Exhibits 35** and **36**.

Ø Surcharge loading

Other methods may be acceptable with prior approval of the Division of Structural Design's Geotechnical Branch.

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GEOTECHNICAL

Section

SETTLEMENT

Subject

Reducing Settlement Magnitudes

MINIMIZING SETTLEMENT

Settlement magnitudes anticipated for deep or highly compressible foundation soils might be so great that they jeopardize the integrity of the proposed structures. In such cases, modification of the foundation may be required. Ground improvement methods used to reduce settlement magnitudes are the same as those used to improve stability or to increase bearing capacity.

Some techniques utilized to minimize settlement are detailed below.

REPLACEMENT OF FOUNDATION MATERIALS

For cohesive soils, the modification usually involves either a full or a partial replacement of the poor foundation materials. Full replacement involves removal of the compressible soil to bedrock and replacement with an incompressible (or less compressible) material.

In other cases, only the upper layers of compressible soil can be removed due to the practical difficulties of making deep excavations. In such cases, the geotechnical engineer must determine the depth of removal required to reduce settlement magnitudes to acceptable levels.

LIGHTWEIGHT FILL

Lightweight fill can sometimes be utilized in areas where soils are highly compressible. Lightweight fill has been utilized in the extension of box culverts. In the past designers utilized a stepped-down method of construction so that toward the end of the culvert, the culvert section was reduced and not designed to carry a full embankment height.

DYNAMIC COMPACTION

In certain situations, dynamic compaction may be used to increase the relative density of the in-place materials.

STONE COLUMN INSTALLATION

Another method of partial replacement is the installation of stone columns. Although only a portion of the foundation is removed, this method can extend to greater depths than the excavate-and-replace techniques. It is well suited, therefore, to situations where most of the settlement occurs in relatively deep layers.

APPROVAL REQUIRED

Documented techniques other than those discussed above may be acceptable. All techniques above and any other techniques require the approval of the Division of Structural Design's Geotechnical Branch.

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GEOTECHNICAL	*Section* RETAINING STRUCTURES & REINFORCED SOIL SLOPES
TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch	*Subject* Overview

RETAINING WALLS

Retaining walls are usually recommended in situations where typical embankment or cut slopes are not feasible. These situations usually occur in areas where right-of-way constraints exist.

REINFORCED SOIL SLOPES

Reinforced soil slopes may be used as an alternative to retaining walls if sufficient right of way is available. Reinforced soil slopes incorporate geogrids to increase the tensile strength of the soil mass. The reinforcement enables steeper slopes to be used.

Methods for designing retaining walls may be found in foundation engineering texts and in FHWA references listed in the succeeding subjects. An example of the Geotechnical Notes Sheet for MSE and Reinforced Concrete Cantilever Walls are shown in **Exhibits 38 and 38-1**.

GEOTECHNICAL	*Section* RETAINING STRUCTURES & REINFORCED SOIL SLOPES
	Subject Gravity, MSE, & Cantilever Retaining Walls

OVERVIEW

This subject pertains to concrete gravity (modular or cast in place), mechanically stabilized earth (MSE), and reinforced concrete cantilever retaining walls. The Geotechnical Branch shall maintain a list of approved proprietary wall vendors in coordination with the Division of Materials. The geotechnical engineer shall provide guidance regarding appropriate wall systems for specific sites.

WHEN WALL ANALYSIS IS REQUIRED

Small walls that meet the requirements presented in Standard Drawing **RGX-002,** "Retaining Wall, Gravity-Type, Non-Reinforced," generally do not require individual site-specific designs. A geotechnical analysis may not be required for walls with total heights of 8 feet or less. All other walls require individual designs for which the following guidelines are applicable.

INTERNAL STABILITY

The geotechnical engineer will determine whether the walls will be founded on soil or rock and will also estimate the strength parameters of the foundation materials. The structure designer determines the internal stability of cantilever walls (concrete and steel reinforcement). The proprietary MSE wall designer determines the internal stability of MSE walls, which is dependent on the width of the wall (length of straps, grids, etc.) being sufficient to prevent pullout failure. The proprietary precast modular gravity block wall designer determines the internal stability of the modular block wall based on the geometric and physical properties of the structure.

Bridge loads shall be founded on deep foundations for non-cast in place abutments. Foundation elements shall be extended through and isolated from the MSE wall reinforced volume. Spread footings to support bridge loads on top of MSE walls require the permission of the Division of Structural Design.

EXTERNAL STABILITY

The geotechnical engineer shall determine the external stability which includes the service limit state (settlement and slope stability) and strength limit state (bearing, sliding, and base eccentricity). The wall designer shall verify stability based on final design.

MODELING

Target safety factors for slope stability analysis are presented in **GT-601-3**, “Target Safety Factors.” It is assumed for external stability calculations that the internal stability of the wall is adequate. This wall area includes the entire reinforced volume of an MSE wall and the soil above the heel of a cantilever retaining wall.

GEOMETRY

The geotechnical engineer shall check settlement, lateral sliding, bearing resistance, and eccentricity by assuming the following:

- For MSE, the minimum reinforced length is 0.7 times the height of the wall or 8 feet, whichever is greater
- For reinforced concrete cantilever walls, the footing width is two-thirds of the wall height
- For concrete gravity walls (precast modular gravity block), as specified by the supplier
- For concrete gravity walls (cast in place), as detailed in Standard Drawing **RGX-002**

If the initial results are unacceptable, the engineer may choose from several options to improve stability including, but not limited to:

- Foundation replacement to increase the bearing resistance (see **Exhibits 38-1 and 38-2**)
- Adjustment of wall dimensions
- Use of granular backfill to decrease lateral loading
- Consideration of other wall types

The wall design shall conform to the requirements of the *AASHTO LRFD Bridge Design Specifications* with the following exception~~s~~:

- For internal backfill of MSE walls, refer to the *Standard Specifications for Road and Bridge Construction,* current edition, **Section 805**, “Reinforced Fill Material.”

- Minimum embedment shall be:

  - 2 feet to the bottom of the footing for cast-in-place walls
  - 2 feet to the top of the leveling pad for walls with precast panels

- Only inextensible reinforcement is allowed for MSE walls unless approved by the Division of Structural Design, Geotechnical Branch.

GEOTECHNICAL	Section RETAINING STRUCTURES & REINFORCED SOIL SLOPES
TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch	Subject Tieback & Soil Nail Retaining Walls

OVERVIEW

This subject discusses applications of tieback and soil nail retaining walls and provides some design references.

Tieback or soil nail wall structures may be used to retain slopes, to underpin structures, or to correct landslides. Tieback and soil nail walls may be used for temporary or permanent applications.

TIEBACK WALL DESIGN

Tieback wall design involves both geotechnical and structural aspects. Geotechnical aspects include, but are not limited to:

- Ø Determining soil and rock parameters
- Ø Choosing proper methods of analysis
- Ø Calculating lateral loads
- Ø Performing global stability analyses
- Ø Determining the size and scope of the wall to be constructed

Design methods and construction techniques vary but should, in general, agree with FHWA's Geotechnical Engineering Circular No. 4 (Ground Anchors and Anchored Systems) and the current edition of AASHTO LRFD Bridge Design Specifications.

The preferred method of contracting is for the Geotechnical Branch or geotechnical consultant to provide loads and geotechnical design parameters and for the specialty wall contractor to perform the detailed structural design.

SOIL NAIL WALL DESIGN

Soil nail wall design involves both geotechnical and structural aspects. Geotechnical aspects include, but are not limited to:

- Determining soil properties
- Choosing proper methods of analyses
- Performing global stability analyse
- Determining the size and scope of the wall to be constructed

Design methods and construction techniques vary but should, in general, agree with FHWA's Geotechnical Engineering Circular No. 7 (Soil Nail Walls).

The preferred method of contracting is for the Geotechnical Branch or geotechnical consultant to provide geotechnical design parameters and for the specialty wall contractor to perform the detailed structural design.

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GEOTECHNICAL

TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch

Section

RETAINING STRUCTURES & REINFORCED SOIL SLOPES

Subject

Railroad Rail Retaining Structures

WHEN TO USE RAILS

Railroad rails installed as drilled-in piling may be used for correction of landslides in sidehill sections or embankments involving the roadway shoulder and a limited portion of the driving lanes. Refer to **Exhibit 34a** for typical railroad rail installation details.

MODELING

Except in cases where slope inclinometers or other instrumentation indicate that a mass of stable soil underlies the failure surface, it will be assumed that the failure surface is located at the top of bedrock. Determination of the depth to bedrock and field soil classifications may be made with disturbed soil borings. Rock cores shall be obtained if disturbed soil borings prove inconclusive in determining top of bedrock. Rails typically should not be used when the distance from the shoulder to the farthest breaks in the pavement is greater than the depth to bedrock.

MINIMUM EMBEDMENT

Minimum embedment into bedrock is approximately half the distance from the ground surface to the bedrock (minimum of 10 feet). A slightly deeper pre-augered hole may be necessary to allow for auger cuttings falling into the hole and possibly preventing the rail from extending to the required embedment depth.

CENTER-TO-CENTER SPACINGS

Minimum center-to-center spacing of the rails is 2 feet. Maximum spacing is 4 feet, since soil arching between the rails may not develop if larger spacings are allowed.

USE OF MULTIPLE ROWS

Multiple rows of rails may be required when conditions warrant. When using this method, a spacing of approximately 2 feet between staggered rows is required to allow the rows to act as a unit in retaining the sliding mass.

ORIENTATION OF THE RAILS

Flanges on the rails shall be positioned perpendicular to the direction of landslide movement to utilize the full strength of the rail cross-section. The Geotechnical Branch will analyze and determine the appropriate design method.

RAIL SIZE

Rails come in multiple sizes (Lbs/YD). The designer shall clearly indicate the minimum size rail required.

BACKFILLING OF HOLES

Installed rails shall be backfilled with concrete, pea gravel, crushed limestone, or crushed sandstone. The granular backfill material shall have 100 percent passing the ½-inch sieve. Drill cuttings are not permitted. Granular backfill shall be shoveled or dropped in small amounts to prevent voids from forming around the rails. Backfilling is incidental to the price per linear foot for installation.

RETENTION OF SOIL BACKFILL

Rails are not to be damaged when placing or compacting backfill behind the rail wall. Retention of the backfill may require the use of lagging. Lagging may be wood, guardrail, or concrete panels. Lagging shall be extended to bedrock or to a minimum depth of 12 feet below the finished grade in front of the retaining structure.

EROSION CONTROL

Severe erosion on the slope below a rail structure could be detrimental to its long-term performance. Suitable erosion control shall be provided as a part of the initial design if there is a potential for severe erosion.

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GEOTECHNICAL	Section RETAINING STRUCTURES & REINFORCED SOIL SLOPES Subject Reinforced Soil Slopes

WHEN TO USE

Situations in which slopes are particularly suited to the use of reinforced soil slopes may include the following cases:

- Ø The on-site materials do not have the necessary strength, and the use of granular materials is not economically feasible.
- Ø Right-of-way restraints require the use of steepened slopes or walls.
- Ø The embankments must span areas of soft foundation soils.

METHODOLOGY

The following FHWA reference presents material characteristics of various reinforcement materials, design consideration and procedures, and cost estimates:

- Ø GEC No.11 – Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Slope

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GEOTECHNICAL	*Section* PILE & DRILLED SHAFT DESIGN
TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch	*Subject* Overview

USE OF PILES

Driven piles are generally recommended at abutments if the depth to bedrock is greater than 15 to 20 feet below roadway grade. This allows a supported length in soil of at least 10 to 14 feet exclusive of the distance from roadway grade to base-of-pile-cap. At abutment and pier locations, the recommended minimum length of pile supported by soil is 10 feet. At times predrilling into bedrock may be required to ensure adequate embedment. Additional pile lengths may be required in areas subject to scour or in areas where the in-situ soils offer little or no lateral resistance.

TYPES OF PILES

The department generally uses steel H-piles in point-bearing applications. 12-inch H-piles of various weights per unit length are the most commonly used, although 14-inch H-piles are used in cases where they may be required to support large vertical or lateral loads or large bending moments.

Steel H-piles, steel pipe piles, or square precast concrete piles (generally prestressed) are used in friction pile applications. Historically, commonly used concrete pile sizes were 14-inch and 16-inch nontapered piles, but these are seldom used because of the difficulty in splicing. Steel pipe piles are gaining popularity as friction piles due to their ease of splicing and resistance to seismic forces. Steel pipe piles of 16-inch diameter with 0.5-inch thick walls are commonly specified; however, with permission other sizes may be selected based on site and loading conditions. Pile points are typically specified to create a closed end condition for the pipe piles.

In some cases, the subsurface conditions may not be suitable for the pile types listed. In these circumstances, other pile types may be appropriate with the consent of the Division of Structural Design, Geotechnical Branch.

DRILLED SHAFTS

Drilled shafts are a foundation alternative to driven piles; however, economic comparisons are necessary to determine the most cost effective method for a specific site. One distinct advantage of drilled shafts is that several large-diameter shafts may be used instead of many small-diameter piles. Drilled shafts are particularly applicable for situations with severe scour, with large applied lateral loads, near railroad tracks, or where excavations for constructing foundations or the use of driven pile foundations is not practical.

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GEOTECHNICAL	Section: PILE & DRILLED SHAFT DESIGN
TRANSPORTATION CABINET, Department of Highways, Division of Structural Design, Geotechnical Branch	Subject: Tip Elevations of Point-Bearing Piles

POLICY

A recommendation for point-bearing piles shall consist of an estimated tip elevation based on rock cores and rockline soundings. If the rockline elevations vary significantly across the width of a pile bent, a recommendation shall provide elevations on each end of the bent (or at both ends and in the center).

The effects of steeply sloping rockline on battered pile lengths shall be considered. It is common practice during construction to drive a "test pile" at each pile bent to confirm the predicted pile lengths. Additional test piles may be necessary in some cases such as sloping rockline, karstic areas, etc.

Pile points are required on all driven point-bearing piles. A recommendation of a required type of pile point may be necessary for breaking through boulders, seating on sloping rockline, etc. Predrilling the piles may be required to reach the required pile tip elevation. In most instances, reinforced pile points are not required for drilled piles.

At locations exhibiting intense karst characteristics the geotechnical engineer may recommend a reduced structural resistance factor to account uncertainty at the bearing elevations.

When steel pipe piles are utilized, pile points are usually specified to create a closed-end condition. Open-end pipe piles, if used, may or may not require cutting shoes depending on the driving conditions and load bearing requirements.

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GEOTECHNICAL	Section PILE & DRILLED SHAFT DESIGN
TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch	Subject Nominal Bearing Resistance of Friction Piles

OVERVIEW

A number of methods have been developed for estimating the driving resistance and nominal bearing resistance of friction piles. The Department requires analysis methods be selected and factored as described in the current edition of AASHTO LRFD Bridge Design Specifications, Section 10.

DETERMINING RESISTANCE

Friction piles shall have a minimum embedment of 10 feet into natural ground. Additional pile lengths may be required in areas subject to scour or in areas where the in-situ soils offer little or no lateral resistance. Any bearing resistance, which might be developed within embankment or scourable materials, shall be ignored. The effects of high water or fluctuations in groundwater levels upon bearing resistance shall be taken into account.

When H-Piles are utilized as friction piles, pile points should not be used.

PRESENTATION OF THE DATA

It is recommended that the factored pile resistance data be developed and presented in a tabular format. Appropriate resistance factors shall be applied to the nominal pile resistance data and the information presented as “Total Factored Geotechnical Axial Resistance” using the resistance factors indicated by AASHTO LRFD Bridge Design Specifications, current edition.

Overburden pressures can affect friction piles. Therefore, the placement of additional fill after pile driving will cause the long-term resistance of the piles to be different than the driving resistance. In some cases these may differ sufficiently to require that the designer be provided with both values. The long-term resistance of piles in scour situations or piles passing through newly constructed embankments will be less than the driving resistance.

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GEOTECHNICAL	Section: PILE & DRILLED SHAFT DESIGN
Transportation Cabinet, Department of Highways, Division of Structural Design, Geotechnical Branch	Subject: Dynamic Pile Testing & Constructability Considerations

ESTIMATING DRIVING STRESS

Piles can be damaged when stresses induced during pile driving exceed the structural capacity of the pile. A wave equation analysis can be used during the design phase (and reevaluated during construction, if necessary) to estimate the pile driving stresses, the pile penetration per blow, and nominal resistance of the pile.

DYNAMIC PILE TESTING

Dynamic testing with signal matching may be used during construction to measure the energy imparted to the pile by the hammer, the stresses in the pile during driving, and the nominal resistance of the pile.

DAMAGE AVOIDANCE

If piles must penetrate layers of dense granular soils, resistance to pile driving may become so great that the piles could be damaged by the driving process. Piles that are intended to bear upon rock must reach the bedrock surface. Friction piles will have some minimum tip elevation that must be reached to allow the piles to resist anticipated lateral loads or to have adequate axial or lateral axial long-term resistance in the event that much of the material in which they are embedded is removed by scour. In such cases, predrilling at the pile locations, or jetting performed during the driving process, may be necessary to allow the pile tips to penetrate the required distance.

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GEOTECHNICAL	Section: PILE & DRILLED SHAFT DESIGN
Transportation Cabinet, Department of Highways, Division of Structural Design, Geotechnical Branch	Subject: Axial Resistance of Drilled Shafts

OVERVIEW

Analysis methods for estimating the axial resistance of individual drilled shafts, as well as allowing for group effects, are presented in FHWA GEC No.10 – Drilled Shafts: Construction Procedures and LRFD Design Methods. For design methodology the designer should refer to the current edition of AASHTO LRFD Bridge Design Specifications. Typically, only the axial resistance of unweathered bedrock is considered for shafts socketed into bedrock; the overburden and weathered bedrock are usually neglected.

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GEOTECHNICAL

TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch

Section

PILE & DRILLED SHAFT DESIGN

Subject

Evaluating Resistance to Lateral Loads

USE OF BATTERED PILES

Deep foundations are generally subjected to both axial and lateral loadings. Battered piles are commonly employed to resist lateral loads; however, consideration of the resistance of vertical piles to lateral loads is increasing. Battered drilled shafts are seldom used due to the difficulty of their construction.

METHODOLOGY

Methodologies for the design of piles or drilled shafts subjected to lateral loads include "Brom's method" and the "p-y (pressure vs. deflection) method." Design procedures are presented in FHWA GEC No.10 – Drilled Shafts: Construction Procedures and LRFD Design Methods. Computer programs are available to assist in performing these analyses. Many software programs can evaluate the resistance of single shafts/piles, or groups of shafts/piles, to lateral loads.

LATERAL LOADS

Generally foundation configurations, loading conditions, and structural details are not known during the geotechnical investigation. Therefore, the geotechnical information necessary for a lateral load analysis should be provided to the structural engineer for conducting soil/structure interaction analyses. The structural engineer should involve the geotechnical engineer in reviewing and assisting in refinement of the analyses.

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GEOTECHNICAL

Section

PILE & DRILLED SHAFT DESIGN

Subject

Pull-Out Resistance

POLICY

Uplift forces may be applied to deep foundations as a result of barge impact, lateral loads, swelling soils, buoyancy, wind loads, seismic, etc. Deep foundations must be designed to withstand applicable tensile forces, and adequate pull-out resistance must be provided.

The factored geotechnical uplift resistance can be calculated by using the appropriate factor from the current edition of AASHTO LRFD Bridge Design Specifications.

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GEOTECHNICAL	Section: PILE & DRILLED SHAFT DESIGN
Transportation Cabinet, Department of Highways, Division of Structural Design, Geotechnical Branch	Subject: Negative Skin Friction (Downdrag)

METHODOLOGY

Downdrag on the piles may be a problem for structures where the foundation soils exhibit slow consolidation rates. In the case of point-bearing piles, the piles are considered to carry downdrag loads if the foundation soils undergo more than 0.4 inch of settlement after the piles are driven. The geotechnical engineer shall adhere to analysis procedures and design considerations discussed in the current edition of *AASHTO LRFD Bridge Design Specifications*.

WHEN DOWNDRAG IS ASSUMED TO OCCUR

In calculating downdrag, it is assumed that 0.4 inch of settlement is required to mobilize the side friction. Therefore, it is necessary to determine the interval from the rock surface upward to the point where 0.4 inch of settlement occurs after the piles are driven. Downdrag loads are not considered to be applicable over this interval. In some cases downdrag loads on point-bearing piles may be neglected.

REDUCING DOWNDRAG LOADS

The piles will be capable of carrying the bridge loads plus the downdrag loads in many cases. Otherwise, it may be necessary to use a cylindrical steel sleeve or a polypropylene sleeve on the portion of the pile in the new embankment so that the downdrag problems can be reduced.

USE OF WAITING PERIOD

It may be possible to use a waiting period between completion of the embankment and pile driving to reduce the downdrag loads. Downdrag loads are considered eliminated if the remaining settlement after the waiting period is less than 0.4 inch. Settlement platforms are required if a waiting period is selected as the method of handling downdrag loads; however, they are not needed if any other method is selected.

DOWNDRAG LOADS ON FRICTION PILES

Friction piles may also be subject to downdrag loads. The loads due to downdrag shall be determined and applied according to the current edition of *AASHTO LRFD Bridge Design Specifications*. The neutral point method described in NCHRP 343 may also be used to determine downdrag loading.

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GEOTECHNICAL

TRANSPORTATION CABINET
Department of Highways
Division of Structural Design
Geotechnical Branch

Section

PILE & DRILLED SHAFT DESIGN

Subject

Settlement of Friction Piles

POLICY

Evaluation of settlement of friction piles is included in service limit design. Geotechnical engineers shall determine the service limit design of driven foundation and drilled shafts as discussed in the current edition of AASHTO LRFD Bridge Design Specifications. Settlement magnitudes for friction piles shall be determined for all dry crossings where embankment settlement analyses are required and for multi-span structures at wet crossings.

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GEOTECHNICAL	Section: PILE & DRILLED SHAFT DESIGN
TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch	Subject: Lateral Squeeze

OVERVIEW

Rotation and horizontal displacement of abutments and piers on piles can be attributed to lateral squeeze. Lateral squeeze is the deformation and displacement of a soft cohesive foundation under embankment loadings.

POLICY

Lateral squeeze shall be checked whenever the weight of the embankment in the vicinity of the bridge abutments is greater than three times the cohesive strength (total stress) of the foundation soils. The determination of lateral squeeze magnitudes and design solutions for preventing damage resulting from lateral squeeze are presented in the FHWA's Soils and Foundations Reference Manual.

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GEOTECHNICAL	Section PILE & DRILLED SHAFT DESIGN
TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch	Subject Load Testing

POLICY

Projects incorporating large numbers of drilled shafts or piles may provide an economic justification for conducting a load test to verify geotechnical resistances as estimated by other methods. Loading procedures and requirements are presented in ASTM D8169. The LRFD design specifications encourage the use of load testing by allowing increased resistance factors for various testing methods, as detailed in the current edition of AASHTO LRFD Bridge Design Specifications.

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GEOTECHNICAL	Section: SCOUR CONSIDERATIONS
TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch	Subject: Scour of Soil

DETERMINING SCOUR POTENTIAL

Flowing water can adversely affect highway structures and embankments. The geotechnical engineer, design engineer, structural engineer, and hydraulic engineer must work together to provide a design that will be resistant to scour-related damages. After the hydraulic engineer provides the geotechnical engineer with the calculated scour potential, the geotechnical engineer determines the appropriate foundation design recommendations.

DESIGNING FOR SCOUR

Potential scour depths shall be determined using the procedures outline in the current edition of AASHTO LRFD Bridge Design Specifications. Design procedures addressing scour are as follows:

Ø **End Bents**—Typically, properly sized slope protection is utilized to neutralize any local scour on bridge approach and spill-through slopes. Deep foundation designs (piles/shafts) shall be checked with no lateral support in the worst-case contraction scour conditions. To check for potential exposed lengths:

1. Construct a vertical line from the toe of the spill-through slope where the stone slope protection terminates, down to the contraction scour depth for the respective end bent

2. Construct a 1H:1V (45°) line (from the above point) back toward the end bent until it intercepts the foundation element line.

The foundation can either be designed to withstand the potential unsupported length, the cap can be set down to that depth to avoid any unsupported length, or a combination of these measures can be employed.

DESIGNING FOR SCOUR (CONT.)

- **Piers**—Foundation elements (piles/shafts) shall be designed for total scour (contraction + local scour) conditions. They can either be designed to withstand the potential unsupported length, the cap can be set down to that depth to avoid any unsupported length, or a combination of these measures can be employed.

- **Walls**—Walls must be analyzed for problems with scour on a case-by-case basis where applicable. Many of the same procedures pertaining to bridges can be utilized in dealing with potential scour at walls.

- **Culverts with paved flowline**—Typically, with the use of paved flow lines, scour is not detrimental at culverts. However, scour holes at culvert outlets can cause problems with wingwall foundations. The hydraulic engineer should analyze outlet velocities and size riprap or design paved outlets to reduce potential scour problems.

- **Three-sided structures**—Where three-sided structures are used (such as box or arch culverts with a natural bottom and no paved flowline), scour should be calculated to ensure that the footings are constructed below any potential scour elevation.

- **Spread Footings** - For bridges, in general, spread footings on soil are not used at wet crossings because of the potential for scour undermining the footings.

Other scour mitigation techniques may be acceptable but require approval by the Division of Structural Design, Geotechnical Branch.

GRAINED SIZE USED TO EVALUATE RESISTANCE TO SCOUR

When required for scour calculations, the geotechnical engineer shall provide the required grain size value to the drainage engineer. The grain size values are obtained from the particle size distribution curve from soil testing.

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GEOTECHNICAL	Section: SCOUR CONSIDERATIONS
TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch	Subject: Scour of Bedrock

REQUIREMENTS FOR FOOTINGS ON SCOUR-PRONE FOUNDATIONS

Spread footings on rock above the maximum calculated scour elevation shall be evaluated to ensure that the bedrock is regarded as scour-resistant. If the bedrock is regarded to be scour-prone, the engineer shall lower the footing to the maximum scour elevation or to a scour-resistant bedrock layer, whichever is higher. Mitigating the scour may be accomplished by embedding the footings 1 to 3 feet into the bedrock and backfilling the entire excavation with mass concrete to the top of the bedrock surface.

EVALUATING SCOUR RESISTANCE

Definitive guidelines relating to susceptibility of rock to scour are not currently available. In the absence of a better method for classifying rock as scour-resistant or scour-prone, the Department recommends that questionable materials be classified as scour-prone. The following criteria should be considered in evaluating susceptibility to scour:

- Existing field conditions
- Lithology
- Rock Quality Designation, Kentucky Method (KY RQD)
- Slake Durability Index
- Jar Slake Test
- Visual inspection of rock cores

EXISTING FIELD CONDITIONS

In evaluating existing field conditions, overall topography of the area should be noted.

In areas with steep gradients, flash floods could produce extremely high flow velocities, possibly scouring some rock that would not be prone to scour in less adverse conditions.

Limestone slabs and other loose rock in stream beds could simply represent mass wastage of hillsides or cliffs bordering the stream, but if similar materials are likely to be present below flowline, they should be taken as an indication that the stream bed could undergo further degradation.

Ponded water in perennial streams will protect their beds from freezing, but the exposed beds of intermittent streams will be subject to freeze-thaw or wetting-drying cycles and associated degradation.

In the case of bridge replacements, the condition of the existing bridge is a good indication of the probability of scour. Evidence that local or contraction scour has affected the existing structure is good evidence that scour potential is high.

Also, joints and fractures in the exposed bedrock should be observed. If present, an evaluation should be made to determine whether their presence and orientation would facilitate the scour process.

LITHOLOGY

Lithology is a principal factor relating to whether or not a particular mass of rock is susceptible to scour. Essentially all of the near-surface rock in Kentucky is sedimentary and can be divided into the following three major groupings:

- **Sandstone:** Massive, firmly cemented sandstones are considered non-scourable. However, friable (nondurable) sandstones, in which the cements binding individual grains are weak, are susceptible to scour.

- **Limestone:** Massive limestones are considered to be scour-resistant for the structure life. However, thinly bedded limestone may be susceptible to scour.

- **Shale:** The scour susceptibility of shales relates to their “durability” as defined by SDI test results. The Geotechnical Branch includes siltstones within the broad shale classification. Hard, massive siltstones are considered to be scour-resistant. Shales with SDI values greater than or equal to 95, termed “durable,” are considered to be scour-resistant. Of the “nondurable” shales, those that have SDI values from 50 through 94 are generally considered potentially scourable. Shale with SDI values less than 50 are considered to be soil-like and, therefore, scourable.

An interbedding of the basic lithologic types also frequently occurs. Thinly interbedded units, where shale layers alternate with thin layers of a more resistant rock type (limestone or sandstone) are considered to be potentially scourable. As the percent of resistant beds increases, the susceptibility to scour decreases.

CORE RECOVERY

Core recovery, the length of core recovered expressed as a percentage of the length of the interval drilled, can be used as a measure of competency of the rock. When core recoveries of less than 85 percent are obtained, the core should be inspected to determine if loss was due to the poor quality of the material or was due to the drilling procedure. Recovery of less than 85 percent may indicate the material is scour-susceptible if losses were due to material quality.

KY RQD

Any rock with a KY RQD of less than 25 is considered to be potentially scourable.

SDI & JAR SLAKE

The Slake Durability Index test (SDI) and the Jar Slake test are applicable to shales and friable sandstones. On occasion, they could be applicable to very argillaceous, shaley limestones. The rapid breakdown that occurs when some shales are immersed in water is an obvious indication that those materials would not be capable of resisting scour. The Jar Slake test will readily identify such units. The SDI test, with its losses occurring as a result of abrasion in an aqueous environment, is a somewhat more subtle measure of resistance to weathering.

VISUAL INSPECTION

Visual inspection of rock cores can provide an indication that characteristics such as cross-bedding, interbedding, partings or laminations, joints, or fissility might provide zones of weakness, which might facilitate scour processes.

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GEOTECHNICAL	Section: SUBGRADES
TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch	Subject: CBR Design Values

USE OF CBR IN PAVEMENT DESIGN

An optimal design of roadway pavements must reflect the amount of support that the pavement receives from the underlying subgrade. A firm subgrade, which is rigid and provides good support, will allow the use of a thinner (lower-strength) pavement. Conversely, if the subgrades are poor, providing little or no support, the pavements must be thicker and of higher strength to avoid rapid deterioration under applied loads.

The CBR (California Bearing Ratio) is a measure of the quality of the subgrade and is used by pavement designers as an indication of how much of the deformation-loading can be transferred to the underlying subgrade and how much must be supported by the pavements. The department uses a modified CBR test method as presented in **KM 64-501**.

TYPICAL MATERIALS

Select Rock Quantities are calculated on roadway projects to assist in determining the availability of rock from roadway excavation.

After areas requiring durable rock (such as embankment, working platforms, slope protection, channel lining, etc.) are satisfied, additional durable rock (limestone, sandstone, or shale with SDI $\geq$ 95) can be used for a 2-foot rock roadbed (if feasible). If a sufficient quantity of durable rock is not available from roadway excavation, a 1-foot soil subgrade or rock borrow is recommended.

Nondurable shale is not recommended in the top 2 feet of the subgrade.

DETERMINING CBR DESIGN VALUES

The pavement design value for a project is determined from laboratory tests on soil samples. CBR and classification tests are performed on bag samples of soil from roadway cut sections. These tests are also performed on bag samples from fill sections whenever applicable.

Typically, the lowest CBR value from laboratory tests is recommended (unless it is an isolated value) as the design value, unless rock roadbed or bank gravel is applicable. On large projects (typically more than 20 CBR tests) Yoder's 90th percentile method is used to calculate an optimum CBR design value. Principles of Pavement Design by Yoder and Witczak provides additional information.

The recommended CBR value is included in the Geotechnical Engineering Report as a design recommendation but not as a geotechnical note. This value is used in determining pavement configurations.

BRIDGE REPLACEMENT PROJECTS

Bridge replacement projects in which a Geotechnical Roadway Report will not be issued may include a recommended CBR design value for pavement in the Structure Report.

TYPICAL VALUES

Following is a range of typical CBR design values. Engineering judgment is important in the selection of an appropriate value.

MATERIAL	ESTIMATED CBR VALUE
Rock (limestone, durable siltstone, durable sandstone)	9 to 11
Rock (non-acidic durable shale, friable sandstone)	7 to 9
Bank gravel	6 to 9
Soil	1 to 6

2 2 2

GEOTECHNICAL

Section

SUBGRADES

Subject

Chemical Stabilization of Subgrades

WHEN STABILIZATION IS NEEDED

Some of the soils along a proposed highway route may have such poor strength characteristics that their occurrence can negatively impact construction operations by rutting and pumping of the subgrade. Stabilization of subgrades is used, when necessary, for the purpose of improving such soils sufficiently to provide an adequate construction platform.

The strength of a stabilized subgrade may be considered in design of the pavement structure, at the discretion of the Division of Highway Design. Stabilization of a soil subgrade shall be considered whenever the CBR design value is less than or equal to 6.0.

CHEMICAL STABILIZATION

Chemical stabilization consists of mixing a reactionary agent such as lime or cement with the soil. This mixture cures into a solid cementitious working platform.

TREATMENT WITH LIME

Clayey soils (plasticity indices greater than 15 and more than 35 percent passing a #200 sieve) are normally treated with lime.

TREATMENT WITH CEMENT

Silty or sandy soils (plasticity indices less than or equal to 25 and less than 35 percent passing a #200 sieve) are normally treated with cement.

GUIDELINES

The appropriate chemical will be determined in accordance with FHWA's Soil Stabilization Manual, **FHWA-IP-80-2**. Subgrade construction using lime and cement shall comply with **Section 208** of the current Standard Specifications for Road and Bridge Construction.

CHEMICAL MODIFICATION

When drying of the soil subgrade is required, chemical modification of the soil can be considered. Chemical modification consists of mixing a chemical modifier such as kiln dust with the soil. This mixture does not increase the design soil strength.

2 2 2

GEOTECHNICAL	Section: SUBGRADES
TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch	Subject: Mechanical Stabilization of Subgrades

STABILIZATION OF SUBGRADES

Soil subgrades with a CBR design value of 6 or less should be considered for stabilization. When chemical stabilization is not deemed practical or economical, one of the following methods of mechanical stabilization may be considered as a viable alternate:

Ø For **Rock Stabilization**, use:

- ♦ 1 foot of rock (KY Coarse Aggregate No. 2s, 3s, or 23s) wrapped with a Geotextile Fabric meeting Geotechnical Branch and current Standard Specifications for Road and Bridge Construction requirements. This will be treated as an additional pavement layer for pavement design

- ♦ 2 feet of rock (KY Coarse Aggregate No. 2s, 3s, or 23s) wrapped with a Geotextile Fabric meeting Geotechnical Branch and current Standard Specifications for Road and Bridge Construction requirements. This will be treated as a 2-foot rock roadbed for pavement design

Ø For **Geogrid Stabilization**, install a layer of Geogrid covered with the necessary quantity and appropriate size of crushed aggregate.

Note: When a separator is required between the subgrade soil and the aggregate to prevent the migration of fines include with a Geotextile Fabric meeting Geotechnical Branch and current Standard Specifications for Road and Bridge Construction requirements.

GUIDELINES

The Geotechnical Branch will determine if stabilization is required and will recommend the appropriate method of treatment. Guidelines are as follows:

CBR 1 to 4: Option 1—Chemical stabilization using lime or cement as applicable

Option 2—12 inches (minimum) of coarse aggregate (2s, 3s, or 23s) wrapped with a Geotextile Fabric meeting Geotechnical Branch and current Standard Specifications for Road and Bridge Construction requirements

CBR 4 to 6: Option 3—Install a layer of Geogrid covered with the necessary thickness and appropriate size of crushed aggregate. If a separator is required install a Geotextile Fabric meeting Geotechnical Branch and current Standard Specifications for Road and Bridge Construction requirements.

2 2 2

GEOTECHNICAL	Section: CUT SLOPES IN ROCK
TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch	Subject: General Guidelines

GUIDELINES

Some cuts may expose several different types of rock (limestones, shales, sandstones, coal seams, etc.), and these lithologies will dictate the appropriate slope configuration to select.

Each cut shall be independently designed by using all subsurface information or field mapping available. This information is used to determine:

- Ø Cut slope angles
- Ø Lift heights
- Ø Bench widths
- Ø Bench Elevations
- Ø Base of rock disintegration zone
- Ø Soil overburden thickness
- Ø Overburden bench requirements

GT 601-4, "Cut Slopes in Soil," discusses cut slopes in overburden and weathered rock.

2 2 2

GEOTECHNICAL	Section: CUT SLOPES IN ROCK
Transportation Cabinet, Department of Highways, Division of Structural Design, Geotechnical Branch	Subject: Rock Cut Slope Configurations

BASIS FOR ROCK CUT SLOPE CONFIGURATION

Cut slopes in rock are influenced by lithology but are primarily based on joint inclination and continuity. Benches, where possible, are located at the top of the least resistant lithologic unit in a given rock cut section. Careful consideration must be given to SDI numbers and Jar Slake test results when designing a cut slope.

CLASS III NONDURABLE SHALE WITH OR WITHOUT LAMINATIONS

Typical cut slope recommendations for Class III nondurable shale are 2H:1V (or flatter) slope from groundline to ditchline. Normally these slopes are designed without a roadside ditch bench, intermediate benches, or overburden benches. (Refer to **Exhibit 14**.)

CLASS II NONDURABLE SHALE

Typical cut slope recommendations for Class II nondurable shale vary from 1H:1V to 3H:2V with roadside ditch benches, intermediate benches typically 18 feet wide, and approximate lift heights of 30 feet depending on rock competency. (Refer to **Exhibit 15**.)

CLASS I NONDURABLE SHALE

Typical cut slope recommendations for Class I nondurable shale vary from 3H:4V to 1H:4V, with approximate 30-foot lift heights, intermediate benches typically 18 feet wide, and a roadside ditch bench. (Refer to **Exhibit 16**.)

DURABLE SHALE

Typical cut slope recommendations for durable shale vary from 1H:2V to 1H:4V (depending on fractures) with roadside ditch benches, typical intermediate benches 18 to 20 feet wide, and approximate lift heights of 30 to 45 feet). (Refer to **Exhibit 17**.)

MASSIVE LIMESTONE OR SANDSTONE

Typical cut slope recommendations for massive limestone or sandstone vary from 1H:2V to 1H:20V. This material is usually stable; however, presence of joints, fractures, solution features, cross bedding, etc., will have as much influence on slope design as lithology. Materials placed on 1H:20V slopes may have lift heights up to 60 feet, with intermediate benches 18 to 20 feet wide. It is desirable to design the first lift above grade on a slope flatter than 1H:20V. (Refer to **Exhibit 18**.)

SHALEY LIMESTONE & SANDSTONES

Typical cut slope recommendations for shaley limestone and sandstone vary from 1H:1V to 1H:2V slope with lift heights from 30 to 45 feet and intermediate benches 18 to 20 feet wide. Flatter slopes may be required depending upon the percent and type of shale present. (Refer to **Exhibit 19**.)

2 2 2

GEOTECHNICAL

TRANSPORTATION CABINET
Department of Highways
GB
Division of Structural Design
Geotechnical Branch

Section

CUT SLOPES IN ROCK

Subject

Intermediate & Overburden Bench Widths

INTERMEDIATE BENCHES

The elevation of most intermediate benches is determined by changes in lithology, with the bench being on top of the least resistant material, where possible. The width of intermediate benches may vary from 15 to 25 feet. Typical bench widths are 18 feet. Intermediate bench widths may be 20 to 25 feet when lifts exceed 30 feet in height or in situations where shale is expected to weather rapidly and undercut a massive bedded material. Coal mine openings with weak roof material or other unstable slopes with anticipated heavy rock fallout may also require wider intermediate benches.

Intermediate benches that intercept ditch grade should be transitioned out within a distance of 150 to 200 feet to avoid leaving a transverse rock wall in the cut slope.

OVERBURDEN BENCHES

Overburden benches are placed on top of rock cuts at the base of the Rock Disintegration Zone (RDZ). Typical overburden benches are 15 feet wide and may be wider in areas where instability is anticipated. The depth to the base of RDZ is measured vertically from groundline and may be highly variable.

Overburden benches are drawn on cross-sections and will have some grade through the cut depending on variations in depth of material. These benches are sometimes omitted in mountainous terrain or in cuts where the overburden is less than 10 feet deep.

2 2 2

GEOTECHNICAL

Section	CUT SLOPES IN ROCK
Subject	Serrated Slopes

USE OF SERRATED SLOPES

Serrated slopes are utilized as a means of controlling erosion and establishing vegetation on soft rock formations, shale, or other material that can be excavated by bulldozing or ripping.

Serrations may be recommended for 1H:1V or flatter cut slopes. Typical step risers will vary from 2 to 4 feet and shall be plotted on the cut stability sections. (Refer to **Exhibit 20**.)

2 2 2

GEOTECHNICAL	Section: CUT SLOPES IN ROCK
TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch	Subject: Roadside Ditch Bench

USE OF ROADSIDE DITCH BENCH

When cut slopes are steeper than 3H:2V and the 30-foot safety clear zone from edge of pavement to the cut slope is not required, a roadside ditch bench is recommended.

Typically the width of the roadside ditch bench from outside edge of shoulder to the cut slope will be 12 feet for cuts less than 30 feet in height, and 14 feet for cuts over 30 feet in height. Standard designs shall include a foreslope of 6H:1V.

2 2 2

GEOTECHNICAL

TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch

Section

CUT SLOPES IN ROCK

Subject

Slope Design without Intermediate Benches & with Catchment Areas

POLICY

Design of a rock cut slope without intermediate benches should be used with a roadside ditch catchment area. The continuous cut slope design should be considered under the following circumstances:

- Rock is homogenous.
- Rock consists of limestones or sandstone of low KY RQD numbers that are interbedded with shale of low SDI numbers.
- Intermediate benches will accumulate debris rapidly, making them ineffective.
- Joints are discontinuous, and massive failures are unlikely.

The roadside ditch catchment area (**Exhibit 21**) shall be designed using the guidelines outlined in the “Rockfall Catchment Area Design Guide” Final Report, which was published by the Oregon Department of Transportation Research Group and FHWA (November 2001) [SPR-3-032 (Report # FHWA-OR-RD-02-04)].

The “Rockfall Catchment Area Design Guide” is a current state-of-the-practice reference for sizing rockfall catchment areas for 40- to 80-foot high cut slopes. A copy of the guidelines is available upon request from the Kentucky Department of Highways, Division of Structural Design, Geotechnical Branch.

2 2 2

# GEOTECHNICAL	*Section* CUT SLOPES IN ROCK
	Subject Summary of Rock Quantities

SUBMITTAL OF QUANTITIES

The design engineer will complete a TC 66-208 form, *Summary of Rock Quantities* (**Exhibit 23**), and submit it to the Geotechnical Branch (and the geotechnical consultant, if applicable) after the rock core inspection and prior to the final geotechnical meeting. Typically, the rock types to be calculated and tabulated on the summary sheet are limestone, durable sandstone, non-durable sandstone, durable shale, nondurable shale Class I, and nondurable shale Class II (excluding thin seams [less than 8 feet] that cannot be practically separated during construction).

A two-foot rock roadbed shall be calculated and shown on the summary sheet assuming the rock extends from shoulder to shoulder in the fills and from ditchline to ditchline in the cuts. In areas where curb and gutter are proposed, the limits of the rock roadbed will extend under the curb and gutter.

OVERALL SITE CONDITIONS

While information derived from each core is important, there are cases where individual cores may not be representative of the site as a whole. The most common cases where this is true relate to lapies or other karstic features developed in limestones. In situations such as this, the rock swell may need to be reduced.

CALCULATION OF QUANTITIES

Projects that are anticipated to have sufficient quantities of desirable materials (such as sandstone or limestone) along with less desirable materials may require calculation of only the quantity of available desirable material. Questions as to the type and thickness of rock to be considered will be resolved at the rock core inspection. **GT-703** discusses rock core meetings.

The Geotechnical Consultant shall submit reduced (11-inch x 17-inch) cut stability sections, with lithology divisions indicated, and the minutes of the rock core meeting to the design engineer. (Refer to **Exhibit 24**.) The design engineer uses the lithology divisions to assist in calculating and tabulating the select rock quantities on the TC 66-208 form. The division lines shall not be indicated on final plans.

STABILITY CONSIDERATIONS

Knowledge of the quantity of rock available allows the geotechnical engineer to effectively make realistic embankment slope stability analyses. The geotechnical engineer will complete the stability analyses and determine where rock is required.

The design engineer will calculate, tabulate, and resubmit these quantities on the TC 66-208 form, *Summary of Rock Quantities* (**Exhibit 23**), as necessary to verify the final quantities of rock required for the geotechnical recommendations.

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GEOTECHNICAL	Section: SPECIAL GEOLOGIC CONSIDERATIONS
TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch	Subject: Sinkholes

USE OF SINKHOLES

All pertinent subsurface information concerning sinkholes shall be shown on the soil profile sheets.

Sinkholes Not Used for Drainage—Construction procedures for stabilizing open sinkholes that are not to be utilized for drainage shall be in accordance with the current methods outlined in the “Treatment of Open Sinkholes” sepia sheet on the Division of Highway Design's sepia sheet list.

The plan sheet presenting the guidelines for sinkholes not used for drainage will be placed in the plans by the Division of Highway Design, as applicable.

Sinkholes Used for Drainage—Sinkholes that will be used for drainage shall have special recommendations and guidelines to follow during construction that have been approved by the Division of Highway Design.

2 2 2

GEOTECHNICAL	Section: SPECIAL GEOLOGIC CONSIDERATIONS
	Subject: Mines

DESIGN CONSIDERATIONS

Design considerations relating to mines must include a determination of whether the mines are below, at, or above grade.

Below Grade—Mines below grade that do not show signs of subsidence are generally left undisturbed.

At Grade—Mines at or near grade may be excavated and replaced with suitable backfill.

Above Grade—Cut slope designs for mines above grade utilize wider benches, shorter lifts, and pneumatic backstowing and leave as much pillar as possible. If the slope is determined to be unstable during construction, the unstable material is excavated, the benches are widened, and the remaining openings are pneumatically backstowed.

2 2 2

GEOTECHNICAL

TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch

Section

SPECIAL GEOLOGIC CONSIDERATIONS

Subject

Dipping Rock

IMPACT UPON DESIGN

Lithologic variations will be more complicated in highly tilted strata depending upon the apparent dip. Therefore, a complete field reconnaissance of each cut section is required prior to slope design. Apparent dip of strata along centerline and cross-sections as well as lithology and character of the strata influence recommendations.

SETTING SLOPES

Normal design criteria for slopes may be utilized when the apparent dip along centerline is less than two degrees and apparent dip on the cross-section is away from the roadway. Intermediate bench elevations should follow apparent dip and will have a slight grade. These benches are drawn horizontally on cross-sections and will cross cut strata in one direction.

INTERMEDIATE BENCHES

Intermediate benches with widths from 18 feet to 25 feet may be utilized and should be designed as horizontal in cuts where the apparent dip along centerline is more than two degrees and the apparent dip on the cross-section is away from the roadway. The benches will cross cut strata in two directions. Cut slopes with a maximum vertical lift of 60 feet are recommended according to the strata encountered in that particular lift.

OMISSION OF BENCHES

Intermediate benches shall be omitted and one pre-split slope is recommended from the top of rock to grade in cuts where the apparent dip on the cross-sections is toward the roadway. Lithology and character of the strata determine this slope. In some areas where a large mass of material could create a major landslide, the design slope should follow the dip of the strata.

2 2 2

GEOTECHNICAL

TRANSPORTATION CABINET Department of Highways GB Division of Structural Design Geotechnical Branch

Section

SPECIAL GEOLOGIC CONSIDERATIONS

Subject

Faults

DESIGN CONSIDERATIONS

Site-specific design considerations relating to faults shall include:

- Location of fault
- Type of fault
- Width of fault or area influenced
- Competence of faulted materials
- Amount of displacement

The effect of the fault on the roadway or structure then must be determined, and appropriate designs and recommendations developed.

2 2 2

GEOTECHNICAL	Section: SPECIAL GEOLOGIC CONSIDERATIONS
Transportation Cabinet, Department of Highways, Division of Structural Design, Geotechnical Branch	Subject: Acid-Producing Shales

OVERVIEW

Special design considerations shall be addressed for acid-producing shales when the following geologic formations are encountered in cut sections or when the shale is used in embankment fill sections. The Geologic Formations of Acid Producing Shales include:

- **New Albany Shale**
- **Chattanooga Shale**
- **Ohio Black Shale**

DESIGN CONSIDERATIONS

In general, for cut sections, the cut slope is over-excavated a minimum of 4.5 feet using a serrated slope on a 1½H:1V or 2H:1V slope (as shown in **Exhibit 20**) and covered with 4 feet of clay soil (unified classification of CL or CH) or nondurable shale to prevent production of acidic runoff and covered with 0.5 feet of top soil to support vegetation.

In general, when the shales are used in embankment fill sections, the acidic shale is encased inside the embankment. The encasement of the acidic shale includes using 2.5 feet (parallel to fill slope) of nondurable shale or clay soil (unified classification of CL or CH) as a barrier to protect the acidic shale from the weathering elements such as water and air.

However, a minimum of 4 feet of nondurable shale or clay soil (unified classification of CL or CH) is recommended on top of the embankment to control corrosion of guardrail, sign post, etc. from the acidic shale. If available, the side slopes shall be dressed with 0.5 feet of top soil to support vegetation.

These are general guidelines and do not cover all of the specific recommendations that are needed in a Geotechnical Report or cover other options available to mitigate the production of acidic runoff conditions.

2 2 2

Typical Slope Configuration
Class III Nondurable Shale

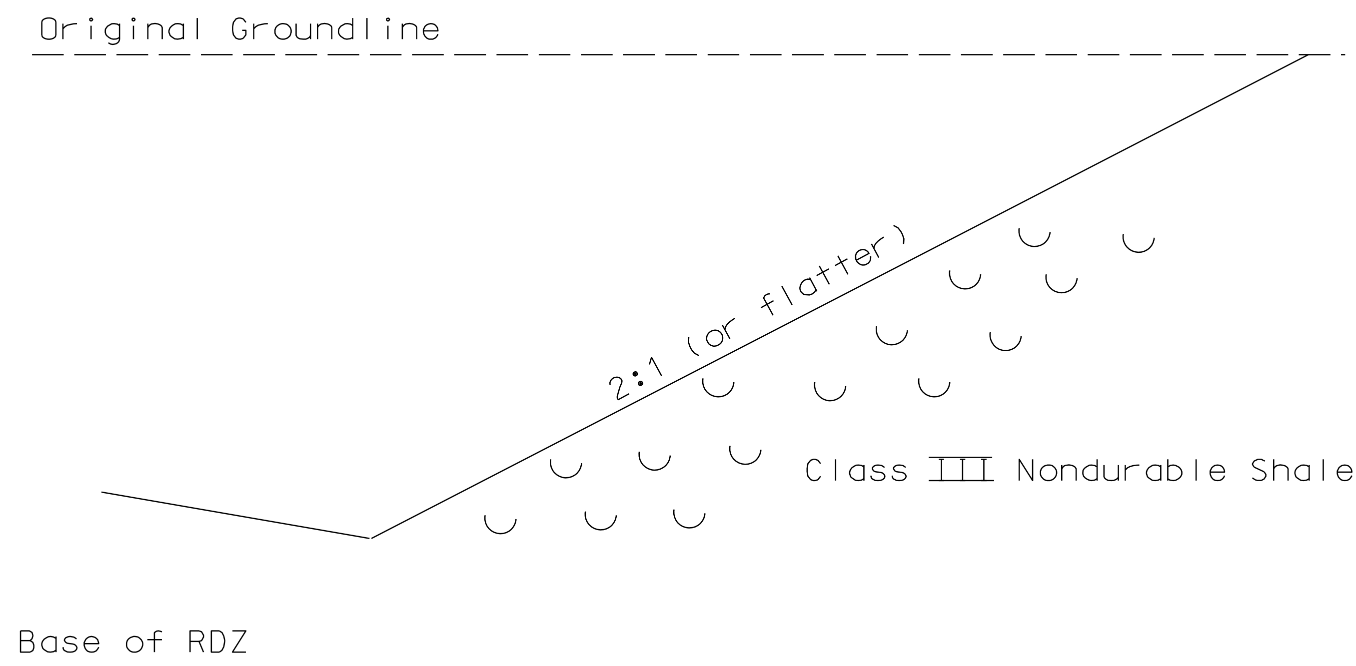
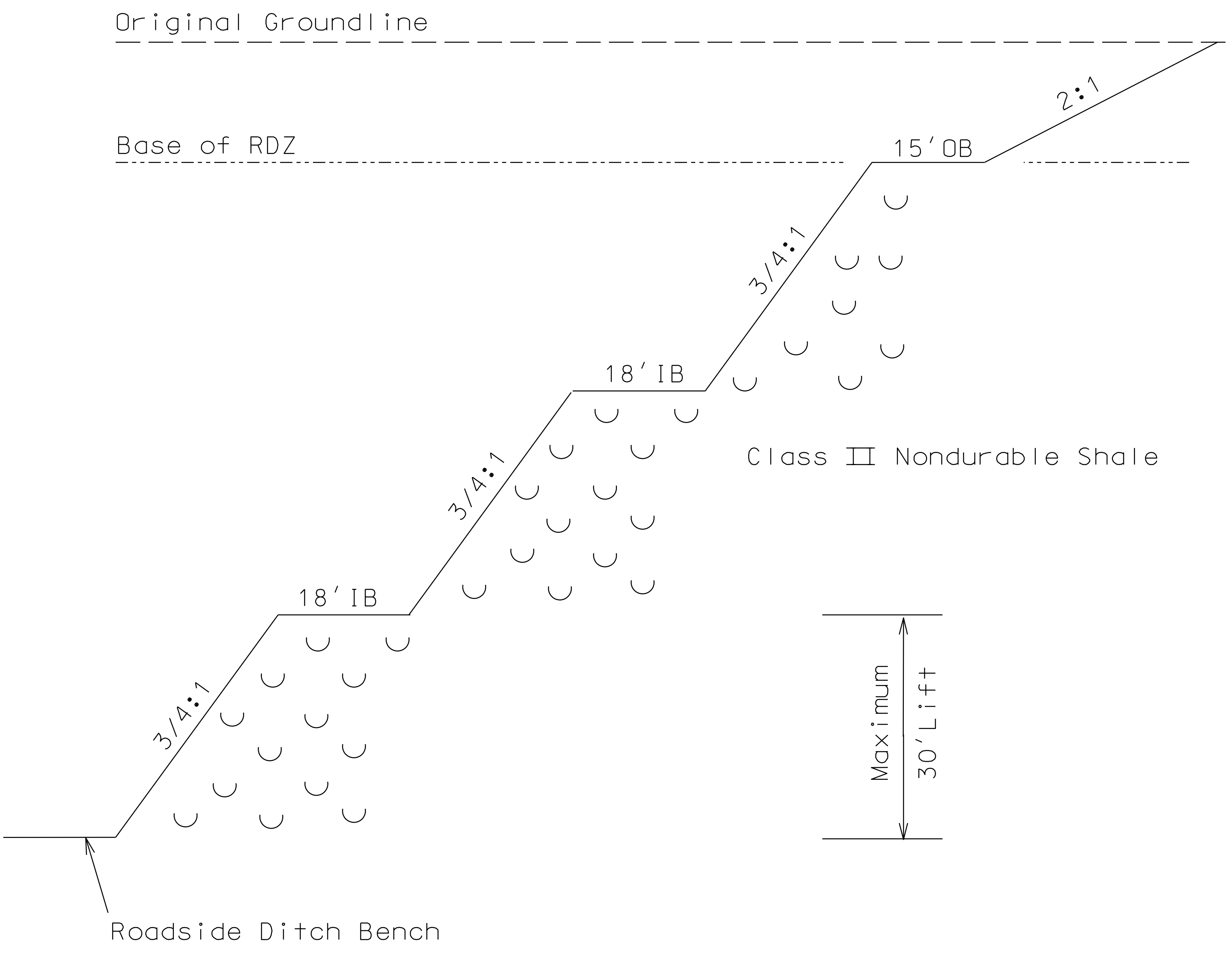


Exhibit 14

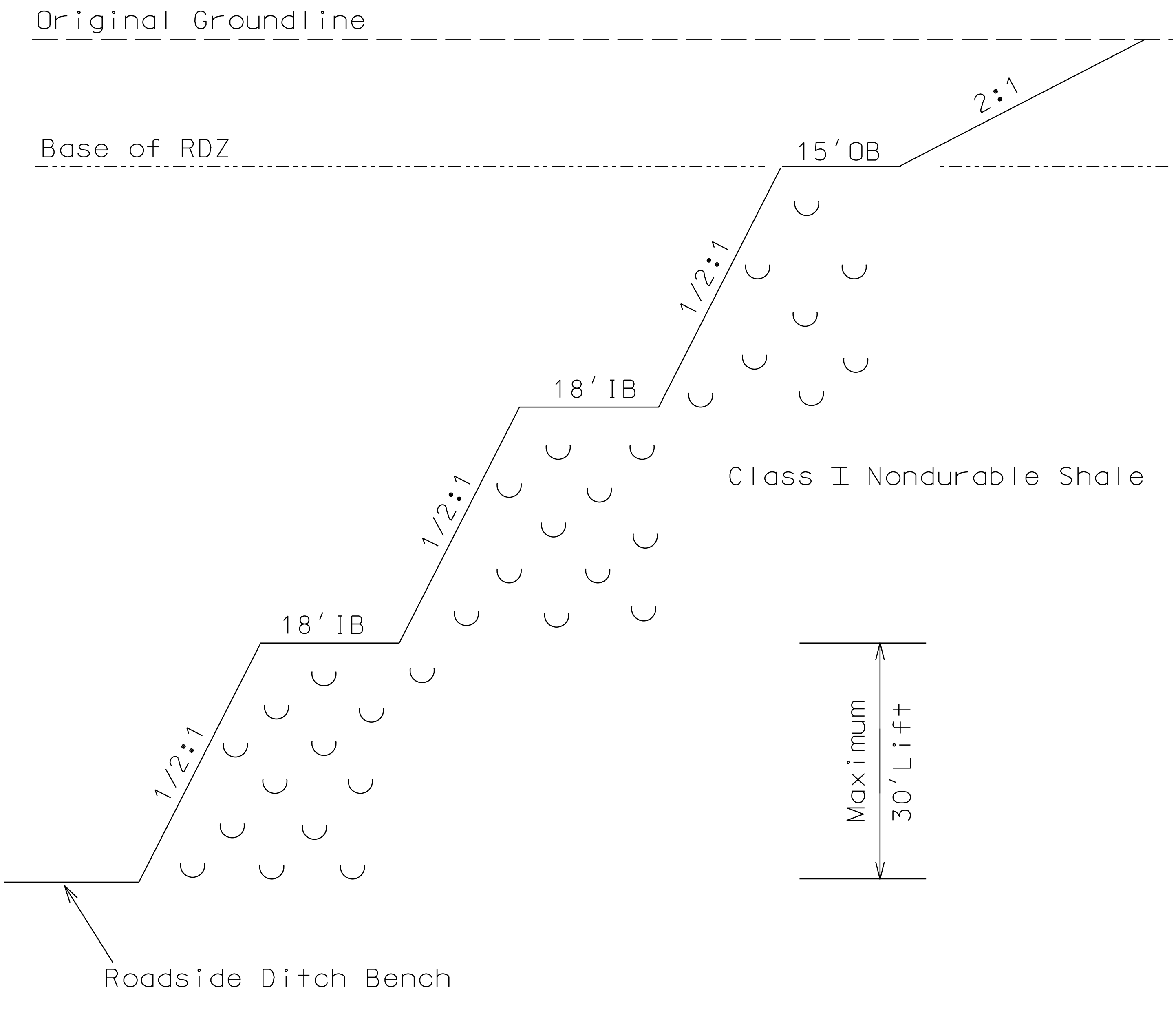
Typical Slope Configuration
Class II Nondurable Shale



NOTE :
IB = Intermediate Bench
OB = Overburden Bench

Exhibit 15

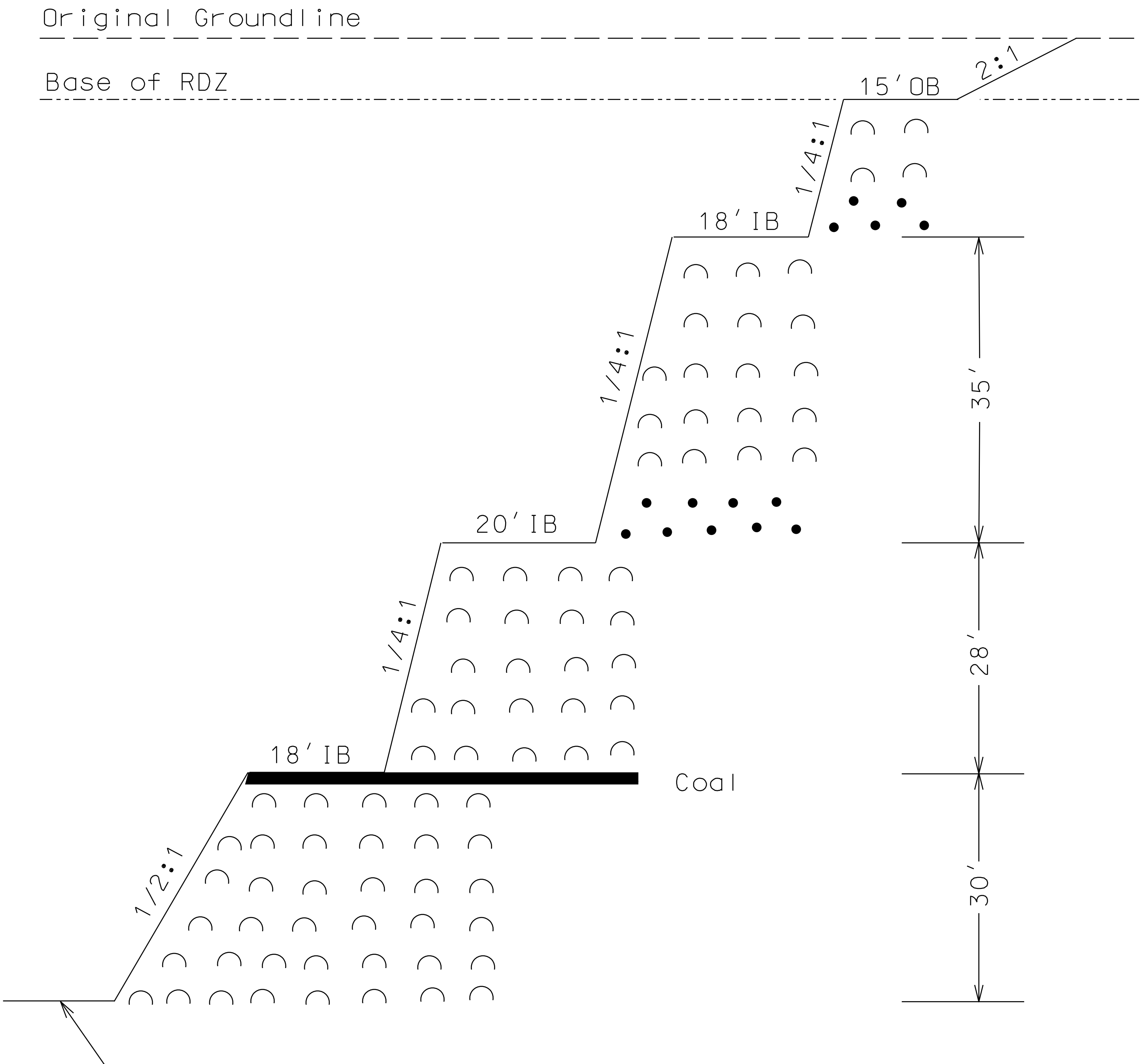
Typical Slope Configuration
Class I Nondurable Shale



NOTE :
IB = Intermediate Bench
OB = Overburden Bench

Exhibit 16

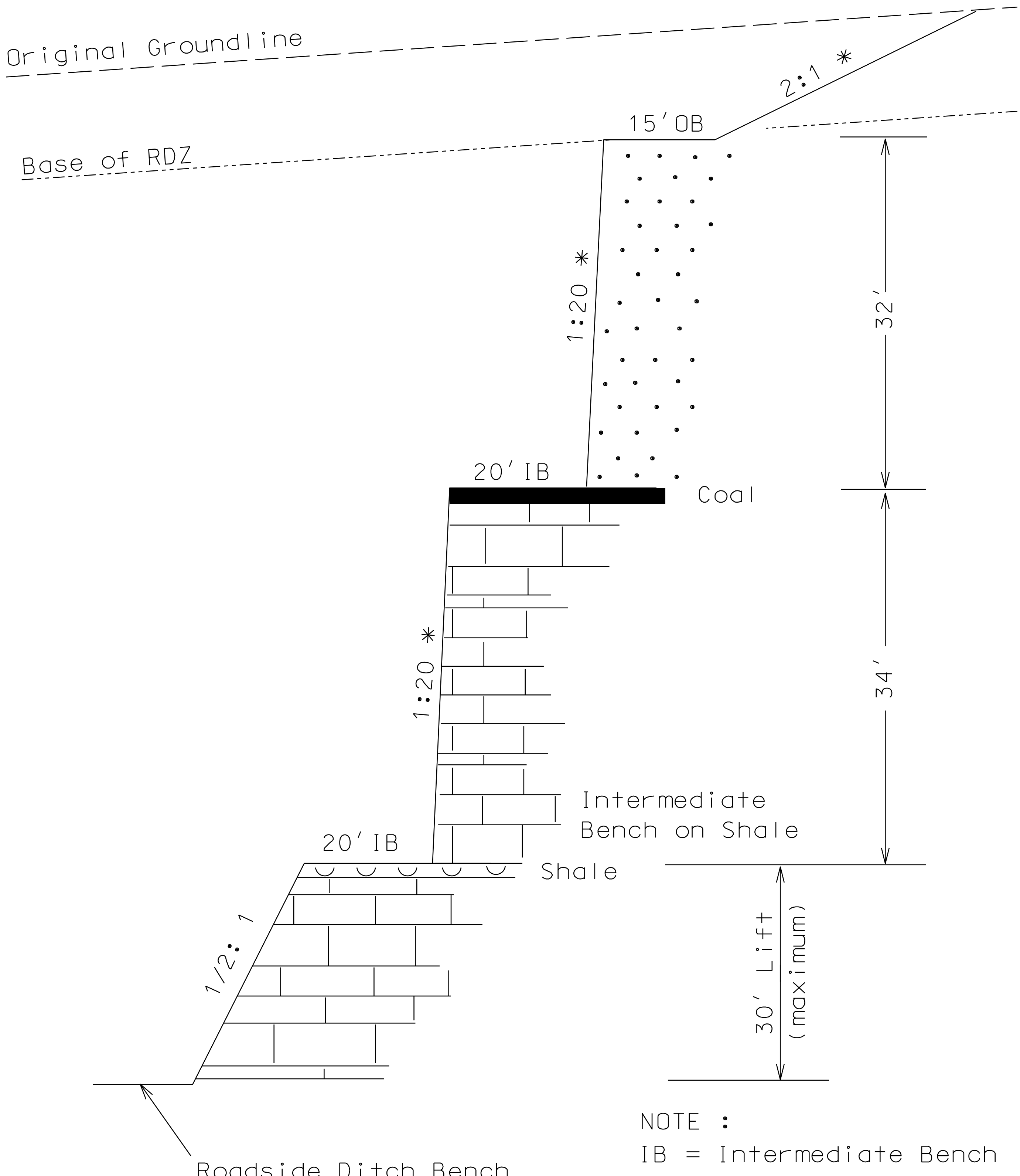
Typical Slope Configuration
Durable Shale



NOTE :
IB = Intermediate Bench
OB = Overburden Bench

Exhibit 17

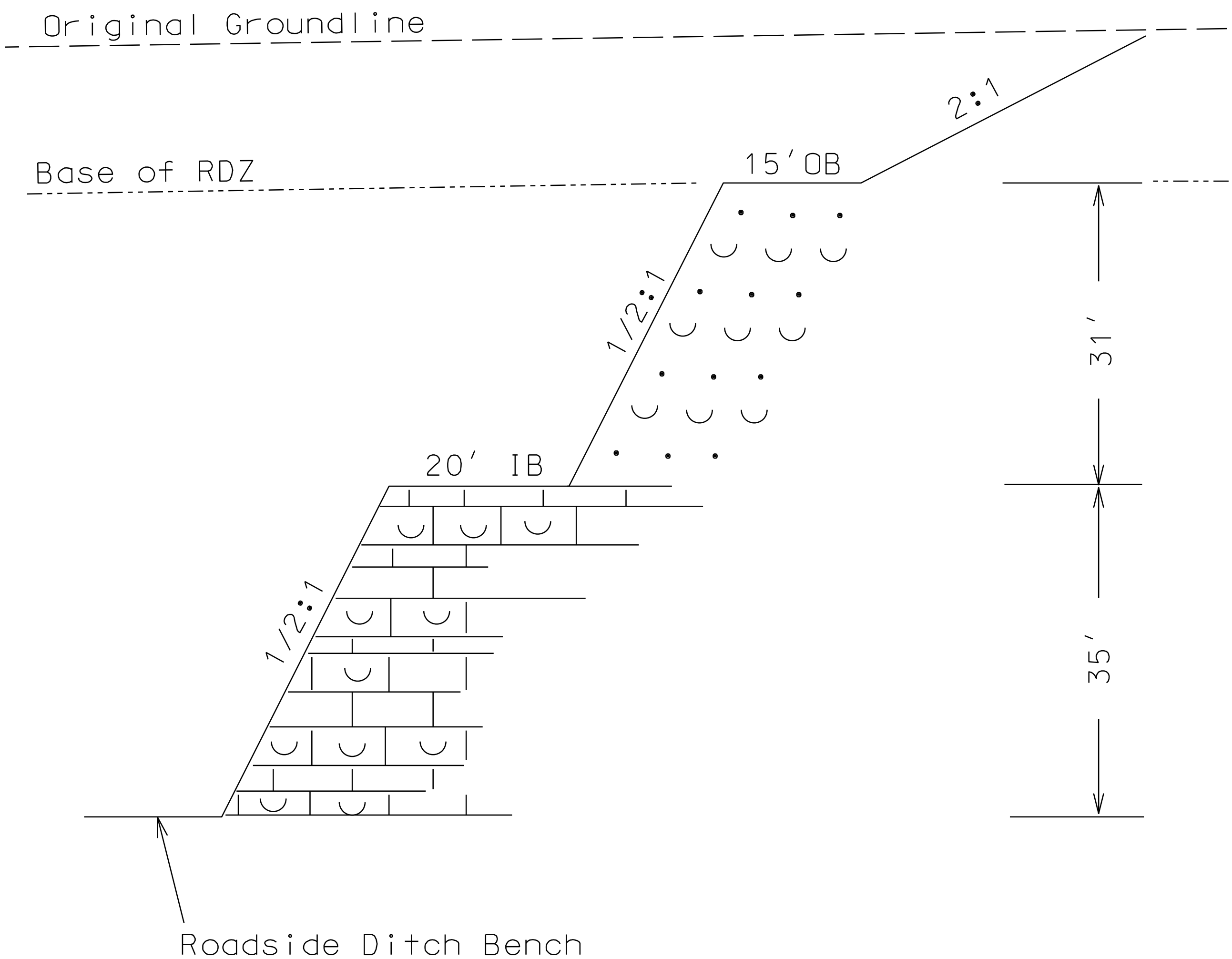
Typical Slope Configuration
Massive Limestone or Sandstone
* Slopes are shown at maximum steepness



NOTE :
IB = Intermediate Bench
OB = Overburden Bench

Exhibit 18

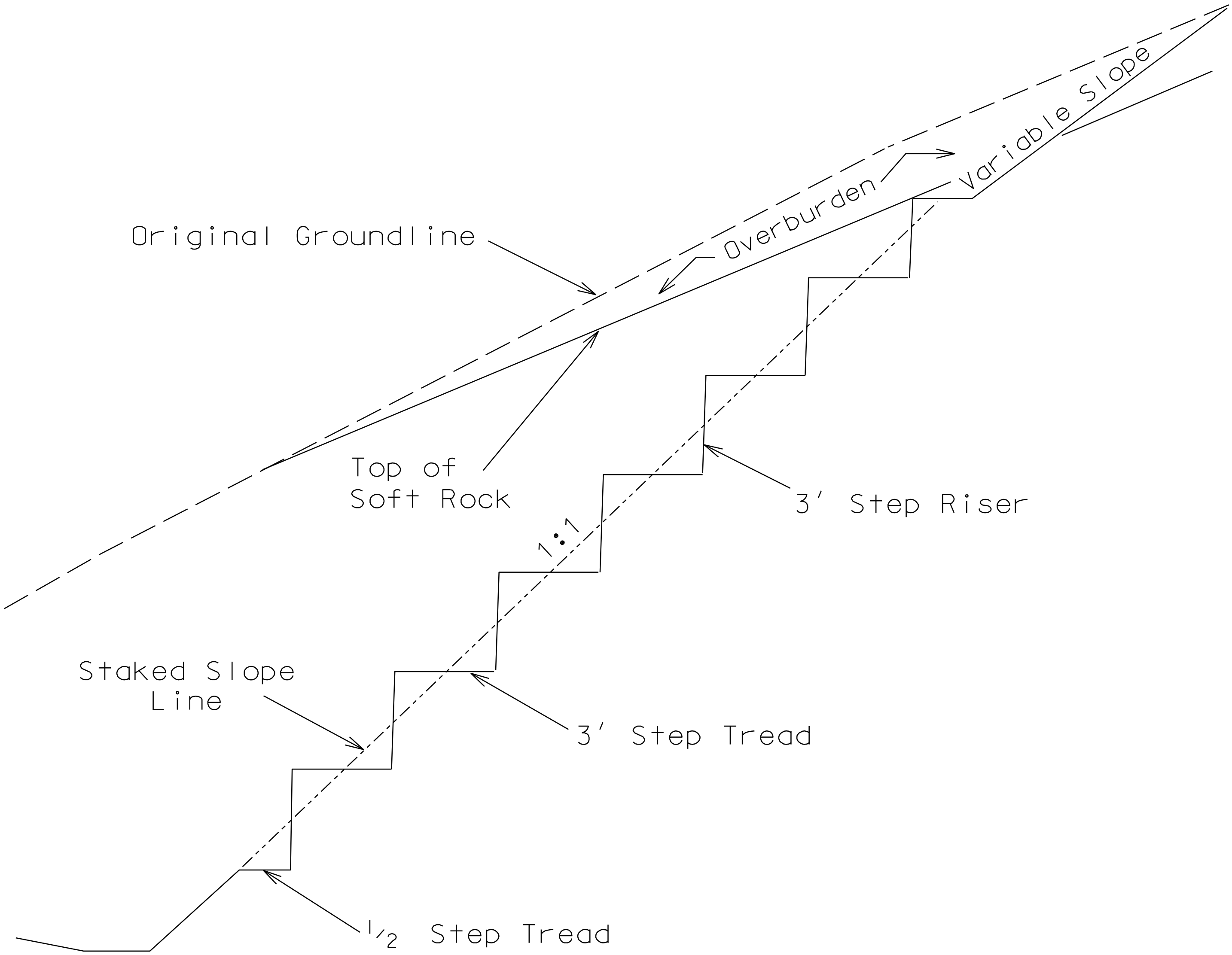
Typical Slope Configuration
Shaley Limestone or Sandstone



NOTE :
IB = Intermediate Bench
OB = Overburden Bench

Exhibit 19

Typical Slope Configuration
1:1 Serrated Slopes



NOTE :
1:1 slope configuration shown.
For a 1 1/2:1 slope (not shown)
use 2′ riser with a 3′ tread
or 4′ riser with a 6′ tread.

Exhibit 20

Roadside Ditch Catchment Area

For a Copy of Guidelines Contact the
Kentucky Department of Highways
Division of Structural Design
Geotechnical Branch

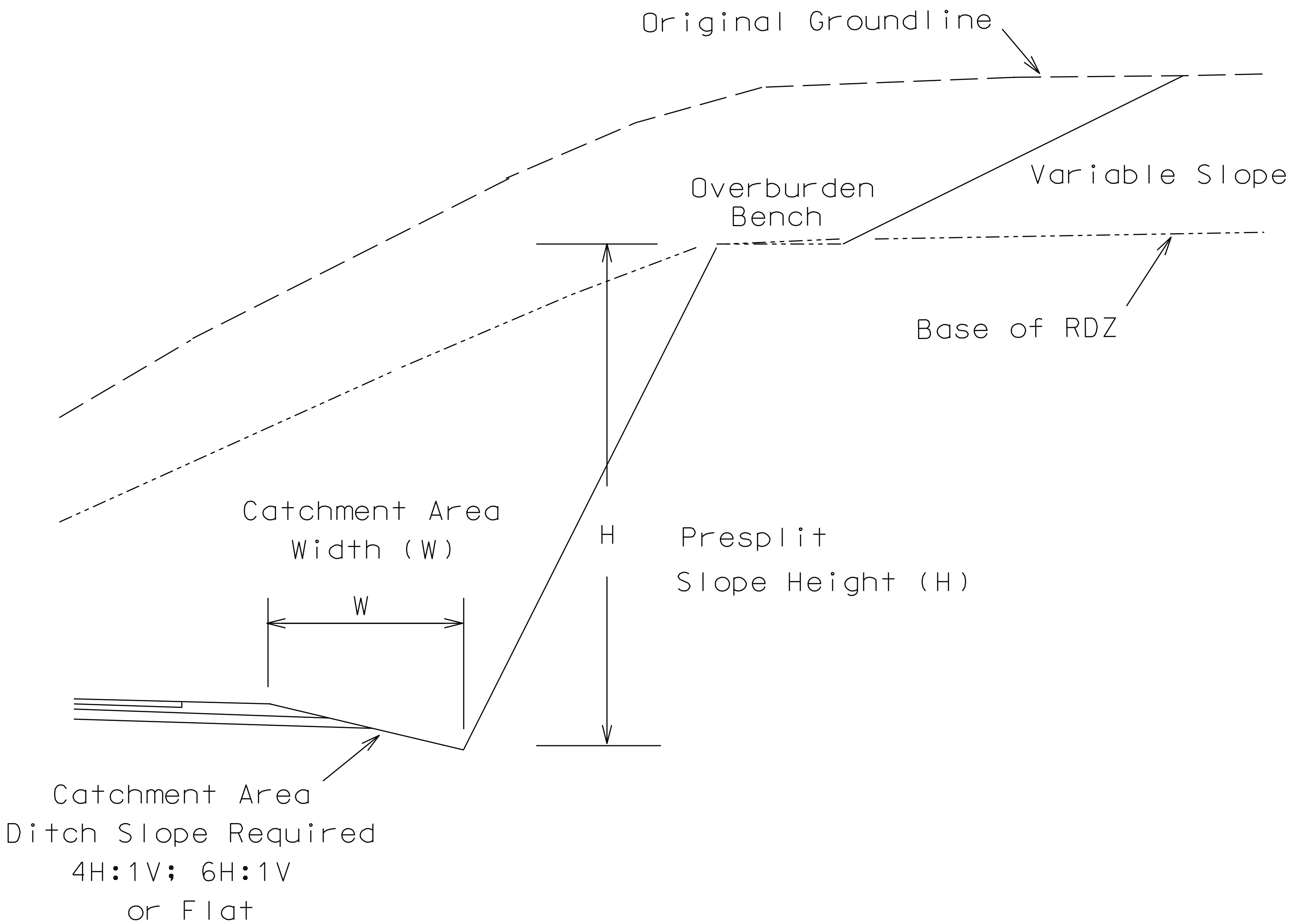


Exhibit 21

KENTUCKY TRANSPORTATION CABINET
Division of Structural Design
Geotechnical Branch

TC 66-208
Rev. 7/2010

County *Springfield*

Page *1* **of** *1*

Item No. *13-765.00*

SUMMARY OF ROCK QUANTITIES
(CUBIC YARDS)

Submittal No. *2*

Project No. *FD52 126 0555 005-023 009D*

Date *7/21/2012*

Sheet Totals Station to Station	2 Foot Rock Roadbed (Required)	Rock Embankment (Required)	Channel Lining (Required)	Type of Excavated Material			
				Sandstone or Limestone	Durable Shale	Nondurable Shale	
						Class I	Class II
MAINLINE SHEETS							
520+00 - 535+00	1,704	17,000					
535+00 - 550+00	6,390	4,000	4,231		1,259		
550+00 - 565+00	6,390				62,240		
565+00 - 580+00	6,309	5,000			1,712		
580+00 - 595+00	6,390				1,209	5,923	17,933
595+00 - 610+00	6,390	4,000				194	1,023
610+00 - 625+00	6,390				128,247	172,935	59525
625+00 - 640+00	6,390	45,000					
640+00 - 655+00	6,390	61,000					
655+00 - 670+00	6,390	72,000					
670+00 - 685+00	6,390	65,000	10,971				
685+00 - 700+00	6,390			17,484	556		10,841
700+00 - 715+00	6,390			2,232	7,023	9,567	10,086
715+00 - 730+00	6,390			17,394	73,640	68,136	4,052
730+00 - 745+00	6,390			53,158	56,582	25,245	
Sheet Total (Cubic Yards)	91,164	273,000	15,202	90,238	332,468	282,000	103,460
Accumulated Total	91,164	273,000	15,202	90,238	332,468	282,000	103,460

Exhibit 23

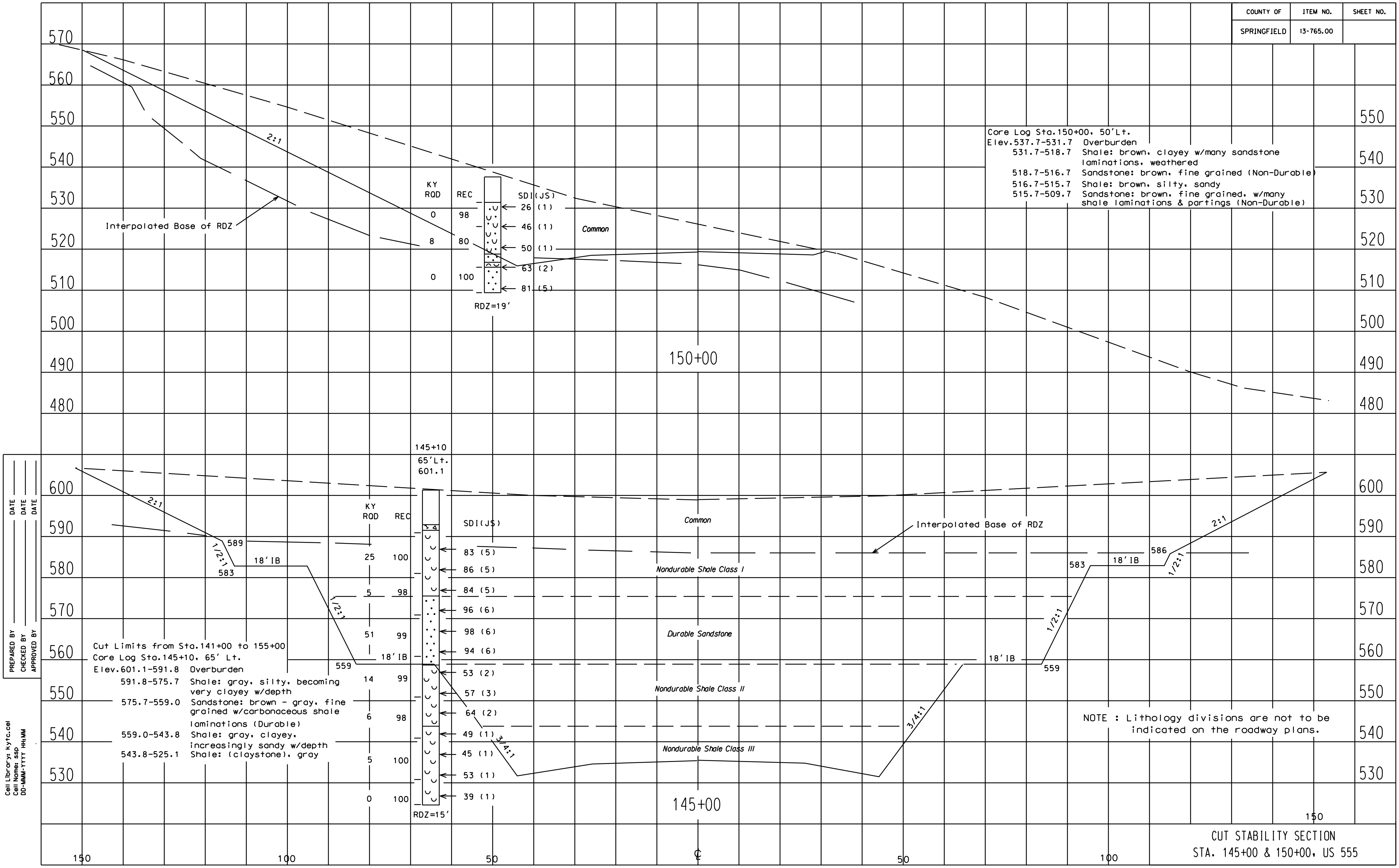
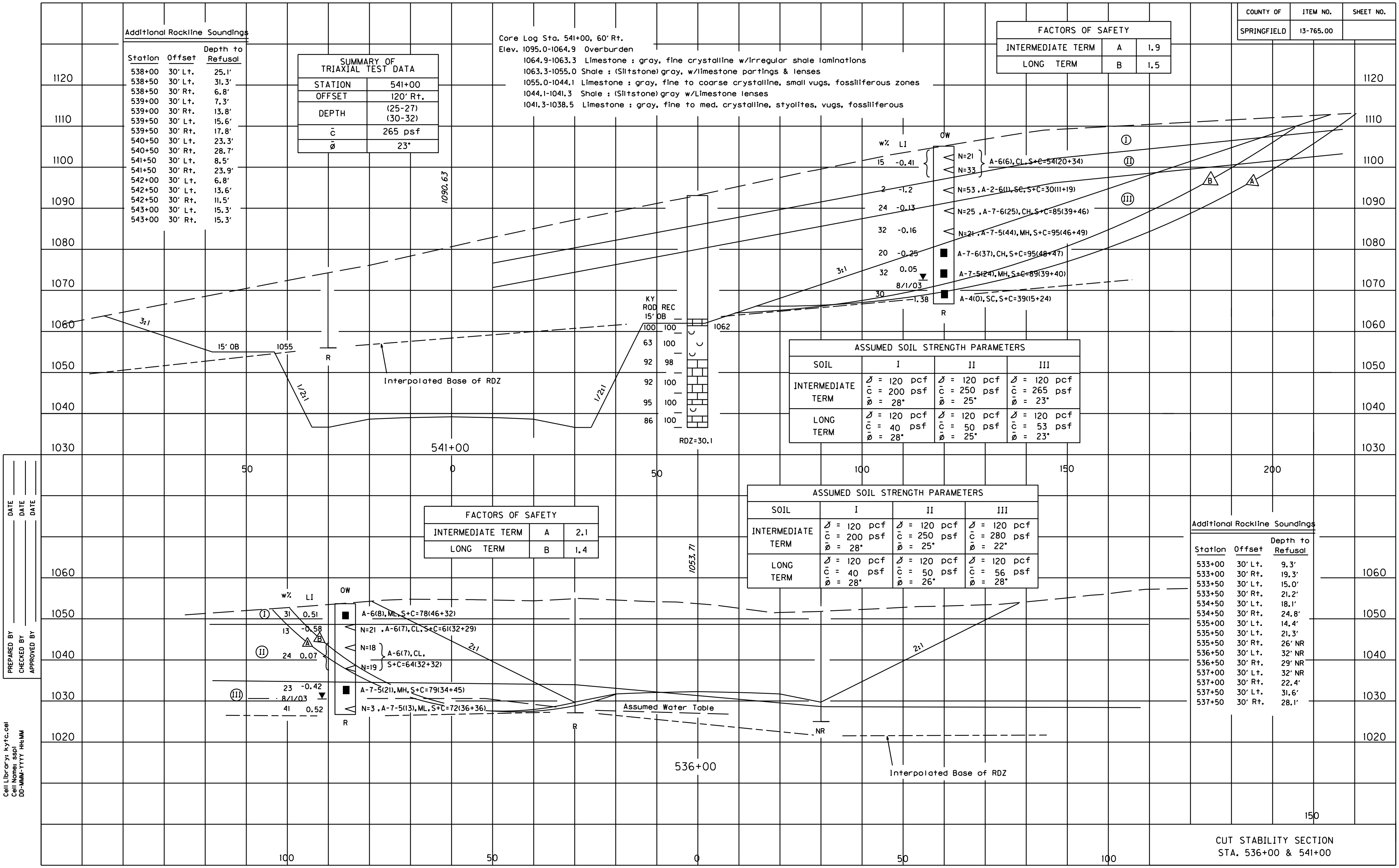

COUNTY OF
ITEM NO.
SHEET NO.
SPRINGFIELD
13-765.00
570
560
550
540
530
520
510
500
490
480
2:1
Interpolated Base of RDZ
KY
RQD
REC
SDI(JS)
0
98
8
80
0
100
26 (1)
46 (1)
50 (1)
63 (2)
81 (5)
RDZ=19'
Common
Core Log Sta.150+00, 50'Lt.
Elev.537.7-531.7 Overburden
531.7-518.7 Shale: brown, clayey w/many sandstone laminations, weathered
518.7-516.7 Sandstone: brown, fine grained (Non-Durable)
516.7-515.7 Shale: brown, silty, sandy
515.7-509.7 Sandstone: brown, fine grained, w/many shale laminations & partings (Non-Durable)
150+00
145+10
65'Lt.
601.1
600
590
580
570
560
550
540
530
2:1
589
1/2:1
583
18'IB
1/2:1
559
KY
RQD
REC
SDI(JS)
25
100
5
98
51
99
18'IB
14
99
6
98
5
100
0
100
RDZ=15'
83 (5)
86 (5)
84 (5)
96 (6)
98 (6)
94 (6)
53 (2)
57 (3)
64 (2)
49 (1)
45 (1)
53 (1)
39 (1)
3/4:1
Common
Nondurable Shale Class I
Durable Sandstone
Nondurable Shale Class II
Nondurable Shale Class III
Interpolated Base of RDZ
586
2:1
583
18'IB
1/2:1
1/2:1
18'IB
559
3/4:1
Cut Limits from Sta.141+00 to 155+00
Core Log Sta.145+10, 65' Lt.
Elev.601.1-591.8 Overburden
591.8-575.7 Shale: gray, silty, becoming very clayey w/depth
575.7-559.0 Sandstone: brown - gray, fine grained w/carbonaceous shale laminations (Durable)
559.0-543.8 Shale: gray, clayey, increasingly sandy w/depth
543.8-525.1 Shale: (claystone), gray
NOTE : Lithology divisions are not to be indicated on the roadway plans.
145+00
150
CUT STABILITY SECTION
STA. 145+00 & 150+00, US 555
150
100
50
℄
50
100
PREPARED BY
CHECKED BY
APPROVED BY
DATE
DATE
DATE
Cell Library: kytc.cel
Cell Name: ssp
DD-MMM-YYYY HH:MM

Exhibit 24

COUNTY OF	ITEM NO.	SHEET NO.
SPRINGFIELD	13-765.00	



Additional Rockline Soundings

Station	Offset	Depth to Refusal
538+00	30' Lt.	25.1'
538+50	30' Lt.	31.3'
538+50	30' Rt.	6.8'
539+00	30' Lt.	7.3'
539+00	30' Rt.	13.8'
539+50	30' Lt.	15.6'
539+50	30' Rt.	17.8'
540+50	30' Lt.	23.3'
540+50	30' Rt.	28.7'
541+50	30' Lt.	8.5'
541+50	30' Rt.	23.9'
542+00	30' Lt.	6.8'
542+50	30' Lt.	13.6'
542+50	30' Rt.	11.5'
543+00	30' Lt.	15.3'
543+00	30' Rt.	15.3'

SUMMARY OF TRIAXIAL TEST DATA

STATION	541+00
OFFSET	120' Rt.
DEPTH	(25-27) (30-32)
$\bar{c}$	265 psf
$\bar{\phi}$	23°

FACTORS OF SAFETY

INTERMEDIATE TERM	A	1.9
LONG TERM	B	1.5

ASSUMED SOIL STRENGTH PARAMETERS

SOIL	I	II	III
INTERMEDIATE TERM	γ = 120 pcf $\bar{c}$ = 200 psf $\bar{\phi}$ = 28°	γ = 120 pcf $\bar{c}$ = 250 psf $\bar{\phi}$ = 25°	γ = 120 pcf $\bar{c}$ = 265 psf $\bar{\phi}$ = 23°
LONG TERM	γ = 120 pcf $\bar{c}$ = 40 psf $\bar{\phi}$ = 28°	γ = 120 pcf $\bar{c}$ = 50 psf $\bar{\phi}$ = 25°	γ = 120 pcf $\bar{c}$ = 53 psf $\bar{\phi}$ = 23°

ASSUMED SOIL STRENGTH PARAMETERS

SOIL	I	II	III
INTERMEDIATE TERM	γ = 120 pcf $\bar{c}$ = 200 psf $\bar{\phi}$ = 28°	γ = 120 pcf $\bar{c}$ = 250 psf $\bar{\phi}$ = 25°	γ = 120 pcf $\bar{c}$ = 280 psf $\bar{\phi}$ = 22°
LONG TERM	γ = 120 pcf $\bar{c}$ = 40 psf $\bar{\phi}$ = 28°	γ = 120 pcf $\bar{c}$ = 50 psf $\bar{\phi}$ = 26°	γ = 120 pcf $\bar{c}$ = 56 psf $\bar{\phi}$ = 28°

FACTORS OF SAFETY

INTERMEDIATE TERM	A	2.1
LONG TERM	B	1.4

Additional Rockline Soundings

Station	Offset	Depth to Refusal
533+00	30' Lt.	9.3'
533+00	30' Rt.	19.3'
533+50	30' Lt.	15.0'
533+50	30' Rt.	21.2'
534+50	30' Lt.	18.1'
534+50	30' Rt.	24.8'
535+00	30' Lt.	14.4'
535+50	30' Lt.	21.3'
535+50	30' Rt.	26' NR
536+50	30' Lt.	32' NR
536+50	30' Rt.	29' NR
537+00	30' Lt.	32' NR
537+00	30' Rt.	22.4'
537+50	30' Lt.	31.6'
537+50	30' Rt.	28.1'

CUT STABILITY SECTION
STA. 536+00 & 541+00

Exhibit 32

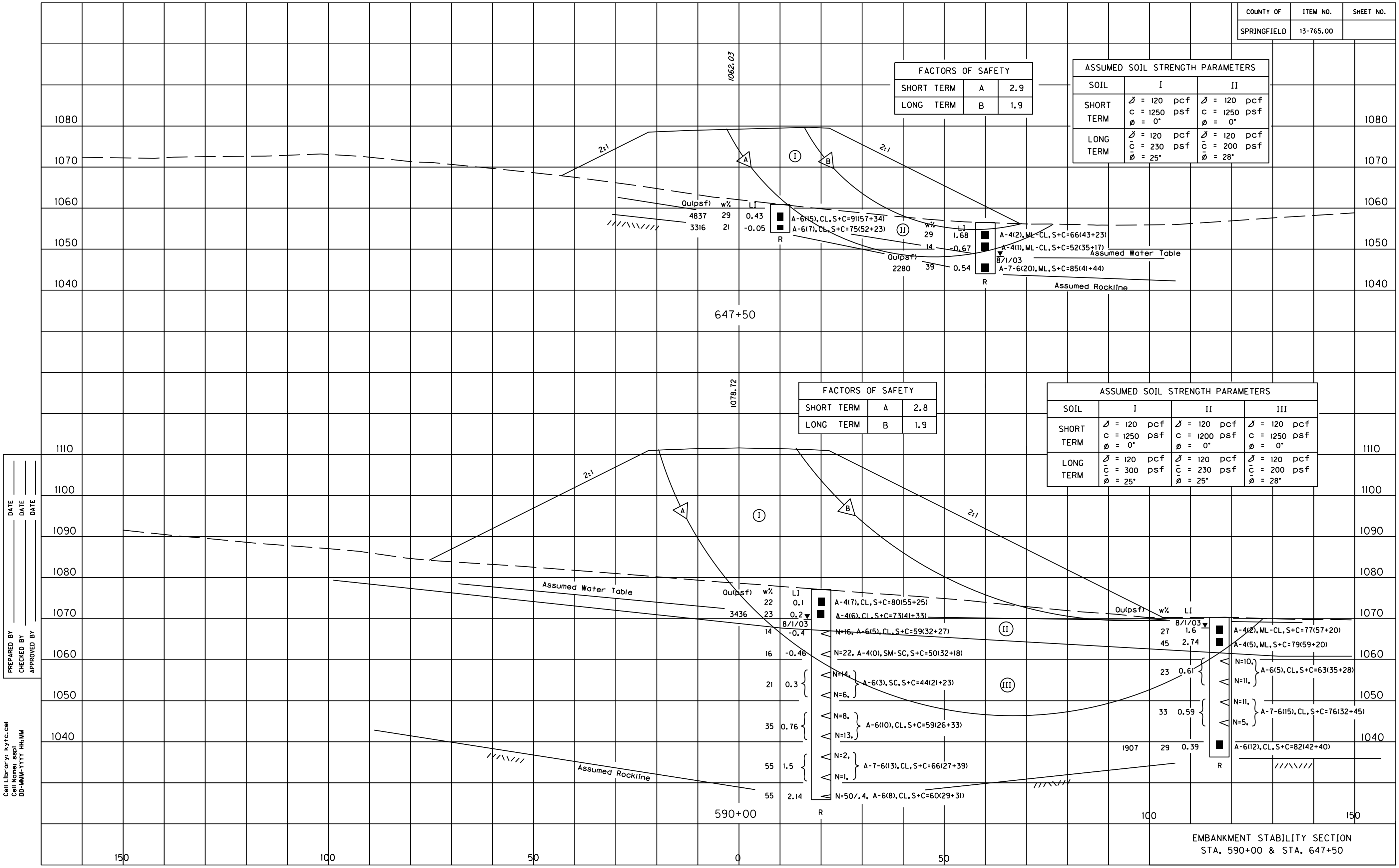
COUNTY OF	ITEM NO.	SHEET NO.
SPRINGFIELD	13-765.00	

FACTORS OF SAFETY		
SHORT TERM	A	2.9
LONG TERM	B	1.9

ASSUMED SOIL STRENGTH PARAMETERS		
SOIL	I	II
SHORT TERM	γ = 120 pcf c = 1250 psf ø = 0°	γ = 120 pcf c = 1250 psf ø = 0°
LONG TERM	γ = 120 pcf c̄ = 230 psf ø̄ = 25°	γ = 120 pcf c̄ = 200 psf ø̄ = 28°

FACTORS OF SAFETY		
SHORT TERM	A	2.8
LONG TERM	B	1.9

ASSUMED SOIL STRENGTH PARAMETERS			
SOIL	I	II	III
SHORT TERM	γ = 120 pcf c = 1250 psf ø = 0°	γ = 120 pcf c = 1200 psf ø = 0°	γ = 120 pcf c = 1250 psf ø = 0°
LONG TERM	γ = 120 pcf c̄ = 300 psf ø̄ = 25°	γ = 120 pcf c̄ = 230 psf ø̄ = 25°	γ = 120 pcf c̄ = 200 psf ø̄ = 28°



EMBANKMENT STABILITY SECTION
STA. 590+00 & STA. 647+50

Exhibit 33

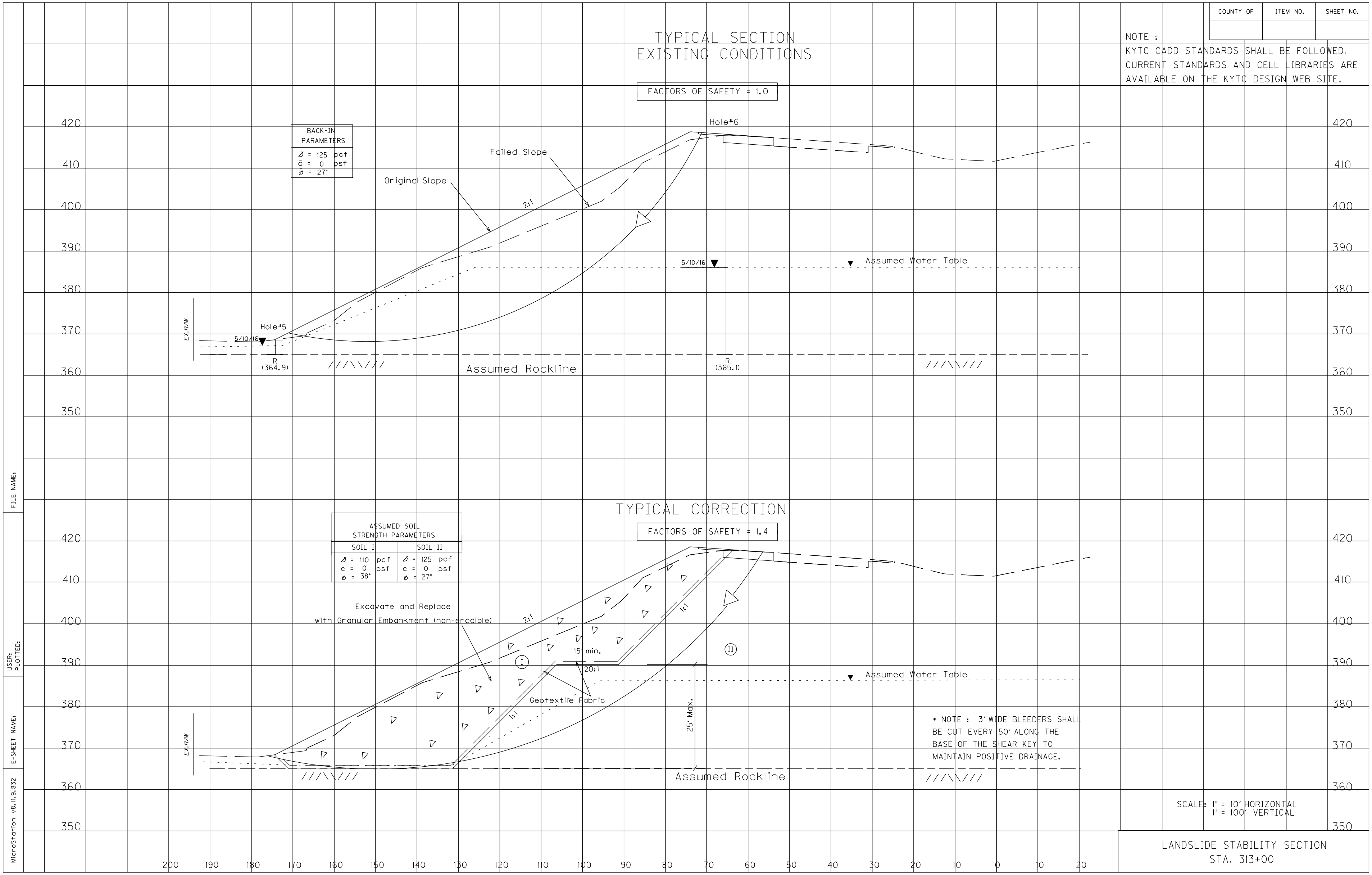
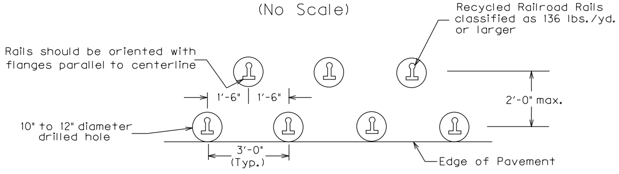

COUNTY OF
ITEM NO.
SHEET NO.
NOTE :
KYTC CADD STANDARDS SHALL BE FOLLOWED.
CURRENT STANDARDS AND CELL LIBRARIES ARE AVAILABLE ON THE KYTC DESIGN WEB SITE.
TYPICAL SECTION
EXISTING CONDITIONS
FACTORS OF SAFETY = 1.0
BACK-IN PARAMETERS
Ø = 125 pcf
c = 0 psf
ø = 27°
Hole#6
Failed Slope
Original Slope
2:1
5/10/16
Assumed Water Table
Hole#5
EX.R/W
R
(364.9)
Assumed Rockline
(365.1)
TYPICAL CORRECTION
FACTORS OF SAFETY = 1.4
ASSUMED SOIL STRENGTH PARAMETERS
SOIL I
Ø = 110 pcf
c = 0 psf
ø = 38°
SOIL II
Ø = 125 pcf
c = 0 psf
ø = 27°
Excavate and Replace
with Granular Embankment (non-erodible)
1:1
15' min.
20:1
I
II
Geotextile Fabric
25' Max.
• NOTE : 3' WIDE BLEEDERS SHALL BE CUT EVERY 50' ALONG THE BASE OF THE SHEAR KEY TO MAINTAIN POSITIVE DRAINAGE.
SCALE: 1" = 10' HORIZONTAL
1" = 100' VERTICAL
LANDSLIDE STABILITY SECTION
STA. 313+00
FILE NAME:
USER:
PLOTTED:
E-SHEET NAME:
MicroStation v8.11.9.832
420
410
400
390
380
370
360
350
200 190 180 170 160 150 140 130 120 110 100 90 80 70 60 50 40 30 20 10 0 10 20

Exhibit 34

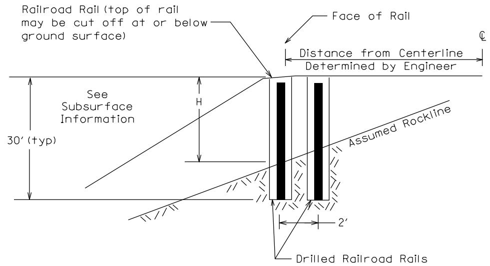
COUNTY OF	ITEM NO.	SHEET NO.
		___

TYPICAL PLAN VIEW
(NO SCALE)

TYPICAL SPACING DETAIL - DOUBLE ROW
(No Scale)

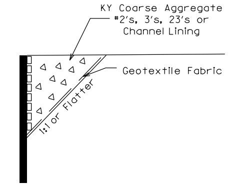


TYPICAL CROSS SECTION DETAIL - DOUBLE ROW
(No Scale)

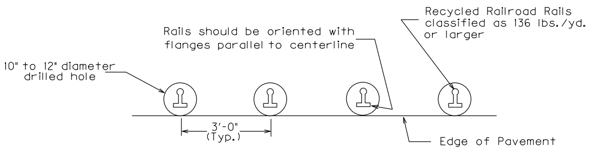


TYPICAL CROSS SECTION WITH BACKFILL
(Use with Either Double or Single Row)

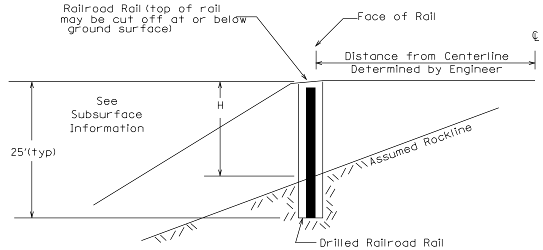
(No Scale)



TYPICAL SPACING DETAIL - SINGLE ROW
(No Scale)



TYPICAL CROSS SECTION DETAIL - SINGLE ROW
(No Scale)



DESIGNED BY: ________
DATE SUBMITTED: ________

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS
COUNTY OF

PROJECT NUMBERS: ________

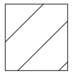
FILE NAME:
USER:
DATE PLOTTED:
E-SHEET NAME: ____
MicroStation v8.11.9.832

Exhibit 34a

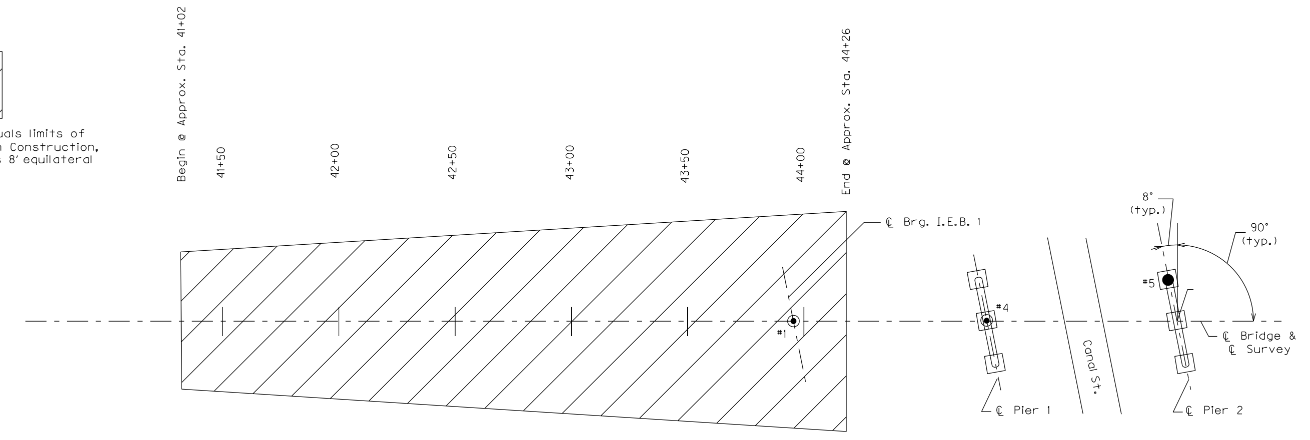
COUNTY OF	ITEM NO.	SHEET NO.
SPRINGFIELD	13-765.00	

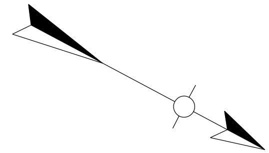
WICK DRAIN LAYOUT

Not To Scale



Symbol equals limits of Wick Drain Construction, spacing is 8' equilateral

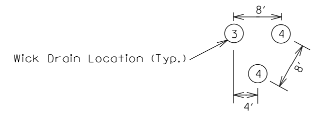




PLAN VIEW - WICK DRAIN LOCATIONS

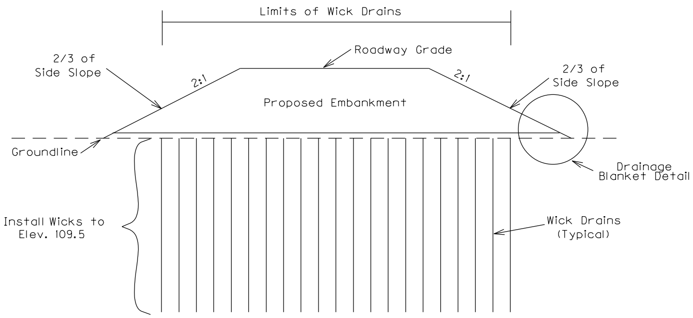
DETAIL - TRIANGULAR SPACING

NOT TO SCALE



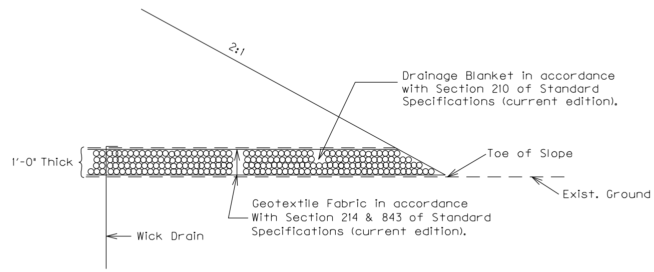
TYPICAL SECTION - WICK DRAINS

Not To Scale



DRAINAGE BLANKET DETAIL

Not To Scale



PREPARED BY ______ DATE ______
CHECKED BY ______ DATE ______
APPROVED BY ______ DATE ______

Cell Library: kytc.cel
Cell Name: sspl
DD-MMM-YYYY HH:MM

KENTUCKY
DEPARTMENT OF HIGHWAYS
COUNTY OF
SPRINGFIELD

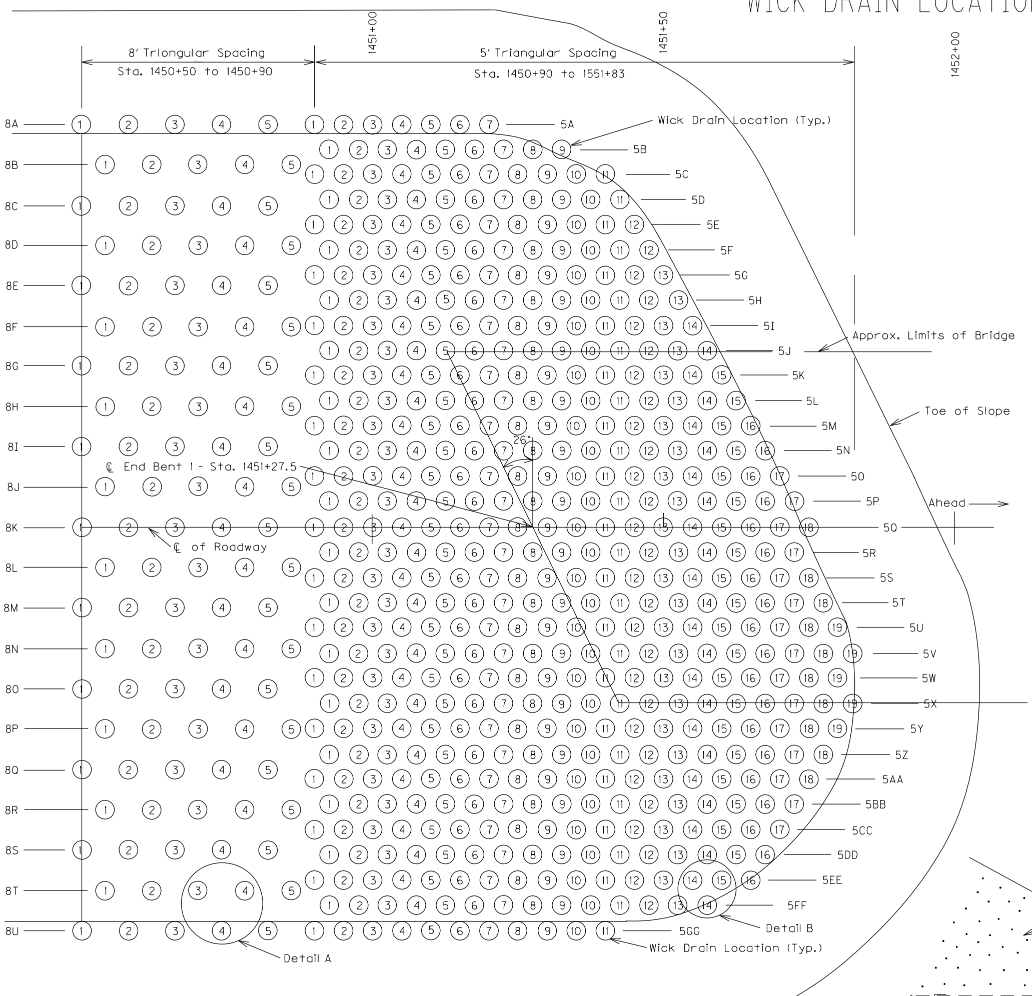
PROJECT NUMBERS ______

WICK DRAIN DETAILS

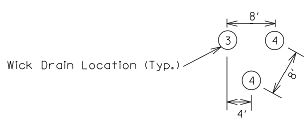
Exhibit 35

COUNTY OF	ITEM NO.	SHEET NO.
SPRINGFIELD	13-765.00	

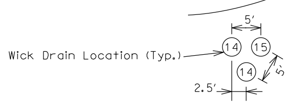
WICK DRAIN LOCATIONS



PLAN VIEW - WICK DRAIN LOCATIONS



DETAIL A - TRIANGULAR SPACING



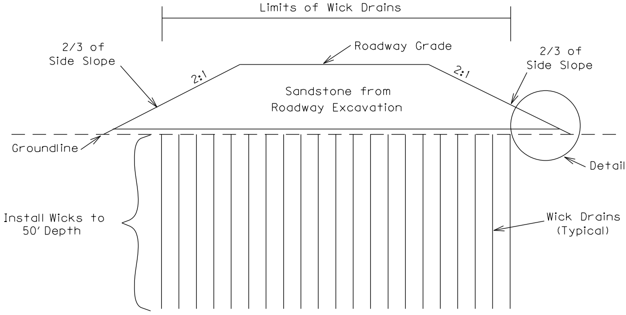
DETAIL B - 5' TRIANGULAR SPACING

5' TRIANGULAR WICK DRAIN SPACINGS

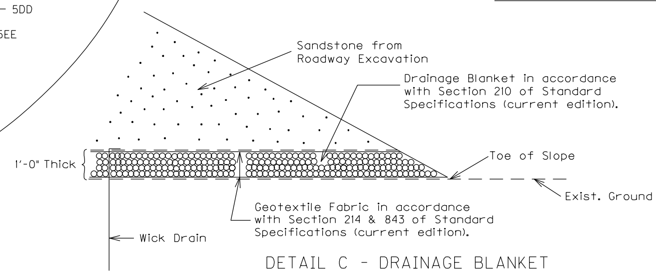
ROW	OFFSET LEFT OF ℄ (FT.)	ROW	OFFSET RIGHT OF ℄ (FT.)
5A	69.3	5R	4.3
5B	65.0	5S	8.7
5C	60.6	5T	13.0
5D	56.3	5U	17.3
5E	52.0	5V	21.7
5F	47.6	5W	26.0
5G	43.3	5X	30.3
5H	39.0	5Y	34.6
5I	34.6	5Z	39.0
5J	30.3	5AA	43.3
5K	26.0	5BB	47.6
5L	21.7	5CC	52.0
5M	17.3	5DD	56.3
5N	13.0	5EE	60.6
5O	8.7	5FF	65.0
5P	4.3	5GG	69.3
5Q	0.0	-	-

8' TRIANGULAR WICK DRAIN SPACINGS

ROW	OFFSET LEFT OF ℄ (FT.)	ROW	OFFSET RIGHT OF ℄ (FT.)
8A	69.3	8L	6.9
8B	62.4	8M	13.9
8C	55.4	8N	20.8
8D	48.5	8O	27.7
8E	41.6	8P	34.6
8F	34.6	8Q	41.6
8G	27.7	8R	48.5
8H	20.8	8S	55.4
8I	13.9	8T	62.4
8J	6.93	8U	69.3
8K	0.0	-	-



TYPICAL SECTION - WICK DRAINS



DETAIL C - DRAINAGE BLANKET

KENTUCKY
DEPARTMENT OF HIGHWAYS
COUNTY OF
SPRINGFIELD

PROJECT NUMBERS

WICK DRAIN DETAILS

PREPARED BY ——— DATE
CHECKED BY ——— DATE
APPROVED BY ——— DATE

Cell Library: kytc.cel
Cell Name: sspl
DD-MMM-YYYY HH:MM

Exhibit 36

GEOTECHNICAL NOTES

for MSE Walls

If the Contractor elects to use an MSE Wall as allowed by the Contract Documents, design the wall (or walls) in accordance with the AASHTO LRFD Bridge Design Specifications. The Contract Documents control where a requirement which is not covered by, or is contrary to, AASHTO exists.

Use only MSE Walls with inextensible reinforcement.

Granular replacement depths (D) versus wall height (H):
For H <= 10 ft. D= 0
For H>10 ft and <= 20 ft. D= 2.0 ft.

Station Interval	Bearing Surface	Nominal Bearing Resistance
10+20 - 11+15	Soil	- ksf
11+15 - 12+15	Gran. Repl.	- ksf

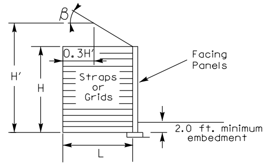
Use the following soil strength parameters for design:

	Cohesion (psf)	Friction Angle (degrees)	Unit Weight (pcf)
Internal Backfill (in reinforced volume)	0	34	115
External Backfill			
Soil Embankment	-	-	-
Granular Embankment	-	-	-
Foundation Soils			
Existing	-	-	-
Granular Replacement	-	-	-

Where granular replacement of existing foundation materials is required, excavate the existing foundation soil and replace with granular material as shown below. Use granular material meeting the requirements of "granular embankment" in Section 805 of the Standard Specifications, current edition, except that the maximum size is 4 inches. Use material that is classified as non-erodible, as defined in Section 805 of the Standard Specifications, current edition. Place Geotextile Fabric in accordance with Sections 214 and 843 of the Standard Specifications, current edition, as shown below.

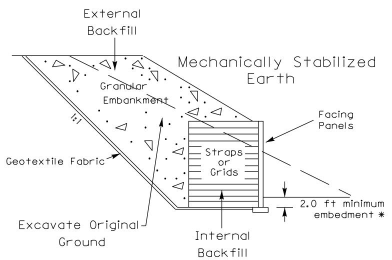
Where external granular backfill is required, place granular material as shown below. Use granular material meeting the requirements of "granular embankment" in Section 805 of the Standard Specifications, current edition, except that the maximum size is 4 inches. Use material that is classified as non-erodible, as defined in Section 805 of the Standard Specifications, current edition. Place Geotextile Fabric in accordance with Sections 214 and 843 of the Standard Specifications, current edition, as shown below.

Temporary shoring, sheeting and/or dewatering may be required during construction.

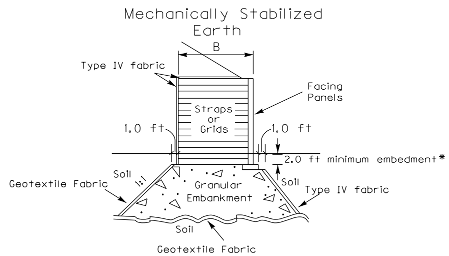


The minimum reinforcment length (L) shall be the greater of:
L > 0.7 H' (Where H' is the effective wall height)
L > 8 ft
H' = H/(1-0.3 tan β) for sloping backfill
H'=H for level backfill

EXCAVATION AND GRANULAR BACKFILL REPLACEMENT

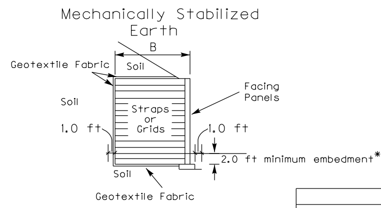


EXCAVATION AND GRANULAR FOUNDATION REPLACEMENT



Geotextile Fabric required only where there is a soil-granular material interface.

MSE WALL ON SOIL



* - Unless Otherwise Noted

REVISION	DATE
DATE: 25-SEPTEMBER-2004	CHECKED BY
DESIGNED BY:	
DETAILED BY: D.CONWAY	J. MOLEN

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
SPRINGFIELD

ROUTE US 555 | CROSSING

S–005–04

ITEM NUMBER
13–765.00

PREPARED BY
Division of Structural Design
GEOTECHNICAL BRANCH

SHEET NO.
DRAWING NO. 00000

Exhibit 38

GEOTECHNICAL NOTES

for Granular Replacement at Reinforced Concrete Cantilever Retaining Walls

The minimum embedment shall be 2 ft, to the bottom of footing for cast in place walls. Walls shall be designed in accordance with the AASHTO LRFD Bridge Design Specifications, current edition.

Size the wall footings at Service Limit State using a Factored Nominal Bearing Resistance of 0.33 times the Nominal Bearing Resistances given below. For checking the Strength and Extreme Limit States, use Resistance Factors of 0.55 and 1.0, respectively.

Granular replacement depths (D) versus wall height (H):
For H <= 10 ft. D= 0
For H>10 ft and <= 20 ft. D= 2.0 ft.

Station Interval	Bearing Surface	Nominal Bearing Resistance
10+20 - 11+15	Soil	- ksf
11+15 - 12+15	Gran. Repl.	- ksf

Use the following soil strength parameters for design:

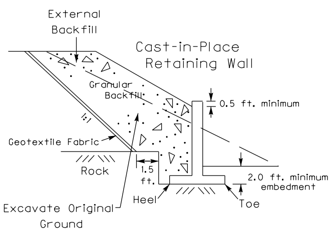
	Cohesion (psf)	Friction Angle (degrees)	Unit Weight (pcf)
External Backfill			
Soil Embankment	-	-	-
Granular Embankment	-	-	-
Foundation Soils			
Existing	-	-	-
Granular Replacement	-	-	-

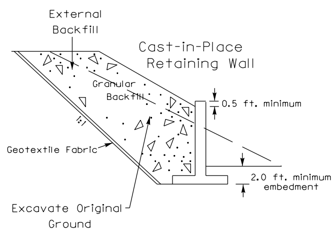
Where granular replacement of existing foundation materials is required, excavate the existing foundation soil and replace with granular material as shown below. Use granular material meeting the requirements of "granular embankment" in Section 805 of the Standard Specifications, current edition, except that the maximum size is 4 inches. Use material that is classified as non-erodible, as defined in Section 805 of the Standard Specifications, current edition. Place Geotextile Fabric in accordance with Sections 214 and 843 of the Standard Specifications, current edition, as shown below.

Where external granular backfill is required, place granular material as shown below. Use granular material meeting the requirements of "granular embankment" in Section 805 of the Standard Specifications, current edition, except that the maximum size is 4 inches. Use material that is classified as non-erodible, as defined in Section 805 of the Standard Specifications, current edition. Place Geotextile Fabric in accordance with Sections 214 and 843 of the Standard Specifications, current edition, as shown below.

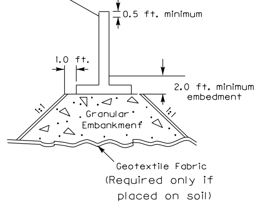
Temporary shoring, sheeting and/or dewatering may be required during construction.

EXTERNAL EXCAVATION AND BACKFILL REPLACEMENT





EXCAVATION AND GRANULAR FOUNDATION REPLACEMENT



REVISION	DATE
DATE: 25-SEPTEMBER-2004	CHECKED BY
DESIGNED BY:	
DETAILED BY: D.CONWAY	J. MOLEN

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
SPRINGFIELD

ROUTE US 555 | CROSSING

S-005-04

ITEM NUMBER
13-765.00

PREPARED BY
Division of Structural Design
GEOTECHNICAL BRANCH

SHEET NO.
DRAWING NO. 00000

Exhibit 38-1