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April 28th, 2020

Dear Dr. Kristina Cantin,

My senior design team partnered with Gresham Smith's Knoxville office to develop roadway plans that will connect the proposed Cherahala Boulevard to Tennessee State Route 162 (Pellissippi Parkway). The goal of this project was to improve the transportation network in the Hardin Valley area by providing access to Pellissippi Parkway for proposed residential developments. Our design also aims to improve safety, particularly during the high traffic volume time periods exhibited during service times at Faith Promise Church. This project not only gave me the opportunity to perform real-world engineering work, but I learned how engineers need to work in a group. The experience was not just about engineering calculations and design, but the planning, organization, and communication required to work as a team. Even for a 7-member team, the scope of the project was very ambitious. These dynamics gave me many opportunities to perform critical tasks that not only contributed to the quality of the design, but also to our team's efficiency. My two main roles, Stormwater Design and Management Lead and Construction Drawings Coordinator, exhibit both of these aspects.

As the leader of the stormwater design, I was responsible for managing the safe and efficient drainage of all rainfall on the site. This position gave me the opportunity to apply engineering calculations and design principles in a real-world scenario. While I had already learned most of the fundamentals of hydrology, this experience required me to research and understand the applicable design guidelines, standards, and requirements specific to our project. Since our project is a contracted Tennessee Department of Transportation (TDOT) roadway design development, most of my design followed the guidelines and requirements of the TDOT Design Division's *Drainage Manual*. The drainage plan depended on existing topography, proposed geometric alignments, and the retaining wall recommendation, so I spent a lot of time coordinating with other team members in order to ensure that my delineated drainage sub basins were based on the proposed design. Using the calculated runoff from these sub basins, I developed a system of roadside ditches, catch basins, and pipes that will safely direct water off site.

I was also responsible for coordinating the submittals of our drawings for each deadline. I reviewed many professional examples and communicated with Gresham Smith in order to make sure our drawing sets were as professional as possible. Our final construction drawings set contains 59 pages compiled from dozens of files in AutoDesk Civil 3D and Microstation. Before this experience, I was the only member of our team who was familiar with Microstation. I helped some other team members complete their tasks (such as vertical alignments in GeoPAK) and draft them in Microstation to ensure uniformity and quality in our presentation. I also drafted work designed by others such as the Right of Way, Erosion Prevention and Sediment Control, and Stripping and

Signage. In addition, I reviewed sheets created in AutoDesk 3D to see if they matched with ones created in Microstation. Finally, I coordinated among our team to ensure that every sheet was accounted for in each submission.

The enclosed document is the final engineering report for the entire project. To find my specific contributions, please reference the attached itemized list of my personal responsibilities and task items. While I was involved in many areas of the project, this list details the items that I "championed."

Sincerely,

Zachary Jerome

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TICKLE COLLEGE OF ENGINEERING

DEPARTMENT OF CIVIL & ENVIRONMENTAL ENGINEERING

Course: CE 400: Senior Design
Instructor: Dr. Jenny Retherford
Team Name: 2020 Vision Engineering
Project Name: Faith Promise Lane: From Cherahala Boulevard to TN-162 MP 1.9

Project Submittals

- 95% Contract Documents: Final Engineering Report
- 95% Contract Documents: Construction Drawings

Role and Personal Task Items

- **Stormwater Design and Management Lead**
- **Construction Drawings Coordinator**
- Final Engineering Report
  - Section 1.1: *Background*
  - Section 1.3: *Purpose*
  - Chapter 4.0: *Stormwater Design & Management*
  - Appendix C: *Stormwater & Drainage Design*
- Construction Drawings
  - Design
    - 2H: Tabulated Quantities
    - 3: Property Map & Layout Sheet
    - 4 & 5: Present Layout & Demolition Sheets
    - 4B & 5B: Proposed Layout Sheets
    - 4C & 5C: Proposed Profile Sheets
      - Stormwater Drainage: Pipe Network
    - 6: Driveway Profiles
    - 7: Erosion Prevention and Sediment Control Notes
    - X1-X30: Roadway Cross Sections
      - Stormwater Drainage: Ditches
  - Drafting
    - 4A & 5A: R.O.W. Detail Sheets
    - 4C & 5C: Proposed Profile Sheets
      - Vertical Curves
    - 7A & 8A: Erosion Prevention and Sediment Control Phase I
    - 7B & 8B: Erosion Prevention and Sediment Control Phase II
    - 10 & 11: Signing and Pavement Marking Plan

Enclosed:

Faith Promise Lane: From Cherahala Boulevard to TN-162 MP 1.9 Final Engineering Report

Faith Promise Lane: From Cherahala Boulevard to TN-162 MP 1.9

FINAL ENGINEERING REPORT

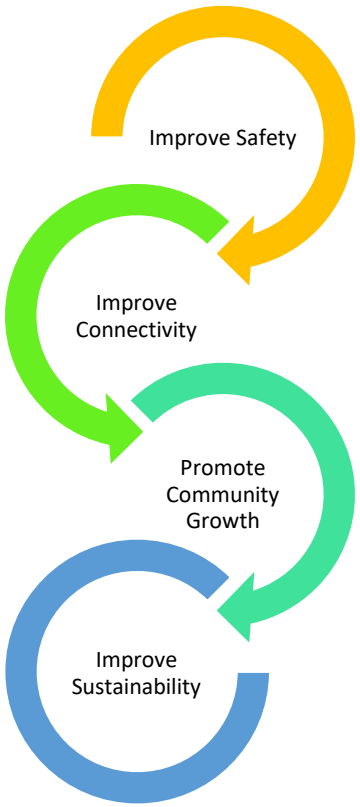
Wesley Darling, Drew Ferguson, Tyler Fox, Zachary Jerome, Collis Lowe, Logan Naifeh, Brady Nutt

UNIVERSITY OF TENNESSEE, KNOXVILLE
TICKLE COLLEGE OF ENGINEERING
DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING

SENIOR DESIGN COURSE EXPERIENCE
CE399S & CE400
FALL 2019 – SPRING 2020

A Note on the NAE Grand Challenges

The work performed by 2020 Vision Engineering to design access redevelopments at milepost 1.9 on TN-162 most closely aligns with the NAE Grand Challenge to restore and improve urban infrastructure. Not only does the project stipulate the design of new roadway infrastructure, but because of the project's planned improvements on community safety, connectivity, and overall alleviation of area traffic, the project closely meets the focus of the National Academy of Engineers on enhancing the sustainability, health, security, and joy of living within the community which the project serves. By working to redevelop access, 2020 Vision Engineering is developing the following impacts for the Hardin Valley community:



It is the ethical obligation of civil engineers to hold the safety, health and welfare of the public paramount in the performance of professional duties. By working to improve the infrastructure people use in their daily lives, 2020 Vision Engineering has also created a solution to better connect the community, reduce overall carbon emissions in the project area, and improve the safety of all users of the proposed and existing infrastructure, fully upholding the code of civil engineers.

Acknowledgements

This project would not have been possible without the help and support of key mentors, teachers and community members. Their guidance and feedback gave 2020 Vision Engineering the tools and technical knowledge to work through the senior design course experience. 2020 Vision Engineering would like to specifically acknowledge the following professionals for their assistance and encouragement on this project:

Patrick Fiveash – Gresham Smith

Jason Brady – Gresham Smith

Jake Graves – Gresham Smith

J. Shap Stiles – Gresham Smith

Donald McCrary – Gresham Smith

Dr. Jenny Retherford – The University of Tennessee, Knoxville

Disclaimer

The work contained in this technical engineering report, its appendices and accompanying engineering drawings was performed by students at the University of Tennessee, Knoxville for a senior design course with limited oversight by a licensed engineer. This work contained in this document meets applicable codes and standards of engineering practice. However, the individuals involved in creating this document assume no liability for construction of the recommended design solution. This project should not be considered for construction without being reviewed in its entirety and approved by a licensed professional engineer.

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1.0 Introduction

This report contains the calculations and conclusions determined by 2020 Vision Engineering for the design of a new connector road and entrance ramp for planned residential subdivisions located in the Hardin Valley area of Knoxville, Tennessee. The design serves to improve connectivity, throughput, and vehicular safety between the community and Tennessee State Route 162 (also known as Pellissippi Parkway), particularly during high traffic volume time periods. The overall design also includes site improvements and an analysis of the effects of the proposed roadway infrastructure on the site itself.

1.1 Background

The project serves as part of a broader plan to modify transportation access to proposed residential developments on Cherahala Boulevard via Faith Promise Lane located in the western part of Knox County. Currently, as shown in Figure 1 and Figure 3, the existing median opening and parkway entrance for Pellissippi Parkway promotes unsafe driving behavior. Drivers from the north direction can only access the site by using the median opening and making a dangerous left turn across on-coming traffic. Drivers exiting the site must either make a left turn across on-coming traffic using the median opening (travelling south) or must turn right onto Pellissippi Parkway through the existing at-grade intersection (travelling north).

High traffic volumes for the community events held regularly at Faith Promise Church create dangerous travelling conditions around the median opening, requiring undesirable temporary traffic control measures such as rolling roadblocks enforced by the Knox County Sheriff's Department, as shown in Figure 2. However, increasing traffic volumes due to the area's rapid population growth are likely to create dangerous driving requirements on weekdays as well in the future, especially for drivers crossing highway traffic to exit the highway or accelerating to free-flow speed in order to merge after passing through the at-grade intersection. The current layout could be dangerous for drivers, and the level of service and traffic safety of the existing infrastructure will continue to decline as average daily and peak hour traffic volumes continue to increase.



Figure 1: Median cut-through on TN-162



Figure 2: Rolling roadblock for regularly held community events

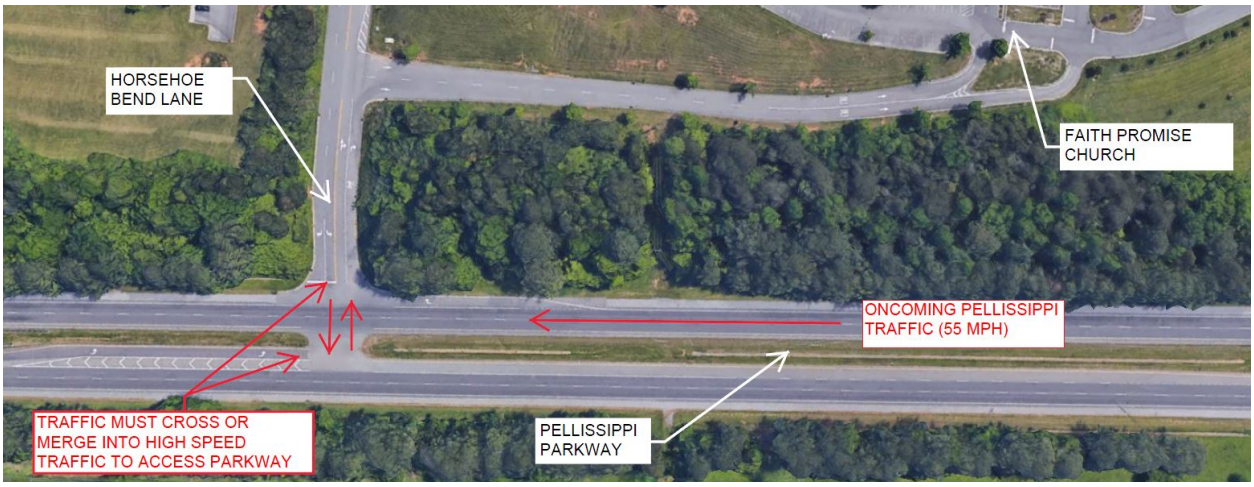


Figure 3: Median cut-through crosses high-speed north-bound traffic lanes

1.2 Existing Site Conditions

The project site has an approximate area of 7 acres and consists of an existing off-ramp from Pellissippi Parkway that connects to Faith Promise Lane (shown in Figure 4), which directs traffic into the 12-acre parking lot for Faith Promise Church located in the middle of the property. Additional existing site features include a detention pond located on the north side of the existing off-ramp and a drainage ditch that flows parallel to the northbound Pellissippi Parkway right-of-way. Traffic entering the parking lot from Pellissippi Parkway flows in a circuit traveling northeast then northwest then southwest around Faith Promise Church towards Horseshoe Bend Lane located on the northwest side of the property. Horseshoe Bend Lane serves as the second entrance and sole exit for Faith Promise Church and the nearby *Aqua Bailers Inc.* manufacturing business. An exception to the standard traffic pattern is on Saturday evenings from 5:00pm-7:00pm and Sunday mornings from 8:45am-12:30pm, when, due to well-attended, regularly held community events, a rolling roadblock is conducted by three traffic officers from the Knox County Sheriff's Department. The rolling roadblocks are implemented to safely direct the increased traffic volume exiting left out of Horseshoe Bend Lane to travel southbound on Pellissippi Parkway in the direction of Hardin Valley.

For the purposes of this senior design project, a roadway proposed by Gresham Smith to extend Cherahala Boulevard to Horseshoe Bend Lane (pictured in Figure 5), as well as planned residential developments, are considered as existing conditions for the site. Though the roadway and subdivision have not yet been constructed, conversations with Gresham Smith indicated that the Cherahala Boulevard extension and residential area will be completed before construction work on Faith Promise Lane begins and should thus be factored into the proposed changes as existing infrastructure.



Figure 4: Northbound view of existing ramp facing towards the Faith Promise Church entrance

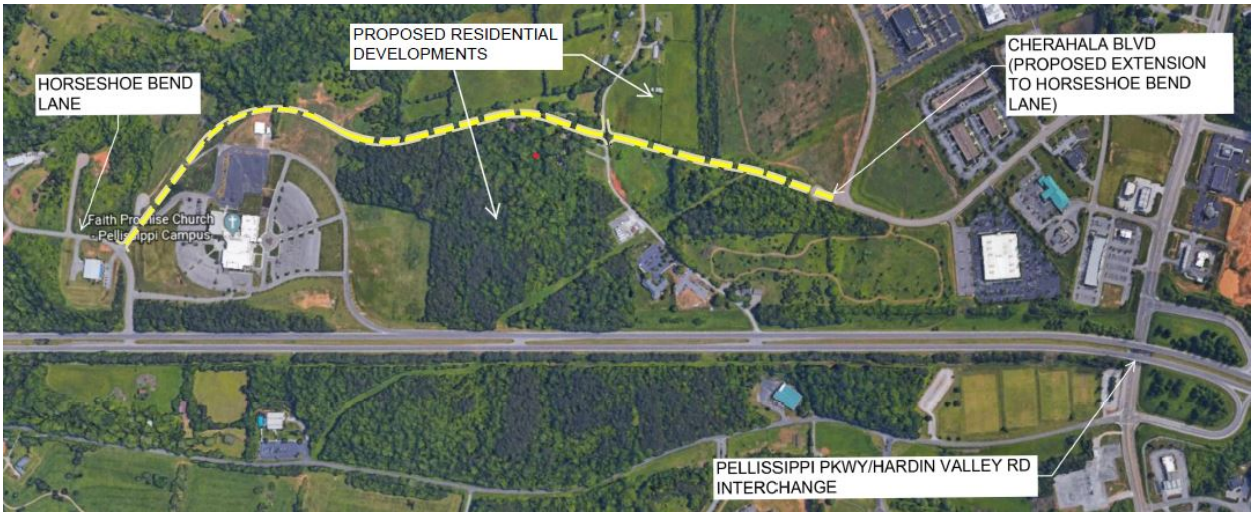


Figure 5: Proposed Cherahala Boulevard (Designed by Gresham Smith)

1.3 Purpose

The purpose of this project is to redesign access to Pellissippi Parkway in order to promote safer driving behaviors, connectivity, and throughput. In addition, redevelopment of the infrastructure of this area will match on-going efforts by the Tennessee Department of Transportation (TDOT) to actively remove median cut-throughs on the Parkway in an effort to increase safety. The existing median opening at Horseshoe Bend is very similar to the median opening removed at Centerpoint Boulevard approximately 1.8 miles further south on Pellissippi Parkway. Therefore, 2020 Vision Engineering has determined two main objectives for the proposed redevelopments (as shown in Figure 6): 1) modify north-bound access to Pellissippi Parkway and 2) provide access to residential developments along Cherahala Boulevard. Redeveloping the north-bound access to Pellissippi Parkway will allow residents and people attending community activities to safely merge into highway traffic on Pellissippi Parkway, and developing a connection between proposed and current residential developments along Cherahala Boulevard and Pellissippi Parkway will replace the need for people having to use the median opening in order to exit the site in the southbound direction. These design improvements will not only promote safer driving actions to access residential communities and community activities but will also decrease weekend congestion and promote safer driving conditions on Pellissippi Parkway.

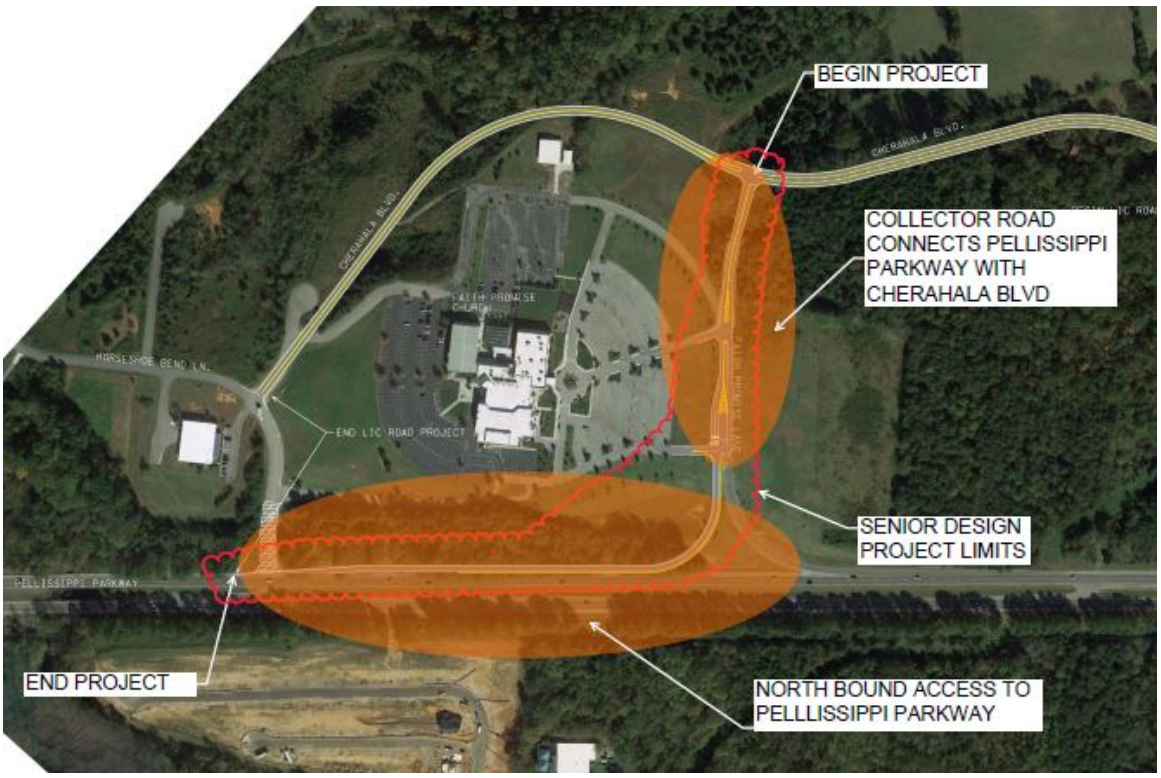


Figure 6: Redevelopment objectives and project site extents

1.4 Project Contact Information

2020 Vision Engineering is comprised of seven civil engineering students who will be graduating in the spring of 2020 with Bachelor of Science in Civil Engineering degrees from the Tickle College of Engineering at the University of Tennessee, Knoxville. The members of 2020 Vision Engineering (pictured in Figure 7) have specialties in the transportation, construction, structural, geotechnical, and environmental disciplines of the civil engineering field and collectively bring 28 years of collegiate education and 7 years of professional experience to the senior design project. Wesley Darling gained construction management experience working for the BNSF Railway for one summer and has several years of management experience from earning a previous degree in Business Administration. Zachary Jerome has spent four summers gaining consulting engineering experience working with Fulghum, Macindoe, & Associates. Collis Lowe held a 4-month co-op position with Messer Construction where he gained construction management and site design experience. Logan Naifeh gained construction experience working for the Lake County Utility District for three summers. Brady Nutt acquired geotechnical engineering experience conducting research for the University of Tennessee, Knoxville during the summer of 2019. Drew Ferguson has a summer of construction management experience working with the Boger Construction firm. Tyler Fox has worked for the Tennessee Valley Authority on their engineering projects for one-and-a-half years. The combined professional experiences of 2020 Vision Engineering (contact information listed in Table 1) provided practical knowledge that closely aligns with the needs of this project. Additional calculation review, mentorship, and general project guidance was given by several members of the Gresham Smith engineering design firm, as well as faculty from the University of Tennessee, Knoxville (contact information provided in Table 2).

Name	***Project Area of Focus***	***Email Address***
Wesley Darling	Project Team Leader; Geotechnical Lead	wdarlin1@vols.utk.edu
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Brady Nutt	Traffic Control Lead	bnutt@vols.utk.edu
Drew Ferguson	Construction Scheduling Lead	bfergu17@vols.utk.edu
Tyler Fox	Structural Lead	nxc511@vols.utk.edu

Table 1: 2020 Vision Engineering Member Contact Information



Figure 7: 2020 Vision Engineering (from left to right: Brady Nutt, Logan Naifeh, Collis Lowe, Zachary Jerome, Tyler Fox, Wesley Darling, Drew Ferguson)

Name	***Organization***	***Email Address***
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Jake Graves	Gresham Smith	jake.graves@greshamsmith.com
Dr. Jenny Retherford	University of Tennessee, Knoxville	jretherf@utk.edu

Table 2: Project Advisor Contact Information

1.5 Technical Engineering Scope of Work

2020 Vision Engineering performed the necessary engineering tasks to design a collector road and an on-ramp connecting the proposed Cherahala Boulevard extension to Pellissippi Parkway and to evaluate the project's impacts on the surrounding area. The geometric design for the new roadways was created by developing horizontal alignments that tied into the Cherahala Boulevard extension, Faith Promise Church's driveways, and the northbound travel lanes of Pellissippi Parkway; vertical alignments that safely traverse the steep grade change between the existing infrastructure; and typical sections and 50-foot cross-sections for the proposed alignments. An earth retaining system was developed by performing a geotechnical analysis of the site soil and designing an economical structural system to support the earthwork necessary for the roadway alignments based on the soil strength, compactness, and response to loading. A traffic control plan for the new roadways was created by performing an initial traffic study of the area, running a simulation of the proposed alignments, and developing a striping and signage plan. The project's impacts on site drainage were evaluated through a stormwater analysis, the development of a drainage plan, and an analysis of the volume change of the on-site detention pond. Construction management tasks performed for this project included quantifying the cut and fill volume requirements for the project, balancing the cut-fill requirements to minimize earthwork, performing an estimate of probable project cost, and developing a construction schedule for the project. All project work was performed in accordance with the Tennessee Department of Transportation (TDOT), the American Association of State Highway and Transportation Officials (AASHTO) standards, Knox County municipal codes, and traditional engineering processes and procedures.

2.0 **Road Design Recommendations**

2020 Vision Engineering designed the horizontal and vertical alignments for the connector road and ramp onto Pellissippi Highway. The geometric design was performed in accordance with the guidelines set by AASHTO's Green Book. Typical sections for the new roadway alignments were created, and 50-foot cross-sections were generated to allow for alignment iteration and cut and fill analysis. The development of the new roadway also required the evaluation of driveway connection tie-ins between Faith Promise Church and the connector road. 2020 Vision Engineering was successful in creating a design that maximizes connectivity to the nearby church, future residential areas, and Pellissippi Parkway for all users while maintaining safety and driver comfort.

Using the criteria described in the AASHTO's Green Book, the horizontal alignments for the proposed roadway were selected, including the minimum radius length for the roadway's design speeds, superelevation, and friction factor as presented in Table 3. The design speed was initially selected based on the current speed of the existing Faith Promise Lane segment and the current approach speed of traffic on Pellissippi Parkway. For the alignment of the ramp, although AASHTO's Green Book only required a minimum radius of 250 ft for the ramp's design specifications, a radius of 325 ft was selected for the horizontal curve on the ramp to allow for a more direct approach to Pellissippi Parkway that would help avoid the complicated terrain of the detention pond as well as increase driver comfort and safety. Additionally, the horizontal curve required a radius small enough to avoid interference with the existing Pellissippi Parkway right of way (see Appendix A). The horizontal alignment was performed in MicroStation and is presented in the project drawings.

Roadway Section	***Superelevation***	***Friction Factor***	***Design Speed***	***Minimum Radius***	***Design Radius***	***Minimum Grade***	***Maximum Grade***	***Taper***
Faith Promise Lane	2%	0.1	30 mph	500 ft	550 ft	0.5 %	2.81%	N/A
On-Ramp	4%	0.2	30 mph	250 ft	325 ft	0.5%	8%	50:1

Table 3: Horizontal alignments design specifications

The vertical curves for the proposed connector road and ramp were designed using the Geopak software tool in accordance with criteria stipulated in the AASHTO's Green Book. The AASHTO criteria include designing the vertical curves to satisfy length criteria pertaining to headlight visibility, general appearance, and rider comfort. According to TDOT standards, on-ramps should be designed to have a maximum downgrade of 5-7%. However, due to the project site's existing topography, which includes a steep decline in elevation of more than 30 ft over the planned length of the ramp, a special exemption in the TDOT standard was utilized, allowing for the ramp to have a downgrade of more than 7% in areas with extreme existing topography. The maximum downgrade for the ramp was selected to be 8% and was only necessary at the steep elevation decline near the retention pond (see Appendix A).

The ramp, connector road, and taper lane were designed with a minimum grade large enough to prevent puddling and flooding as suggested in the AASHTO's Green Book. Because the proposed roadway is at the bottom of a sag curve, flooding on the roadway can present a driving hazard and

must be avoided. Using AASHTO's recommended minimum grade for the vertical alignments will mitigate any potential puddling related to the terrain of the project area. The taper-type design selected to connect the ramp to Pellissippi Parkway satisfies all of AASHTO's Green Book requirements (see Appendix A). Using the Geopak software, the visualizations for the vertical alignment profile for Faith Promise Lane and the on-ramp were generated and are presented in the project drawings.

Typical sections were designed for the planned roadway alignments by modifying TDOT standard drawings to include drainage, safety, and mobility features. A width of 12 feet was chosen for each lane. The typical section design included curb and gutter features to satisfy water drainage for the collector road given its sag vertical curves. Additionally, the inclusion of a sidewalk and buffer provide pedestrian infrastructure for the area. Overall, the 40-ft right of way from the centerline included 2.5-ft curb and gutter, 12-ft lane widths, and allocates 6-ft for sidewalks, all corresponding to TDOT standards. The typical sections for the entrance ramp were modified to include superelevation, foreslope and backslope, lane widths, rounding, shoulder widths, and slope and construction easements. Foreslope and backslope incorporated a 6:1 slope ratio per TDOT standards for a one-lane ramp. The lane width for the ramp was selected to be 16-ft with the inclusion of a 4-ft paved low side shoulder and a 6-ft paved high side shoulder. A 4-ft rounding for both low side and high sides was additionally added to the typical section design.

Using Geopak, 50-foot cross-sections of the proposed roadway were created by using information from the horizontal alignment and vertical profiles created. The horizontal alignment determined the widths of the travel way that were drawn in the cross-sections. The cross-sections at the driveway tie-ins for Faith Promise Church were also aligned with the horizontal alignments, as the special circumstances altered the cross-sections for those locations. The typical sections were utilized to draw the width of the sidewalk, curb and gutter, and buffer to ensure that these features met the standard design criteria. The typical sections were also utilized to draw the slopes of the pavement and subgrade of the proposed roadway to ensure that these features also met the standard design criteria. All elevations in the cross-sections were determined from the respective vertical profiles. Slopes of 6:1 were used along the sides of the travel way whenever possible to serve as an area for drivers that accidentally drift off the road to safely recover. Slopes between 2:1 and 4:1 were used on the side opposite those with roadside ditches. Gentler slopes of 3:1 or 4:1 were used where the existing ground profile permitted, and a 2:1 slope was used in more extreme terrain areas.

The right of way for the project was determined to be 40 feet from the left and right centerline of Faith Promise Lane. Furthermore, the team designed the roadway width to be confined within this 80-foot threshold, so alternation of the easement grant from Faith Promise Church was not necessary. Regarding the new ramp, the majority of the roadway alignments fell outside the border of the property owned by Faith Promise Church and within the existing right of way of Pellissippi Parkway, so there is no need to purchase this part of the property.

3.0 **Retaining Wall Recommendations**

2020 Vision Engineering designed a retaining wall to remove the need for site grade changes which accompany the planned roadway developments including the (19) cut cross-sections that traverse a 35-foot elevation change over 360 feet of the proposed entrance ramp's length. A geotechnical analysis of the project site was performed allowing for recommendations regarding an earth retaining system to be determined. Different types of retaining systems were considered and one was selected based on site right-of-way restrictions, cost, and other secondary criteria. A final retaining wall design which minimizes the proposed roadway's effects on the detention pond and area drainage was then determined.

3.1 **Geotechnical Analysis**

Geotechnical data of the Hardin Valley area was provided to 2020 Vision Engineering by GEOServices, Inc. The data contained boring sample information from tests taken at project areas near Faith Promise Lane as shown in Figure 8, and it was reasonably assumed that the soils at the Faith Promise site were comparable in their mechanical properties and classification. Using the data provided, recommendations for earthwork, such as site preparation, backfill material types, compaction requirements, grading and drainage, and other construction considerations, were determined. Additionally, the soil strength, settlement, and load-transfer properties of the soils were calculated using the geotechnical data to facilitate the design of an earth retaining structure.

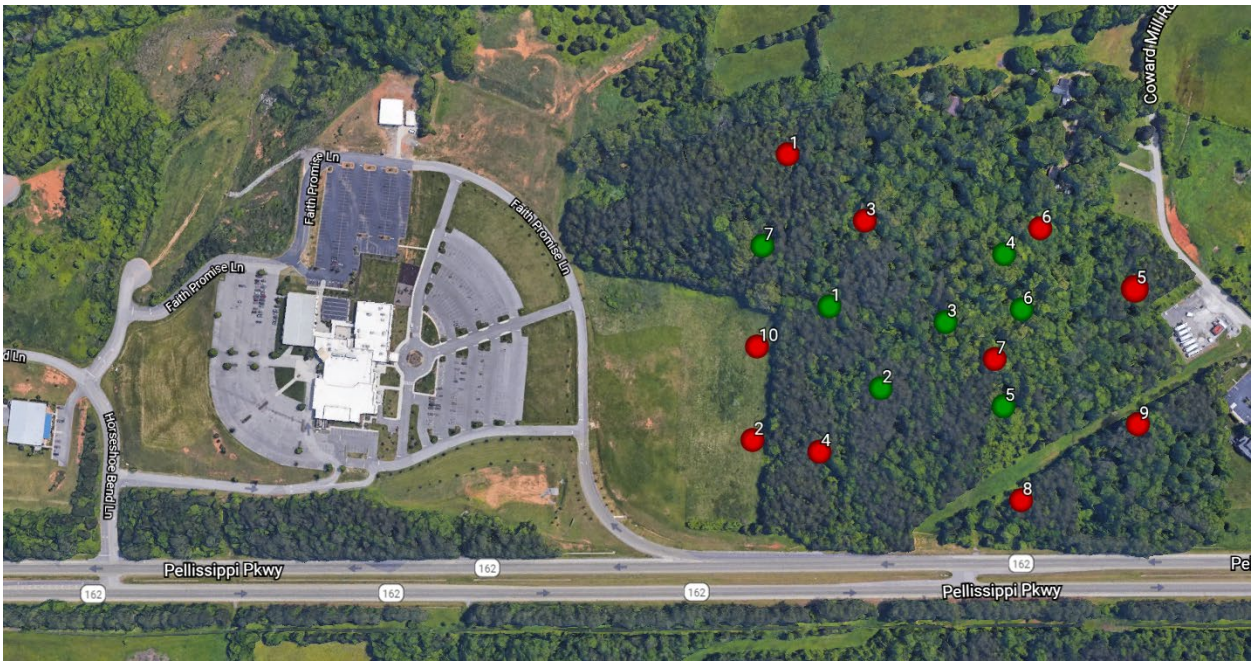


Figure 8: GEOServices boring locations in relation to the project site

The bore logs from the geotechnical reports contained soil characteristic data for the locations shown, including the soil classification, standard penetration test N-values, water content, liquid limit, and plasticity index from the surface to a depth of an average 30 feet. The geotechnical information determined across the 10 boring locations had little variation numerically, and only one soil layer was identified: a layer of lean clay with chert fragments running from below the

topsoil to depth. Based on the roadway cross-sections, an excavation of no more than 20 feet will be required for the proposed roadway design (Appendix B). Because no boring information is available for the actual project site, average values were determined for the first 20 feet of samples across the 10 boring locations and are presented in Table 4.

Soil Characteristic	***Average Bore Log Value***
Soil Classification	CL (Lean Clay, firm to very stiff)
SPT N-Value	14 blows
Water Content	25.1%
Liquid Limit	47%
Plasticity Limit	26%

Table 4: Averaged site soil characteristics

The groundwater table was not reached at any of the boring locations. Because there is no evidence of groundwater retention at the project site, the effects of seepage and pore water pressure in the existing condition were considered negligible during the initial design of the retaining wall. However, to account for potential changes in site conditions such as those related to the detention pond filling during storm events, the earth retaining system selected was required to have mechanisms that allow for adequate drainage integrated into the final design. Additionally, boring location 9 was the only test to reach auger refusal, so it is assumed that a layer of rock is at a depth of 44.1 feet below the surface. This depth is below the proposed roadway excavation limits and thus was also disregarded during retaining wall bearing capacity calculations. By using the soil information given as well as comparable soil sample data and ranges of suitable numbers gleaned from Federal Highway Administration and Naval Facilities Engineering Command reference materials, the conservative values presented in Table 5 were determined (Appendix B).

Soil Characteristic	***Estimated Value***
Dry Unit Weight of Soil	94 lb/ft^3
Unit Weight of Soil	118 lb/ft^3
Soil Friction Angle	28°
Soil Cohesion	2000 lb/ft^2

Table 5: Estimated site soil characteristics

The soil in the project area is classified under the USCS system as a low-plasticity Lean Clay (CL) and under the AASHTO system as group A-6. Though the soil does not undergo large volume changes due to its low plasticity index, it does have low permeability and poor bearing strength. Per AASHTO, this soil is not suitable for use as subgrade material without alteration. In order to improve the drainage of the subgrade post-construction, a system of subsurface collector pipes was designed to run the length of the paved portion of the right of way and remove water from the subgrade-base layer interface and direct it towards the roadside drainage ditches. Additionally, a geotextile fabric was selected to separate the subgrade from the pavement base course, assisting with drainage and helping to better distribute loads from the pavement across the entirety of the subgrade. It is also recommended that the subgrade be compacted with a sheepsfoot roller before paving to increase the soil density and strength.

Because the site soil has low permeability, a free-draining granular material is recommended to be used as backfill for the earth retaining system selected to reduce porewater pressure from forming

behind the retaining system. In conjunction with a piping and weep-hole drainage system running the length of the retaining structure, increasing the drainage of the retained earth along the length of the proposed roadway will mitigate potential soil stability issues. Incorporating the drainage features described in the project drawings and using a backfill material such as that described in Table 6 will also allow for a more economical retaining structure to be designed as drainage helps to reduce loads related to horizontal earth surcharge.

Parameter	***Value***
Backfill Aggregate Type/Grading	Class A Grading D
Unit Weight of Backfill	120 lb/ft^3
Backfill Friction Angle	30°
Backfill Cohesion	0 lb/ft^2

Table 6: Backfill material requirements

The earth retaining structure was designed with the assumption that all structural components would bear on the existing site soils while all soil retained by the structure would be the specified backfill. Vesic's soil bearing capacity equation was applied to determine the allowable bearing pressure for the lean clay as well as a minimum footing dimension, and Terzaghi one-dimensional consolidation settlement equations were used to determine the total and differential settlement of the structure. The ultimate coefficient of sliding friction for concrete on clay was gleaned from a Naval Facilities Engineering Command reference material (Appendix B). These properties, which are listed in Table 7, were the limiting parameters for the design of the retaining wall and were factored into the iterative LRFD design process of the retaining wall system.

Parameter	***Value***
Net allowable bearing pressure	6510 lb/ft^2
Minimum retaining wall slab dimension	6 feet
Approximate total settlement	1.47 inches
Estimated differential settlement	0.86 inches
Ultimate coefficient of sliding friction	0.45

Table 7: Retaining Wall Soil Foundation Design Parameters

3.2 Retaining Structure Alternative Analysis

The development of an earth-retaining system required selecting the type of system which best fit the project. The topography of the proposed entrance ramp and the project right-of way restrictions were the initial decision criteria. Budgetary limitations and material costs were additionally considered. The last selection criteria for the retaining structure used was based on each retaining system's response to seepage under potential, new site drainage conditions. After the alternative solutions were considered, a final selection for the retaining structure was made.

The type of retaining wall selected for use along the proposed roadway was determined initially by the area's soil characteristics and strength. Because of the retaining wall's intended function as a means to reduce the necessary excavation of the existing berms of the on-site detention pond, the height of the section cuts requisite of the proposed entrance ramp also contributed to the selection process. Based on these factors, the preliminary list of applicable earth retaining systems was narrowed to a mechanically stabilized earth (MSE) wall or a cast-in-place cantilevered wall.

Given the cross-section information of the available retaining systems, the selection of an MSE wall system would require anchors at varying lengths of up to 70% of the wall height to be utilized in order to meet LRFD stability criteria. Though both cantilever and MSE wall systems require a large quantity of engineered fill, due to the anchor length, the fill required for the MSE wall was greater than the amount needed for the cast-in-place cantilever wall and would involve significantly more cut material (Appendix B).

Another factor considered in the retaining structure selection process was the overall cost of each alternative. The cast-in-place concrete cantilever wall was priced according to the constituent materials. The MSE wall was priced per square foot of exposed wall, which includes all structural materials and engineered retaining fill (Appendix B). The final materials cost of the two retaining systems differentiated greatly as shown in Table 8, with the cast-in-place concrete cantilever wall being the most economical choice.

Cast-In-Place Concrete Cantilever Wall			
	Price	***Quantity Needed***	***Total Cost***
Class A Concrete	$144/cub.yd.	477 cub. yd.	$68,700
Reinforcing Steel Bars	$0.90/lb	87670 lb	$78,900
Granular Backfill	$31.90/ton	355 ton	$11,325
Select Sand Backfill	$19.00/ton	3811 ton	$72,410
		TOTAL:	$231,335
MSE Wall			
	Price	***Quantity Needed***	***Total Cost***
MSE Wall	$100/sq.ft.	3130 sq.ft.	$313,000
		TOTAL:	$313,000

Table 8: Material cost comparison of Cast-In-Place Concrete Wall and MSE Wall

The last factor considered when selecting the earth-retaining system was the system's response to soil seepage, as changes made to the project site could impact the area drainage and influences the soil saturation. Seepage would be a concern for MSE walls under saturated conditions as the presence of subsurface water affects the wall anchors' bond with the reinforced fill. For a comparable MSE wall solution, an extensive storm drain system would need to be installed and the lengths of the anchors and quantity of reinforced fill would need to be increased beyond the 70% of the wall height standard in order to account for the potential increase in lateral stress. For concrete cantilever walls, however, seepage can be neglected due to the presence of free-draining granular backfill, the placement of weep-holes along the face of the wall, and a drainage piping system along the stem of the wall. From the analysis of all stated factors, it was determined that a cast-in-place concrete cantilever retaining wall would be the most economical and practical option to be sufficient for soil loading demands while maintaining a minimal footprint and requiring minimal additional excavations and property easements.

3.3 Cast-In-Place Concrete Cantilever Retaining Wall Design

A cast-in-place concrete cantilever retaining wall was designed to minimize right of way impacts and materials usage while providing sufficient strength to meet all limit states. Applicable load types that affect the wall were determined, and factored load combinations were calculated. An iterative design process was used to select a retaining wall cross-section, calculate the strength capacity of the design and check limit states, evaluate the proposed cross-section's effects on the area's soils, and revise as necessary. With all stated factors considered, a final design for the retaining wall was selected and a proposed layout along the roadway was developed.

Using the AASHTO Load and Resistance Factor Design (AASHTO LRFD) manual, applicable loads and load factors were determined. The supported soil will experience earth surcharge loads in the horizontal and vertical direction in addition to dead loads from the weight of the wall and the soil the wall is supporting. The maximum wind pressure that could affect the wall was also determined. Because the site supported by the retaining wall is intended to remain vacant and will receive no vehicular or pedestrian traffic, it was determined that no live loads would affect the soil and thus the additional stresses from this type of loading were ignored. The project site's seismic activity level was determined and because it falls significantly below the magnitude required for analysis in the LRFD manual, earthquake loads were also ignored (Appendix B). The Strength Ia, Strength Ib, Strength III, and Serviceability load combinations (shown in Table 9) were selected for analysis and lead to the development of a wall design that satisfies stability, soil bearing strength, sliding, and overturning checks.

	Load Factors			
Type of Load	***Strength Ia***	***Strength Ib***	***Strength III***	***Serviceability***
Passive Earth Surcharge Load	1.35	1.35	1.35	1
Horizontal Active Earth Surcharge Load	1.5	1.5	1.5	1
Vertical Earth Load	1	1.35	1	1
Self-Weight of Wall	0.9	1.25	0.9	1
Wind Load	–	–	1.4	0.4

Table 9: LFRD load factors and load combinations for retaining wall

The preliminary design of the retaining wall required determining the wall stem heights necessary for retaining the soil of the drainage pond berms. The stem of the retaining wall was required to extend 2 feet into the soil to place the footing below the frost depth for eastern Tennessee (Appendix B). The stem of the wall was also designed to extend an additional foot above the backslope of the fill to allow for the creation of a drainage trench to run along the length of the top of the wall. The stem of the wall was drawn to be placed at the edge of the shoulder for the proposed ramp. To minimize the required property easements necessary for the wall's maintenance and construction, the geometric "rule of thumb" of designing one-third of the wall footer as the toe and two-thirds of the footer extending into the engineered fill as the heel was used. From calculations,

the “rule of thumb” was found to provide sufficient stability while staying within the right of way along the length of the wall (Appendix B).

The entrance ramp’s location is on a hill starting at station 3+20 and extending to station 6+80 and requires a wall of varying height and backslope as the elevation decreases. The selected layout for the retaining wall is presented in the project drawings. Because the base footer of the retaining wall must maintain uniform dimensions along its entire length for ease of constructability, the worst-case cross-section was selected to design the footer. However, half of the wall’s layout requires a wall height below 10 feet, as shown in Table 10, and thus does not necessitate the same size footing required to support the 21 foot tall section. Therefore, two worst-case cross-sections (Table 11) were selected for design – one for stations requiring a wall height over 10 feet and one for stations requiring a wall height less than 10 feet – which will reduce the material quantities required for the wall’s construction.

Required Wall Height	***Length of Wall***
3 – 5 feet	60 feet
5 – 10 feet	120 feet
10 – 15 feet	20 feet
15 – 21 feet	160 feet

Table 10: Required wall height groupings

Wall Parameter	***Dimension***
Required Stem Lengths	10 feet 20.66 feet
Backslope	18 Degrees
Depth of Stem	2 feet
Length of Toe	B/3

Table 11: Constraining parameters for the worst-case retaining wall sections

During the wall sizing process, the inclusion of a shear key was considered to reduce the sliding potential for the wall and to decrease the slab thickness. However, due to the importance of project construction simplicity, particularly along the entrance ramp section which requires several lane closures during construction, it was determined that the casting of a shear key for the wall slab would greatly increase the construction time and complexity. Additionally, the shear key could be omitted with only minimal added footing thickness. The final slab sizing and wall thicknesses presented in Table 12 were determined through the LRFD design process and each supports the worst-case heights of retained backfill and associated loadings affecting the wall while maintaining adequate safety factors against sliding, rotation, and bearing capacity failure (Appendix B).

Parameter	*Wall Design 1*	*Wall Design 2*
Wall Height	3 – 10 feet	10 – 21 feet
Applicable Stations	3+20 to 4+40 6+20 to 6+80	4+40 to 6+20
Stem Thickness	1 foot	2 foot
Footing Thickness	1 foot	2 foot
Footing Length	6 foot	14 foot

Table 12: Final designed retaining wall sizes

Based on the load cases identified, the American Concrete Institute (ACI) Building Code Requirements for Structural Concrete 318-14 were used to design the steel reinforcements of the cast-in-place concrete cantilever retaining wall. The stem of the reinforced concrete cantilever wall was designed to resist flexural bending from the maximum factored computed moment. Though wind loads were incorporated into the calculations of the maximum moment in the stem, the wind pressure never reversed the moment reaction created by the earth surcharge forces, therefore flexural reinforcement was only required on the soil-facing side of the wall. The footing of the wall was designed to resist the weight of the backfill soil and the weight of the wall itself. Due to the shear stresses experienced in the stem and footing of the wall, to provide sufficient resistance to loading, their dimensions were calculated to be identical in terms of width and depth for each wall design, further contributing to a simplified construction process (Appendix B). The typical wall reinforcements are presented in the project drawings.

2020 Vision Engineering's analysis into drainage of the project area determined that the earth retained will be susceptible to some degree of water loads. Proper water drainage away from the wall will be provided through the inclusion of a geotextile drainage core and a perforated drainage pipe along the stem of the wall. These features will ensure that water will not collect and impact the structure's integrity. Water ponding factors such as corrosion of reinforcement or freeze-thaw cracking of the concrete is thus protected by this design. To further improve drainage for the wall, weep-holes are included along the stem at specified spacings throughout the entire construction. Expansion and contraction joints were also included to additionally support the wall through freeze-thaw cycles and general changing weather conditions. The expansion and contraction joints will provide the necessary tolerances to help protect against potentially severe structural implications and will promote overall sustainability for the entirety of the retaining wall

4.0 **Stormwater Design & Management**

2020 Vision Engineering performed engineering calculations to manage the stormwater drainage of the project site in accordance with the TDOT Design Division Drainage Manual, which was developed based on the Model Drainage Manual from the American Association of State Highway and Transportation Officials (AASHTO). By referencing this manual, proposed geometric alignments, and the retaining wall recommendation, the change in area runoff was calculated and a drainage plan developed to move stormwater away from the site in a safe and efficient manner through ditches, inlets, and pipes. Additionally, an erosion prevention and sediment control plan (EPSC) using TDOT standards was developed to prevent project construction from disturbing nearby surface water bodies.

Because Faith Promise Lane will be a collector road, the drainage plan designed was required to provide site resilience for a 50-year storm event, including all inlet spreads, ditches, roadway freeboards, and culverts. Drainage subbasins and outfall locations, presented in Table 13, were determined using existing topography, proposed vertical curves, cross sections, and grade work. The site has 19 drainage basins (10 "on-site" and 9 "off-site") and 3 outfall locations. Runoff coefficients and time of concentration values were determined using the Drainage Manual, which also provides balanced stormwater rainfall distribution. Using the Rational method, flow rates were calculated for each basin (Appendix C).

	Drainage Subbasin	***Outfall***	***Runoff Coefficient***	***Time of Concentration (min)***	***Area (Acres)***	***Q (cfs)***
On-Site	SB 0	3	.40	5	0.1853	0.58
	SB 1	2	.85	5	0.0985	0.65
	SB 2	2	.69	5	0.6703	2.48
	SB 3	2	.85	5	0.3041	2.02
	SB 4	2	.85	5	0.2867	1.91
	SB 5	1	.85	5	0.1113	0.74
	SB 6	1	.85	5	0.1115	0.74
	SB 7	1	.40	5	0.281	0.88
	SB 8	1	.85	5	0.1015	0.68
	SB 9	1	.85	5	0.1015	0.68
Off-Site	OB 1	3	.49	15.00	10.7370	27.61
	OB 2	3	.50	15.33	8.2667	21.00
	OB 3	2	N/A	N/A	3.2312	N/A
	OB 4	2	N/A	N/A	1.4876	N/A
	OB 5	2	.82	5	0.5979	3.83
	OB 6	2	.66	11.35	1.9850	7.68
	OB 7	1	.71	11.18	0.3957	1.05
	OB 8	1	.40	5	3.9055	0.88
	OB 9	1	.40	5	0.1600	0.50

Table 13: Drainage Area Summary

The flow rates from each subbasin were used to size ditches, space inlets, and place and size pipes that safely and efficiently redirect water off-site. Ditches were sized in accordance with Chapter 5 of the TDOT Drainage Manual: Roadside Ditches and Streams. The roadside ditches are designed to hold the flow runoff from the subbasins they are serving, as shown in Table 14. Different linings (concrete, riprap, or vegetation) were selected to reduce shear stress on the ditch depending on the slope, velocity, flow rate, and capacity of the sized ditch. Appendix C supplies information about the different parameters surrounding each lining choice and the specific design components for each ditch are presented in the project drawings.

2020 Vision Engineering designed the storm water pipe system in accordance with Chapter 7 of the TDOT Drainage Manual: Storm Drainage Systems. Inlet placement was determined from a minimum allowable spread of 8-ft and a maximum spacing of 400 feet. Inlets were placed in order to fully capture all runoff from the road and applicable ditches. The team used the Rational method to calculate pipe sizing and slopes, as shown in Table 14. As explained in section 7.04.6 in the TDOT Drainage Manual, storm drainage systems that are designed for 50-year storm events do not require a hydraulic grade line computation. Therefore, no HGL check was performed (Appendix C).

	Pipe / Ditch	*Subbasin(s) & Outfall(s)*	*Manning's N*	*Lining*	*Velocity (ft/s)*	*Slope*	*Size*
Ditches	FPL South Sta. 0+00 to 2+00	OB 9	0.830	Vegetation Class B	0.18	2.81%	1.46' depth, 1:3 slopes
	FPL North Sta. 0+00 to 4+25	SB 7	0.484	Vegetation Class B	0.31	2.81%	1.52' depth, 1:3 slopes
	FPL North Sta. 4+25 to 4+75	OB 7	0.475	Vegetation Class B	0.31	2.36%	1.58' depth, 1:3 slopes
	Driveway A	OB 7	0.484	Vegetation Class B	0.31	2.81%	1.52' depth, 1:3 slopes
	FPL North Sta. 5+00 to 8+00	OB 6	0.156	Vegetation Class B	0.81	1.17%	2.28' depth, 1:3 slopes
	Driveway B	OB 5	0.219	Vegetation Class B	0.76	2.36%	1.81' depth, 1:3 slopes
	Ramp North Sta. 2+40 to 3+00	SB 2 & EW-2	0.072	Riprap Class A-1	3.89	8.00%	1.62' depth, 1:3 and 1:6 slopes
	Ramp South Sta. 2.90 to 5+90	SB 0	0.124	Riprap Class A-1	1.12	8.00%	0.89' depth, 1:4 and 1:6 slopes
	Ramp North Sta. 7+00 to End	OB 1 and Outfall 2	0.013	Concrete	15.95	4.85%	2.06' depth, 1:2 and 1:6 slopes
Pipes	CB-1 to EW-1	SB 1, 2, 3, 4	0.013	RCP III	8.40	2.00%	18" dia.
	CB-2 to CB-1	SB 2, 3, 4	0.013	RCP III	7.28	1.50%	18" dia.
	CB-3 to CB-2	SB 3	0.013	RCP III	5.94	1.00%	18" dia.
	CB-4 to CB-2	SB 4	0.013	RCP III	5.82	0.96%	18" dia.
	CB-5 to EW-2	SB 5, 6, 7 OB 7, 8	0.013	RCP III	7.28	1.50%	18" dia.
	CB-6 to CB-5	SB 6, 7 OB 7, 8	0.013	RCP III	7.28	1.50%	18" dia.
	CB-7 to CB-6	SB 7 OB 7, 8	0.013	RCP III	7.28	1.50%	18" dia.
	CB-8 CB-7	SB 8	0.013	RCP III	9.86	2.75%	18" dia.
	CB-9 to CB-8	SB 9	0.013	RCP III	4.20	0.50%	18" dia.

Table 14: Pipes and Ditches Summary

Stormwater from the proposed development ultimately drains into Beaver Creek. The Erosion Prevention and Sediment Control Plan (EPSC) facilitates the management of erosion and sediment during construction and evaluates future impacts of the proposed work. This plan provides recommendations for best management practices (BMPs) in order to contain all sediment and natural materials within the site and to maintain the structural integrity of the soil. These measures, which are listed in Table 15, will prevent sediment particles and other erosion items from the site from draining into Beaver Creek. Additionally, a construction entrance is planned to further

minimize sediment deposition on nearby roadways and reduce the amount of sediment in runoff of the project site. The designed layout for the EPSC plan is presented in the project drawings.

EPSC Measure	***Quantity***
Silt Fence	2,313 feet
Slope Protection	14,520 square yards
Rock Check Dam	25 count
Inlet Protection	9 count
Catch Basin Protection	9 count
Culvert Protection	1 count

Table 15: Erosion Prevention and Sediment Control containment measures

5.0 **Striping & Signing Recommendations**

The process of redesigning Faith Promise Lane and the construction of the new entrance ramp required alterations to the roadway striping and signing throughout the project area. Due to the addition of the entrance ramp, 2020 Vision Engineering reconfigured the lane space provided to motorists throughout Faith Promise Lane by following guidelines listed in the MUTCD and throughout TDOT Standards while also considering the addition of pedestrian infrastructure. Upon determining the new lane configuration, 2020 Vision Engineering created a permanent striping and signing plan for Faith Promise Lane and the entrance ramp that directs traffic behavior efficiently while maintaining a standard of safety throughout the project area.

Because of the addition of the new entrance ramp, the lane configuration along Faith Promise Lane had to be redesigned in order to adapt to the increase in average vehicular traffic brought from Cherahala Boulevard as well as that of alternate modes of travel originating from the planned residential developments. As a result of the expected increase in vehicular traffic, it was determined that vehicular storage along Faith Promise Lane would be an issue for motorists due to turn movements into the Faith Promise Church campus. To account for this, two turn lanes have been added to Faith Promise Lane in the redesign to facilitate the constant movement of traffic and prevent traffic breakdowns. One turn lane allows vehicular storage for those turning left into the Faith Promise Church campus via the driveway closest to Cherahala Boulevard, while the other turn lane provides storage for vehicles turning right into the campus at the ramp intersection. This second lane also provides a dedicated through lane for those entering Pellissippi Parkway via the new entrance ramp (Figure 9). The addition of the two turn lanes minimizes the chances of through traffic having to stop for a turning vehicle, and this reduction will minimize back-ups along Faith Promise Lane, Cherahala Boulevard, and the existing ramp exiting Pellissippi Parkway.

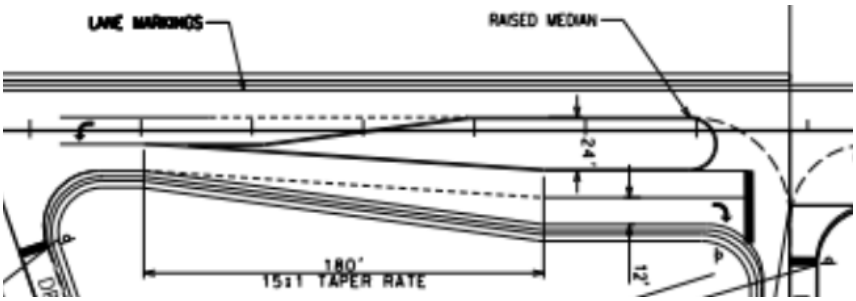


Figure 9: Turn lanes for vehicle storage on Faith Promise Lane

Pedestrian infrastructure was also implemented in the redevelopment of Faith Promise Lane in order to add features that promote alternate forms of travel along the new, actively designed street. Pedestrians were primarily considered due to the proximity of the proposed residential developments around the Faith Promise area. The pedestrian infrastructure was designed based off the assumption that the residential developments will result in higher pedestrian activity. For the case of Faith Promise Lane, these sidewalks may be used casually or as a passage from the residential developments into the Faith Promise Church campus. The sidewalks designed span along the entirety of Faith Promise Lane on both sides and have a width of 6 feet. TDOT Standard Drawing MM-SW-1 was referenced in the designing of this infrastructure (Appendix D). Overall,

the addition of pedestrian infrastructure and the reconfiguration of lane space along Faith Promise Lane makes the street more user-friendly for all potential modes of travel.

The striping and signing plan for the project area was designed corresponding to the new lane configuration. The plan maintains the focus on user safety and vehicular storage to avoid traffic breakdowns during peak travel hours. To facilitate this, two medians were selected for Faith Promise Lane: one raised concrete median towards the Pellissippi Parkway ramp intersection, and one channelizing median between the driveway closest to Cherahala Boulevard and Cherahala Boulevard itself (Figure 10). This space was not converted into a consecutive turning lane along Faith Promise Lane for the purpose of regulating speeds by dividing the lanes of traffic. For the channelizing median, yellow pavement markings with interior striping angled at 45 degrees were selected, as specified by the MUTCD, to redirect traffic into their appropriate directions of travel in the case that the driver coasts out of his or her lane (Appendix D). For the remainder of pavement markings, (4) white stop bars and (2) turning arrows were selected to add emphasis to the correct traffic movements (Figure 11). Additionally, each stop bar is accompanied by a stop sign. All stop motions were warranted by guidelines set by the MUTCD, and all markings were sized appropriately according to the MUTCD (Appendix D). Additionally, for the entrance ramp, consecutively dashed lines were utilized between the border of the ramp and the highway to indicate the merge movement onto Pellissippi Parkway (Figure 12).

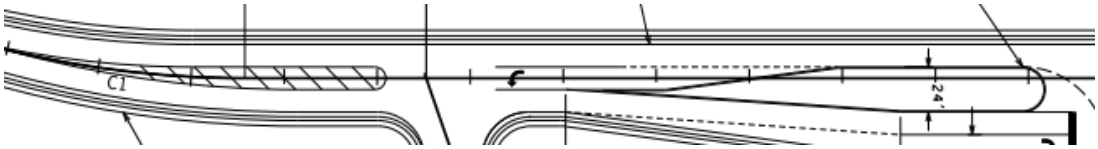


Figure 10: Channelizing (left) and raised (right) medians on Faith Promise Lane

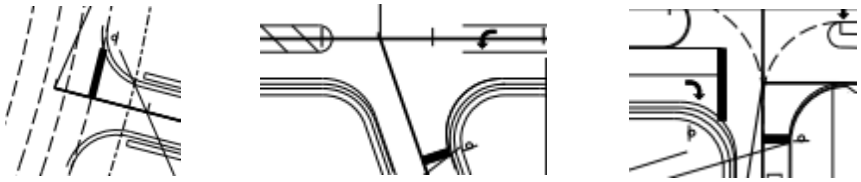


Figure 11: Pavement markings and signage used throughout Faith Promise Lane (From left to right: Cherahala Boulevard intersection, west driveway, ramp intersection)

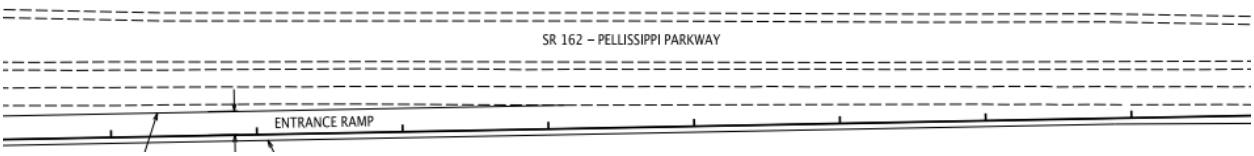


Figure 12: Merge of the entrance ramp and Pellissippi Parkway

6.0 Construction Management Plan

2020 Vision Engineering developed a construction management plan for the proposed roadway project, including a construction schedule, traffic control plan, and a final quantities and costs estimate for the project. A full construction schedule was developed based on the required tasks defined in the project scope of work, and a total project duration was determined using RSMeans, a construction estimating database. A traffic control plan was designed to facilitate safe throughput of area drivers during the construction of the entrance ramp and the repaving of Faith Promise Lane. The quantity of the different materials required for the project were determined based on the final design drawings. A final cost estimate was then determined using the material quantities and referencing the average TDOT bidding prices for Region 1.

6.1 Construction Scheduling Recommendations

A project construction schedule was created for the construction process of the final proposed roadway design. The schedule was developed by dividing the major construction tasks into three separate phases, which will ensure that community members have continuous access to the area throughout construction. An equipment list was constructed to determine what was needed to complete each task in a timely manner and determine which tasks can occur concurrently to minimize downtime. A Gantt chart was developed to visually present the ideal chronology of the phases and tasks. From the performed construction scheduling work, a total project length in days was determined. Use of this final schedule will help minimize delays between construction tasks and maximize project efficiency.

The phasing of the project was developed by analyzing the major aspects of the project scope of work. Three phases were determined: the first phase includes the redevelopment of Faith Promise Lane and the driveways that tie-in along its length; the second phase is the proposed entrance ramp merging onto Pellissippi Parkway from Faith Promise Lane, which includes the construction of the retaining wall; and the final phase of the project is the closing and scarification of Horseshoe Bend and the completion of final minor tasks on the project. The order of these phases was selected based on the existing usage of the site. It is important that vehicular access be maintained to the Faith Promise Church campus and that drivers have the ability to enter Pellissippi Parkway throughout the duration of construction. Construction on Faith Promise Lane takes place first as the redesigned striping and signage plan will allow access to the church to be maintained in later stages due to its additional turning lane. Construction of the ramp follows suit with the exit of Horseshoe Bend staying open to provide consistent access to the site. The final stage of closing Horseshoe Bend will only take place once the ramp is completed and driver access both in and out of the project site is fully available.

The types and quantities of equipment required for the construction of the proposed roadway designs were determined using “Construction Planning, Equipment, and Methods” textbook as well as the project material quantities. Through examining the available usage of each piece of equipment as described in the referenced textbook in relation to the material quantities of the project, the specific equipment needs to complete all construction tasks could be determined. A

total list of heavy equipment required for the project was compiled and used for the development of the final construction schedule (Appendix E).

A Gantt chart was created for the project and presents the chronological order that the construction tasks should be completed for ideal equipment and labor usage. Each of the three phases of the project construction were broken down into their constituent construction tasks. Major construction tasks were identified using the General Notes and Special Notes sheets presented in the final project drawings. The length of each construction task was determined using RSMeans, an online construction estimation database. The task lengths were calculated by taking the project quantity for each specific task and dividing that by the daily crew output recommended by RSMeans. The final Gantt chart was constructed by laying out each task at its calculated duration and organizing tasks based on their dependency to other tasks (Appendix E).

The total duration of the proposed roadway project was determined using the Gantt chart task durations, as well as several assumptions based on worker output and site conditions. During calculations it was assumed that each crew of laborers works 8-hours each workday and that there are 5 workdays a week. It was also assumed that the project site was not affected by weather delays or overtime. Based on this, contractors would be able to add or subtract workdays based off their company's work schedule, project budget changes, or the presence of rain days estimated over the timeline of the project. The total project length was calculated by summing the duration of the construction tasks along the critical path (Appendix E). From the construction schedule developed, the total project length was determined to be 93 days.

6.2 Traffic Control Plan

2020 Vision Engineering developed two traffic control plans that will maintain commuter and construction worker safety throughout the project construction processes of both the entrance ramp and Faith Promise Lane. The two plans were developed to regulate the traffic flow through the respective project construction areas—the right shoulder of Pellissippi Parkway, and the western half of Faith Promise Lane. By referencing TDOT traffic control standards and MUTCD guidelines, a safe, temporary traffic control plan for drivers on both Pellissippi Parkway and Faith Promise Lane was developed. The appropriate quantities and placements of the necessary traffic control devices were calculated and distributed throughout the project area based on recommendations provided in the MUTCD.

A two-week time allotment of the construction schedule has been dedicated for earthwork along the shoulder of Pellissippi Parkway, which includes the removal of trees and the cutting and filling of soil, for the construction of the entrance ramp. Conversely, the construction of the raised median and the tie-in of pavement from the entrance ramp to the west end of Faith Promise Lane requires vehicular access be prohibited in the construction activity area while still allowing for the majority of the street to remain open to the public for transportation. Due to the nature of this work and the proximity to flowing traffic, it was determined that certain closures within proximity of the two construction areas were required, as stated in the MUTCD standards. Regarding Pellissippi Parkway, the right lane must be closed for the duration of the earthwork for placement of the entrance ramp. This closure is necessary to provide a clearance zone between the operating machinery, construction workers, and flowing traffic. By indicating a work zone and a lane closure

with ample temporary traffic control signage placed up to 1 mile ahead of the construction activity area along both directions of travel, this closure minimizes the risk of construction related accidents occurring between workers and motorists.

The specified distances from the project area for all traffic control devices used along Pellissippi Parkway (Table 16) have been determined based on Figure 6H-33 provided in the MUTCD. Along with temporary signage, channelizing drums were selected to guide motorists through the work zone by tapering the two lanes of Pellissippi Parkway into one and acting as a physical barrier between the work zone and the traffic lane (Table 17). Drum spacing was determined directly from the MUTCD Sections 6C.08 and 6F.63 (Appendix E). The developed traffic control plan facilitates the earthwork process of the project construction while providing sufficient warning for incoming traffic traveling in both directions of Pellissippi Parkway at off-peak hours of vehicular flow.

Sign	Length from South Threshold of Activity Area (ft)	Length from North Threshold of Activity Area (ft)
Type 3 Arrow Sign	500	N/A
Merge Left	1000	N/A
Right Lane Closed ½ Mile	2500	N/A
Road Work 1 Mile	5140	N/A
End Road Work	500	500
Road Work Ahead	N/A	2640

Table 16: Signs with corresponding placement distances for temporary traffic control devices for construction along Pellissippi Parkway

Section	Length (ft)	Spacing (ft)	Quantity
Taper prior to Activity Area	780	20	39
Activity Area	1520		76
Total	2300		115

Table 17: Channelizing drum spacing and required quantities for construction along Pellissippi Parkway

2020 Vision Engineering's traffic control plan for Faith Promise Lane prohibits access to the construction activity area by closing the west-bound lane from the driveway closest to Cherahala Boulevard to the new ramp for a maximum duration of one week. Although a portion of the road will be closed, the selected closure minimizes the impacts of construction on local transportation by providing motorists with continued access to the Faith Promise Church campus and Cherahala Boulevard. A temporary detour for one-direction travel adapted from Figure 6H-9 of the MUTCD was utilized for Faith Promise Lane (Appendix E). Recommended traffic control devices from Figure 6H-9, including type III barricades, signs, and channelizing drums (Table 18), were also placed along the street. Although the majority of temporary signs did not require specific placements, the locations of the "Detour Ahead" and "Road Closed Ahead" signs were determined based on MUTCD specifications (Table 19). Because only the west-bound segment of Faith Promise Lane will be closed, traffic entering from Pellissippi Parkway will be properly accommodated and avoid the need for a full road closure, which would result in a complicated, lengthy detour for drivers hoping to enter Pellissippi Parkway.

Section	Length (ft)	Spacing (ft)	Quantity
Activity Zone	340	20	17

Table 18: Channelizing drum spacing and required quantity for construction on Faith Promise Lane

Sign	Distance from East Driveway (ft)
Detour Ahead	100
Road Closed Ahead	200

Table 19: Specified distances for temporary signage from east driveway on Faith Promise Lane

6.3 Project Quantities Estimate

The material quantities for the proposed roadway project were compiled in order to determine the equipment and labor needs for the project as well as to accurately estimate the final project cost. Earthwork and pavement quantities were calculated using AutoCAD and applying the average-end method. The materials required for the retaining wall were quantified using the cross-section drawings, the designed reinforcement sizing and spacing, and the average wall height for each section. The material quantities for the remaining items used for the project scope of work were determined through counting on the project drawings and using MicroStation to calculate land areas. All quantities were converted to units compatible with those presented in the TDOT average material bid prices document.

Because the excavation and embankment of earth around the project site requires the use of heavy machinery, roads adjacent to the areas of the project at which earthwork is taking place must be shut down. The pavement laying process requires similar roadway closures, so by accurately quantifying the materials necessary for these major construction tasks, a more precise timeline for project construction could be developed. The quantities of cuts and fills needed for each roadway cross section were determined by examining the existing site conditions and new design infrastructure in AutoCAD to measure the required area of cut or fill, and then using the average-end method to calculate the total quantity cut and total quantity fill for the project. A shrink/swell factor of 20% was incorporated into the earthwork calculations in accordance with TDOT standards. The quantities of pavement materials used for the project were calculated by using MicroStation to calculate the total paved roadway area and multiplying this value by the thickness of the pavement layer. Quantification of these materials determined the site to be a waste site with an excess volume of 18,799 cubic yards of earth (Appendix E).

The quantity of materials required for the construction of the retaining wall were calculated in order to better inform the earth retaining system selection process, which resulted in the selection of a cast-in-place concrete cantilever system. The area of concrete for the wall was calculated by determining the area of the footer, adding the area of the wall stem, and then multiplying this by the length of each wall section (20 feet). Because the wall height changes continuously, the average wall height for each section was used in the calculation. The average wall height for each section was also used in determining the lengths of flexural steel reinforcing bar in the stem as well as the

number of longitudinal bars, as each longitudinal bar has a 16-inch spacing. The uniform footer sizes were used to calculate the lengths of flexural reinforcement and the number of longitudinal bars in the footer. The number of longitudinal bars used in the stem and footer were multiplied by the 20-foot length of each wall section. All lengths of reinforcing bar were multiplied by their standard pound-per-foot weight as described in ACI-318. The quantity of granular and select sand backfill for the wall were calculated using AutoCAD to measure the area behind each wall as described in the TDOT standard drawing for retaining walls and the average-end method (Appendix E).

The quantity of materials for the drainage plan, erosion prevention and sediment control plan, signing and striping plan, and traffic control plan were determined using the project drawings. Count items such as the standard catch basins used in the drainage plan were quantified by simply counting the number of items as presented in the final project drawings. Area quantity for items such as the erosion control blanket was calculated by using a topographic site map in MicroStation and selecting the boundaries of the application areas to create a polygon, the areas of which were summed and tabulated (Appendix E).

6.4 Project Cost Estimate

An estimate of the probable project cost was completed to provide decision makers with the budgetary information necessary to determine the investment required for the proposed infrastructural changes. Unit prices for the project materials were obtained from the TDOT average bid price list and the cost per material was calculated based on the previously calculated quantities. Materials were categorized into major construction tasks allowing for a subtotal cost per task to be determined. The summation of these subtotals allowed for the final project cost estimate to be calculated.

The individual material item unit prices were obtained from the 2019 TDOT average bid price document. The unit prices for Region 1 were selected for the cost estimation as the project site is located in this region. For items with no Region 1 average price available, the State average bid price was used. Per TDOT standards, all labor and transportation costs associated with the use of the material are included within the average bid price for that material. For this reason, the price of the retaining wall was not estimated based on its constituent parts, as costs associated with concrete testing and transportation, formwork fabrication and use, and all associated labor would not be properly represented. The TDOT line item “Retaining Wall” was selected for the cost estimation, which is based on the area of wall face, including that which is beneath the ground surface. The unit prices for all items were multiplied by the quantities of those materials, allowing for the cost per material to be calculated (Appendix E).

The costs per material were categorized into the major construction tasks for which they are planned to be used. This allowed for the subtotals for each major construction task to be calculated. This information can be utilized by decision makers to identify the more costly elements of the roadway project and potentially focus on developing cost-saving alternatives for future iterations of the project design. By summing the subtotal costs for each major construction task, a total estimated project cost was determined, as presented in Table 20.

Major Construction Task	***Subtotal Cost***
Site Preparation	$151,614.56
Erosion Prevention & Sediment Control	$41,720.70
Earthwork	$266,227.20
Traffic Control	$5,398.21
Roadway Development	$507,320.61
Retaining Wall	$529,991.00
Drainage	$113,367.47
Signage & Striping	$1,963.23
TOTAL PROJECT COST	**$1,617,602.63**

Table 20: Cost of major construction tasks and total project cost estimate

7.0 **Conclusions**

The redevelopments for access designed by 2020 Vision Engineering provide the necessary infrastructural support for existing and future traffic volumes. Hardin Valley is the fastest growing area of Knox County, and as it continues to thrive, new traffic challenges will develop while the infrastructure catches up to better support the region's population size. TDOT is actively working to redevelop the major access points of state highways in the area like Pellissippi Parkway into restricted-access interchanges and are actively scarifying median cut-throughs and dangerous at-grade intersections. The work 2020 Vision Engineering has done to plan for redeveloping the access to Pellissippi Parkway at Faith Promise Lane into an entrance ramp and collector road closely aligns with TDOT's goals for a safer experience for drivers in Tennessee and the restored infrastructure helps move Knox County one step closer to achieving the Vision Zero of no traffic fatalities.

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Appendix A: Road Design

Table 3-7. Minimum Radius Using Limiting Values of *e* and *f*

Metric						U.S. Customary					
Design Speed (km/h)	Maximum *e* (%)	Maximum *f*	Total (*e*/100 + *f*)	Calculated Radius (m)	Rounded Radius (m)	Design Speed (mph)	Maximum *e* (%)	Maximum *f*	Total (*e*/100 + *f*)	Calculated Radius (ft)	Rounded Radius (ft)
15	4.0	0.40	0.44	4.0	4	10	4.0	0.38	0.42	15.9	16
20	4.0	0.35	0.39	8.1	8	15	4.0	0.32	0.36	41.7	42
30	4.0	0.28	0.32	22.1	22	20	4.0	0.27	0.31	86.0	86
40	4.0	0.23	0.27	46.7	47	25	4.0	0.23	0.27	154.3	154
50	4.0	0.19	0.23	85.6	86	30	4.0	0.20	0.24	250.0	250
60	4.0	0.17	0.21	135.0	135	35	4.0	0.18	0.22	371.2	371
70	4.0	0.15	0.19	203.1	203	40	4.0	0.16	0.20	533.3	533
80	4.0	0.14	0.18	280.0	280	45	4.0	0.15	0.19	710.5	711
90	4.0	0.13	0.17	375.2	375	50	4.0	0.14	0.18	925.9	926
100	4.0	0.12	0.16	492.1	492	55	4.0	0.13	0.17	1186.3	1190
						60	4.0	0.12	0.16	1500.0	1500
15	6.0	0.40	0.46	3.9	4	10	6.0	0.38	0.44	15.2	15
20	6.0	0.35	0.41	7.7	8	15	6.0	0.32	0.38	39.5	39
30	6.0	0.28	0.34	20.8	21	20	6.0	0.27	0.33	80.8	81
40	6.0	0.23	0.29	43.4	43	25	6.0	0.23	0.29	143.7	144
50	6.0	0.19	0.25	78.7	79	30	6.0	0.20	0.26	230.8	231
60	6.0	0.17	0.23	123.2	123	35	6.0	0.18	0.24	340.3	340
70	6.0	0.15	0.21	183.7	184	40	6.0	0.16	0.22	484.8	485
80	6.0	0.14	0.20	252.0	252	45	6.0	0.15	0.21	642.9	643
90	6.0	0.13	0.19	335.7	336	50	6.0	0.14	0.20	833.3	833
100	6.0	0.12	0.18	437.4	437	55	6.0	0.13	0.19	1061.4	1060
110	6.0	0.11	0.17	560.4	560	60	6.0	0.12	0.18	1333.3	1330
120	6.0	0.09	0.15	755.9	756	65	6.0	0.11	0.17	1656.9	1660
130	6.0	0.08	0.14	950.5	951	70	6.0	0.10	0.16	2041.7	2040
						75	6.0	0.09	0.15	2500.0	2500
						80	6.0	0.08	0.14	3047.6	3050
15	8.0	0.40	0.48	3.7	4	10	8.0	0.38	0.46	14.5	14
20	8.0	0.35	0.43	7.3	7	15	8.0	0.32	0.40	37.5	38
30	8.0	0.28	0.36	19.7	20	20	8.0	0.27	0.35	76.2	76
40	8.0	0.23	0.31	40.6	41	25	8.0	0.23	0.31	134.4	134
50	8.0	0.19	0.27	72.9	73	30	8.0	0.20	0.28	214.3	214
60	8.0	0.17	0.25	113.4	113	35	8.0	0.18	0.26	314.1	314
70	8.0	0.15	0.23	167.8	168	40	8.0	0.16	0.24	444.4	444
80	8.0	0.14	0.22	229.1	229	45	8.0	0.15	0.23	587.0	587
90	8.0	0.13	0.21	303.7	304	50	8.0	0.14	0.22	757.6	758
100	8.0	0.12	0.20	393.7	394	55	8.0	0.13	0.21	960.3	960
110	8.0	0.11	0.19	501.5	501	60	8.0	0.12	0.20	1200.0	1200
120	8.0	0.09	0.17	667.0	667	65	8.0	0.11	0.19	1482.5	1480
130	8.0	0.08	0.16	831.7	832	70	8.0	0.10	0.18	1814.8	1810
						75	8.0	0.09	0.17	2205.9	2210
						80	8.0	0.08	0.16	2666.7	2670
15	10.0	0.40	0.50	3.5	4	10	10.0	0.38	0.48	13.9	14
20	10.0	0.35	0.45	7.0	7	15	10.0	0.32	0.42	35.7	36
30	10.0	0.28	0.38	18.6	19	20	10.0	0.27	0.37	72.1	72
40	10.0	0.23	0.33	38.2	38	25	10.0	0.23	0.33	126.3	126
50	10.0	0.19	0.29	67.9	68	30	10.0	0.20	0.30	200.0	200
60	10.0	0.17	0.27	105.0	105	35	10.0	0.18	0.28	291.7	292
70	10.0	0.15	0.25	154.3	154	40	10.0	0.16	0.26	410.3	410
80	10.0	0.14	0.24	210.0	210	45	10.0	0.15	0.25	540.0	540
90	10.0	0.13	0.23	277.3	277	50	10.0	0.14	0.24	694.4	694
100	10.0	0.12	0.22	357.9	358	55	10.0	0.13	0.23	876.8	877
110	10.0	0.11	0.21	453.7	454	60	10.0	0.12	0.22	1090.9	1090
120	10.0	0.09	0.19	596.8	597	65	10.0	0.11	0.21	1341.3	1340
130	10.0	0.08	0.18	739.3	739	70	10.0	0.10	0.20	1633.3	1630
						75	10.0	0.09	0.19	1973.7	1970
						80	10.0	0.08	0.18	2370.4	2370
15	12.0	0.40	0.52	3.4	3	10	12.0	0.38	0.50	13.3	13
20	12.0	0.35	0.47	6.7	7	15	12.0	0.32	0.44	34.1	34
30	12.0	0.28	0.40	17.7	18	20	12.0	0.27	0.39	68.4	68
40	12.0	0.23	0.35	36.0	36	25	12.0	0.23	0.35	119.0	119
50	12.0	0.19	0.31	63.5	64	30	12.0	0.20	0.32	187.5	188
60	12.0	0.17	0.29	97.7	98	35	12.0	0.18	0.30	272.2	272
70	12.0	0.15	0.27	142.9	143	40	12.0	0.16	0.28	381.0	381
80	12.0	0.14	0.26	193.8	194	45	12.0	0.15	0.27	500.0	500
90	12.0	0.13	0.25	255.1	255	50	12.0	0.14	0.26	641.0	641
100	12.0	0.12	0.24	328.1	328	55	12.0	0.13	0.25	806.7	807
110	12.0	0.11	0.23	414.2	414	60	12.0	0.12	0.24	1000.0	1000
120	12.0	0.09	0.21	539.9	540	65	12.0	0.11	0.23	1224.6	1220
130	12.0	0.08	0.20	665.4	665	70	12.0	0.10	0.22	1484.8	1480
						75	12.0	0.09	0.21	1785.7	1790
						80	12.0	0.08	0.20	2133.3	2130

Note: In recognition of safety considerations, use of e_{max} = 4.0% should be limited to urban conditions.

U.S. Customary										
Highway design speed (mph)	30	35	40	45	50	55	60	65	70	75
Ramp design speed (mph)										
Upper range (85%)	25	30	35	40	45	48	50	55	60	65
Middle range (70%)	20	25	30	33	35	40	45	45	50	55
Lower range (50%)	15	18	20	23	25	28	30	30	35	40
Corresponding minimum radius (ft)	see Table 3-7									

U.S. Customary

$$HSO = R\left[1 - \cos\left(\frac{28.65S}{R}\right)\right] \quad \text{(3-36)}$$

where:

HSO = Horizontal sight line offset, ft

S = Stopping sight distance, ft

R = Radius of curve, ft

U.S. Customary

When S is less than L,

$$L = \frac{AS^2}{100\left(\sqrt{2h_1} + \sqrt{2h_2}\right)^2} \quad (3\text{-}41)$$

When S is greater than L,

$$L = 2S - \frac{200\left(\sqrt{h_1} + \sqrt{h_2}\right)^2}{A} \quad (3\text{-}42)$$

where:

L = length of vertical curve, ft

A = algebraic difference in grades, percent

S = sight distance, ft

h_1 = height of eye above roadway surface, ft

h_2 = height of object above roadway surface, ft

U.S. Customary

When S is less than L,

$$L = \frac{AS^2}{2158} \tag{3-43}$$

When S is greater than L,

$$L = 2S - \frac{2158}{A} \tag{3-44}$$

U.S. Customary

When S is less than L,

$$L = \frac{AS^2}{200\left[2.0 + S\left(\tan 1^\circ\right)\right]} \quad \text{(3-47)}$$

or,

$$L = \frac{AS^2}{400 + 3.5S} \quad \text{(3-48)}$$

When S is greater than L,

$$L = 2S - \frac{200\left[2.0 + S\left(\tan 1^\circ\right)\right]}{A} \quad \text{(3-49)}$$

or,

$$L = 2S - \frac{400 + 3.5S}{A} \quad \text{(3-50)}$$

where:

L = length of sag vertical curve, ft

A = algebraic difference in grades, percent

S = light beam distance, ft

U.S. Customary

$$L = \frac{AV^2}{46.5} \qquad (3\text{-}51)$$

where:

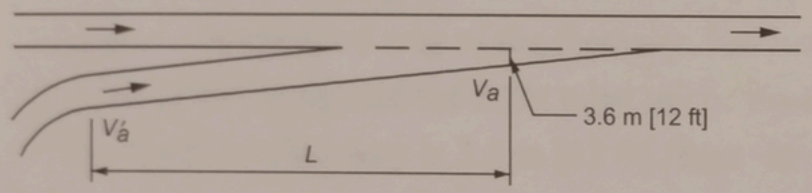
L = length of sag vertical curve, ft

A = algebraic difference in grades, percent

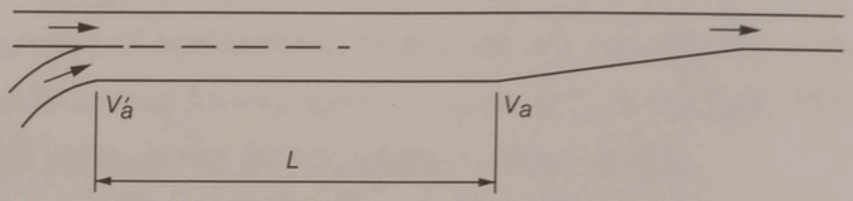
V = design speed, mph

U.S. Customary										
Acceleration Length, L (ft) for Entrance Curve Design Speed (mph)										
Highway		Stop Condition	15	20	25	30	35	40	45	50
Design Speed, V (mph)	Speed Reached, V_a (mph)	and Initial Speed, V'_a (mph)								
		0	14	18	22	26	30	36	40	44
30	23	180	140	—	—	—	—	—	—	—
35	27	280	220	160	—	—	—	—	—	—
40	31	360	300	270	210	120	—	—	—	—
45	35	560	490	440	380	280	160	—	—	—
50	39	720	660	610	550	450	350	130	—	—
55	43	960	900	810	780	670	550	320	150	—
60	47	1200	1140	1100	1020	910	800	550	420	180
65	50	1410	1350	1310	1220	1120	1000	770	600	370
70	53	1620	1560	1520	1420	1350	1230	1000	820	580
75	55	1790	1730	1630	1580	1510	1420	1160	1040	780

Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1,300 ft.



Taper Type



Parallel Type

Table 10-4. Speed Change Lane Adjustment Factors as a Function of Grade (Continued)

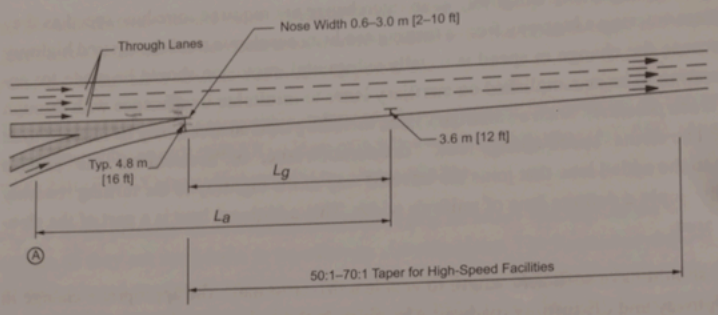
U.S. Customary					
Design Speed of Highway (mph)	Deceleration Lanes				
	Ratio of Length on Grade to Length on Level for Design Speed of Turning Curve (mph)[a]				
All Speeds	3 to 4% upgrade 0.9			3 to 4% downgrade 1.2	
All Speeds	5 to 6% upgrade 0.8			5 to 6% downgrade 1.35	
Design Speed of Highway (mph)	Acceleration Lanes				
	Ratio of Length on Grade to Length on Level for Design Speed of Turning Curve (mph)[a]				
	20	30	40	50	All Speeds
3 to 4% Upgrade					3 to 4% Downgrade
40	1.3	1.3	—	—	0.7
45	1.3	1.35	—	—	0.675
50	1.3	1.4	1.4	—	0.65
55	1.35	1.45	1.45	—	0.625
60	1.4	1.5	1.5	1.6	0.6
65	1.45	1.55	1.6	1.7	0.6
70	1.5	1.6	1.7	1.8	0.6
5 to 6% Upgrade					5 to 6% Downgrade
40	1.5	1.5	—	—	0.6
45	1.5	1.6	—	—	0.575
50	1.5	1.7	1.9	—	0.55
55	1.6	1.8	2.05	—	0.525
60	1.7	1.9	2.2	2.5	0.5
65	1.85	2.05	2.4	2.75	0.5
70	2.0	2.2	2.6	3.0	0.5

[a] Ratio from this table multiplied by the length in Table 10-3 or Table 10-5 gives length of speed change lane on grade.

Single-Lane Free-Flow Terminals, Exits

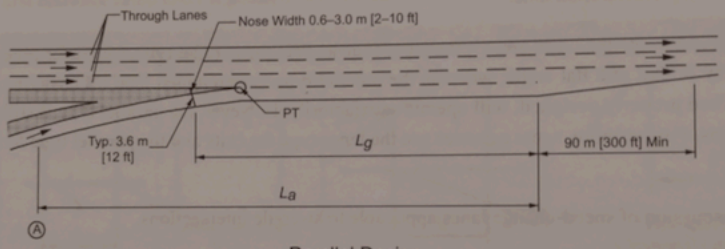
Taper-type exits—The taper-type exit fits the direct path preferred by most drivers, per follow an easy path within the diverging area. The taper-type exit terminal beginning wit alignment break usually provides a clear indication of the point of departure from the th has generally been found to operate smoothly on high-volume freeways. The divergence between 2 and 5 degrees.

Studies of this type of terminal show that most vehicles leave the through lane at relativ thereby reducing the potential for rear-end collisions as a result of deceleration on the th speed change can be achieved off the traveled way as the exiting vehicle moves along th ramp proper. Figure 10-70A shows a typical design for a taper-type exit.



Tapered Design

– A –



Parallel Design

– B –

Notes:

1. L_a is the required acceleration length as shown in Table 10-3 or as adjusted by Table 10-4.
2. Point A controls speed on the ramp. L_a should not start back on the curvature of the ramp unless the radius equals 300 m [1,000 ft] or more.
3. L_g is the required gap acceptance length. L_g should be a minimum of 150 m [300 ft to 500 ft] depending on the nose width.
4. The value of L_a or L_g, whichever produces the greater distance downstream from where the nose equals 0.6 m [2 ft], is suggested for use in the design of the ramp distance.

Figure 10-69. Typical Single-Lane Entrance Ramps

Length of Ramp | Senior Design | Logan Naffah | 10/2/19

Curve Design Speed = 30 mph

Highway Design Speed = 55 mph = V

Speed Reached = V_a = 43 mph

By Table 10-3 (Pg. 10-110) → Acceleration Length = L = 670 ft

Elevation At start of curve (estimate) = 994 ft

Elevation at End of curve (estimate) = 906 ft

Elevation change (estimate) = 88 ft

Length of curve based on ramp radius = 1633.708 ft

Downgrade = $100\% \cdot \frac{88\ ft}{1633.708\ ft} \approx 5\%$

Per Table 10-4, Length adjustment factor = 0.525

Therefore → Acceleration Length = 0.525 · 670 ft = 351.75 ft

Based on projected drawing based on ramp radius → Acceleration length = 1143 ft > 351.75 ft ✓

Per Figure 10-69, L_g = 500 ft = gap acceptance length

Nose width = 10 ft

Taper = 40:1

* see Revisions from 10/23 *

Horizontal Alignment | Senior Design | Logan Naifeh 10/23/19

Curve	Radius
1	550 ft
2	250 ft

→ From Microstation

* curve 1 is curve closest to connector road, curve 2 is curve closest to ramp *
3-31 Horizontal curve is measured from centerline

For curve 1 use EQ 3-9 and Table 3-7

$V = 30$ mph

$R_{min} = 500$ ft

e and f ?

Assume $e = 2\%$

$2\% \cdot 0.01 + f = \frac{30^2}{15 \cdot 500}$

$f = 0.1$
$e = 2\%$
$V = 30$ mph
$R_{min} = 500$ ft
Curve 1

For curve 2 use EQ 3-9 and Table 3-7 and Table 10-1

From Table 10-1 *assuming Highway Design speed refers to Mississippi*, use a ramp speed between 28-48 mph

use $V = 30$ mph

$V = 30$ mph

$R_{min} = 250$ ft

From Table 3-7 → $e = 4\%$ $f = 0.2$

$(0.01 \cdot 4\%) + 0.2 = \frac{30^2}{15 \cdot R_{min}}$

$R_{min} = 250$ ft

* 3-57, turning Roadways' radii are measured from edge of roadway (3.3.7) *

$V = 30$ mph
$R_{min} = 250$ ft ≤ 250 ft
$e = 4\%$
$f = 0.2$
Curve 2

Length of Ramp | Senior Design | Logan Naifeh 10/23/19

Curve Design Speed = 30 mph
Highway Design Speed = 55 mph

Taper: 50:1 min for High Speed (measure from nose to end (merge))

1. Assume smallest nose
   *10-69A → 16 ft wide lane at nose

2. From Pelissippi Edge of pavement to Ramp Edge of pavement:
   2 ft (nose) + 16 ft (ramp lane) = 18 ft

3. 18ft for 50:1 taper = 900 ft taper

4. Check L_g
   L_g is from nose to where EOP - EOP = 12 ft

5. Barring L_a, distance from nose to merge is at minimum:
   12 ft (50:1) = 600 ft (minimum from end L_g to merge)
   + L_g (min 300 ft to 500 ft depending on nose width)

   Minimum nose to merge length = 600 ft + 300 ft = 900 ft

6. Check taper from nose to end:
   18 ft EOP-EOP @ Nose
   − 12 ft @ End L_g
   6 ft

7. 6 ft (50:1) = 300 ft (within 300-500 ft range)

8. Calculate L_a: Table 10-3 (Pg. 10-110) → 30 ↔ 55 mph = 670 ft
   L_a = 670 ft

Length of Ramp | Senior Design | Logan Naifeh 10/23/19

9. Assume L_g is controlled by PT of curve:

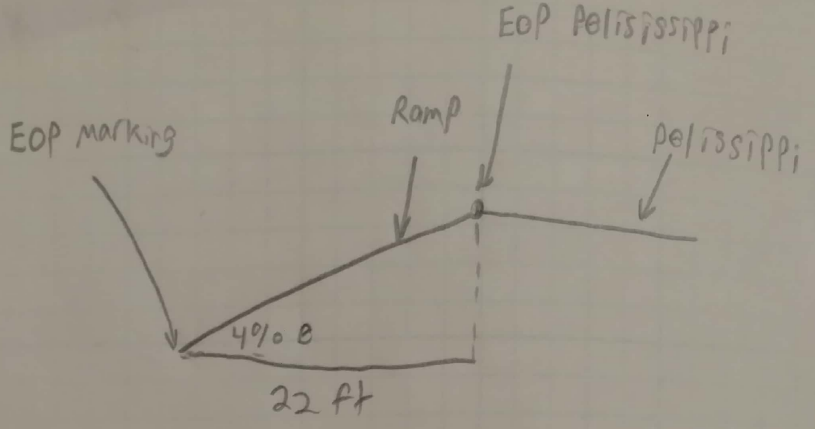
Distance from PT to End L_g = 670 ft

Distance from End L_g → End merge = 600 ft

670 ft + 600 ft = 1270 ft

Based on the 50:1 taper, Length from PT to your End Point (merge) = **1270 ft**

Downgrade calculations | Senior Design | Logan Naifeh 10/23/19



EOP Pelissippi Elevation = 963.2 ft

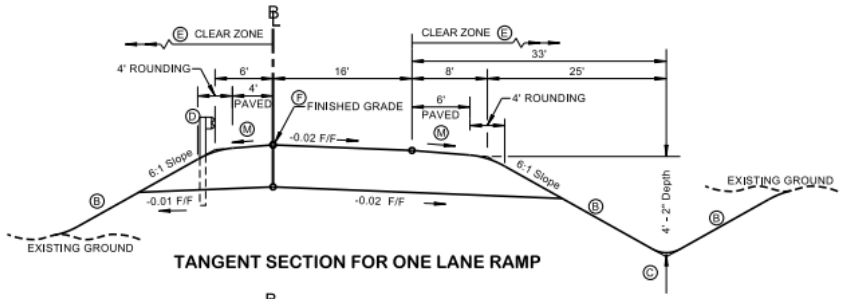
EOP Marking Elevation = 962.32 ft

Beginning of Ramp Elevation = 994.2 ft

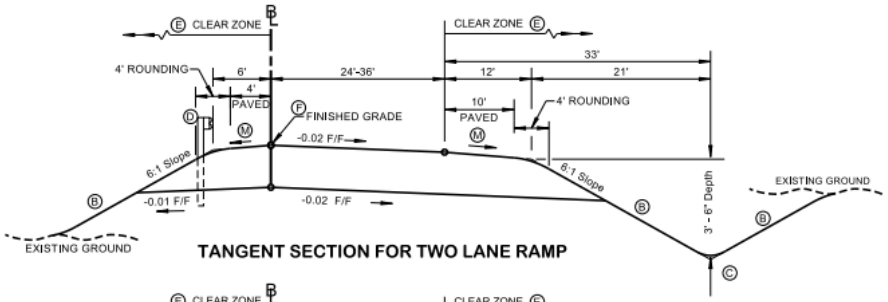
994.2 ft - 962.3 = 31.9 ft

Length along ramp = 419.7632 ft

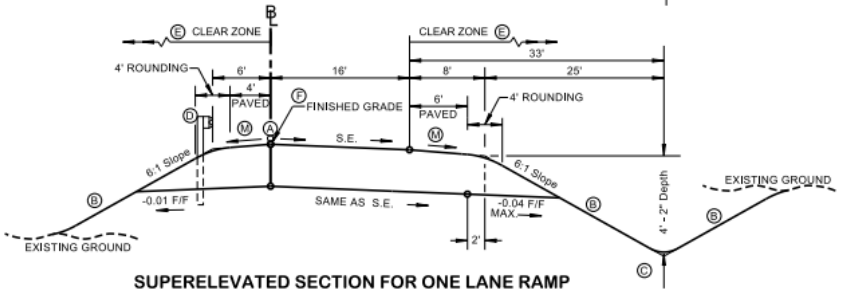
$$\text{Downgrade} = \frac{31.9\text{ ft}}{419.7632\text{ ft}} \cdot 100\% = \boxed{7.6\%}$$



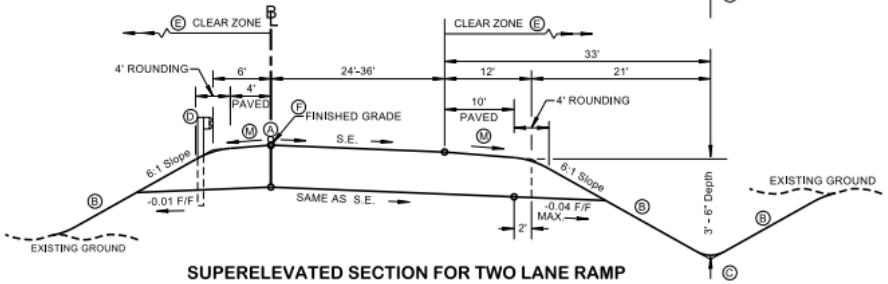
TANGENT SECTION FOR ONE LANE RAMP



TANGENT SECTION FOR TWO LANE RAMP



SUPERELEVATED SECTION FOR ONE LANE RAMP



SUPERELEVATED SECTION FOR TWO LANE RAMP

TABLE I GUIDE VALUES FOR RAMP DESIGN SPEED AS RELATED TO HIGHWAY DESIGN SPEED (SEE PAGE 10-89)

(H) HIGHWAY DESIGN (G) SPEEDS (MPH)	30	35	40	45	50	55	60	65	70	REMARKS
RAMP DESIGN SPEED (MPH) (J)										
UPPER RANGE (85%)	25	30	35	40	45	48	50	55	60	SEE PAGE 10-89
MIDDLE RANGE (70%)	20	25	30	33	35	40	45	45	50	
LOWER RANGE (50%)	15	18	20	23	25	28	30	30	35	

TABLE II STOPPING SIGHT DISTANCE FOR RAMPS (SEE PAGE 3-4) (I)

DESIGN SPEEDS (MPH)	15	20	25	30	35	40	45	50	55	60	65	70
MINIMUM, FEET	80	115	155	200	250	305	360	425	495	570	645	730

TABLE III LENGTHS OF CURVE FOR DIFFERENT COMPOUND CURVE RADII (SEE PAGE 3-58) (K)

RADIUS (FEET)	100	150	200	250	300	400	500 OR MORE
MINIMUM LENGTH OF HORIZONTAL CURVE							
ACCEPTABLE, FEET	40	50	60	80	100	120	140
DESIRABLE, FEET	60	70	90	120	140	180	200

DESIGN NOTES

(A) THE SLOPE OF THE SHOULDER AND THE ROADWAY PAVEMENT SHOULD NOT EXCEED AN ALGEBRAIC DIFFERENCE OF 7%.

(B) SEE STANDARD DRAWING RD11-S-11 FOR FILL AND CUT SLOPE TABLES. ROUNDING ON TOP OF CUT SLOPES AND TOE ROUNDING ON TOP OF CUT SLOPES AND TOE OF FILL SLOPES SPECIAL ROCK TREATMENT AND SUBGRADE ROUNDING IF APPLICABLE.

(C) SEE STANDARD DRAWING RD11-S-11A FOR ROUNDING OF ROADSIDE DITCH SLOPES.

(D) SEE STANDARD DRAWING S-PL-6 FOR TYPICAL GUARDRAIL PLACEMENT.

(E) SEE STANDARD DRAWING S-CZ-1 FOR CLEAR ZONE CRITERIA. SEE THE "ROADSIDE DESIGN GUIDE", AASHTO, 2011, FOR FURTHER INFORMATION REGARDING CLEAR ZONE.

(F) WHEN THE RAMP PAVEMENT IS ADJACENT TO MAINLINE ROADWAY PAVEMENT, THE PROFILE GRADE WILL BE LOCATED ALONG THE MAINLINE EDGE OF PAVEMENT.

(G) DOES NOT PERTAIN TO THE RAMP TERMINALS WHICH SHOULD BE PROPERLY TRANSITIONED AND PROVIDED WITH SPEED CHANGE FACILITIES ADEQUATE FOR THE HIGHWAY SPEED INVOLVED.

(H) THE HIGHER HIGHWAY DESIGN SPEED SHOULD BE THE CONTROL.

(I) THE SIGHT DISTANCE ON A FREEWAY PRECEDING THE APPROACH NOSE OF AN EXIT RAMP SHOULD EXCEED THE MINIMUM FOR THE THROUGH TRAFFIC DESIGN SPEED DESIRABLY BY 25 PERCENT OR MORE.

(J) SEMI-DIRECT CONNECT AND DIRECT CONNECT AND 2-LANE RAMPS, USE MIDDLE RANGE AS MINIMUM. UPPER RANGE IS PREFERRED. LOWER RANGE MAY BE USED FOR RAMPS NOT CONNECTING TO FREEWAYS OR EXPRESSWAYS. LOOP DESIGN SPEED SHALL NOT BE LESS THAN 25 MILES PER HOUR.

(K) FOR RAMPS WITH COMPOUND CURVES, THE PREFERRED RATIO OF THE FLATTER RADIUS TO THE SHARPER RADIUS IS NOT TO EXCEED 1.75:1; HOWEVER, A 2:1 MINIMUM RATIO MAY BE USED (SEE PAGE 3-58).

(L) MAY BE 2% GREATER IN SPECIAL CASES OR WHERE TOPOGRAPHY LIMITS CONDITIONS. DOWN GRADES SHOULD DESIRABLY BE LIMITED TO 3 OR 4 PERCENT ON RAMPS WITH SHARP HORIZONTAL CURVATURE AND SIGNIFICANT HEAVY TRUCK OR BUS TRAFFIC.

(M) FOR CONCRETE RAMPS USE CONSTANT CROSS SLOPE FOR LANES AND SHOULDERS, FOR ASPHALT RAMPS USE .04 F/F FOR TANGENT SHOULDERS AND A CROSS SLOPE FOR SHOULDERS ON THE HIGH SIDE OF SUPERELEVATION SUCH THAT THE ALGEBRAIC DIFFERENCE BETWEEN CROSS SLOPE DOES NOT EXCEED 7%, THE CROSS SLOPE ON THE LOW SIDE SHALL BE THE MINIMUM OF 4% OR THE SUPERELEVATION RATE.

TABLE IV
RECOMMENDED MAXIMUM DESIGN GRADES ON RAMPS (SEE PAGE 10-93)

DESIGN SPEED (MPH)	UPGRADE OR DOWNGRADE (L)
15-25	6-8%
25-30	5-7%
40	4-6%
45 OR GREATER	3-5%

GENERAL NOTES

1. PAGE NUMBERS REFERRED TO ON THIS DRAWING, UNLESS OTHERWISE NOTED AND FOR SPECIFIC CONDITIONS NOT COVERED ON THIS DRAWING , REFEREE TO "A POLICY OF GEOMETRIC DESIGN OF HIGHWAYS AND STREETS" AASHTO, 2011 (GREEN BOOK).
2. REFERENCE SHOULD ALSO BE MADE TO THE AASHTO "ROADSIDE DESIGN GUIDE," AASHTO, 2011.
3. SECTIONS DRAWN LOOKING IN DIRECTION OF TRAVEL.

STATE OF TENNESSEE
STANDARD DRAWING
DEPARTMENT OF TRANSPORTATION

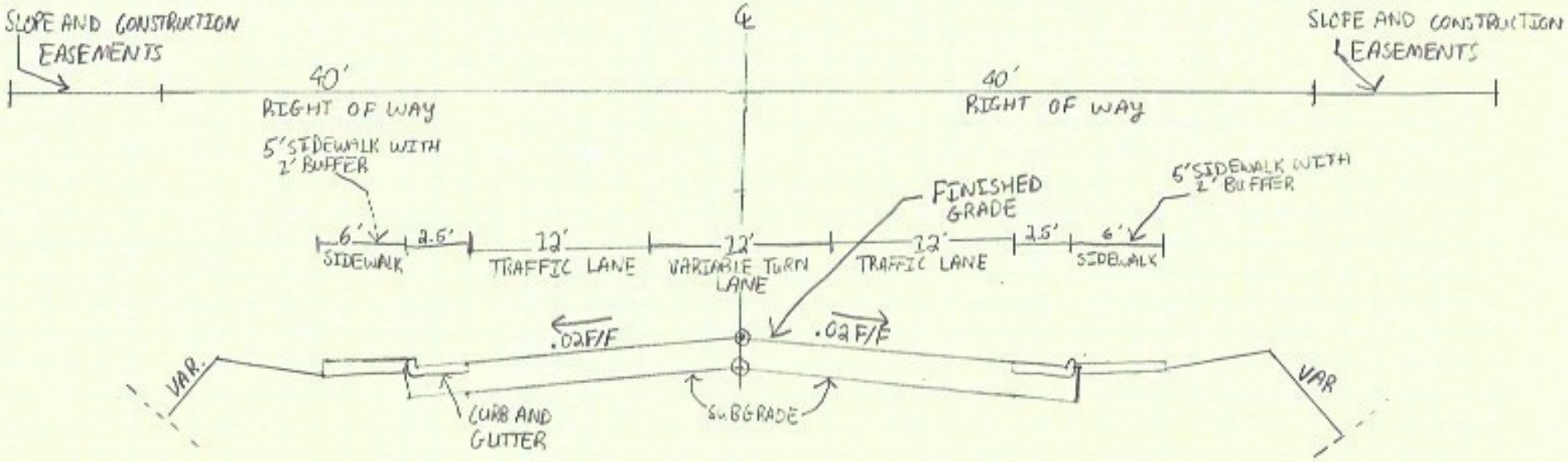
DESIGN STANDARDS FOR ARTERIAL AND FREEWAY RAMPS (1, 2 AND 3 LANE)

10-01-2019 RD11-TS-4

NOT TO SCALE

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LLP

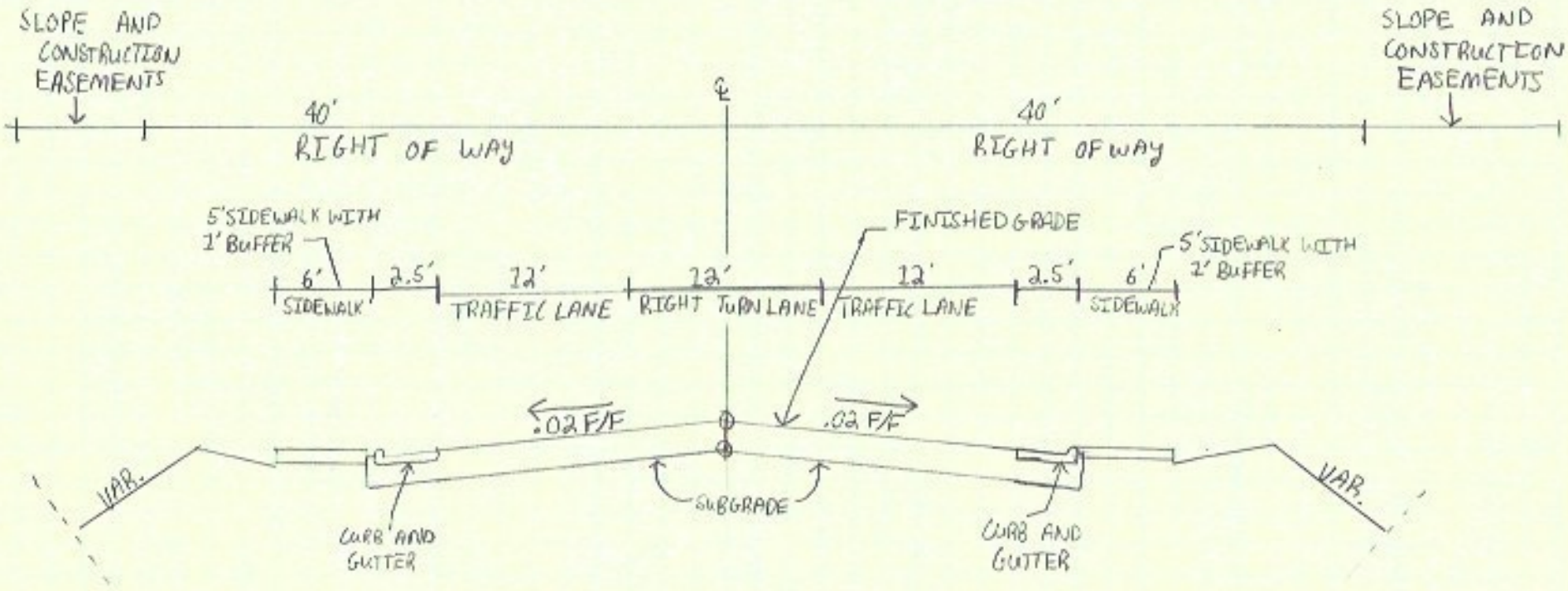
TYPICAL SECTIONS | FAITH PROMISE LANE | 2020 VISION ENG.



TANGENT SECTION

FAITH PROMISE LANE

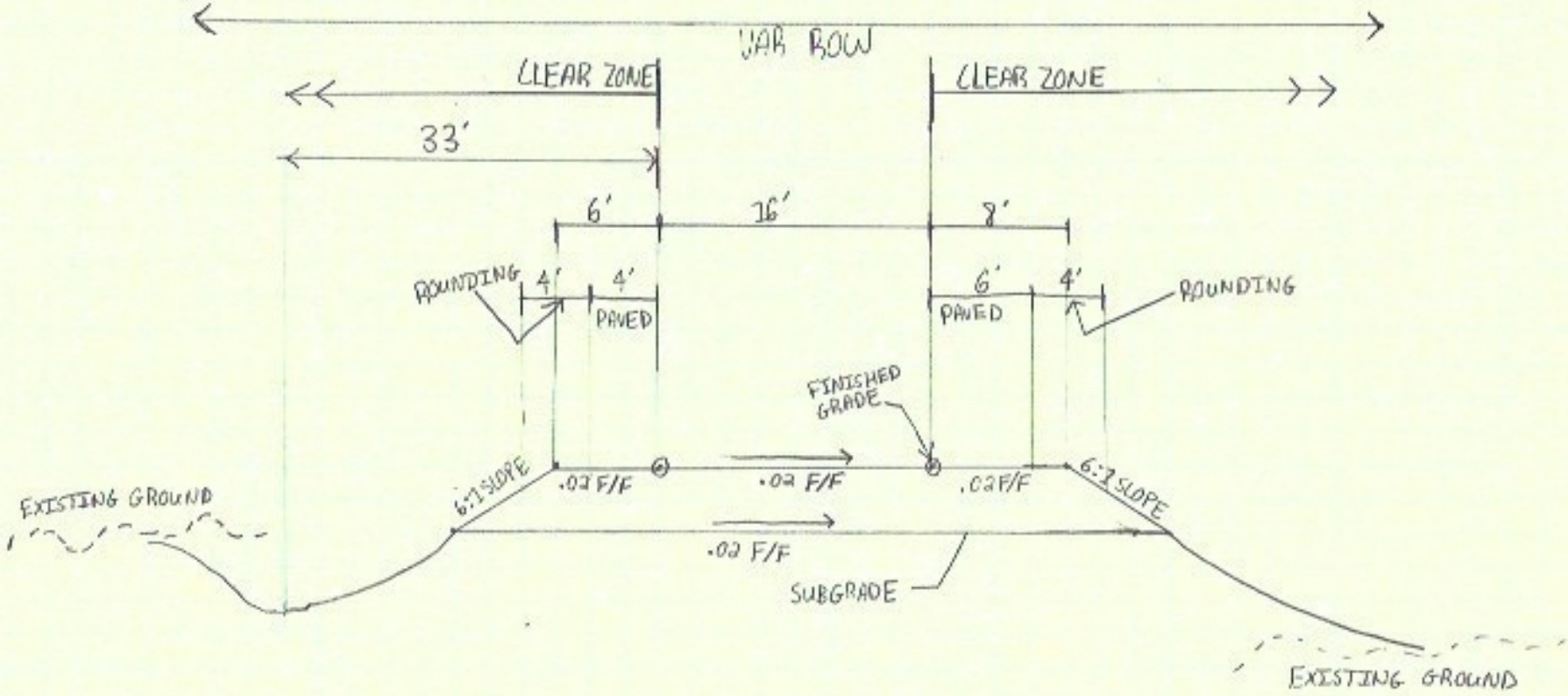
TYPICAL SECTIONS | FAITH PROMISE LANE | 2020 VISION ENG.



TANGENT SECTION

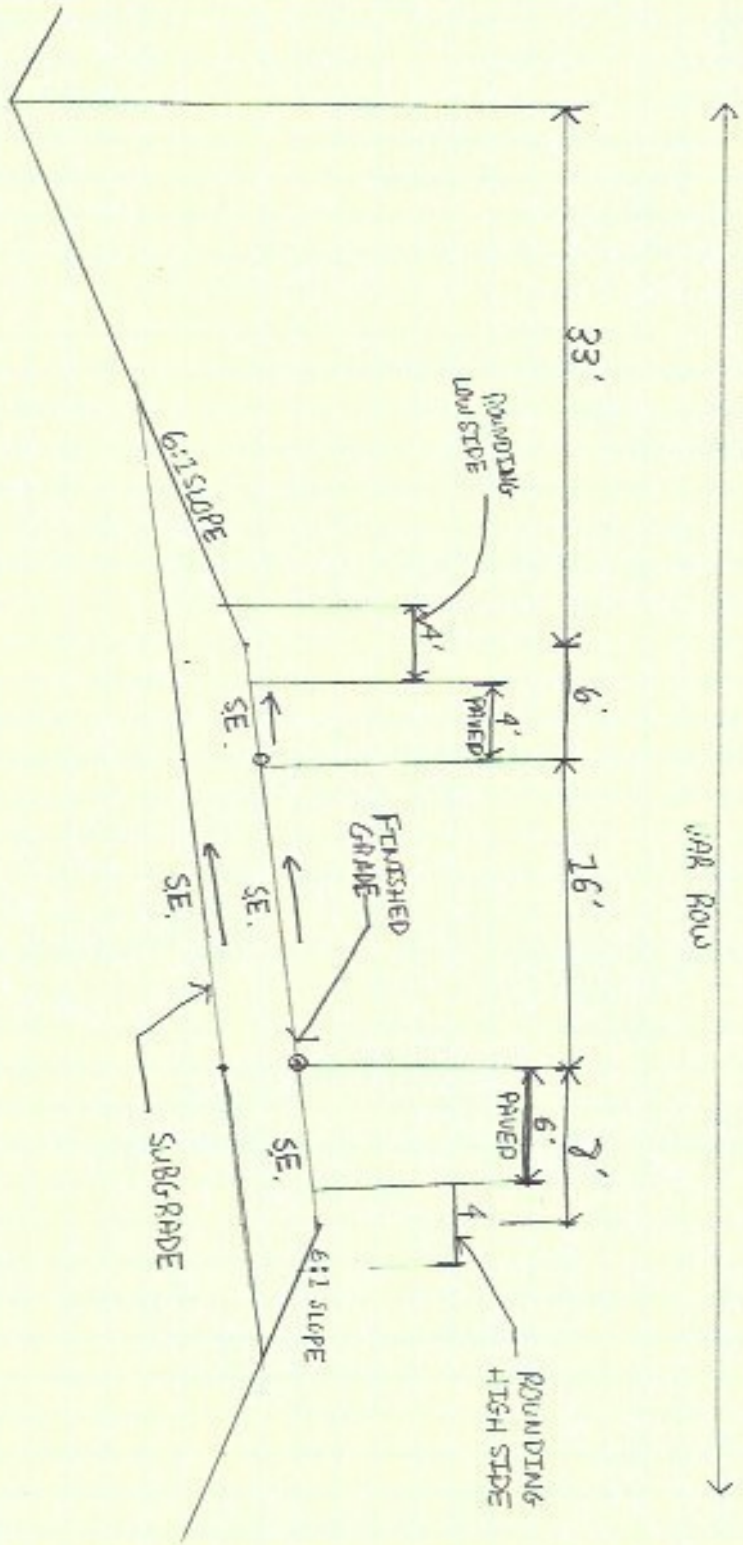
FAITH PROMISE LANE

TYPICAL SECTIONS · FAITH PROMISE LANE TO PELLISSIPPI RAMP | 2020 VISION ENG.



TANGENT SECTION

TYPICAL SECTIONS | FAITH PROMISE LANE TO PELISSIPPI RAMP | 2020 VISION ENG.



SUPERELEVATED SECTION

U.S. Customary

$$0.01e + f = \frac{V^2}{15R} \qquad (3\text{-}9)$$

where:

$V = V_D$ = design speed, mph

$e = e_{max}$ = maximum superelevation, percent

$f = f_{max}$ = maximum allowable side friction factor

$R = R_{min}$ = minimum radius, ft

Appendix B: Retaining Wall Design

Soil Map—Knox County, Tennessee



Map Scale: 1:3,630 if printed on A landscape (11" x 8.5") sheet.

N

Meters: 0 50 100 200 300

Feet: 0 150 300 600 900

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 16N WGS84

MAP LEGEND

Area of Interest (AOI)

- Area of Interest (AOI)

Soils

- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features

- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features

- Streams and Canals

Transportation

- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background

- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:12,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Knox County, Tennessee
Survey Area Data: Version 14, Sep 16, 2018

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Mar 8, 2016—Oct 26, 2017

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
EvB	Etowah-Minvale complex, 2 to 5 percent slopes	13.1	20.5%
FvC	Fullerton-Minvale complex, 5 to 12 percent slopes	22.6	35.3%
MfD	Minvale-Fullerton complex, 12 to 25 percent slopes, stony	10.4	16.3%
MfE	Minvale-Bodine-Fullerton complex, 25 to 50 percent slopes, stony	4.4	6.9%
Ro	Rockdell gravelly loam, 0 to 4 percent slopes, rarely flooded	2.5	3.9%
Uu	Urban land-Udorthents complex	10.9	17.1%
Totals for Area of Interest		**63.9**	**100.0%**

Engineering Properties

This table gives the engineering classifications and the range of engineering properties for the layers of each soil in the survey area.

Hydrologic soil group is a group of soils having similar runoff potential under similar storm and cover conditions. The criteria for determining Hydrologic soil group is found in the National Engineering Handbook, Chapter 7 issued May 2007(http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17757.wba). Listing HSGs by soil map unit component and not by soil series is a new concept for the engineers. Past engineering references contained lists of HSGs by soil series. Soil series are continually being defined and redefined, and the list of soil series names changes so frequently as to make the task of maintaining a single national list virtually impossible. Therefore, the criteria is now used to calculate the HSG using the component soil properties and no such national series lists will be maintained. All such references are obsolete and their use should be discontinued. Soil properties that influence runoff potential are those that influence the minimum rate of infiltration for a bare soil after prolonged wetting and when not frozen. These properties are depth to a seasonal high water table, saturated hydraulic conductivity after prolonged wetting, and depth to a layer with a very slow water transmission rate. Changes in soil properties caused by land management or climate changes also cause the hydrologic soil group to change. The influence of ground cover is treated independently. There are four hydrologic soil groups, A, B, C, and D, and three dual groups, A/D, B/D, and C/D. In the dual groups, the first letter is for drained areas and the second letter is for undrained areas.

The four hydrologic soil groups are described in the following paragraphs:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

Depth to the upper and lower boundaries of each layer is indicated.

Texture is given in the standard terms used by the U.S. Department of Agriculture. These terms are defined according to percentages of sand, silt, and clay in the fraction of the soil that is less than 2 millimeters in diameter. "Loam," for example, is soil that is 7 to 27 percent clay, 28 to 50 percent silt, and less than 52 percent sand. If the content of particles coarser than sand is 15 percent or more, an appropriate modifier is added, for example, "gravelly."

Classification of the soils is determined according to the Unified soil classification system (ASTM, 2005) and the system adopted by the American Association of State Highway and Transportation Officials (AASHTO, 2004).

The Unified system classifies soils according to properties that affect their use as construction material. Soils are classified according to particle-size distribution of the fraction less than 3 inches in diameter and according to plasticity index, liquid limit, and organic matter content. Sandy and gravelly soils are identified as GW, GP, GM, GC, SW, SP, SM, and SC; silty and clayey soils as ML, CL, OL, MH, CH, and OH; and highly organic soils as PT. Soils exhibiting engineering properties of two groups can have a dual classification, for example, CL-ML.

The AASHTO system classifies soils according to those properties that affect roadway construction and maintenance. In this system, the fraction of a mineral soil that is less than 3 inches in diameter is classified in one of seven groups from A-1 through A-7 on the basis of particle-size distribution, liquid limit, and plasticity index. Soils in group A-1 are coarse grained and low in content of fines (silt and clay). At the other extreme, soils in group A-7 are fine grained. Highly organic soils are classified in group A-8 on the basis of visual inspection.

If laboratory data are available, the A-1, A-2, and A-7 groups are further classified as A-1-a, A-1-b, A-2-4, A-2-5, A-2-6, A-2-7, A-7-5, or A-7-6. As an additional refinement, the suitability of a soil as subgrade material can be indicated by a group index number. Group index numbers range from 0 for the best subgrade material to 20 or higher for the poorest.

Percentage of rock fragments larger than 10 inches in diameter and 3 to 10 inches in diameter are indicated as a percentage of the total soil on a dry-weight basis. The percentages are estimates determined mainly by converting volume percentage in the field to weight percentage. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Percentage (of soil particles) passing designated sieves is the percentage of the soil fraction less than 3 inches in diameter based on an ovendry weight. The sieves, numbers 4, 10, 40, and 200 (USA Standard Series), have openings of 4.76, 2.00, 0.420, and 0.074 millimeters, respectively. Estimates are based on laboratory tests of soils sampled in the survey area and in nearby areas and on estimates made in the field. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Liquid limit and *plasticity index* (Atterberg limits) indicate the plasticity characteristics of a soil. The estimates are based on test data from the survey area or from nearby areas and on field examination. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

References:

American Association of State Highway and Transportation Officials (AASHTO). 2004. Standard specifications for transportation materials and methods of sampling and testing. 24th edition.

American Society for Testing and Materials (ASTM). 2005. Standard classification of soils for engineering purposes. ASTM Standard D2487-00.

Report—Engineering Properties

Absence of an entry indicates that the data were not estimated. The asterisk '*' denotes the representative texture; other possible textures follow the dash. The criteria for determining the hydrologic soil group for individual soil components is found in the National Engineering Handbook, Chapter 7 issued May 2007(http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17757.wba). Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Engineering Properties–Knox County, Tennessee														
Map unit symbol and soil name	Pct. of map unit	Hydrologic group	Depth	USDA texture	Classification		Pct Fragments		Percentage passing sieve number—				Liquid limit	Plasticity index
					Unified	AASHTO	>10 inches	3-10 inches	4	10	40	200		
			In				*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*
EvB—Etowah-Minvale complex, 2 to 5 percent slopes														
Etowah	55	B	0-10	Loam, silt loam	CL, CL-ML, ML, SC-SM, SC, SM	A-4	0- 0- 0	0- 0- 0	80-90-100	75-88-100	70-83-95	45-58-70	20-25-30	3-7 -10
			10-64	Silty clay loam, clay loam, clay	CL, MH, ML, CH	A-6, A-7	0- 0- 0	0- 0- 0	80-90-100	75-88-100	70-83-95	65-75-85	39-50-60	15-20-25
Minvale	30	B	0-9	Gravelly silty clay loam, gravelly silt loam, loam	CL, CL-ML, GC, GC-GM	A-4, A-6	0- 0- 0	0- 0- 0	80-90-100	75-88-100	40-55-70	36-51-65	20-30-40	5-10-15
			9-60	Gravelly clay loam, very gravelly silty clay loam, gravelly clay	CL, GC, ML, SC	A-4, A-6, A-7	0- 0- 0	0- 3- 5	55-68-80	50-63-75	40-55-70	36-51-65	25-38-50	7-15-23

Engineering Properties–Knox County, Tennessee														
Map unit symbol and soil name	Pct. of map unit	Hydrologic group	Depth	USDA texture	Classification		Pct Fragments		Percentage passing sieve number—				Liquid limit	Plasticity index
					Unified	AASHTO	>10 inches	3-10 inches	4	10	40	200		
			In				*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*
FvC—Fullerton-Minvale complex, 5 to 12 percent slopes														
Fullerton	53	B	0-20	Gravelly loam, gravelly silt loam, fine sandy loam	CL, CL-ML, GC, GC-GM, SM, SC, SC-SM	A-2, A-4	0- 0- 0	2- 9- 15	60-77-94	45-63-80	40-58-75	30-50-70	18-24-30	3-7 -10
			20-60	Gravelly clay, gravelly silty clay	GM, MH, ML, SM	A-2, A-7	0- 0- 0	2-10- 18	60-75-90	45-63-80	40-58-75	30-53-75	48-63-78	20-31-42
Minvale	38	B	0-9	Gravelly silty clay loam, gravelly silt loam, loam	CL, CL-ML, GC, GC-GM	A-4, A-6	0- 0- 0	0- 0- 0	80-90-100	75-88-100	40-55-70	36-51-65	20-30-40	5-10-15
			9-60	Gravelly clay loam, very gravelly silty clay loam, gravelly clay	CL, GC, ML, SC	A-4, A-6, A-7	0- 0- 0	0- 3- 5	55-68-80	50-63-75	40-55-70	36-51-65	25-38-50	7-15-23

Engineering Properties–Knox County, Tennessee														
Map unit symbol and soil name	Pct. of map unit	Hydrologic group	Depth	USDA texture	Classification		Pct Fragments		Percentage passing sieve number—				Liquid limit	Plasticity index
					Unified	AASHTO	>10 inches	3-10 inches	4	10	40	200		
			In				*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*
MfD—Minvale-Fullerton complex, 12 to 25 percent slopes, stony														
Minvale	70	B	0-9	Gravelly silty clay loam, gravelly silt loam, loam	CL, CL-ML, GC, GC-GM	A-4, A-6	0- 0- 0	0- 0- 0	80-90-100	75-88-100	40-55-70	36-51-65	20-30-40	5-10-15
			9-60	Gravelly clay loam, very gravelly silty clay loam, gravelly clay	SC, CL, GC, ML	A-4, A-6, A-7	0- 0- 0	0- 3- 5	55-68-80	50-63-75	40-55-70	36-51-65	25-38-50	7-15-23
Fullerton	23	B	0-20	Fine sandy loam, gravelly silt loam, gravelly loam	CL, CL-ML, GC, GC-GM, SM, SC, SC-SM	A-2, A-4	0- 0- 0	2- 9- 15	60-77-94	45-63-80	40-58-75	30-50-70	18-24-30	3-7 -10
			20-60	Gravelly clay, gravelly silty clay	GM, MH, ML, SM	A-2, A-7	0- 0- 0	2-10- 18	60-75-90	45-63-80	40-58-75	30-53-75	48-63-78	20-31-42

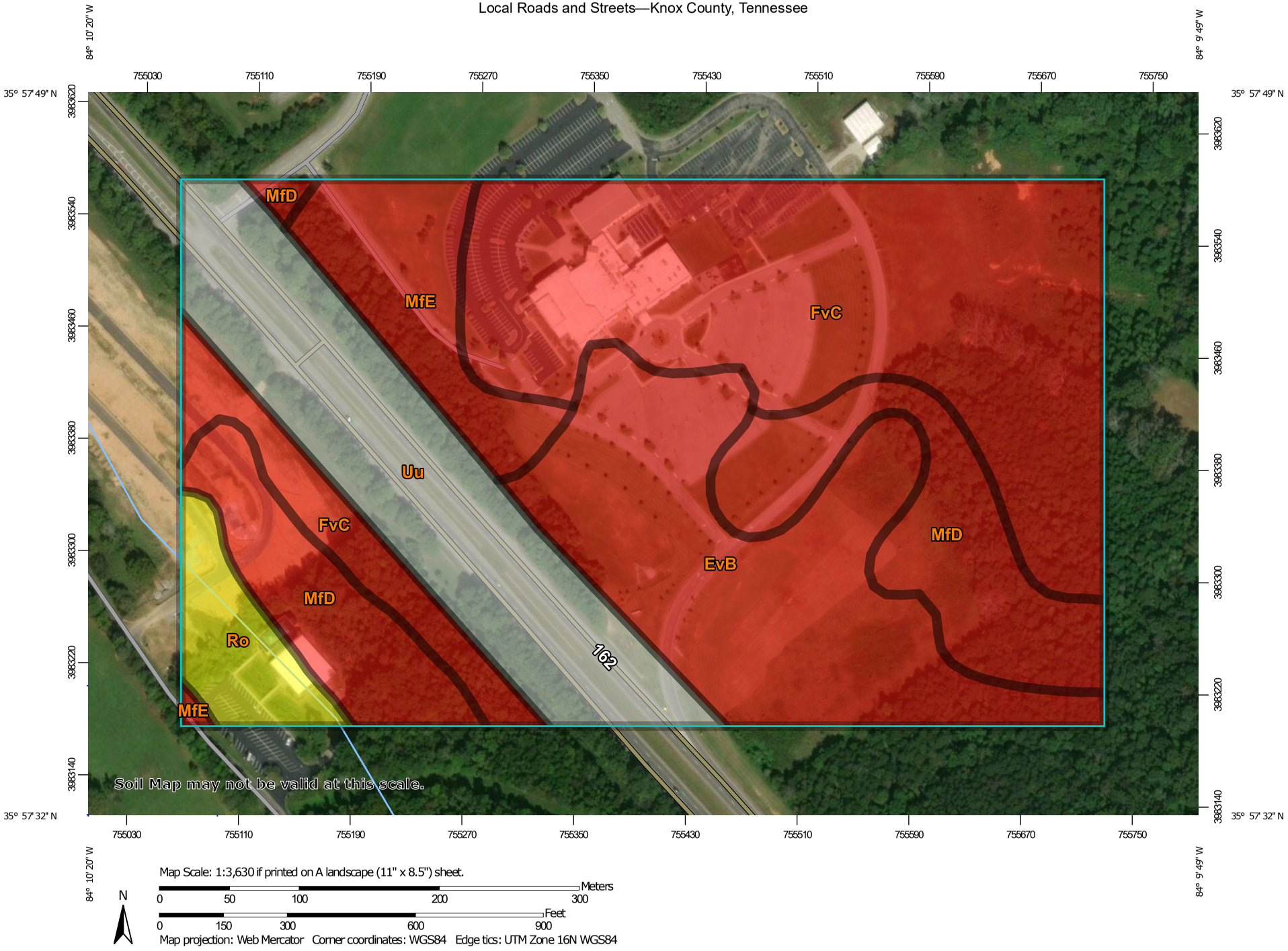
Engineering Properties–Knox County, Tennessee														
Map unit symbol and soil name	Pct. of map unit	Hydrologic group	Depth	USDA texture	Classification		Pct Fragments		Percentage passing sieve number—				Liquid limit	Plasticity index
					Unified	AASHTO	>10 inches	3-10 inches	4	10	40	200		
			In				*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*
MfE—Minvale-Bodine-Fullerton complex, 25 to 50 percent slopes, stony														
Minvale	48	B	0-9	Gravelly silty clay loam, gravelly silt loam, loam	CL, CL-ML, GC, GC-GM	A-4, A-6	0- 0- 0	0- 0- 0	80-90-100	75-88-100	40-55-70	36-51-65	20-30-40	5-10-15
			9-60	Gravelly clay loam, very gravelly silty clay loam, gravelly clay	CL, GC, ML, SC	A-4, A-6, A-7	0- 0- 0	0- 3- 5	55-68-80	50-63-75	40-55-70	36-51-65	25-38-50	7-15-23
Bodine	25	A	0-1	Highly decomposed plant material	PT	A-1	0- 0- 0	0- 0- 0	100-100-100	100-100-100	—	—	—	—
			1-16	Very gravelly silt loam, extremely gravelly loam, very gravelly loam	GC, GC-GM, GM, SC, SC-SM, SM	A-1, A-2, A-4, A-6	0- 0- 0	10-23-35	30-50-70	20-43-65	20-38-55	15-30-45	20-29-38	3-9 -15
			16-60	Very gravelly silty clay loam, extremely gravelly clay loam, very gravelly loam, very gravelly clay	GC, GM, GW-GC, GW-GM, SC, SM	A-2	0- 0- 0	10-23-35	20-45-70	15-40-65	15-30-45	12-24-35	26-34-42	8-12-16
Fullerton	20	B	0-20	Fine sandy loam, gravelly silt loam, gravelly loam	CL, CL-ML, GC, GC-GM, SM, SC, SC-SM	A-2, A-4	0- 0- 0	2- 9- 15	60-77-94	45-63-80	40-58-75	30-50-70	18-24-30	3-7 -10
			20-60	Gravelly clay, gravelly silty clay	GM, MH, ML, SM	A-2, A-7	0- 0- 0	2-10- 18	60-75-90	45-63-80	40-58-75	30-53-75	48-63-78	20-31-42

Engineering Properties–Knox County, Tennessee														
Map unit symbol and soil name	Pct. of map unit	Hydrologic group	Depth	USDA texture	Classification		Pct Fragments		Percentage passing sieve number—				Liquid limit	Plasticity index
					Unified	AASHTO	>10 inches	3-10 inches	4	10	40	200		
			In				*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*
Ro—Rockdell gravelly loam, 0 to 4 percent slopes, rarely flooded														
Rockdell	80	B	0-10	Gravelly loam, cobbly fine sandy loam	GM, SC-SM, SM, GC-GM, SM	A-2, A-4	0- 0- 0	0-10- 20	50-63-75	50-55-60	40-48-55	30-40-49	15-20-25	NP-4 -7
			10-31	Very gravelly sandy clay loam, very cobbly sandy loam, extremely gravelly loam, gravelly loam	SC, GC, SC-SM	A-2, A-4, A-6	0- 0- 0	0-10- 20	50-63-75	20-40-60	20-35-50	15-28-40	25-33-40	7-11-15
			31-60	Very cobbly clay loam, gravelly clay	GM, MH, SM	A-7	0- 0- 0	35-53-70	65-75-85	55-65-75	45-58-70	40-50-60	70-75-80	30-35-40

Data Source Information

Soil Survey Area: Knox County, Tennessee
Survey Area Data: Version 14, Sep 16, 2018

Local Roads and Streets—Knox County, Tennessee



Map Scale: 1:3,630 if printed on A landscape (11" x 8.5") sheet.

N

Meters: 0 50 100 200 300

Feet: 0 150 300 600 900

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 16N WGS84

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils

Soil Rating Polygons
- Very limited
- Somewhat limited
- Not limited
- Not rated or not available

Soil Rating Lines
- Very limited
- Somewhat limited
- Not limited
- Not rated or not available

Soil Rating Points
- Very limited
- Somewhat limited
- Not limited
- Not rated or not available

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:12,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Knox County, Tennessee
Survey Area Data: Version 14, Sep 16, 2018

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Mar 8, 2016—Oct 26, 2017

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Local Roads and Streets

Map unit symbol	Map unit name	Rating	Component name (percent)	Rating reasons (numeric values)	Acres in AOI	Percent of AOI
EvB	Etowah-Minvale complex, 2 to 5 percent slopes	Very limited	Etowah (55%)	Low strength (1.00)	13.1	20.5%
FvC	Fullerton-Minvale complex, 5 to 12 percent slopes	Very limited	Fullerton (53%)	Low strength (1.00)	22.6	35.3%
				Shrink-swell (0.05)		
				Slope (0.04)		
MfD	Minvale-Fullerton complex, 12 to 25 percent slopes, stony	Very limited	Minvale (70%)	Slope (1.00)	10.4	16.3%
			Fullerton (23%)	Slope (1.00)		
				Low strength (1.00)		
				Shrink-swell (0.05)		
MfE	Minvale-Bodine-Fullerton complex, 25 to 50 percent slopes, stony	Very limited	Minvale (48%)	Slope (1.00)	4.4	6.9%
			Bodine (25%)	Slope (1.00)		
				Soluble bedrock (0.03)		
			Fullerton (20%)	Slope (1.00)		
				Low strength (1.00)		
				Shrink-swell (0.05)		
Ro	Rockdell gravelly loam, 0 to 4 percent slopes, rarely flooded	Somewhat limited	Rockdell (80%)	Flooding (0.40)	2.5	3.9%
Uu	Urban land-Udorthents complex	Not rated	Urban land (70%)		10.9	17.1%
			Udorthents (30%)			
Totals for Area of Interest					**63.9**	**100.0%**

Rating	Acres in AOI	Percent of AOI
Very limited	50.5	79.0%
Somewhat limited	2.5	3.9%
Null or Not Rated	10.9	17.1%

Rating	**Acres in AOI**	**Percent of AOI**
Totals for Area of Interest	**63.9**	**100.0%**

Description

Local roads and streets have an all-weather surface and carry automobile and light truck traffic all year. They have a subgrade of cut or fill soil material; a base of gravel, crushed rock, or soil material stabilized by lime or cement; and a surface of flexible material (asphalt), rigid material (concrete), or gravel with a binder. The ratings are based on the soil properties that affect the ease of excavation and grading and the traffic-supporting capacity. The properties that affect the ease of excavation and grading are depth to bedrock or a cemented pan, hardness of bedrock or a cemented pan, depth to a water table, ponding, flooding, the amount of large stones, and slope. The properties that affect the traffic-supporting capacity are soil strength (as inferred from the AASHTO group index number), subsidence, linear extensibility (shrink-swell potential), the potential for frost action, depth to a water table, and ponding.

The ratings are both verbal and numerical. Rating class terms indicate the extent to which the soils are limited by all of the soil features that affect the specified use. "Not limited" indicates that the soil has features that are very favorable for the specified use. Good performance and very low maintenance can be expected. "Somewhat limited" indicates that the soil has features that are moderately favorable for the specified use. The limitations can be overcome or minimized by special planning, design, or installation. Fair performance and moderate maintenance can be expected. "Very limited" indicates that the soil has one or more features that are unfavorable for the specified use. The limitations generally cannot be overcome without major soil reclamation, special design, or expensive installation procedures. Poor performance and high maintenance can be expected.

Numerical ratings indicate the severity of individual limitations. The ratings are shown as decimal fractions ranging from 0.01 to 1.00. They indicate gradations between the point at which a soil feature has the greatest negative impact on the use (1.00) and the point at which the soil feature is not a limitation (0.00).

The map unit components listed for each map unit in the accompanying Summary by Map Unit table in Web Soil Survey or the Aggregation Report in Soil Data Viewer are determined by the aggregation method chosen. An aggregated rating class is shown for each map unit. The components listed for each map unit are only those that have the same rating class as listed for the map unit. The percent composition of each component in a particular map unit is presented to help the user better understand the percentage of each map unit that has the rating presented.

Other components with different ratings may be present in each map unit. The ratings for all components, regardless of the map unit aggregated rating, can be viewed by generating the equivalent report from the Soil Reports tab in Web Soil Survey or from the Soil Data Mart site. Onsite investigation may be needed to validate these interpretations and to confirm the identity of the soil on a given site.

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Higher

Table 10.4.2-1—Minimum Number of Exploration Points and Depth of Exploration (modified after Sabatini et al., 2002)

Application	Minimum Number of Exploration Points and Location of Exploration Points	Minimum Depth of Exploration
Retaining Walls	A minimum of one exploration point for each retaining wall. For retaining walls more than 100 ft in length, exploration points spaced every 100 to 200 ft with locations alternating from in front of the wall to behind the wall. For anchored walls, additional exploration points in the anchorage zone spaced at 100 to 200 ft. For soil-nailed walls, additional exploration points at a distance of 1.0 to 1.5 times the height of the wall behind the wall spaced at 100 to 200 ft.	Investigate to a depth below bottom of wall at least to a depth where stress increase due to estimated foundation load is less than ten percent of the existing effective overburden stress at that depth and between one and two times the wall height. Exploration depth should be great enough to fully penetrate soft highly compressible soils, e.g., peat, organic silt, or soft fine grained soils, into competent material of suitable bearing capacity, e.g., stiff to hard cohesive soil, compact dense cohesionless soil, or bedrock.
Shallow Foundations	For substructure, e.g., piers or abutments, widths less than or equal to 100 ft, a minimum of one exploration point per substructure. For substructure widths greater than 100 ft, a minimum of two exploration points per substructure. Additional exploration points should be provided if erratic subsurface conditions are encountered.	Depth of exploration should be: • great enough to fully penetrate unsuitable foundation soils, e.g., peat, organic silt, or soft fine grained soils, into competent material of suitable bearing resistance, e.g., stiff to hard cohesive soil, or compact to dense cohesionless soil or bedrock ; • at least to a depth where stress increase due to estimated foundation load is less than ten percent of the existing effective overburden stress at that depth; and • if bedrock is encountered before the depth required by the second criterion above is achieved, exploration depth should be great enough to penetrate a minimum of 10 ft into the bedrock, but rock exploration should be sufficient to characterize compressibility of infill material of near-horizontal to horizontal discontinuities. Note that for highly variable bedrock conditions, or in areas where very large boulders are likely, more than 10 ft or rock core may be required to verify that adequate quality bedrock is present.
Deep Foundations	For substructure, e.g., bridge piers or abutments, widths less than or equal to 100 ft, a minimum of one exploration point per substructure. For substructure widths greater than 100 ft, a minimum of two exploration points per substructure. Additional exploration points should be provided if erratic subsurface conditions are encountered, especially for the case of shafts socketed into bedrock. To reduce design and construction risk due to subsurface condition variability and the potential for construction claims, at least one exploration per shaft should be considered for large diameter shafts (e.g., greater than 5 ft in diameter), especially when shafts are socketed into bedrock.	In soil, depth of exploration should extend below the anticipated pile or shaft tip elevation a minimum of 20 ft, or a minimum of two times the minimum pile group dimension, whichever is deeper. All borings should extend through unsuitable strata such as unconsolidated fill, peat, highly organic materials, soft fine-grained soils, and loose coarse-grained soils to reach hard or dense materials. For piles bearing on rock, a minimum of 10 ft of rock core shall be obtained at each exploration point location to verify that the boring has not terminated on a boulder. For shafts supported on or extending into rock, a minimum of 10 ft of rock core, or a length of rock core equal to at least three times the shaft diameter for isolated shafts or two times the minimum shaft group dimension, whichever is greater, shall be extended below the anticipated shaft tip elevation to determine the physical characteristics of rock within the zone of foundation influence. Note that for highly variable bedrock conditions, or in areas where very large boulders are likely, more than 10 ft or rock core may be required to verify that adequate quality bedrock is present.

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Following bid letting, the awarded contractor prepares, then submits to TDOT, the retaining wall shop drawings. The preparation of the retaining wall shop drawings must be in accordance with the state's contract which includes, but is not limited to, geotechnical design parameters, notes furnished in RETAINING WALL DETAILS sheets, and Special Provision 624 - Retaining Walls (SP624). The Contractor submits retaining wall shop drawings to TDOT for review, comment and approval. Once the retaining wall shop drawings are finalized, the Contractor installs the wall in accordance with the shop drawings stamped "Approved" by the Structures Division.

Drilling Requirements

Subsurface explorations are to be conducted in accordance with the latest edition of the AASHTO LRFD Bridge Design Specifications (Table 10.4.2-1 in 6th Ed.). In addition to being in compliance with AASHTO guidance, typical TDOT subsurface exploration plans include borings advanced a minimum of 1.5 times the wall height below the proposed bottom of the footing if in soil. This rule of thumb will generally result in getting data that is within 2 times the foundation width. If initial drilling indicates soft soil conditions, test borings should extend to 2 times the maximum wall height, to rock, or to firm or dense material is encountered. Split spoon samples shall be taken where needed. Representative Shelby tube samples shall be taken of each soil type expected beneath the foundation. If bedrock is encountered relatively shallow, consider advancing borings to refusal to identify the bedrock depth, especially if foundation improvements could include excavating \ backfilling to bedrock, or a soldier pile could be socketed. If rock is encountered less than about 10 feet below the approximate proposed bottom of footing, the rock below the footing support elevation shall be cored to obtain 7 to 10 feet of competent rock sufficient to support all retaining wall types feasible for the site.

Generally, a TDOT subsurface exploration plan would consist of advancing test borings on approximate fifty to one-hundred feet along the wall face alignment depending upon the expected subsurface soil and rock consistency. This spacing may be expanded or reduced, or a minimum of three borings, depending upon the consistency of conditions encountered and the judgment of the geotechnical engineer or engineering geologist. Consideration should also be given to advancing test borings behind the wall to characterize conditions for anchors, soil nail, or temporary excavation purposes.

The geotechnical engineer should consult the appropriate references such as ASHTO LRFD Bridge Design or other NHI publications on specific wall types for further details of recommended drilling/sampling/testing requirements.

10.4.3—Laboratory Tests

10.4.3.1—Soil Tests

Laboratory testing should be conducted to provide the basic data with which to classify soils and to measure their engineering properties.

When performed, laboratory tests shall be conducted in accordance with the AASHTO, ASTM, or Owner-supplied procedures applicable to the design properties needed.

C10.4.3.1

Laboratory tests of soils may be grouped broadly into two general classes:

- Classification or index tests. These may be performed on either disturbed or undisturbed samples.
- Quantitative or performance tests for permeability, compressibility and shear strength. These tests are generally performed on undisturbed samples, except for materials to be placed as controlled fill or materials that do not have a stable soil-structure, e.g., cohesionless materials. In these cases, tests should be performed on specimens prepared in the laboratory.

Detailed information regarding the types of tests needed for foundation design is provided in Geotechnical Engineering Circular #5—Evaluation of Soil and Rock Properties (Sabatini et al., 2002).

10.4.3.2—Rock Tests

If laboratory strength tests are conducted on intact rock samples for classification purposes, they should be considered as upper bound values. If laboratory compressibility tests are conducted, they should be considered as lower bound values. Additionally, laboratory tests should be used in conjunction with field tests and field characterization of the rock mass to give estimates of rock mass behavioral characteristics. When performed, laboratory tests shall be conducted in accordance with the ASTM or Owner-supplied procedures applicable to the design properties needed.

C10.4.3.2

Rock samples small enough to be tested in the laboratory are usually not representative of the entire rock mass. Laboratory testing of rock is used primarily for classification of intact rock samples, and, if performed properly, serves a useful function in this regard.

Detailed information regarding the types of tests needed and their use for foundation design is provided in Geotechnical Engineering Circular #5—Evaluation of Soil and Rock Properties, April 2002 (Sabatini et al., 2002).

10.4.4—In-Situ Tests

In-situ tests may be performed to obtain deformation and strength parameters of foundation soils or rock for the purposes of design and/or analysis. In-situ tests should be conducted in soils that do not lend themselves to undisturbed sampling as a means to estimate soil design parameters. When performed, in-situ tests shall be conducted in accordance with the appropriate ASTM or AASHTO standards.

Where in-situ test results are used to estimate design properties through correlations, such correlations should be well established through long-term widespread use or through detailed measurements that illustrate the accuracy of the correlation.

C10.4.4

Detailed information on in-situ testing of soils and rock and their application to geotechnical design can be found in Sabatini et al. (2002) and Wyllie (1999).

Correlations are in some cases specific to a geological formation. While this fact does not preclude the correlation from being useful in other geologic formations, the applicability of the correlation to those other formations should be evaluated.

For further discussion, see Article 10.4.6.

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If *SPT N* values are used, unless otherwise specified for the design method or correlation being used, they shall be corrected for the effects of overburden pressure determined as:

$$N1 = C_N N \qquad (10.4.6.2.4\text{-}1)$$

$N1$ = *SPT* blow count corrected for overburden pressure, σ'_v (blows/ft)

C_N = $[0.77 \log_{10}(40/\sigma'_v)]$, and $C_N < 2.0$

σ'_v = vertical effective stress (ksf)

N = uncorrected *SPT* blow count (blows/ft)

SPT N values should also be corrected for hammer efficiency, if applicable to the design method or correlation being used, determined as:

$$N_{60} = (ER/60\%)N \qquad (10.4.6.2.4\text{-}2)$$

where:

N_{60} = *SPT* blow count corrected for hammer efficiency (blows/ft)

ER = hammer efficiency expressed as percent of theoretical free fall energy delivered by the hammer system actually used

N = uncorrected *SPT* blow count (blows/ft)

When *SPT* blow counts have been corrected for both overburden effects and hammer efficiency effects, the resulting corrected blow count shall be denoted as $N1_{60}$, determined as:

$$N1_{60} = C_N N_{60} \qquad (10.4.6.2.4\text{-}3)$$

The drained friction angle of granular deposits should be determined based on the following correlation.

Table 10.4.6.2.4-1—Correlation of *SPT* $N1_{60}$ Values to Drained Friction Angle of Granular Soils (modified after Bowles, 1977)

$N1_{60}$	ϕ_f
<4	25–30
4	27–32
10	30–35
30	35–40
50	38–43

relative density estimated from the available in-situ data. The test specimen should be large enough to allow the full grain size range of the soil to be included in the specimen. This may not always be possible, and if not possible, it should be recognized that the shear strength measured would likely be conservative.

A method using the results of *SPT* testing is presented. Other in-situ tests such as *CPT* and *DMT* may be used. For details on determination of ϕ_f from these tests, refer to Sabatini et al. (2002).

The use of automatic trip hammers is increasing. In order to use correlations based on standard rope and cathead hammers, the *SPT N* values must be corrected to reflect the greater energy delivered to the sampler by these systems.

Hammer efficiency (ER) for specific hammer systems used in local practice may be used in lieu of the values provided. If used, specific hammer system efficiencies shall be developed in general accordance with ASTM D4945 for dynamic analysis of driven piles or other accepted procedure.

The following values for ER may be assumed if hammer specific data are not available, e.g., from older boring logs:

ER = 60 percent for conventional drop hammer using rope and cathead

ER = 80 percent for automatic trip hammer

Corrections for rod length, hole size, and use of a liner may also be made if appropriate. In general, these are only significant in unusual cases or where there is significant variation from standard procedures. These corrections may be significant for evaluation of liquefaction. Information on these additional corrections may be found in Youd and Idriss (1997).

The $N1_{60}$-ϕ_f correlation used is modified after Bowles (1977). The correlation of Peck, Hanson, and Thornburn (1974) falls within the ranges specified. Experience should be used to select specific values within the ranges. In general, finer materials or materials with significant silt-sized material will fall in the lower portion of the range. Coarser materials with less than five percent fines will fall in the upper portion of the ranges. The geologic history and angularity of the particles may also need to be considered when selecting a value for ϕ_f.

Care should be exercised when using other correlations of *SPT* results to soil parameters. Some published correlations are based on corrected values ($N1_{60}$) and some are based on uncorrected values (N).

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For gravels and rock fill materials where *SPT* testing is not reliable, Figure 10.4.6.2.4-1 should be used to estimate the drained friction angle.

Rock Fill Grade	Particle Unconfined Compressive Strength (ksf)
A	>4610
B	3460– 4610
C	2590– 3460
D	1730– 2590
E	≤1730

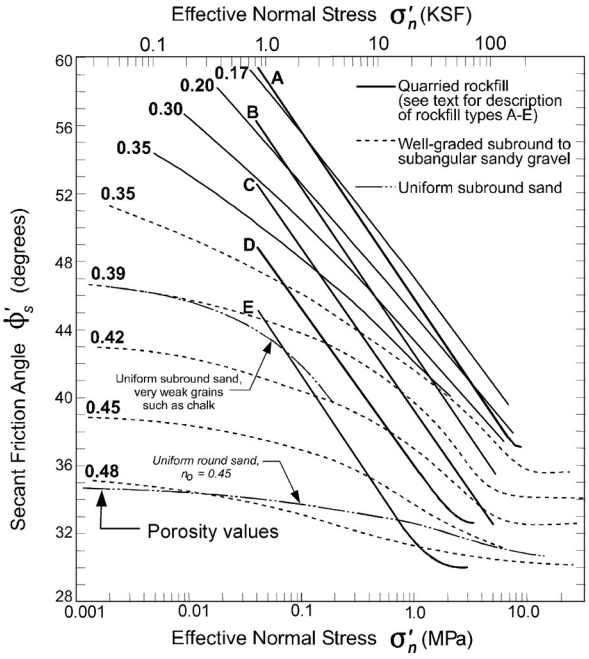


Figure 10.4.6.2.4-1—Estimation of Drained Friction Angle of Gravels and Rock Fills (modified after Terzaghi, Peck, and Mesri, 1996)

The designer should ascertain the basis of the correlation and use either N160 or N as appropriate.

Care should also be exercised when using *SPT* blow counts to estimate soil shear strength if in soils with coarse gravel, cobbles, or boulders. Large gravels, cobbles, or boulders could cause the *SPT* blow counts to be unrealistically high.

The secant friction angle derived from the procedure to estimate the drained friction angle of gravels and rock fill materials depicted in Figure 10.4.6.2.4-1 is based on a straight line from the origin of a Mohr diagram to the intersection with the strength envelope at the effective normal stress. Thus the angle derived is applicable only to analysis of field conditions subject to similar normal stresses. See Terzaghi, Peck, and Mesri (1996) for additional details regarding this procedure.

10.4.6.3—Soil Deformation

Consolidation parameters C_c, C_r, C_α should be determined from the results of one-dimensional consolidation tests. To assess the potential variability in the settlement estimate, the average, upper and lower bound values obtained from testing should be considered.

C10.4.6.3

It is important to understand whether the values obtained are computed based on a void ratio definition or a strain definition. Computational methods vary for each definition.

For preliminary analyses or where accurate prediction of settlement is not critical, values obtained from correlations to index properties may be used. Refer to Sabatini et al. (2002) for discussion of the various correlations available. If correlations for prediction of

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Table C10.4.6.3-1—Elastic Constants of Various Soils (modified after U.S. Department of the Navy, 1982; Bowles, 1988)

Soil Type	Typical Range of Young's Modulus Values, E_s (ksi)	Poisson's Ratio, ν (dim)
Clay:		
Soft sensitive	0.347–2.08	0.4–0.5 (undrained)
Medium stiff	2.08–6.94	
to stiff	6.94–13.89	
Very stiff		
Loess	2.08–8.33	0.1–0.3
Silt	0.278–2.78	0.3–0.35
Fine Sand:		
Loose	1.11–1.67	0.25
Medium dense	1.67–2.78	
Dense	2.78–4.17	
Sand:		
Loose	1.39–4.17	0.20–0.36
Medium dense	4.17–6.94	
Dense	6.94–11.11	0.30–0.40
Gravel:		
Loose	4.17–11.11	0.20–0.35
Medium dense	11.11–13.89	
Dense	13.89–27.78	0.30–0.40

Estimating E_s from *SPT N* Value	
Soil Type	E_s (ksi)
Silts, sandy silts, slightly cohesive mixtures	$0.056\ N1_{60}$
Clean fine to medium sands and slightly silty sands	$0.097\ N1_{60}$
Coarse sands and sands with little gravel	$0.139\ N1_{60}$
Sandy gravel and gravels	$0.167\ N1_{60}$

Estimating E_s from q_c (static cone resistance)	
Sandy soils	$0.028q_c$

The modulus of elasticity for normally consolidated granular soils tends to increase with depth. An alternative method of defining the soil modulus for granular soils is to assume that it increases linearly with depth starting at zero at the ground surface in accordance with the following equation:

$$E_s = nh \times z \qquad \text{(C10.4.6.3-1)}$$

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where:

E_s = the soil modulus at depth z (ksi)
n_h = rate of increase of soil modulus with depth as defined in Table C10.4.6.3-2 (ksi/ft)
z = depth below the ground surface (ft)

Table C10.4.6.3-2—Rate of Increase of Soil Modulus with Depth n_h (ksi/ft) for Sand

Consistency	Dry or Moist	Submerged
Loose	0.417	0.208
Medium	1.11	0.556
Dense	2.78	1.39

The potential for soil swell that may result in uplift on deep foundations or heave of shallow foundations should be evaluated based on Table 10.4.6.3-1.

Table 10.4.6.3-1—Method for Identifying Potentially Expansive Soils (Reese and O'Neill, 1988)

Liquid Limit *LL* (%)	Plastic Limit *PL* (%)	Soil Suction (ksf)	Potential Swell (%)	Potential Swell Classification
>60	>35	>8	>1.5	High
50–60	25–35	3–8	0.5–1.5	Marginal
<50	<25	<3	<0.5	Low

The formulation provided in Eq. C10.4.6.3-1 is used primarily for analysis of lateral response or buckling of deep foundations.

10.4.6.4—Rock Mass Strength

The strength of intact rock material should be determined using the results of unconfined compression tests on intact rock cores. If point load strength index tests are used to assess intact rock compressive strength in lieu of a full suite of unconfined compression tests on intact rock cores, the point load test results should be calibrated to unconfined compression strength tests obtained from the same geologic unit.

Except as noted for design of spread footings in rock, for a rock mass of "randomly" oriented discontinuities such that it that contains a sufficient number behaves as an isotropic mass, and thus, its behavior is largely independent of the direction of the applied loads, the strength of the rock mass should first be classified using its geological strength index (GSI) as described in Figures 10.4.6.4-1 and 10.4.6.4-2, and then assessed using the Hoek-Brown failure criterion.

C10.4.6.4

Point load strength index tests rely on empirical correlations to intact rock compressive strength. The correlation provided in the ASTM point load test procedure (ASTM D5731) is empirically based and may not be valid for the specific rock type under consideration. Therefore, a site specific correlation with uniaxial compressive strength test results from the same geologic unit is recommended. Point load strength index tests should not be used for weak to very weak rocks (< 2200 psi /15 MPa).

Because the engineering behavior of rock is strongly influenced by the presence and characteristics of discontinuities, emphasis is placed on visual assessment of the rock and the rock mass. The application of a rock mass classification system essentially assumes that the rock mass contains a sufficient number of "randomly" oriented discontinuities such that it behaves as an isotropic mass, and thus its behavior is largely independent of the direction of the applied loads. It is generally not appropriate to use such classification systems for rock masses with well defined, dominant structural fabrics or where the orientation of discrete, persistent discontinuities controls behavior to loading.

The *GSI* was introduced by Hoek et al. (1995) and Hoek and Brown (1997), and updated by Hoek et al.

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Regular Service—Condition excluding the presence of special permit vehicles, wind exceeding 55 mph, and extreme events, including scour.

Rehabilitation—A process in which the resistance of the bridge is either restored or increased.

Resistance Factor—A statistically-based multiplier applied to nominal resistance accounting primarily for variability of material properties, structural dimensions and workmanship, and uncertainty in the prediction of resistance, but also related to the statistics of the loads through the calibration process.

Service Life—The period of time that the bridge is expected to be in operation.

Service Limit States—Limit states relating to stress, deformation, and cracking under regular operating conditions.

Strength Limit States—Limit states relating to strength and stability during the design life.

1.3—DESIGN PHILOSOPHY

1.3.1—General

Bridges shall be designed for specified limit states to achieve the objectives of constructibility, safety, and serviceability, with due regard to issues of inspectability, economy, and aesthetics, as specified in Article 2.5.

Regardless of the type of analysis used, Eq. 1.3.2.1-1 shall be satisfied for all specified force effects and combinations thereof.

C1.3.1

The limit states specified herein are intended to provide for a buildable, serviceable bridge, capable of safely carrying design loads for a specified lifetime.

The resistance of components and connections is determined, in many cases, on the basis of inelastic behavior, although the force effects are determined by using elastic analysis. This inconsistency is common to most current bridge specifications as a result of incomplete knowledge of inelastic structural action.

1.3.2—Limit States

1.3.2.1—General

Each component and connection shall satisfy Eq. 1.3.2.1-1 for each limit state, unless otherwise specified. For service and extreme event limit states, resistance factors shall be taken as 1.0, except for bolts, for which the provisions of Article 6.5.5 shall apply, and for concrete columns in Seismic Zones 2, 3, and 4, for which the provisions of Articles 5.10.11.3 and 5.10.11.4.1b shall apply. All limit states shall be considered of equal importance.

$$\sum \eta_i \gamma_i Q_i \le \phi R_n = R_r \qquad (1.3.2.1\text{-}1)$$

in which:

For loads for which a maximum value of γ_i is appropriate:

$$\eta_i = \eta_D \eta_R \eta_I \ge 0.95 \qquad (1.3.2.1\text{-}2)$$

For loads for which a minimum value of γ_i is appropriate:

$$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \le 1.0 \qquad (1.3.2.1\text{-}3)$$

C1.3.2.1

Eq. 1.3.2.1-1 is the basis of LRFD methodology.

Assigning resistance factor $\phi = 1.0$ to all nonstrength limit states is a default, and may be over-ridden by provisions in other Sections.

Ductility, redundancy, and operational classification are considered in the load modifier η. Whereas the first two directly relate to physical strength, the last concerns the consequences of the bridge being out of service. The grouping of these aspects on the load side of Eq. 1.3.2.1-1 is, therefore, arbitrary. However, it constitutes a first effort at codification. In the absence of more precise information, each effect, except that for fatigue and fracture, is estimated as ±5 percent, accumulated geometrically, a clearly subjective approach. With time, improved quantification of ductility, redundancy, and operational classification, and their interaction with system reliability, may be attained, possibly leading to a rearrangement of Eq. 1.3.2.1-1, in which these effects may appear on either side of the equation or on both sides.

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where:

γ_i = load factor: a statistically based multiplier applied to force effects

ϕ = resistance factor: a statistically based multiplier applied to nominal resistance, as specified in Sections 5, 6, 7, 8, 10, 11, and 12

η_i = load modifier: a factor relating to ductility, redundancy, and operational classification

η_D = a factor relating to ductility, as specified in Article 1.3.3

η_R = a factor relating to redundancy as specified in Article 1.3.4

η_I = a factor relating to operational classification as specified in Article 1.3.5

Q_i = force effect

R_n = nominal resistance

R_r = factored resistance: ϕR_n

The influence of η on the girder reliability index, β, can be estimated by observing its effect on the minimum values of β calculated in a database of girder-type bridges. Cellular structures and foundations were not a part of the database; only individual member reliability was considered. For discussion purposes, the girder bridge data used in the calibration of these Specifications was modified by multiplying the total factored loads by $\eta = 0.95$, 1.0, 1.05, and 1.10. The resulting minimum values of β for 95 combinations of span, spacing, and type of construction were determined to be approximately 3.0, 3.5, 3.8, and 4.0, respectively. In other words, using $\eta > 1.0$ relates to a β higher than 3.5.

A further approximate representation of the effect of η values can be obtained by considering the percent of random normal data less than or equal to the mean value plus $\lambda\ \sigma$, where λ is a multiplier, and σ is the standard deviation of the data. If λ is taken as 3.0, 3.5, 3.8, and 4.0, the percent of values less than or equal to the mean value plus $\lambda\ \sigma$ would be about 99.865 percent, 99.977 percent, 99.993 percent, and 99.997 percent, respectively.

The Strength I Limit State in the *AASHTO LRFD Design Specifications* has been calibrated for a target reliability index of 3.5 with a corresponding probability of exceedance of 2.0E-04 during the 75-yr design life of the bridge. This 75-yr reliability is equivalent to an annual probability of exceedance of 2.7E-06 with a corresponding annual target reliability index of 4.6. Similar calibration efforts for the Service Limit States are underway. Return periods for extreme events are often based on annual probability of exceedance and caution must be used when comparing reliability indices of various limit states.

1.3.2.2—Service Limit State

The service limit state shall be taken as restrictions on stress, deformation, and crack width under regular service conditions.

C1.3.2.2

The service limit state provides certain experience-related provisions that cannot always be derived solely from strength or statistical considerations.

1.3.2.3—Fatigue and Fracture Limit State

The fatigue limit state shall be taken as restrictions on stress range as a result of a single design truck occurring at the number of expected stress range cycles.

The fracture limit state shall be taken as a set of material toughness requirements of the AASHTO Materials Specifications.

C1.3.2.3

The fatigue limit state is intended to limit crack growth under repetitive loads to prevent fracture during the design life of the bridge.

1.3.2.4—Strength Limit State

Strength limit state shall be taken to ensure that strength and stability, both local and global, are provided to resist the specified statistically significant load combinations that a bridge is expected to experience in its design life.

C1.3.2.4

The strength limit state considers stability or yielding of each structural element. If the resistance of any element, including splices and connections, is exceeded, it is assumed that the bridge resistance has been exceeded. In fact, in multigirder cross-sections there is significant elastic reserve capacity in almost all such bridges beyond such a load level. The live load cannot be positioned to maximize

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Table 3.4.1-1—Load Combinations and Load Factors

Load Combination Limit State	*DC* *DD* *DW* *EH* *EV* *ES* *EL* *PS* *CR* *SH*	*LL* *IM* *CE* *BR* *PL* *LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time				
										EQ	*BL*	*IC*	*CT*	*CV*
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	γ_p	γEQ	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—	—
Service III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—	—
Fatigue I— *LL*, *IM* & *CE* only	—	1.50	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue II— *LL*, *IM* & *CE* only	—	0.75	—	—	—	—	—	—	—	—	—	—	—	—

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Table 3.4.1-2—Load Factors for Permanent Loads, γ_p

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor Maximum	Load Factor Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.4	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (1999) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for anchored walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures			
○ Metal Box Culverts, Structural Plate Culverts with Deep Corrugations, and Fiberglass Culverts		1.5 1.3	0.9 0.9
○ Thermoplastic Culverts		1.95	0.9
○ All others			
ES: Earth Surcharge		1.50	0.75

Table 3.4.1-3—Load Factors for Permanent Loads Due to Superimposed Deformations, γ_p

Bridge Component	*PS*	*CR, SH*
Superstructures—Segmental Concrete Substructures supporting Segmental Superstructures (see 3.12.4, 3.12.5)	1.0	See γ_P for *DC*, Table 3.4.1-2
Concrete Superstructures—non-segmental	1.0	1.0
Substructures supporting non-segmental Superstructures		
• using I_g	0.5	0.5
• using $I_{effectuve}$	1.0	1.0
Steel Substructures	1.0	1.0

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within the soil mass in accordance with Article 3.11.3. Appropriate drainage provisions shall be provided to prevent hydrostatic and seepage forces from developing behind the wall in accordance with the provisions of Section 11. In no case shall highly plastic clay be used for backfill.

3.11.5.3—Active Lateral Earth Pressure Coefficient, k_a

Values for the coefficient of active lateral earth pressure may be taken as:

$$k_a = \frac{\sin^2(\theta + \phi'_f)}{\Gamma\,[\sin^2\theta \sin(\theta - \delta)]} \qquad (3.11.5.3\text{-}1)$$

in which:

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi'_f + \delta)\sin(\phi'_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right]^2 \qquad (3.11.5.3\text{-}2)$$

where:

δ = friction angle between fill and wall taken as specified in Table 3.11.5.3-1 (degrees)
β = angle of fill to the horizontal as shown in Figure 3.11.5.3-1 (degrees)
θ = angle of back face of wall to the horizontal as shown in Figure 3.11.5.3-1 (degrees)
ϕ'_f = effective angle of internal friction (degrees)

For conditions that deviate from those described in Figure 3.11.5.3-1, the active pressure may be calculated by using a trial procedure based on wedge theory using the Culmann method (e.g., see Terzaghi et al., 1996).

C3.11.5.3

The values of k_a by Eq. 3.11.5.3-1 are based on the Coulomb earth pressure theories. The Coulomb theory is necessary for design of retaining walls for which the back face of the wall interferes with the development of the full sliding surfaces in the backfill soil assumed in Rankine theory (Figure C3.11.5.3-1 and Article C3.11.5.8). Either Coulomb or Rankine wedge theory may be used for long heeled cantilever walls shown in Figure C3.11.5.3-1a. In general, Coulomb wedge theory applies for gravity, semigravity and prefabricated modular walls with relatively steep back faces, and concrete cantilever walls with short heels.

For the cantilever wall in Figure C3.11.5.3-1b, the earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of soil to the left of the vertical plane is considered as part of the wall weight.

The differences between the Coulomb theory currently specified, and the Rankine theory specified in the past is illustrated in Figure C3.11.5.3-1. The Rankine theory is the basis of the equivalent fluid method of Article 3.11.5.5.

Silt and lean clay should not be used for backfill where free-draining granular materials are available. When using poorly draining silts or cohesive soils, extreme caution is advised in the determination of lateral earth pressures assuming the most unfavorable conditions. Consideration must be given for the development of pore water pressure within the soil mass in accordance with Article 3.11.3. Appropriate drainage provisions should be provided to prevent hydrostatic and seepage forces from developing behind the wall in accordance with the provisions in Section 11. In no case should highly plastic clay be used for backfill.

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The resultant lateral earth load due to the weight of the backfill shall be assumed to act at a height of $H/3$ above the base of the wall, where H is the total wall height, measured from the surface of the ground at the back of the wall to the bottom of the footing or the top of the leveling pad (for MSE walls).

3.11.5.2—At-Rest Lateral Earth Pressure Coefficient, k_o

For normally consolidated soils, vertical wall, and level ground, the coefficient of at-rest lateral earth pressure may be taken as:

$$k_o = 1 - \sin \phi'_f \qquad (3.11.5.2\text{-}1)$$

where:

ϕ'_f = effective friction angle of soil
k_o = coefficient of at-rest lateral earth pressure

For overconsolidated soils, the coefficient of at-rest lateral earth pressure may be assumed to vary as a function of the overconsolidation ratio or stress history, and may be taken as:

$$k_o = (1 - \sin \phi'_f)\,(OCR)^{\sin \phi'_f} \qquad (3.11.5.2\text{-}2)$$

where:

OCR = overconsolidation ratio

Silt and lean clay shall not be used for backfill unless suitable design procedures are followed and construction control measures are incorporated in the construction documents to account for their presence. Consideration must be given for the development of pore water pressure earth pressure may act as high as $0.4H$ above the base of the wall for a mass concrete gravity retaining wall, where H is the total wall height measured from the top of the backfill to the base of the footing, where the wall deflects laterally, i.e., translates, in response to lateral earth loading. For such structures, the backfill behind the wall must slide down along the back of the wall for the retained soil mass to achieve the active state of stress. Experimental results indicate that the backfill arches against the upper portion of the wall as the wall translates, causing an upward shift in the location at which the resultant of the lateral earth load is transferred to the wall (Terzaghi, 1934; Clausen and Johansen et al., 1972; Sherif et al., 1982). Such walls are not representative of typical gravity walls used in highway applications.

For most gravity walls which are representative of those used in highway construction, nongravity cantilever retaining walls or other flexible walls which tilt or deform laterally in response to lateral loading, e.g., MSE walls, as well as walls which cannot translate or tilt, e.g., integral abutment walls, significant arching of the backfill against the wall does not occur, and the resultant lateral load due to earth pressure acts at a height of $H/3$ above the base of the wall. Furthermore, where wall friction is not considered in the analysis, it is sufficiently conservative to use a resultant location of $H/3$ even if the wall can translate.

C3.11.5.2

For typical cantilevered walls over 5.0 ft high with structural grade backfill, calculations indicate that the horizontal movement of the top of the wall due to a combination of structural deformation of the stem and rotation of the foundation is sufficient to develop active conditions.

In many instances, the OCR may not be known with enough accuracy to calculate k_o using Eq. 3.11.5.2-2. Based on information on this issue provided by Holtz and Kovacs (1981), in general, for lightly overconsolidated sands (OCR = 1 to 2), k_o is in the range of 0.4 to 0.6. For highly overconsolidated sand, k_o can be on the order of 1.0.

The evaluation of the stress induced by cohesive soils is highly uncertain due to their sensitivity to shrink-swell, wet-dry and degree of saturation. Tension cracks can form, which considerably alter the assumptions for the estimation of stress. Extreme caution is advised in the determination

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11.6.3.3—Eccentricity Limits

For foundations on soil, the location of the resultant of the reaction forces shall be within the middle two-thirds of the base width.

For foundations on rock, the location of the resultant of the reaction forces shall be within the middle nine-tenths of the base width.

C11.6.3.3

The specified criteria for the location of the resultant, coupled with investigation of the bearing pressure, replace the investigation of the ratio of stabilizing moment to overturning moment. Location of the resultant within the middle two-thirds of the base width for foundations on soil is based on the use of plastic bearing pressure distribution for the limit state.

11.6.3.4—Subsurface Erosion

For walls constructed along rivers and streams, scour of foundation materials shall be evaluated during design, as specified in Article 2.6.4.4.2. Where potential problem conditions are anticipated, adequate protective measures shall be incorporated in the design.

The provisions of Article 10.6.1.2 shall apply.

The hydraulic gradient shall not exceed:

- For silts and cohesive soils: 0.20
- For other cohesionless soils: 0.30

Where water seeps beneath a wall, the effects of uplift and seepage forces shall be considered.

C11.6.3.4

The measures most commonly used to ensure that piping does not occur are:

- Seepage control,
- Reduction of hydraulic gradient, and
- Protective filters.

Seepage effects may be investigated by constructing a flow net, or in certain circumstances, by using generally accepted simplified methods.

11.6.3.5—Passive Resistance

Passive resistance shall be neglected in stability computations, unless the base of the wall extends below the depth of maximum scour, freeze-thaw or other disturbances. In the latter case, only the embedment below the greater of these depths shall be considered effective.

Where passive resistance is utilized to ensure adequate wall stability, the calculated passive resistance of soil in front of abutments and conventional walls shall be sufficient to prevent unacceptable forward movement of the wall.

The passive resistance shall be neglected if the soil providing passive resistance is, or is likely to become soft, loose, or disturbed, or if the contact between the soil and wall is not tight.

C11.6.3.5

Unacceptable deformations may occur before passive resistance is mobilized. Approximate deformations required to mobilize passive resistance are discussed in Article C3.11.1, where H in Table C3.11.1-1 is the effective depth of passive restraint.

11.6.3.6—Sliding

The provisions of Article 10.6.3.4 shall apply.

11.6.4—Safety against Structural Failure

The structural design of individual wall elements and wall foundations shall comply with the provisions of Sections 5, 6, 7, and 8.

The provisions of Article 10.6.1.3 shall be used to determine the distribution of contact pressure for structural design of footings.

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Table 3.11.5.3-1—Friction Angle for Dissimilar Materials (U.S. Department of the Navy, 1982a)

Interface Materials	Friction Angle, δ (degrees)	Coefficient of Friction, tan δ (dim.)
Mass concrete on the following foundation materials:		
• Clean sound rock	35	0.70
• Clean gravel, gravel-sand mixtures, coarse sand	29 to 31	0.55 to 0.60
• Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29	0.45 to 0.55
• Clean fine sand, silty or clayey fine to medium sand	19 to 24	0.34 to 0.45
• Fine sandy silt, nonplastic silt	17 to 19	0.31 to 0.34
• Very stiff and hard residual or preconsolidated clay	22 to 26	0.40 to 0.49
• Medium stiff and stiff clay and silty clay	17 to 19	0.31 to 0.34
Masonry on foundation materials has same friction factors.		
Steel sheet piles against the following soils:		
• Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22	0.40
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17	0.31
• Silty sand, gravel or sand mixed with silt or clay	14	0.25
• Fine sandy silt, nonplastic silt	11	0.19
Formed or precast concrete or concrete sheet piling against the following soils:		
• Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22 to 26	0.40 to 0.49
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17 to 22	0.31 to 0.40
• Silty sand, gravel or sand mixed with silt or clay	17	0.31
• Fine sandy silt, nonplastic silt	14	0.25
Various structural materials:		
• Masonry on masonry, igneous and metamorphic rocks:		
○ dressed soft rock on dressed soft rock	35	0.70
○ dressed hard rock on dressed soft rock	33	0.65
○ dressed hard rock on dressed hard rock	29	0.55
• Masonry on wood in direction of cross grain	26	0.49
• Steel on steel at sheet pile interlocks	17	0.31

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

For noncohesive soils, values of the coefficient of passive lateral earth pressure may be taken from Figure 3.11.5.4-1 for the case of a sloping or vertical wall with a horizontal backfill or from Figure 3.11.5.4-2 for the case of a vertical wall and sloping backfill. For conditions that deviate from those described in Figures 3.11.5.4-1 and 3.11.5.4-2, the passive pressure may be calculated by using a trial procedure based on wedge theory, e.g., see Terzaghi et al. (1996). When wedge theory is used, the limiting value of the wall friction angle should not be taken larger than one-half the angle of internal friction, ϕ_f.

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

The movement required to mobilize passive pressure is approximately 10.0 times as large as the movement needed to induce earth pressure to the active values. The movement required to mobilize full passive pressure in loose sand is approximately five percent of the height of the face on which the passive pressure acts. For dense sand, the movement required to mobilize full passive pressure is smaller than five percent of the height of the face on which the passive pressure acts, and five percent represents a conservative estimate of the movement required to mobilize the full passive pressure. For poorly compacted cohesive soils, the movement required to mobilize full passive pressure is larger than five percent of the height of the face on which the pressure acts.

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$$p_p = k_p \gamma_s z + 2c\sqrt{k_p} \qquad (3.11.5.4\text{-}1)$$

Wedge solutions are inaccurate and unconservative for larger values of wall friction angle.

where:

p_p = passive lateral earth pressure (ksf)
γ_s = unit weight of soil (kcf)
z = depth below surface of soil (ft)
c = soil cohesion (ksf)
k_p = coefficient of passive lateral earth pressure specified in Figures 3.11.5.4-1 and 3.11.5.4-2, as appropriate

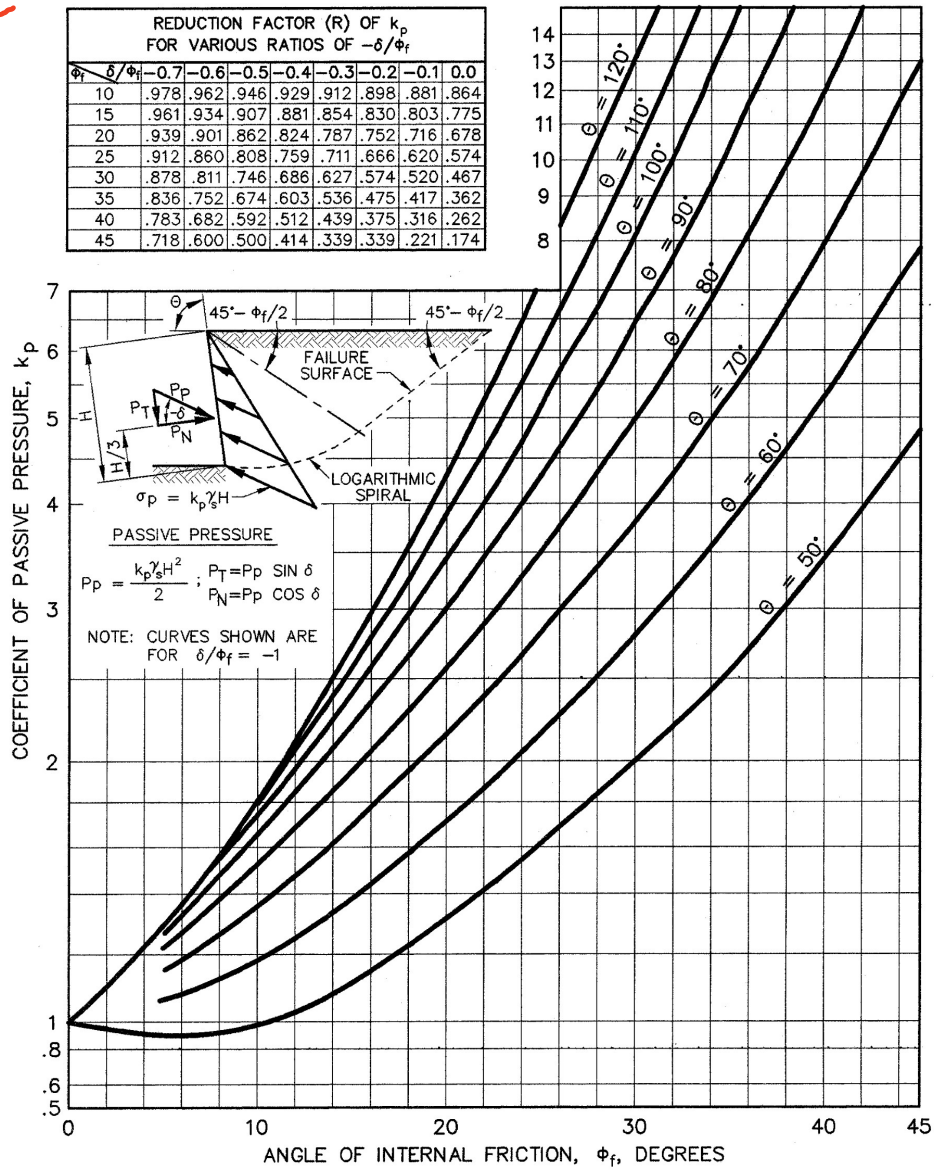


Figure 3.11.5.4-1—Computational Procedures for Passive Earth Pressures for Vertical and Sloping Walls with Horizontal Backfill (U.S. Department of the Navy, 1982a)

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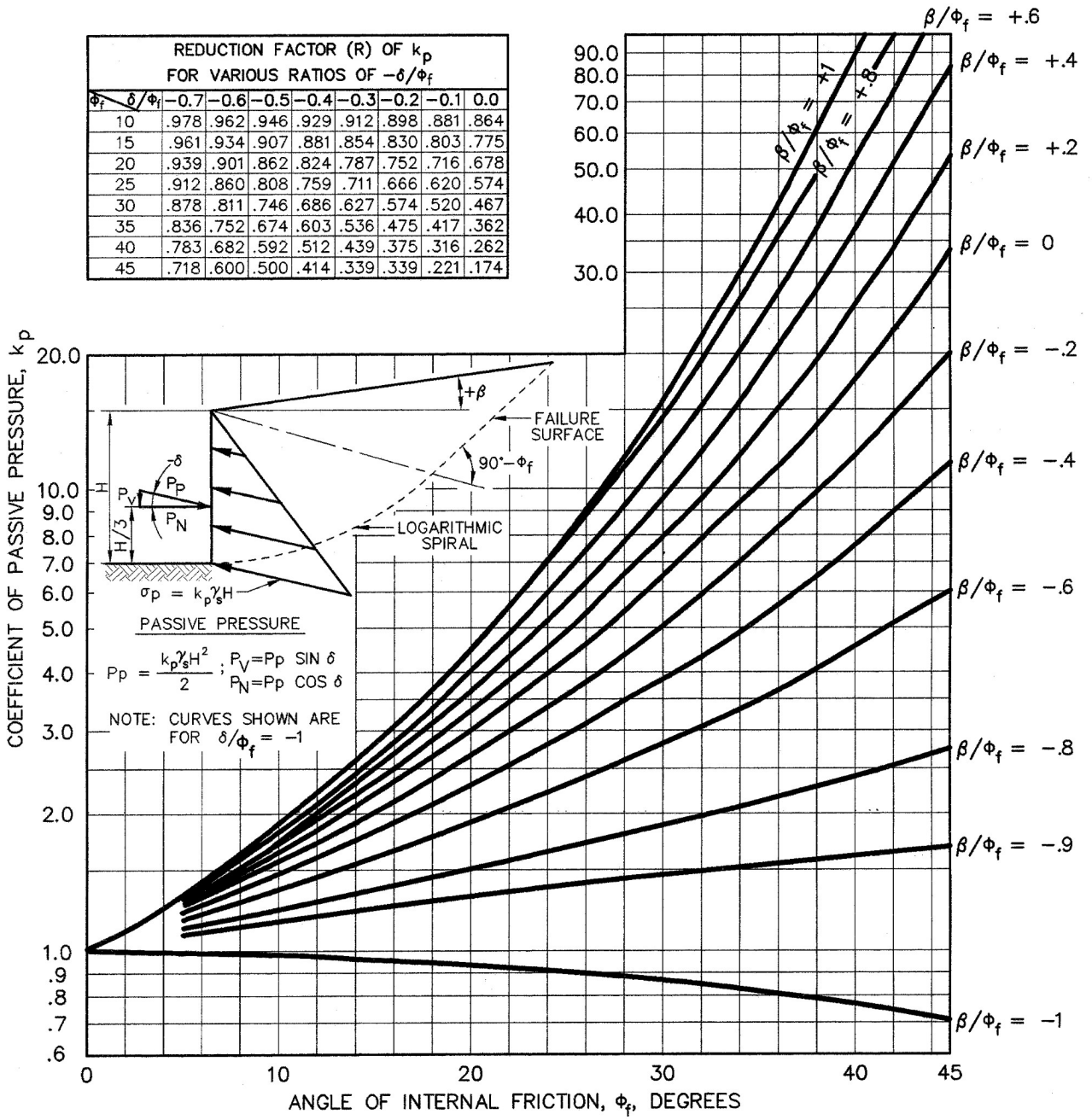


Figure 3.11.5.4-2—Computational Procedures for Passive Earth Pressures for Vertical Wall with Sloping Backfill (U.S. Department of the Navy, 1982a)

3.11.5.5—Equivalent-Fluid Method of Estimating Rankine Lateral Earth Pressures

The equivalent-fluid method may be used where Rankine earth pressure theory is applicable.

The equivalent-fluid method shall only be used where the backfill is free-draining. If this criterion cannot be satisfied, the provisions of Articles 3.11.3, 3.11.5.1 and 3.11.5.3 shall be used to determine horizontal earth pressure.

C3.11.5.5

Applicability of Rankine theory is discussed in Article C3.11.5.3.

Values of the unit weights of equivalent fluids are given for walls that can tolerate very little or no movement as well as for walls that can move as much as 1.0 in. in 20.0 ft. The concepts of equivalent fluid unit weights have taken into account the effect of soil creep on walls.

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Where the equivalent-fluid method is used, the basic earth pressure, p (ksf), may be taken as:

$$p = \gamma_{eq}\, z \qquad (3.11.5.5\text{-}1)$$

where:

γ_{eq} = equivalent fluid unit weight of soil, not less than 0.030 (kcf)
z = depth below surface of soil (ft)

The resultant lateral earth load due to the weight of the backfill shall be assumed to act at a height of $H/3$ above the base of the wall, where H is the total wall height, measured from the surface of the ground to the bottom of the footing.

Typical values for equivalent fluid unit weights for design of a wall of height not exceeding 20.0 ft may be taken from Table 3.11.5.5-1, where:

Δ = movement of top of wall required to reach minimum active or maximum passive pressure by tilting or lateral translation (ft)
H = height of wall (ft)
β = angle of fill to the horizontal (degrees)

The magnitude of the vertical component of the earth pressure resultant for the case of sloping backfill surface may be determined as:

$$P_v = P_h \tan\beta \qquad (3.11.5.5\text{-}2)$$

where:

$$P_h = 0.5\gamma_{eq}H^2 \qquad (3.11.5.5\text{-}3)$$

If the backfill qualifies as free-draining (i.e., granular material with <5 percent passing a No. 200 sieve), water is prevented from creating hydrostatic pressure.

For discussion on the location of the resultant of the lateral earth force see Article C3.11.5.1.

The values of equivalent fluid unit weight presented in Table 3.11.5.5-1 for Δ/H = 1/240 represent the horizontal component of active earth pressure based on Rankine earth pressure theory. This horizontal earth pressure is applicable for cantilever retaining walls for which the wall stem does not interfere with the sliding surface defining the Rankine failure wedge within the wall backfill (Figure C3.11.5.3-1). The horizontal pressure is applied to a vertical plane extending up from the heel of the wall base, and the weight of soil to the left of the vertical plane is included as part of the wall weight.

For the case of a sloping backfill surface in Table 3.11.5.5-1, a vertical component of earth pressure also acts on the vertical plane extending up from the heel of the wall.

Table 3.11.5.5-1—Typical Values for Equivalent Fluid Unit Weights of Soils

	Level Backfill		Backfill with β = 25 degrees	
Type of Soil	At-Rest γ_{eq} (kcf)	Active Δ/H = 1/240 γ_{eq} (kcf)	At-Rest γ_{eq} (kcf)	Active Δ/H = 1/240 γ_{eq} (kcf)
Loose sand or gravel	0.055	0.040	0.065	0.050
Medium dense sand or gravel	0.050	0.035	0.060	0.045
Dense sand or gravel	0.045	0.030	0.055	0.040

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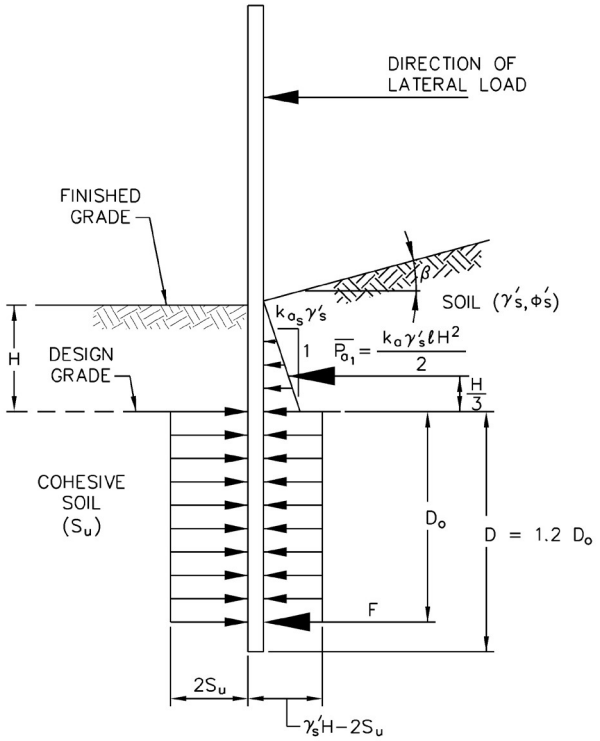


Figure 3.11.5.10-5—Unfactored Simplified Earth Pressure Distributions for Continuous Vertical Wall Elements Embedded in Cohesive Soil Modified after Teng (1962)

3.11.6—Surcharge Loads: *ES* and *LS*

The factored soil stress increase behind or within the wall caused by concentrated surcharge loads or stresses shall be the greater of (1) the unfactored surcharge loads or stresses multiplied by the specified load factor, *ES*, or (2) the factored loads for the structure as applied to the structural element causing the surcharge load, setting *ES* to 1.0. The load applied to the wall due to the structural element above the wall shall not be double factored.

C3.11.6

Concentrated surcharge loads induced by foundations are typically the result of dead load, live load, wind load, and possibly other loads that are associated with load factors other than *ES*. However, the controlling uncertainty in load prediction for surcharges is the transmission of the surcharge load through the soil to the wall or other structure below the surcharge. Hence, *ES* should be applied to the unfactored concentrated surcharge loads, unless the combined effect of the factored loads applicable to the foundation unit transmitting load to the top of the wall is more conservative. In this latter case, *ES* should be set equal to 1.0 and the factored footing loads used as the concentrated surcharge load in the wall design.

3.11.6.1—Uniform Surcharge Loads (*ES*)

Where a uniform surcharge is present, a constant horizontal earth pressure shall be added to the basic earth pressure. This constant earth pressure may be taken as:

$$\Delta_p = k_s q_s \qquad (3.11.6.1\text{-}1)$$

where:

Δ_p = constant horizontal earth pressure due to uniform surcharge (ksf)

C3.11.6.1

When the uniform surcharge is produced by an earth loading on the upper surface, the load factor for both vertical and horizontal components shall be taken as specified in Table 3.4.1-2 for earth surcharge.

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k_s = coefficient of earth pressure due to surcharge
q_s = uniform surcharge applied to the upper surface of the active earth wedge (ksf)

For active earth pressure conditions, k_s shall be taken as k_a, and for at-rest conditions, k_s shall be taken as k_o. Otherwise, intermediate values appropriate for the type of backfill and amount of wall movement may be used.

3.11.6.2—Point, Line, and Strip Loads (*ES*): Walls Restrained from Movement

The horizontal pressure, Δ_{ph} in ksf, on a wall resulting from a uniformly loaded strip parallel to the wall may be taken as:

$$\Delta_{ph} = \frac{2p}{\pi}\left[\delta - \sin\delta\cos(\delta + 2\alpha)\right] \qquad (3.11.6.2\text{-}1)$$

where:

p = uniform load intensity on strip parallel to wall (ksf)
α = angle specified in Figure 3.11.6.2-1 (rad)
δ = angle specified in Figure 3.11.6.2-1 (rad)

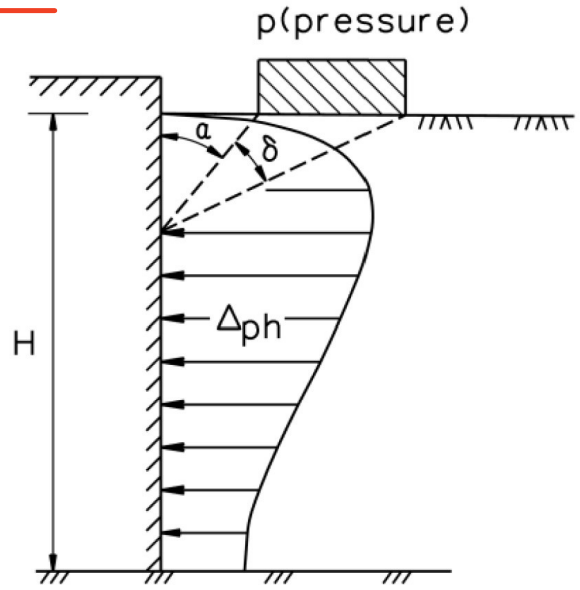


Figure 3.11.6.2-1—Horizontal Pressure on Wall Caused by a Uniformly Loaded Strip

The horizontal pressure, Δ_{ph} in ksf, on a wall resulting from a point load may be taken as:

$$\Delta_{ph} = \frac{P}{\pi R^2}\left[\frac{3ZX^2}{R^3} - \frac{R(1-2\nu)}{R+Z}\right] \qquad (3.11.6.2\text{-}2)$$

Wall movement needed to mobilize extreme active and passive pressures for various types of backfill can be found in Table C3.11.1-1.

C3.11.6.2

Eqs. 3.11.6.2-2, 3.11.6.2-3, 3.11.6.2-4, and 3.11.6.2-5 are based on the assumption that the wall does not move, i.e., walls which have a high degree of structural rigidity or restrained at the top combined with an inability to slide in response to applied loads. For flexible walls, this assumption can be very conservative. Additional guidance regarding the ability of walls to move is provided in Article C3.11.1.

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where:

P = point load (kip)
R = radial distance from point of load application to a point on the wall as specified in Figure 3.11.6.2-2 where $R = (x^2 + y^2 + z^2)^{0.5}$ (ft)
X = horizontal distance from back of wall to point of load application (ft)
Y = horizontal distance from point on the wall under consideration to a plane, which is perpendicular to the wall and passes through the point of load application measured along the wall (ft)
Z = vertical distance from point of load application to the elevation of a point on the wall under consideration (ft)
ν = Poisson's ratio (dim.)

The point on the wall does not have to lie in a plane which is perpendicular to the wall and passes through the point of load application.

Poisson's ratio for soils varies from about 0.25 to 0.49, with lower values more typical for granular and stiff cohesive soils and higher values more typical for soft cohesive soils.

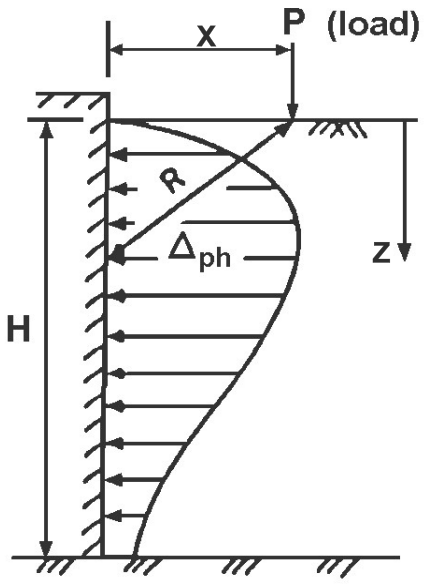


Figure 3.11.6.2-2—Horizontal Pressure on a Wall Caused by a Point Load

The horizontal pressure, Δ_{ph} in ksf, resulting from an infinitely long line load parallel to a wall may be taken as:

$$\Delta_{ph} = \frac{4Q}{\pi}\frac{X^2 Z}{R^4} \qquad (3.11.6.2\text{-}3)$$

where:

Q = load intensity in kip/ft

and all other notation is as defined above and shown in Figure 3.11.6.2-3.

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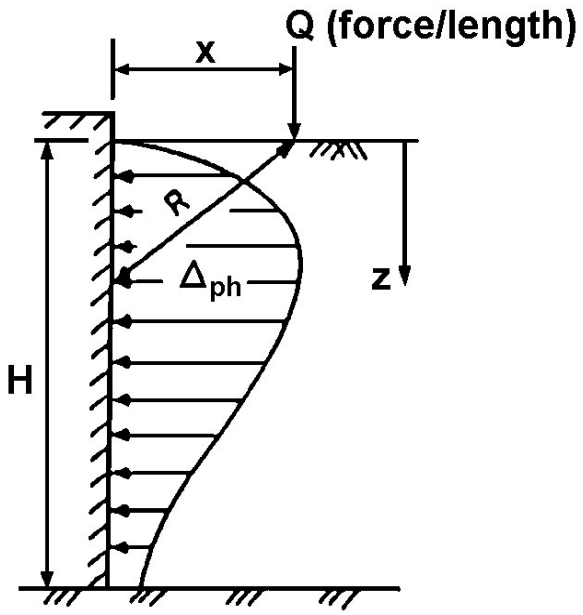


Figure 3.11.6.2-3—Horizontal Pressure on a Wall Caused by an Infinitely Long Line Load Parallel to the Wall

The horizontal pressure distribution, Δ_{ph} in ksf, on a wall resulting from a finite line load perpendicular to a wall may be taken as:

$$\Delta_{ph} = \frac{Q}{\pi Z}\left(\frac{1}{A^3}-\frac{1-2\nu}{A+\frac{Z}{X_2}}-\frac{1}{B^3}+\frac{1-2\nu}{B+\frac{Z}{X_1}}\right) \quad (3.11.6.2\text{-}4)$$

in which:

$$A = \sqrt{1+\left(\frac{Z}{X_2}\right)^2} \quad (3.11.6.2\text{-}5)$$

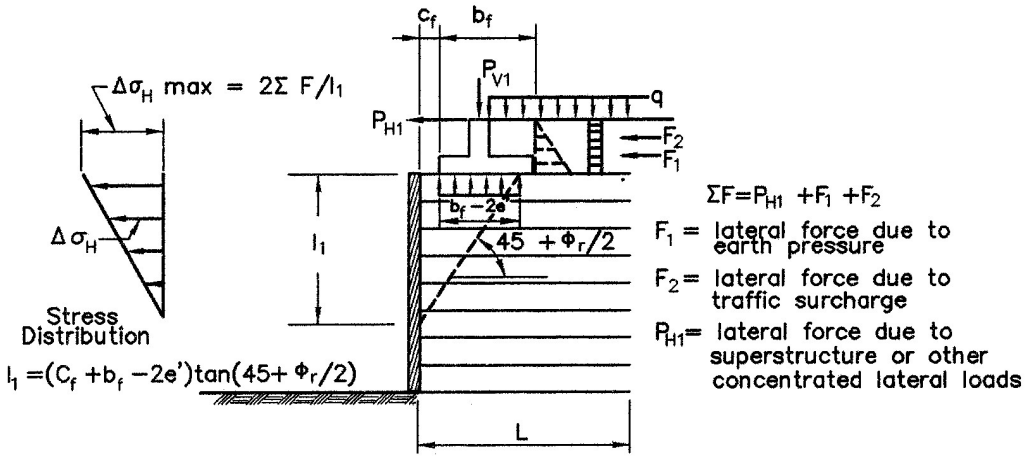
$$B = \sqrt{1+\left(\frac{Z}{X_1}\right)^2} \quad (3.11.6.2\text{-}6)$$

where:

X_1 = distance from the back of the wall to the start of the line load as specified in Figure 3.11.6.2-4 (ft)
X_2 = length of the line load (ft)
Z = depth from the ground surface to a point on the wall under consideration (ft)
ν = Poisson's Ratio (dim.)
Q = load intensity (kip/ft)

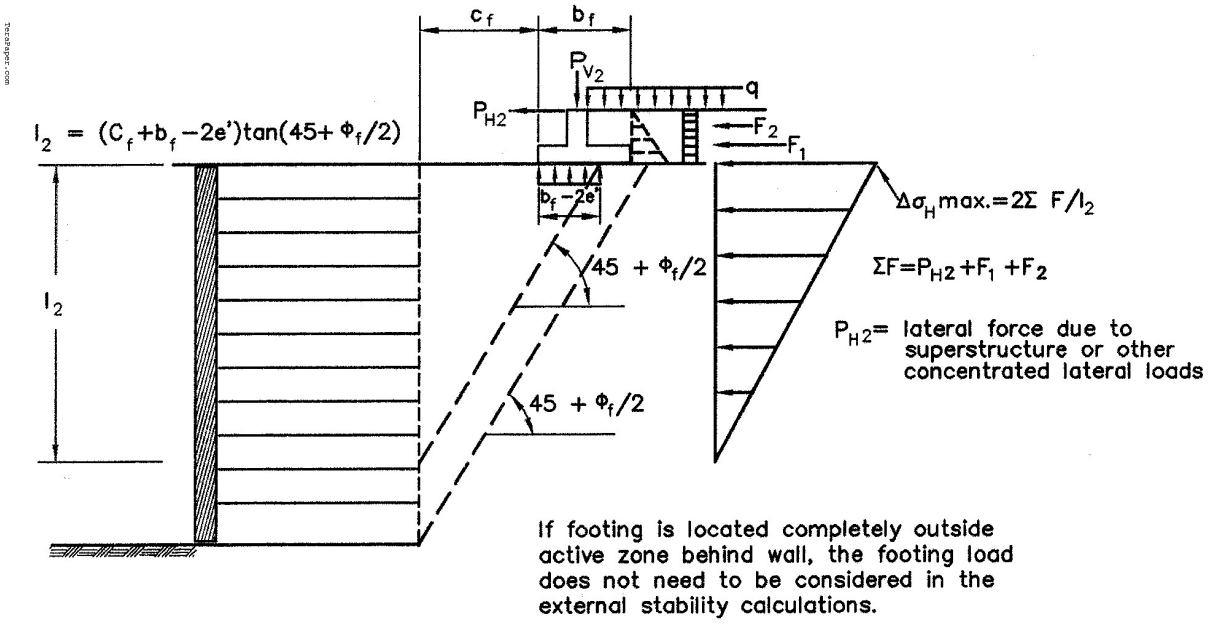
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Only Necessary if retaining wall is designed on left side of ramp



e′ = eccentricity of load on footing (see Figure 11.10.10.1-1 for example of how to calculate this)

a—Distribution of Stress for Internal Stability Calculations



b—Distribution of Stress for External Stability Calculations

Figure 3.11.6.3-2—Distribution of Stress from Concentrated Horizontal Loads

3.11.6.4—Live Load Surcharge (*LS*)

A live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall. If the surcharge is for a highway, the intensity of the load shall be consistent with the provisions of Article 3.6.1.2. If the surcharge is for other than a highway, the Owner shall specify and/or approve appropriate surcharge loads.

The increase in horizontal pressure due to live load surcharge may be estimated as:

C3.11.6.4

The tabulated values for h_{eq} were determined by evaluating the horizontal force against an abutment or wall from the pressure distribution produced by the vehicular live load of Article 3.6.1.2. The pressure distributions were developed from elastic half-space solutions using the following assumptions:

- Vehicle loads are distributed through a two-layer system consisting of pavement and soil subgrade

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$$\Delta_p = k\gamma_s h_{eq} \qquad (3.11.6.4\text{-}1)$$

where:

Δ_p = constant horizontal earth pressure due to live load surcharge (ksf)
γ_s = total unit weight of soil (kcf)
k = coefficient of lateral earth pressure
h_{eq} = equivalent height of soil for vehicular load (ft)

Equivalent heights of soil, h_{eq}, for highway loadings on abutments and retaining walls may be taken from Tables 3.11.6.4-1 and 3.11.6.4-2. Linear interpolation shall be used for intermediate wall heights.

The wall height shall be taken as the distance between the surface of the backfill and the bottom of the footing along the pressure surface being considered.

Table 3.11.6.4-1—Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

Abutment Height (ft)	h_{eq} (ft)
5.0	4.0
10.0	3.0
≥20.0	2.0

Table 3.11.6.4-2—Equivalent Height of Soil for Vehicular Loading on Retaining Walls Parallel to Traffic

Retaining Wall Height (ft)	h_{eq} (ft) Distance from wall backface to edge of traffic	
	0.0 ft	1.0 ft or Further
5.0	5.0	2.0
10.0	3.5	2.0
≥20.0	2.0	2.0

The load factor for both vertical and horizontal components of live load surcharge shall be taken as specified in Table 3.4.1-1 for live load surcharge.

3.11.6.5—Reduction of Surcharge

If the vehicular loading is transmitted through a structural slab, which is also supported by means other than earth, a corresponding reduction in the surcharge loads may be permitted.

- Poisson's ratio for the pavement and subgrade materials are 0.2 and 0.4, respectively
- Wheel loads were modeled as a finite number of point loads distributed across the tire area to produce an equivalent tire contact stress
- The process for equating wall moments resulting from the elastic solution with the equivalent surcharge method used a wall height increment of 0.25 ft.

The value of the coefficient of lateral earth pressure k is taken as k_o, specified in Article 3.11.5.2, for walls that do not deflect or move, or k_a, specified in Articles 3.11.5.3, 3.11.5.6 and 3.11.5.7, for walls that deflect or move sufficiently to reach minimum active conditions.

The analyses used to develop Tables 3.11.6.4-1 and 3.11.6.4-2 are presented in Kim and Barker (1998).

The values for h_{eq} given in Tables 3.11.6.4-1 and 3.11.6.4-2 are generally greater than the traditional 2.0 ft of earth load historically used in the AASHTO specifications, but less than those prescribed in previous editions (i.e., before 1998) of this specification. The traditional value corresponds to a 20.0-kip single unit truck formerly known as an H10 truck, Peck et al. (1974). This partially explains the increase in h_{eq} in previous editions of this specification. Subsequent analyses, i.e., Kim and Barker (1998) show the importance of the direction of traffic, i.e., parallel for a wall and perpendicular for an abutment on the magnitude of h_{eq}. The magnitude of h_{eq} is greater for an abutment than for a wall due to the proximity and closer spacing of wheel loads to the back of an abutment compared to a wall.

The backface of the wall should be taken as the pressure surface being considered. Refer to Article C11.5.5 for application of surcharge pressures on retaining walls.

C3.11.6.5

This Article relates primarily to approach slabs which are supported at one edge by the backwall of an abutment, thus transmitting load directly thereto.

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For the Extreme Event I limit state, the peak seismic lateral pressures acting on the wall should not be based on the maximum ground water elevation due to the low probability that the design peak seismic acceleration would be combined with the maximum ground water level. Instead, it is more appropriate to use the time-averaged mean groundwater elevation or a reasonable engineering estimate of this elevation.

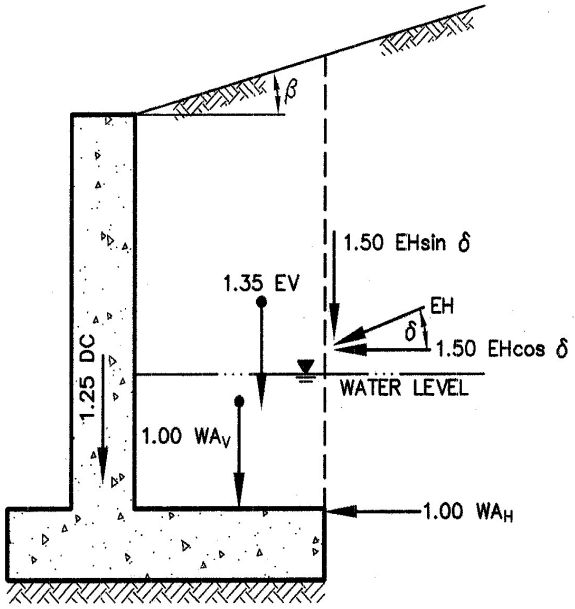


Figure C11.5.6-1—Typical Application of Load Factors for Bearing Resistance

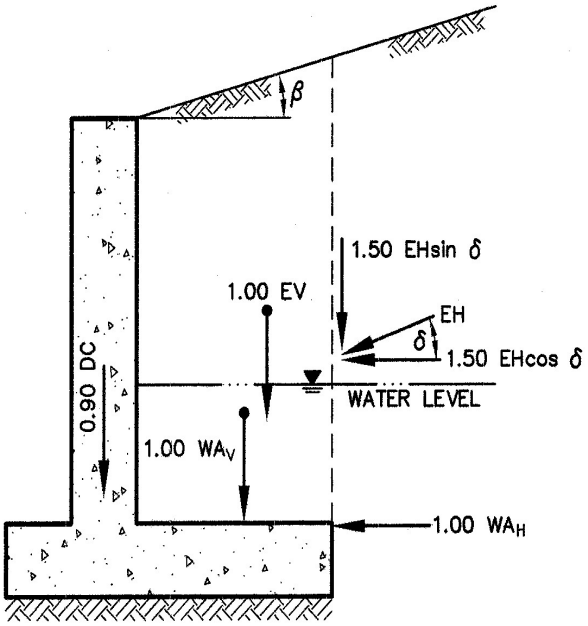
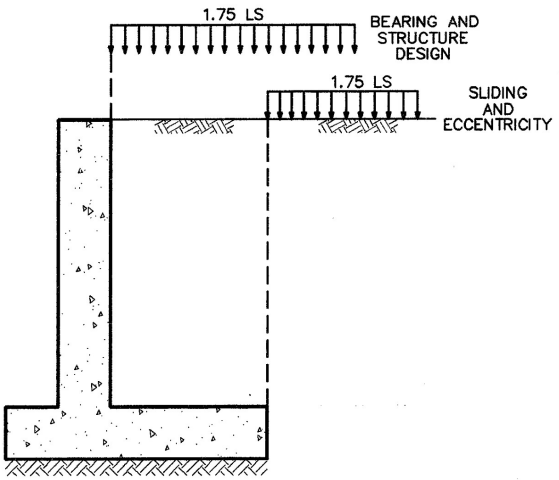


Figure C11.5.6-2—Typical Application of Load Factors for Sliding and Eccentricity

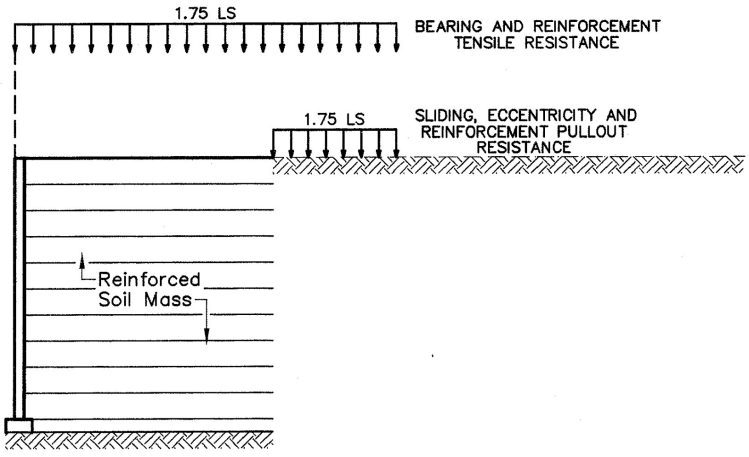
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(a) CONVENTIONAL STRUCTURE



(b) MECHANICALLY STABILIZED EARTH STRUCTURE

Figure C11.5.6-3—Typical Application of Live Load Surcharge

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The foundation resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this Article.

10.5.5.2.2—Spread Footings

The resistance factors provided in Table 10.5.5.2.2-1 shall be used for strength limit state design of spread footings, with the exception of the deviations allowed for local practices and site specific considerations in Article 10.5.5.2.

Note that not all of the resistance factors provided in this Article have been derived using statistical data from which a specific β value can be estimated, since such data were not always available. In those cases, where data were not available, resistance factors were estimated through calibration by fitting to past allowable stress design safety factors, e.g., the AASHTO *Standard Specifications for Highway Bridges* (2002).

Additional discussion regarding the basis for the resistance factors for each foundation type and limit state is provided in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5. Additional, more detailed information on the development of the resistance factors for foundations provided in this Article, and a comparison of those resistance factors to previous Allowable Stress Design practice, e.g., AASHTO (2002), is provided in Allen (2005).

Scour design for the design flood must satisfy the requirement that the factored foundation resistance after scour is greater than the factored load determined with the scoured soil removed. The resistance factors will be those used in the Strength Limit State, without scour.

C10.5.5.2.2

Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

Method/Soil/Condition			Resistance Factor
Bearing Resistance	φ_b	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *CPT*	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *SPT*	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	φ_τ	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

The resistance factors in Table 10.5.5.2.2-1 were developed using both reliability theory and calibration by fitting to Allowable Stress Design (ASD). In general, ASD safety factors for footing bearing capacity range from 2.5 to 3.0, corresponding to a resistance factor of approximately 0.55 to 0.45, respectively, and for sliding, an ASD safety factor of 1.5, corresponding to a resistance factor of approximately 0.9. Calibration by fitting to ASD controlled the selection of the resistance factor in cases where statistical data were limited in quality or quantity.

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Table 10.5.5.2.4-1—Resistance Factors for Geotechnical Resistance of Drilled Shafts

Method/Soil/Condition			Resistance Factor
Nominal Axial Compressive Resistance of Single-Drilled Shafts, φ_{stat}	Side resistance in clay	α-method (Brown et al., 2010)	0.45
	Tip resistance in clay	Total Stress (Brown et al., 2010)	0.40
	Side resistance in sand	β-method (Brown et al., 2010)	0.55
	Tip resistance in sand	Brown et al. (2010)	0.50
	Side resistance in cohesive IGMs	Brown et al. (2010)	0.60
	Tip resistance in cohesive IGMs	Brown et al. (2010)	0.55
	Side resistance in rock	Kulhawy et al. (2005) Brown et al. (2010)	0.55
	Side resistance in rock	Carter and Kulhawy (1988)	0.50
	Tip resistance in rock	Canadian Geotechnical Society (1985) Pressuremeter Method (Canadian Geotechnical Society, 1985) Brown et al. (2010)	0.50
Block Failure, φ_{b1}	Clay		0.55
Uplift Resistance of Single-Drilled Shafts, φ_{up}	Clay	α-method (Brown et al., 2010)	0.35
	Sand	β-method (Brown et al., 2010)	0.45
	Rock	Kulhawy et al. (2005) Brown et al. (2010)	0.40
Group Uplift Resistance, φ_{ug}	Sand and clay		0.45
Horizontal Geotechnical Resistance of Single Shaft or Shaft Group	All materials		1.0
Static Load Test (compression), φ_{load}	All Materials		0.70
Static Load Test (uplift), φ_{upload}	All Materials		0.60

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Table 11.5.7-1—Resistance Factors for Permanent Retaining Walls

Wall-Type and Condition		Resistance Factor
Nongravity Cantilevered and Anchored Walls		
Axial compressive resistance of vertical elements		Article 10.5 applies
Passive resistance of vertical elements		0.75
Pullout resistance of anchors [1]	• Cohesionless (granular) soils • Cohesive soils • Rock	0.65 [1] 0.70 [1] 0.50 [1]
Pullout resistance of anchors [2]	• Where proof tests are conducted	1.0 [2]
Tensile resistance of anchor tendon	• Mild steel (e.g., ASTM A615 bars) • High strength steel (e.g., ASTM A722 bars)	0.90 [3] 0.80 [3]
Flexural capacity of vertical elements		0.90
Mechanically Stabilized Earth Walls, Gravity Walls, and Semigravity Walls		
Bearing resistance	• Gravity and semigravity walls • MSE walls	0.55 0.65
Sliding		1.0
Tensile resistance of metallic reinforcement and connectors	Strip reinforcements [4] • Static loading Grid reinforcements [4] [5] • Static loading	 0.75 0.65
Tensile resistance of geosynthetic reinforcement and connectors	• Static loading	0.90
Pullout resistance of tensile reinforcement	• Static loading	0.90
Prefabricated Modular Walls		
Bearing		Article 10.5 applies
Sliding		Article 10.5 applies
Passive resistance		Article 10.5 applies

(1) Apply to presumptive ultimate unit bond stresses for preliminary design only in Article C11.9.4.2.

(2) Apply where proof test(s) are conducted on every production anchor to a load of 1.0 or greater times the factored load on the anchor.

(3) Apply to maximum proof test load for the anchor. For mild steel apply resistance factor to F_y. For high-strength steel apply the resistance factor to guaranteed ultimate tensile strength.

(4) Apply to gross cross-section less sacrificial area. For sections with holes, reduce gross area in accordance with Article 6.8.3 and apply to net section less sacrificial area.

(5) Applies to grid reinforcements connected to a rigid facing element, e.g., a concrete panel or block. For grid reinforcements connected to a flexible facing mat or which are continuous with the facing mat, use the resistance factor for strip reinforcements.

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11.6.2—Movement and Stability at the Service Limit State

11.6.2.1—Abutments

The provisions of Articles 10.6.2.4, 10.6.2.5, 10.7.2.3 through 10.7.2.5, 10.8.2.2 through 10.8.2.4, and 11.5.2 shall apply as applicable.

11.6.2.2—Conventional Retaining Walls

The provisions of Articles 10.6.2.4, 10.6.2.5, 10.7.2.3 through 10.7.2.5, 10.8.2.2 through 10.8.2.4, and 11.5.2 apply as applicable.

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11.6.2.3—Overall Stability

The overall stability of the retaining wall, retained slope and foundation soil or rock shall be evaluated for all walls using limiting equilibrium methods of analysis. The overall stability of temporary cut slopes to facilitate construction shall also be evaluated. Special exploration, testing and analyses may be required for bridge abutments or retaining walls constructed over soft deposits.

The evaluation of overall stability of earth slopes with or without a foundation unit should be investigated at the Service I Load Combination and an appropriate resistance factor. In lieu of better information, the resistance factor, ϕ, may be taken as:

- Where the geotechnical parameters are well defined, and the slope does not support or contain a structural element..... 0.75
- Where the geotechnical parameters are based on limited information, or the slope contains or supports a structural element 0.65

C11.6.2.2

For a conventional reinforced concrete retaining wall, experience suggests that differential wall settlements on the order of 1 in 500 to 1 in 1,000 may overstress the wall.

C11.6.2.3

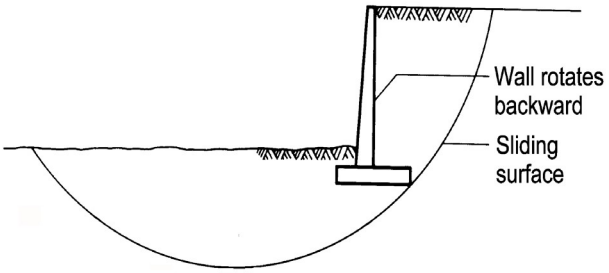


Figure C11.6.2.3-1—Retaining Wall Overall Stability Failure

Figure C11.6.2.3-1 shows a retaining wall overall stability failure. Overall stability is a slope stability issue, and, therefore, is considered a service limit state check.

The Modified Bishop, simplified Janbu or Spencer methods of analysis may be used.

Soft soil deposits may be subject to consolidation and/or lateral flow which could result in unacceptable long-term settlements or horizontal movements.

With regard to selection of a resistance factor for evaluation of overall stability of walls, examples of structural elements supported by a wall that may justify the use of the 0.65 resistance factor include a bridge or pipe arch foundation, a building foundation, a pipeline, a critical utility, or another retaining wall. If the structural element is located beyond the failure surface for external stability behind the wall illustrated conceptually in Figure 11.10.2-1, or if the wall does not support a structural element, a resistance factor of 0.75 may be used.

Available slope stability programs produce a single factor of safety, *FS*. The specified resistance factors are essentially the inverse of the *FS* that should be targeted in the slope stability program.

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Regarding impact to the wall itself, differential settlement along the length of the wall and to some extent from front to back of wall is the best indicator of the potential for retaining wall structural damage or overstress. Wall facing stiffness and ability to adjust incrementally to movement affect the ability of a given wall system to tolerate differential movements. The total and differential vertical deformation of a retaining wall should be small for rigid gravity and semigravity retaining walls, and for soldier pile walls with a cast-in-place facing. For walls with anchors, any downward movement can cause significant stress relaxation of the anchors.

MSE walls can tolerate larger total and differential vertical deflections than rigid walls. The amount of total and differential vertical deflection that can be tolerated depends on the wall facing material, configuration and timing of facing construction. A cast-in-place facing has the same vertical deformation limitations as the more rigid retaining wall systems. However, an MSE wall with a cast-in-place facing can be specified with a waiting period before the cast-in-place facing is constructed so that vertical (as well as horizontal) deformations have time to occur. An MSE wall with welded wire or geosynthetic facing can tolerate the most deformation. An MSE wall with multiple precast concrete panels cannot tolerate as much vertical deformation as flexible welded wire or geosynthetic facings because of potential damage to the precast panels and unsightly panel separation.

11.5.3—Strength Limit State

Abutments, walls, and piers shall be investigated at the strength limit states using Eq. 1.3.2.1-1 for:

- Bearing resistance failure,
- Lateral sliding,
- Loss of base contact due to eccentric loading,
- Pullout failure of anchors or soil reinforcements, and
- Structural failure.

11.5.4—Extreme Event Limit State

11.5.4.1—General Requirements

Abutments, walls, and piers shall be investigated at the extreme event limit state for:

- Overall stability failure,
- Bearing resistance failure,
- Lateral sliding,
- Loss of base contact due to eccentric loading,

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11.6.3—Bearing Resistance and Stability at the Strength Limit State

11.6.3.1—General

Abutments and retaining walls shall be proportioned to ensure stability against bearing capacity failure, overturning, and sliding. Safety against deep-seated foundation failure shall also be investigated, in accordance with the provisions of Article 10.6.2.5.

11.6.3.2—Bearing Resistance

Bearing resistance shall be investigated at the strength limit state using factored loads and resistances, assuming the following soil pressure distributions:

- Where the wall is supported by a soil foundation:

  the vertical stress shall be calculated assuming a uniformly distributed pressure over an effective base area as shown in Figure 11.6.3.2-1.

  The vertical stress shall be calculated as follows:

$$\sigma_v = \frac{\sum V}{B - 2e} \qquad (11.6.3.2\text{-}1)$$

  where:

  ΣV = the summation of vertical forces, and the other variables are as defined in Figure 11.6.3.2-1

- Where the wall is supported by a rock foundation:

  the vertical stress shall be calculated assuming a linearly distributed pressure over an effective base area as shown in Figure 11.6.3.2-2. If the resultant is within the middle one-third of the base:

$$\sigma_{vmax} = \frac{\sum V}{B}\left(1 + 6\frac{e}{B}\right) \qquad (11.6.3.2\text{-}2)$$

$$\sigma_{vmin} = \frac{\sum V}{B}\left(1 - 6\frac{e}{B}\right) \qquad (11.6.3.2\text{-}3)$$

  where the variables are as defined in Figure 11.6.3.2-2. If the resultant is outside the middle one-third of the base:

$$\sigma_{vmax} = \frac{2\sum V}{3[(B/2) - e)]} \qquad (11.6.3.2\text{-}4)$$

$$\sigma_{vmin} = 0 \qquad (11.6.3.2\text{-}5)$$

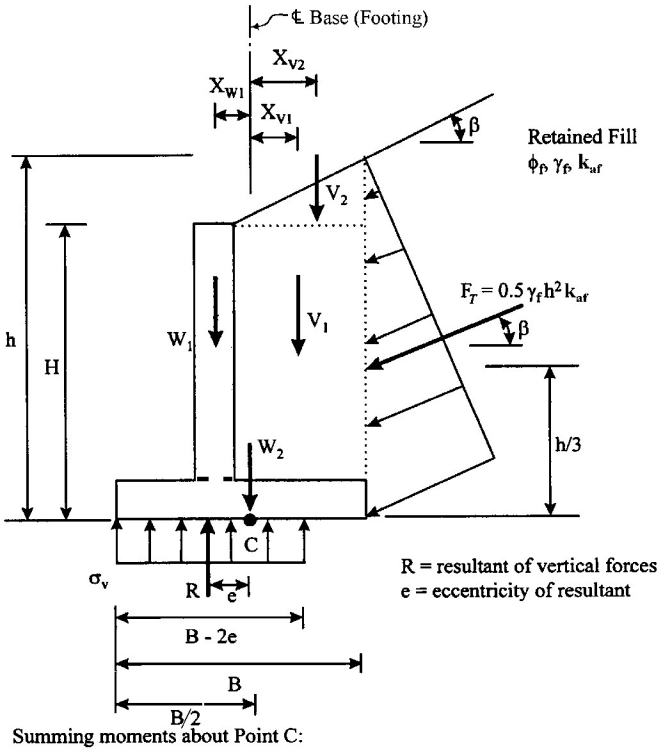
  where the variables are as defined in Figure 11.6.3.2-2.

C11.6.3.2

See Figure 11.10.10.1-1 for an example of how to calculate the vertical bearing stress where the loading is more complex. Though this figure shows the application of superposition principles to mechanically stabilized earth walls, these principles can also be directly applied to conventional walls.

See Article C11.5.5 for application of load factors for bearing resistance and eccentricity.

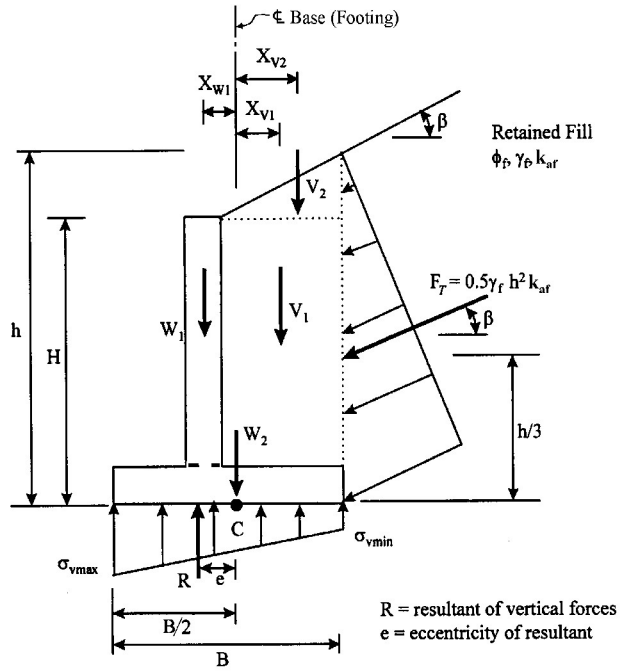
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Summing moments about Point C:

$$e = \frac{\left(F_T Cos\beta\right)h/3 - \left(F_T \sin\beta\right)B/2 - V_1X_{V_1} - V_2X_{V2} + W_1X_{W1}}{V_1 + V_2 + W_1 + W_2 + F_T \sin\beta}$$

Figure 11.6.3.2-1—Bearing Stress Criteria for Conventional Wall Foundations on Soil



If e > B/6, σ_{vmin} will drop to zero, and as "e" increases, the portion of the heel of the footing which has zero vertical stress increases.

Summing moments about Point C:

$$e = \frac{\left(F_T Cos\beta\right)h/3 - \left(F_T \sin\beta\right)B/2 - V_1X_{V_1} - V_2X_{V2} + W_1X_{W1}}{V_1 + V_2 + W_1 + W_2 + F_T \sin\beta}$$

Figure 11.6.3.2-2—Bearing Stress Criteria for Conventional Wall Foundations on Rock

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Geotechnical Aspects of Pavements Reference Manual

https://www.fhwa.dot.gov/engineering/geotech/pubs/05037/05a.cfm

Table 5-17. Typical compacted densities and optimum moisture contents for USCS soil types (after Carter and Bentley, 1991).

Soil Description	USCS Class	Compacted Dry Unit Weight		Optimum Moisture Content (%)
		(lb/ft3)	(kN/m3)	
Gravel/sand mixtures:				
well-graded, clean	GW	125-134	19.6-21.1	8-11
poorly-graded, clean	GP	115-125	18.1-19.6	11-14
well-graded, small silt content	GM	119-134	18.6-21.1	8-12
well-graded, small clay content	GC	115-125	18.1-19.6	9-14
Sands and sandy soils:				
well-graded, clean	SW	109-131	17.2-20.6	9-16
poorly-graded, small silt content	SP	94-119	15.7-18.6	12-21
well-graded, small silt content	SM	109-125	17.2-19.6	11-16
well-graded, small clay content	SC	106-125	16.7-19.6	11-19
Fined-grained soils of low plasticity:				
silts	ML	94-119	14.7-18.6	12-24
clays	CL	94-119	14.7-18.6	12-24
organic silts	OL	81-100	12.7-15.7	21-33
Fine-grained soils of high plasticity:				
silts	MH	69-94	10.8-14.7	24-40
clays	CH	81-106	12.7-18.6	19-36
organic clays	OH	66-100	10.3-15.7	21-45

Table 5-18. Typical compacted densities and optimum moisture contents for AASHTO soil types (after Carter and Bentley, 1991).

Soil Description	AASHTO Class	Compacted Dry Unit Weight		Optimum Moisture Content (%)
		(lb/ft3)	(kN/m3)	
Well-graded gravel/sand mixtures	A-1	115-134	18.1-21.1	5-15
Silty or clayey gravel and sand	A-2	109-134	17.2-21.1	9-18
Poorly-graded sands	A-3	100-119	15.7-18.6	5-12
Low plasticity silty sands and gravels	A-4	94-125	14.7-19.6	10-20
Diatomaceous or micaceous silts	A-5	84-100	13.2-15.7	20-35
Plastic clay, sandy clay	A-6	94-119	14.7-18.6	10.30
Highly plastic clay	A-7	81-115	12.7-18.1	15-35

Table 5-24. Estimation of swell potential (Holtz and Gibbs, 1956).

% finer than 0.001mm	Atterberg Limits		Probable expansion, % total volume change*	Potential for expansion
	PI (%)	SL (%)		
> 28	> 35	< 11	> 30	Very high
20-31	25-41	7-12	20-30	High
13-23	15-28	10-16	10-30	Medium
< 15	< 18	>15	< 10	Low

Table 4-14. Summary of soil characteristics as a pavement material.
(from NCHRP 1-37A Pavement Design Guide)

Major Divisions		Name	Subgrade Strength when Not Subject to Frost Action	Potential Frost Action	Compressibility & Expansion	Drainage Characteristics
Gravel And Gravelly Soils	GW	Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	None to very slight	Almost none	Excellent
	GP	Poorly graded gravels or gravel-sand mixtures, little or no fines	Good to excellent	None to very slight	Almost none	Excellent
	*d GM---u	Silty gravels, gravel-sand silt mixtures	Good to excellent	Slight to medium	Very slight	Fair to poor
			Good	Slight to medium	Slight	Poor to practically impervious
	GC	Clayey gravels, gravel-sand-clay mixture	Good	Slight to medium	Slight	Poor to practically impervious
Sand and Sandy Soils	SW	Well-graded sands or gravelly sands, little or no fines	Good	None to very slight	Almost none	Excellent
	SP	Poorly graded sands or gravelly sands, little or no fines	Fair to good	None to very slight	Almost none	Excellent
	*d SM---u	Silty sands, sand-silt mixtures	Fair to good	Slight to high	Very slight	Fair to poor
			Fair	Slight to high	Slight to medium	Poor to practically impervious
	SC	Clayey sands, sand-clay mixtures	Poor to fair	Slight to high	Slight to medium	Poor to practically impervious
Silts & Clays with Liquid Limit Less Than 50	ML	Inorganic silts & very fine sand, rock flour, silty or clayey fine sand or clayey silts with slight plasticity	Poor to Fair	Medium to Very High	Slight to medium	Fair to Poor
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Poor to Fair	Medium to High	Slight to medium	Practically Impervious
	OL	Organic silts & organic silt-clays or low plasticity	Poor	Medium to High	Medium to high	Poor
Silts & Clays with Liquid Limit Greater Than 50	MH	Inorganic silts, micaceous or diatomaceous fine sand or silty soils, elastic silts	Poor	Medium to Very High	High	Fair to Poor

Table 2. Engineering properties of bulk soil samples

Route	Ky 994	Ky 11	Pennyrile Parkway	Ky 10	U.S. 25	U.S.31W
County	McCracken	Lee	Henderson	Campbell	Fayette	Hardin
Soil Description	Gray Silt	Brown Silty Clay	Brown Silt	Greenish-gray Clay	Red-Brownish Clay	Red Clay
Liquid Limit (%)	26.5	34.4	28.2	41.1	47.7	52.3
Plasticity Index (%)	5.8	7.6	8.5	18.9	19.0	26.7
Specific Gravity	2.64	2.76	2.69	2.76	2.89	2.73
Percent Finer (%):						
No. 10 sieve	98.5	90.6	100.0	95.9	96.4	95.1
No. 200 sieve	80.3	70.1	99.4	91.4	83.7	79.7
0.002mm	21.8	21.2	20.0	40.8	50.5	52.8
Classification:						
AASHTO	A-4(3)	A-4(4)	A-4(7)	A-7-6(18)	A-7-6(17)	A-7-6(22)
Unified	ML-CL	ML	CL	CL	CL	CH
CUw/PP[1]: Effective Stress Parameter, ϕ' (deg.)	32.4	28.1	33.4	24.1	28.1	21.0
Effective Stress Parameter, c' (psf)	0	372.8	0	431.2	324.4	737.3
KYCBR[2]-as compacted	27.3	12.1	22.1	17.1	51.8	12.3
KYCBR[3]—soaked according to standard	3.9	3.6	10.8	2.0	6.6	4.6
KYCBR[4]—soaked until swell ceases	6.9	3.9	8.2	1.9	1.0	5.4
AASHTO[5]-as compacted	17.9	30.6	12.5	11.6	20.0	11.1
AASHTO[3]-soaked according to standard	4.2	2.9	9.1	1.6	1.3	1.2
AASHTO[4]-soaked until swell ceases	7.1	3.1	11.2	0.4	1.2	1.3
AASHTO--5 % Hydrated Lime		21.0		9.6	22.5	30.3
AASHTO—10 % Cement	71.5		18.6			

1. Consolidated-undrained triaxial compression tests with pore pressure measurements. Specimens compacted to 95 % of maximum dry density and optimum moisture content obtained from AAHTO T-99.
2. CBR test performed on the "as compacted" Specimen.
3. CBR specimen allowed to soak and swell according to criteria specified by the Ky CBR method and the AASHTO method, T-193-99.
4. CBR specimens allowed to swell until swelling ceased.

https://uknowledge.uky.edu/cgi/viewcontent.cgi?article=1115&context=ktc_researchreports

While this is not the exact soil we have, this is a good one to compare our standard parameters to because its LL, PI, and soil description are similar

Carter, Michael Bentley, Stephen P.. (2016). *Soil Properties and Their Correlations (2nd Edition).* John Wiley & Sons. Retrieved from https://app.knovel.com/hotlink/toc/id:kpSPTCE001/soil-properties-their/soil-properties-their

Table 6.3 Typical values of the effective angle of shearing resistance of compacted clays.

Soil description	Class*	φ' (°)
Silty clays, sand-silt mix	SM	34
Clayey sands, sand-clay mix	SC	31
Silts and clayey silts	ML	32
Clays of low plasticity	CL	28
Clayey silts, elastic silts	MH	25
Clays of high plasticity	CH	19

*Unified classification system.

https://www.tugraz.at/fileadmin/user_upload/Institute/IAG/Files/24_NAVFAC_DM_7_02.pdf

FOUNDATIONS ANTD EARTH STRUCTURES Naval Facilities Engineering Command SEPTEMBER 1986

TABLE 1
Ultimate Friction Factors and Adhesion for Dissimilar Materials

Interface Materials	Friction factor, tan [delta]	Friction angle [delta] degrees
Mass concrete on the following foundation materials:		
Clean sound rock	0.70	35
Clean gravel, gravel-sand mixtures, coarse sand	0.55 to 0.60	29 to 31
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	0.45 to 0.55	24 to 29
Clean fine sand, silty or clayey fine to medium sand	0.35 to 0.45	19 to 24
Fine sandy silt, nonplastic silt	0.30 to 0.35	17 to 19
Very stiff and hard residual or preconsolidated clay	0.40 to 0.50	22 to 26
Medium stiff and stiff clay and silty clay	0.30 to 0.35	17 to 19
(Masonry on foundation materials has same friction factors.)		
Steel sheet piles against the following soils:		
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	0.40	22
Clean sand, silty sand-gravel mixture, single size hard rock fill	0.30	17
Silty sand, gravel or sand mixed with silt or clay	0.25	14
Fine sandy silt, nonplastic silt	0.20	11
Formed concrete or concrete sheet piling against the following soils:		
Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	0.40 to 0.50	22 to 26
Clean sand, silty sand-gravel mixture, single size hard rock fill	0.30 to 0.40	17 to 22
Silty sand, gravel or sand mixed with silt or clay	0.30	17
Fine sandy silt, nonplastic silt	0.25	14
Various structural materials:		
Masonry on masonry, igneous and metamorphic rocks:		
Dressed soft rock on dressed soft rock	0.70	35
Dressed hard rock on dressed soft rock	0.65	33
Dressed hard rock on dressed hard rock	0.55	29
Masonry on wood (cross grain)	0.50	26
Steel on steel at sheet pile interlocks	0.30	17

Interface Materials (Cohesion)	Adhesion c+a, (psf)
Very soft cohesive soil (0 - 250 psf)	0 - 250
Soft cohesive soil (250 - 500 psf)	250 - 500
Medium stiff cohesive soil (500 - 1000 psf)	500 - 750
Stiff cohesive soil (1000 - 2000 psf)	750 - 950
Very stiff cohesive soil (2000 - 4000 psf)	950 - 1,300



NOTES:

1.) BASE MAP: USGS QUADRANGLE (LOVELL, TENNESSEE)

GEOServices, LLC-Geotechnical and Materials Engineers
2561 Willow Point Way
Knoxville, Tennessee 37931
Office: 865-539-8242
Fax: 865-539-8252

SITE VICINITY MAP

Cowards Mill

Subdivision

KNOXILLE, TENNESSEE

DRAWN BY:	MTB
APPROVED BY:	MBH
SCALE:	N.T.S.
JOB NO.:	21-18148
DATE:	3-28-2018

FIGURE

1



FIGURE 2

GEOServices, LLC-Geotechnical and Materials Engineers

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Knoxville, Tennessee 37931

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Fax: 865-539-8252

DRAWN BY:	MTB
APPROVED BY:	MBH
SCALE:	N.T.S.
JOB NO.:	21-18148
DATE:	3-28-2018

BORING LOCATION PLAN

Cowards Mill

Subdivision

KNOXVILLE, TENNESSEE

NOTES:

1.) BORING LOCATIONS ARE SHOWN IN GENERAL ARRANGMENT ONLY.
2.) DO NOT USE BORING LOCATIONS FOR DETERMINATIONS OF DISTANCES OR QUANTITIES.
3.) BASE MAP PROVIDED BY: Ball Homes

LOCATION OF SOIL TEST BORING

LOCATION OF AUGER PROBE SOUNDINGS

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Coward Mill Subdivison
Knoxville, Tennessee
GEOServices Project # 21-18148

LOG OF BORING B-1
SHEET 1 OF 2

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-1 DRY ON COMPLETION ? Yes

DATE March 9, 2018 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 35.0 FT. 10.7 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 35.0 FT. ELEV. -35.0 FT.

BORING ADVANCED BY: POWER AUGERING X

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. ____ FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. ____ FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. ____ FT.

PROPOSED FFE: ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											Topsoil (3 Inches)
		1.0	2.5	1	SS	8				19.4	
2.5	-2.5										
		3.5	5.0	2	SS	12				17.8	Lean CLAY (CL) - with trace root structure in the upper 3', chert fragments from 12' to 22', and abundant chert fragments at depth - reddish brown - moist - firm to very stiff (RESIDUUM)
5.0	-5.0										
		6.0	7.5	3	SS	10				18.8	
7.5	-7.5										
		8.5	10.0	4	SS	19*		48	28	32.8	
10.0	-10.0										
12.5	-12.5										
		13.5	15.0	5	SS	16*		80	54	23.7	Fat CLAY (CH) - with trace root structure in the upper 3', chert fragments from 12' to 22', and abundant chert fragments at depth - reddish brown - moist - firm to very stiff (RESIDUUM)
15.0	-15.0										
17.5	-17.5										
		18.5	20.0	6	SS	26*				17.7	
20.0	-20.0										

Continued

REMARKS: (*) indicate N-values elevated due to the increased rock content

GEOS
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Coward Mill Subdivison
Knoxville, Tennessee
GEOServices Project # 21-18148

LOG OF BORING B-1
SHEET 2 OF 2

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-1 DRY ON COMPLETION ? Yes

DATE March 9, 2018 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 35.0 FT. 10.7 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 35.0 FT. ELEV. -35.0 FT.

BORING ADVANCED BY: POWER AUGERING X

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. ____ FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. ____ FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. ____ FT.

PROPOSED FFE: ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											(contiuned)
22.5	-22.5										
		23.5	25.0	7	SS	7				41.5	
25.0	-25.0										
27.5	-27.5										Fat CLAY (CH) - with trace root structure in the upper 3', chert fragments from 12' to 22', and abundant chert fragments at depth - reddish brown - moist - firm to very stiff (RESIDUUM)
		28.5	30.0	8	SS	16				31.7	
30.0	-30.0										
32.5	-32.5										
		33.5	35.0	9	SS	8				45.1	
35.0	-35.0										Boring Terminated at 35 Feet
37.5	-37.5										
40.0	-40.0										

REMARKS: (*) indicate N-values elevated due to the increased rock content

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LOG OF BORING B-2
SHEET 1 OF 2

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-2 DRY ON COMPLETION ? Yes

DATE March 9, 2018 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 30.0 FT. 9.1 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 30.0 FT. ELEV. -30.0 FT.

BORING ADVANCED BY: POWER AUGERING X

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. ____ FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. ____ FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. ____ FT.

PROPOSED FFE: ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											Topsoil (3 Inches)
		1.0	2.5	1	SS	11					
2.5	-2.5										
		3.5	5.0	2	SS	17					
5.0	-5.0										
		6.0	7.5	3	SS	18					
7.5	-7.5										
		8.5	10.0	4	SS	20					
10.0	-10.0										Lean CLAY (CL) - with abundant chert fragments at depth - reddish brown - moist - stiff to very stiff (RESIDUUM)
12.5	-12.5										
		13.5	15.0	5	SS	15					
15.0	-15.0										
17.5	-17.5										
		18.5	20.0	6	SS	11					
20.0	-20.0										

Continued

REMARKS: ____

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Coward Mill Subdivison
Knoxville, Tennessee
GEOServices Project # 21-18148

LOG OF BORING B-2
SHEET 2 OF 2

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-2 DRY ON COMPLETION ? Yes

DATE March 9, 2018 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 30.0 FT. 9.1 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 30.0 FT. ELEV. -30.0 FT.

BORING ADVANCED BY: POWER AUGERING X

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. ____ FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. ____ FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. ____ FT.

PROPOSED FFE: ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											(contiuned)
22.5	-22.5										
		23.5	25.0	7	SS	15					Lean CLAY (CL) - with abundant chert fragments at depth - reddish brown - moist - stiff to very stiff (RESIDUUM)
25.0	-25.0										
27.5	-27.5										
		28.5	30.0	8	SS	24					
30.0	-30.0										Boring Terminated at 30 Feet
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS: ____

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GEOServices Project # 21-18148

LOG OF BORING B-3
SHEET 1 OF 1

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-3 DRY ON COMPLETION ? Yes

DATE March 12, 2018 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 15.0 FT. 4.6 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 15.0 FT. ELEV. -15.0 FT.

BORING ADVANCED BY: POWER AUGERING X

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. ____ FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. ____ FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. ____ FT.

PROPOSED FFE: ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											Topsoil (4 Inches)
		1.0	2.5	1	SS	4					
2.5	-2.5										
		3.5	5.0	2	SS	11					
5.0	-5.0										
		6.0	7.5	3	SS	10					
7.5	-7.5										Lean CLAY (CL) - with chert fragments - reddish brown and brown - moist - soft to stiff (RESIDUUM)
		8.5	10.0	4	SS	9					
10.0	-10.0										
12.5	-12.5										
		13.5	15.0	5	SS	12					
15.0	-15.0										Boring Terminated at 15 Feet
17.5	-17.5										
20.0	-20.0										

REMARKS: ____

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Knoxville, Tennessee
GEOServices Project # 21-18148

LOG OF BORING B-4
SHEET 1 OF 4

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-4 DRY ON COMPLETION ? Yes

DATE March 9, 2018 SURFACE ELEV. FT.
REFUSAL: Yes DEPTH 81.7 FT. ELEV. -81.7 FT.
SAMPLED 81.7 FT. 24.9 M
TOP OF ROCK DEPTH FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 81.7 FT. ELEV. -81.7 FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. FT.

BORING ADVANCED BY: POWER AUGERING X PROPOSED FFE: FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											Topsoil (3 Inches)
		1.0	2.5	1	SS	6				25.1	
2.5	-2.5										
		3.5	5.0	2	SS	7				27.5	
5.0	-5.0										
		6.0	7.5	3	SS	8				26.8	
7.5	-7.5										
		8.5	10.0	4	SS	9				27.1	Lean CLAY (CL) - with trace root structure in the upper 3' and abundant chert fragments from 15' to depth - reddish brown and orangish brown - moist to wet - firm to very soft (RESIDUUM)
10.0	-10.0										
12.5	-12.5										
		13.5	15.0	5	SS	10*				27.0	
15.0	-15.0										
17.5	-17.5										
		18.5	20.0	6	SS	16*				22.6	
20.0	-20.0										

(continued)

REMARKS: (*) indicate N-values elevated due to the increased rock content

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GEOServices Project # 21-18148

LOG OF BORING B-4
SHEET 2 OF 4

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-4 DRY ON COMPLETION ? Yes

DATE March 9, 2018 SURFACE ELEV. FT.
REFUSAL: Yes DEPTH 81.7 FT. ELEV. -81.7 FT.
SAMPLED 81.7 FT. 24.9 M
TOP OF ROCK DEPTH FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 81.7 FT. ELEV. -81.7 FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. FT.

BORING ADVANCED BY: POWER AUGERING X PROPOSED FFE: FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											(contiuned)
22.5	-22.5										
		23.5	25.0	7	SS	11*				22.5	
25.0	-25.0										
27.5	-27.5										
		28.5	30.0	8	SS	11*				21.7	Lean CLAY (CL) - with trace root structure in the upper 3' and abundant chert fragments from 15' to depth - reddish brown and orangish brown - moist to wet - firm to very soft (RESIDUUM)
30.0	-30.0										
32.5	-32.5										
		33.5	35.0	9	SS	22*				28.4	
35.0	-35.0										
37.5	-37.5										
		38.5	40.0	10	SS	14*				26.8	
40.0	-40.0										(contiuned)

REMARKS: (*) indicate N-values elevated due to the increased rock content

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Coward Mill Subdivison
Knoxville, Tennessee
GEOServices Project # 21-18148

LOG OF BORING B-4
SHEET 3 OF 4

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-4 DRY ON COMPLETION ? Yes

DATE February 20, 2018 SURFACE ELEV. FT.
REFUSAL: Yes DEPTH 81.7 FT. ELEV. -81.7 FT.
SAMPLED 81.7 FT. 24.9 M
TOP OF ROCK DEPTH FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 81.7 FT. ELEV. -81.7 FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. FT.

BORING ADVANCED BY: POWER AUGERING X PROPOSED FFE: FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											(contiuned)
42.5	-42.5										
		43.5	45.0	11	SS	18*				26.8	
45.0	-45.0										
47.5	-47.5										
		48.5	50.0	12	SS	13*				29.4	Lean CLAY (CL) - with trace root structure in the upper 3' and abundant chert fragments from 15' to depth - reddish brown and orangish brown - moist to wet - firm to very soft (RESIDUUM)
50.0	-50.0										
52.5	-52.5										
		53.5	55.0	13	SS	13*				11.6	
55.0	-55.0										
57.5	-57.5										
		58.5	60.0	14	SS	9				34.4	
60.0	-60.0										(contiuned)

REMARKS: (*) indicate N-values elevated due to the increased rock content

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LOG OF BORING B-4
SHEET 4 OF 4

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-4 DRY ON COMPLETION ? Yes

DATE February 20, 2018 SURFACE ELEV. FT.
REFUSAL: Yes DEPTH 81.7 FT. ELEV. -81.7 FT.
SAMPLED 81.7 FT. 24.9 M
TOP OF ROCK DEPTH FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 81.7 FT. ELEV. -81.7 FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. FT.

BORING ADVANCED BY: POWER AUGERING X PROPOSED FFE: FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											(contiuned)
62.5	-62.5										
		63.5	65.0	15	SS	8				24.1	
65.0	-65.0										
67.5	-67.5										
		68.5	70.0	16	SS	WOH				39.4	Lean CLAY (CL) - with trace root structure in the upper 3' and abundant chert fragments from 15' to depth - reddish brown and orangish brown - moist to wet - firm to very soft (RESIDUUM)
70.0	-70.0										
72.5	-72.5										
		73.5	75.0	17	SS	1				36.3	
75.0	-75.0										
77.5	-77.5										
		78.5	80.0	18	SS	WOH				51.9	
80.0	-80.0										Boring Terminated 81.7 Feet

REMARKS:

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Coward Mill Subdivison
Knoxville, Tennessee
GEOServices Project # 21-18148

LOG OF BORING B-5
SHEET 1 OF 1

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-5 DRY ON COMPLETION ? Yes

DATE March 13, 2018 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 15.0 FT. 4.6 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 15.0 FT. ELEV. -15.0 FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. ____ FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. ____ FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. ____ FT.

BORING ADVANCED BY: POWER AUGERING X
PROPOSED FFE: ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											Topsoil (4 Inches)
		1.0	2.5	1	SS	14					
2.5	-2.5										
		3.5	5.0	2	SS	18					
5.0	-5.0										
		6.0	7.5	3	SS	21					Lean CLAY (CL) - with chert fragments in the upper 12' - reddish brown and brown - moist - stiff to very stiff (RESIDUUM)
7.5	-7.5										
		8.5	10.0	4	SS	15					
10.0	-10.0										
12.5	-12.5										
		13.5	15.0	5	SS	12					
15.0	-15.0										Boring Terminated at 15 Feet
17.5	-17.5										
20.0	-20.0										

REMARKS: ____

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Coward Mill Subdivison
Knoxville, Tennessee
GEOServices Project # 21-18148

LOG OF BORING B-6
SHEET 1 OF 2

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-6 DRY ON COMPLETION ? Yes

DATE March 12, 2018 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 30.0 FT. 9.1 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 30.0 FT. ELEV. -30.0 FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. ____ FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. ____ FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. ____ FT.

BORING ADVANCED BY: POWER AUGERING X PROPOSED FFE: ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											Topsoil (6 Inches)
		1.0	2.5	1	SS	6					
2.5	-2.5										
		3.5	5.0	2	SS	12*					
5.0	-5.0										
		6.0	7.5	3	SS	11*					
7.5	-7.5										
		8.5	10.0	4	SS	17*					
10.0	-10.0										Lean CLAY (CL) - with chert fragments in the upper 20' - reddish brown and orangish brown - moist - firm to very stiff (RESIDUUM)
12.5	-12.5										
		13.5	15.0	5	SS	15*					
15.0	-15.0										
17.5	-17.5										
		18.5	20.0	6	SS	12*					
20.0	-20.0										

Continued

REMARKS: (*) indicate N-values elevated due to the increased rock content

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LOG OF BORING B-6
SHEET 2 OF 2

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-6 DRY ON COMPLETION ? Yes

DATE March 12, 2018 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 30.0 FT. 9.1 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 30.0 FT. ELEV. -30.0 FT.

BORING ADVANCED BY: POWER AUGERING X

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. ____ FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. ____ FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. ____ FT.

PROPOSED FFE: ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											(contiuned)
22.5	-22.5										
		23.5	25.0	7	SS	8					Lean CLAY (CL) - with chert fragments in the upper 20' - reddish brown and orangish brown - moist - firm to very stiff (RESIDUUM)
25.0	-25.0										
27.5	-27.5										
		28.5	30.0	8	SS	8					
30.0	-30.0										Boring Terminated at 30 Feet
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS: (*) indicate N-values elevated due to the increased rock content

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Knoxville, Tennessee
GEOServices Project # 21-18148

LOG OF BORING B-7
SHEET 1 OF 2

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-7 DRY ON COMPLETION ? Yes

DATE March 12, 2018 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 40.0 FT. 12.2 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 40.0 FT. ELEV. -40.0 FT.

BORING ADVANCED BY: POWER AUGERING X

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. ____ FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. ____ FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. ____ FT.

PROPOSED FFE: ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											Topsoil (5 Inches)
		1.0	2.5	1	SS	12				26.2	
2.5	-2.5										
		3.5	5.0	2	SS	15		46	22	24.6	
5.0	-5.0										
		6.0	7.5	3	SS	10				22.0	Lean CLAY (CL) - with trace chert fragments in the upper 10' - reddish brown and orangish brown - moist - stiff (RESIDUUM)
7.5	-7.5										
		8.5	10.0	4	SS	13				22.9	
10.0	-10.0										
12.5	-12.5										
		13.5	15.0	5	SS	15		31	14	17.4	
15.0	-15.0										
17.5	-17.5										Lean CLAY (CL) - with abundant chert fragments increasing with depth - reddish brown and orangish brown - moist to wet - stiff to very stiff (RESIDUUM)
		18.5	20.0	6	SS	13				36.3	
20.0	-20.0										

(continued)

REMARKS: (*) indicate N-values elevated due to the increased rock content

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LOG OF BORING B-7
SHEET 2 OF 2

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-7 DRY ON COMPLETION ? Yes

DATE March 12, 2018 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 40.0 FT. 12.2 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 40.0 FT. ELEV. -40.0 FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. ____ FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. ____ FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. ____ FT.

BORING ADVANCED BY: POWER AUGERING X
PROPOSED FFE: ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											(contiuned)
22.5	-22.5										
		23.5	25.0	7	SS	16*				35.6	
25.0	-25.0										
27.5	-27.5										
		28.5	30.0	8	SS	15*				33.8	
30.0	-30.0										Lean CLAY (CL) - with abundant chert fragments increasing with depth - reddish brown and orangish brown - moist to wet - stiff to very stiff (RESIDUUM)
32.5	-32.5										
		33.5	35.0	9	SS	12*				28.0	
35.0	-35.0										
37.5	-37.5										
		38.5	40.0	10	SS	14*				32.4	
40.0	-40.0										Boring Terminated at 40 Feet

REMARKS: (*) indicate N-values elevated due to the increased rock content

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LOG OF BORING B-8
SHEET 1 OF 2

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-8 DRY ON COMPLETION ? Yes

DATE March 13, 2018 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 30.0 FT. 9.1 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 30.0 FT. ELEV. -30.0 FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. ____ FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. ____ FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. ____ FT.

BORING ADVANCED BY: POWER AUGERING X PROPOSED FFE: ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											Topsoil (4 Inches)
		1.0	2.5	1	SS	16*					
2.5	-2.5										
		3.5	5.0	2	SS	7					
5.0	-5.0										
		6.0	7.5	3	SS	9					
7.5	-7.5										
		8.5	10.0	4	SS	10					Lean CLAY (CL) - with trace root structure in the upper 6' and chert fragments in the upper 3' and at depth - reddish brown and orangish brown - moist - firm to very stiff (RESIDUUM)
10.0	-10.0										
12.5	-12.5										
		13.5	15.0	5	SS	12					
15.0	-15.0										
17.5	-17.5										
		18.5	20.0	6	SS	5					
20.0	-20.0										

Continued

REMARKS: (*) indicate N-values elevated due to the increased rock content

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LOG OF BORING B-8
SHEET 2 OF 2

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-8 DRY ON COMPLETION ? Yes

DATE March 13, 2018 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 30.0 FT. 9.1 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 30.0 FT. ELEV. -30.0 FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. ____ FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. ____ FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. ____ FT.

BORING ADVANCED BY: POWER AUGERING X
PROPOSED FFE: ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											(contiuned)
22.5	-22.5										
		23.5	25.0	7	SS	6					Lean CLAY (CL) - with trace root structure in the upper 6' and chert fragments in the upper 3' and at depth - reddish brown and orangish brown - moist - firm to very stiff (RESIDUUM)
25.0	-25.0										
27.5	-27.5										
		28.5	30.0	8	SS	22*					
30.0	-30.0										Boring Terminated at 30 Feet
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS: (*) indicate N-values elevated due to the increased rock content

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LOG OF BORING B-9
SHEET 1 OF 3

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-9 DRY ON COMPLETION ? Yes

DATE March 13, 2018 SURFACE ELEV. FT.
REFUSAL: Yes DEPTH 44.1 FT. ELEV. -44.1 FT.
SAMPLED 44.1 FT. 13.4 M
TOP OF ROCK DEPTH FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 44.1 FT. ELEV. -44.1 FT.

BORING ADVANCED BY: POWER AUGERING X

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. FT.

PROPOSED FFE: FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											Topsoil (5 Inches)
		1.0	2.5	1	SS	6				26.4	
2.5	-2.5										
		3.5	5.0	2	SS	12				37.5	
5.0	-5.0										
		6.0	7.5	3	SS	10				27.8	
7.5	-7.5										Lean CLAY (CL) - with trace chert fragments in the upper 3' - reddish brown and orangish brown - moist - firm to very stiff (RESIDUUM)
		8.5	10.0	4	SS	15				40.5	
10.0	-10.0										
12.5	-12.5										
		13.5	15.0	5	SS	16				43.9	
15.0	-15.0										
17.5	-17.5										Lean CLAY (CL) - with abundant chert fragments in the upper 5' - brown and tan - moist - firm to very stiff (RESIDUUM)
		18.5	20.0	6	SS	15*				36.0	
20.0	-20.0										

(continued)

REMARKS: (*) indicate N-values elevated due to the increased rock content

GEOServices, LLC-Geotechnical and Materials Engineers

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LOG OF BORING B-9
SHEET 2 OF 3

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-9 DRY ON COMPLETION ? Yes

DATE March 13, 2018 SURFACE ELEV. FT.
REFUSAL: Yes DEPTH 44.1 FT. ELEV. -44.1 FT.
SAMPLED 44.1 FT. 13.4 M
TOP OF ROCK DEPTH FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 44.1 FT. ELEV. -44.1 FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. FT.

BORING ADVANCED BY: POWER AUGERING X PROPOSED FFE: FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											(contiuned)
22.5	-22.5										
		23.5	25.0	7	SS	14*				35.7	
25.0	-25.0										
27.5	-27.5										
		28.5	30.0	8	SS	12*				39.6	Lean CLAY (CL) - with abundant chert fragments in the upper 5' - brown and tan - moist - firm to very stiff (RESIDUUM)
30.0	-30.0										
32.5	-32.5										
		33.5	35.0	9	SS	13*				41.1	
35.0	-35.0										
37.5	-37.5										
		38.5	40.0	10	SS	5				50.3	
40.0	-40.0										(contiuned)

REMARKS: (*) indicate N-values elevated due to the increased rock content

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LOG OF BORING B-9
SHEET 3 OF 3

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-9 DRY ON COMPLETION ? Yes

DATE March 13, 2018 SURFACE ELEV. FT.
REFUSAL: Yes DEPTH 44.1 FT. ELEV. -44.1 FT.
SAMPLED 44.1 FT. 13.4 M
TOP OF ROCK DEPTH FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 44.1 FT. ELEV. -44.1 FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. FT.

BORING ADVANCED BY: POWER AUGERING X PROPOSED FFE: FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											(contiuned)
42.5	-42.5										Lean CLAY (CL) - with abundant chert fragments in the upper 5' - brown and tan - moist - firm to very stiff (RESIDUUM)
		43.5	45.0	11	SS	50/2					
45.0	-45.0										Auger Refusal at 44.1 Feet
47.5	-47.5										
50.0	-50.0										
52.5	-52.5										
55.0	-55.0										
57.5	-57.5										
60.0	-60.0										

REMARKS: (*) indicate N-values elevated due to the increased rock content

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LOG OF BORING B-10
SHEET 1 OF 2

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-10 DRY ON COMPLETION ? Yes

DATE March 9, 2018 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. -25.0 FT.

BORING ADVANCED BY: POWER AUGERING X

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. ____ FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. ____ FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. ____ FT.

PROPOSED FFE: ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											Topsoil (4 Inches)
		1.0	2.5	1	SS	WOH				21.7	
2.5	-2.5										
		3.5	5.0	2	SS	6				29.8	
5.0	-5.0										
		6.0	7.5	3	SS	16				26.5	
7.5	-7.5										
		8.5	10.0	4	SS	16*				35.0	Lean CLAY (CL) - with trace root structures in the upper 8' and chert fragments increasing with depth - reddish brown and brown - moist - very soft to very stiff (RESIDUUM)
10.0	-10.0										
12.5	-12.5										
		13.5	15.0	5	SS	17*				38.6	
15.0	-15.0										
17.5	-17.5										
		18.5	20.0	6	SS	9*				42.3	
20.0	-20.0										

Continued

REMARKS: (*) indicate N-values elevated due to the increased rock content and Orangic Content for SS-2 is 6.4% by weight.
W.o.h. = weight of hammer (essentially 0 bpf)

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LOG OF BORING B-10
SHEET 2 OF 2

DRILLER Shannon / Jordan
ON-SITE REP.

BORING NO. / LOCATION B-10 DRY ON COMPLETION ? Yes

DATE March 9, 2018 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. -25.0 FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT.
ELEV. ____ FT.
AFTER 1 HRS: DEPTH TNP FT.
ELEV. ____ FT.
AFTER 24 HRS. DEPTH TNP FT.
ELEV. ____ FT.

BORING ADVANCED BY: POWER AUGERING X
PROPOSED FFE: ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qu	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											(contiuned)
22.5	-22.5										Lean CLAY (CL) - with trace root structures in the upper 8' and chert fragments increasing with depth - reddish brown and brown - moist - very soft to very stiff (RESIDUUM)
		23.5	25.0	7	SS	8*				57.7	
25.0	-25.0										Boring Terminated at 25 Feet
27.5	-27.5										
30.0	-30.0										
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS: (*) indicate N-values elevated due to the increased rock content

concrete, all masonry (dry mortar), as indicated or directed, will be classified and paid for as Rock Excavation (Bridges).

Cemented gravel, cemented chert, soft shale, or soft slate, even though requiring the use of explosives for economical excavation, will not be classified as rock.

(e) Bridge Excavation (Unclassified)

Bridge Excavation (Unclassified) shall be structure excavation and foundation preparation performed either above or below the datum line (established by elevation and definitely set out in the plans) as stipulated in **Subsection 204.08(a)** and **204.10(a)** regardless of the nature of the material excavated.

(f) Rock Drilling (Bridges).

Rock Drilling as herein set out covers the drilling or sinking of test holes through or in rock in order to verify the condition of the foundation.

(g) Bedding Material for Support for Pipe Culverts.
Class A-Portland Cement Concrete, Class A.
Class B-Specially selected granular soil.

MATERIALS

204.03-Foundation Fill Material. Material for foundation fill shall consist of suitably graded sand, gravel, slag or stone, as approved by the Engineer.

204.04-Bedding Material. Material for Class B bedding for pipe culverts shall consist of sand or a natural sandy soil, all of which passes a 3/8-in. (9.5 mm) sieve and not more than 10% passes a No. 200 (75 µm) sieve, or stone, gravel, chert or slag meeting the grading requirements for either Grading C, D, or E of **Subsection 903.05**.

In rock cuts or other areas designated by the Engineer where a free drainage bedding or backfill material is required, the material shall be crushed stone, crushed slag, or washed gravel meeting the requirements of **Subsection 903.17**.

204.05-Concrete. Concrete shall conform to the requirements of **Section 604** (Concrete Structures). Unless otherwise shown on the Plans or in the Special Provisions, Class A concrete shall be used for foundation seals and shall meet the requirements of **Subsection 604.19**. Concrete for culvert pipe cradles shall be Class A.

204.06-Backfill Material. Material for backfill shall be fine compatible soil selected from structure excavation if approved by the Engineer as being suitable. Additional material needed shall be obtained from roadway or borrow excavation as described in **Section 203.**

Granular Backfill Material for Structures shall be Type A aggregate, Grading D meeting the requirements of **Subsection 903.05.**

903

Grading Table for Class A and Class B Aggregate for Mineral Aggregate Base and Surface Courses

Total Per Cent by Weight, Passing Sieves

Sieve Size	Grading A	Grading B	Grading C	Grading D	Grading E
63 mm (2-1/2")	100				
50 mm (2")	95-100	100			
37.5 mm (1-1/2")		95-100	100	100	
25 mm (1")			90-100	85-100	100
19 mm (3/4")		65-95		60-95	90-100
9.5 mm (3/8)	35-65		45-74	50-80	65-100
4.75 mm (No. 4)		35-55	30-55	40-65	
1.18 mm (No. 16)		15-45		20-40	
150 µm (No. 100)	0-10	4-15	4-15	9-18	5-15

903.06-Aggregate for Plant Mix Base and Leveling Courses (Hot Mix). Aggregate for plant mix base and leveling courses shall consist of coarse aggregate, fine aggregate, and mineral filler when required.

If at any time the sources of materials are changed, a new mix design will be required in accordance with Subsection 407.03.

(a) Coarse Aggregate.

Coarse aggregate (aggregate retained on the 4.75 mm (No. 4) sieve) shall be crushed stone, crushed granite, crushed gravel, crushed slag or combination of these materials. This material shall conform to the quality requirements of ASTM D 692 except that the sodium sulfate soundness loss on limestone shall not exceed nine percent, and the aggregate shall contain no more than five percent soft or nondurable particles.

Crushed gravel shall consist of siliceous particles processed from washed material. At least seventy percent by count of the gravel retained on the 4.75 mm (No. 4) sieve shall have a minimum of two fractured faces, one of which must be fractured for the approximate average diameter or thickness of the particle. The addition of pea gravel or uncrushed particles shall not be permitted.

Virgin coarse aggregate for Grading A, A-CRL and A-S mixes shall be crushed stone, crushed slag, or a combination thereof.



Figure 5. Photo. Mold (0.1 ft^3) for funnel and vibratory table tests.

Table 7. Unit weight of aggregates.

Sample	Minimum (pcf)	Maximum (pcf)
No. 5	94.9	109.6
No. 56	100.6	103.8
No. 57	95.4	108.7
No. 6	101.0	110.3
No. 67	106.2	124.10
No. 68	96.9	115.9
No. 7	103.3	120.9
No. 78	92.3	109.6
No. 8A	85.7	101.3
No. 8B	—	—
No. 8C	—	—
No. 8D	—	—
No. 8E	97.9	112.8
No. 89	88.4	108.2
No. 9	92.3	110.7
No. 10	115.8	146.3

— = Not measured.

Table 11. SDS test results using MC approach.

Sample	Percent Passing No. 4 Sieve (figure 4)	$\phi'_{t,SDS}$ (degree)	c'_{SDS} (psi)
No. 5	0.46	50.1	1.7
No. 56	2.58	51.7	6.5
No. 57	3.67	56.5	1.9
No. 6	3.72	51.7	4.2
No. 67	3.96	57.3	7.1
No. 68	7.40	42.8	9.5
No. 7	10.26	54.6	7.8
No. 78	6.18	51.3	9.7
No. 8A	26.50	53.0	5.2
No. 8B	—	—	—
No. 8C	—	—	—
No. 8D	—	—	—
No. 8E	—	—	—
No. 89	47.01	54.0	6.1
No. 9	94.22	50.2	3.5
No. 10	98.96	50.6	3.3

$\phi'_{t,SDS}$ = The friction angle using the Mohr-Coulomb (MC) approach.
c'_{SDS} = The shear stress intercept from the best-fit linear MC failure envelope.
— = Not measured.

Note that using a linear MC interpretation results in a cohesion value for each test because the y-intercept of the best-fit approximation is not zero (figure 19, for example); however, these aggregates are cohesionless. This inconsistency is attributed to the fact that the failure envelope is actually nonlinear; there is also aggregate interlocking and dilation that occurs during shearing, depending on the applied normal stress.[54] In reality, the friction angle decreases with increased applied normal stress in a logarithmic function. For the range of stresses tested, the secant friction angle corresponding to the peak for each applied normal stress was larger than that approximated from the MC approach. Because of these issues, the use of an alternative ZDA method is proposed to determine the design friction angle, which is based on the critical state or constant volume concept.[3]

ZDA Approach: The ZDA method requires knowledge about the dilation behavior of the sample. The vertical displacement during the consolidation and shearing phases was measured using an LVDT. Comparing the vertical displacement to the horizontal displacement during shear provides an indication of the dilation behavior of the aggregates. An example for the No. 57 aggregate is shown in figure 20. Positive vertical displacement indicates compression of the sample, whereas negative displacement indicates dilation. The dilation angle (ψ) is found according to equation 14.

8.27 THE RELATIONSHIP BETWEEN UNDRAINED SHEAR STRENGTH AND EFFECTIVE OVERBURDEN PRESSURE

It has been discussed in previous sections that the shear strength is a function of effective consolidation pressure. If a relationship between undrained shear strength, c_u, and effective consolidation pressure p' can be established, we can determine c_u if p' is known and vice versa. If a soil stratum in nature is normally consolidated the existing effective overburden pressure p_0' can be determined from the known relationship. But in overconsolidated natural clay deposits, the preconsolidation pressure p_c' is unknown which has to be estimated by any one of the available methods. If there is a relationship between p_c' and c_u, c_u can be determined from the known value of p_c'. Alternatively, if c_u is known, p_c' can be determined. Some of the relationships between c_u and p' are presented below. A typical variation of undrained shear strength with depth is shown in Fig. 8.36 for both normally consolidated and heavily overconsolidated clays. The higher shear strength as shown in Fig. 8.36(a) for normally consolidated clays close to the ground surface is due to desiccation of the top layer of soil.

Skempton (1957) established a relationship which may be expressed as

$$\frac{c_u}{p'} = 0.10 + 0.004\, I_p \qquad (8.55)$$

From :

Geotechnical Engineering: Principles and Practices of Soil Mechanics and Foundation Engineering

V.N.S. Murthy

The capability of soil to bear loadings is different depending on the type of soil. Generally, fine grained soils have a relative smaller capacity in bearing of load than the coarser grained soils. Hence fine grained soils therefore have greater degree of compressibility. Values of compression index C_c vary for different type of soils. Table 1 below indicates compression index C_c valves of several kind of soils:

Kind of soil	**compression index C_c**
Dense sand	0.0005- 0.01
Loose sand	0.025- 0.05
Firm clay	0.03- 0.06
Stiff clay	0.06- 0.15
Medium soft clay	0.15- 1.0
Organic soil	1.0-4.5

Table No:1 Primary Compression Index (C_c) for several kind of Soils

In literature several correlations have been proposed whereby compressibility characteristic like compression index have been evaluated using liquid limit, natural moisture content(W_n), initial void ratio (e_o), plasticity index, specific gravity, void ration at liquid limit and several other properties of soil.

Skempton (1944) conducted consolidation test on a number of clays collected from different locations and gave the following equation for the compression index for a remolded sample:

Skempton model $C_c = 0.007\ (w_L - 10\%)$

http://ijiet.com/wp-content/uploads/2015/06/41.pdf

Equation No. 1 $C_r = 0.0007LLe_o + 0.01$

http://www.foundationperformance.org/archived_2005/october_2005.cfm

9.4.3 Primary Consolidation Settlement of Overconsolidated Fine-Grained Soils

If the soil is overconsolidated, we have to consider two cases depending on the magnitude of $\Delta\sigma_z$. We will approximate the curve in the σ'_z (log scale) versus e space as two straight lines, as shown in Figure. 9.5. In Case 1, the increase in $\Delta\sigma_z$ is such that $\sigma'_{fin} = \sigma'_{zo} + \Delta\sigma_z$ is less than σ'_{zc} (Figure. 9.5a). In this case, consolidation occurs along the URL and

$$\rho_{pc} = \frac{H_o}{1 + e_o} C_r \log \frac{\sigma'_{fin}}{\sigma'_{zo}}; \quad \sigma'_{fin} < \sigma'_{zc} \tag{9.17}$$

From **Soil Mechanics and Foundations** by Muni Budi

1. Use Factors of 1 for all to calculate the ASD FS
2. Use LL and DL factors to calculate LRFD concrete design

Dimensions of Wall

	Length of Stem	10	ft
	Coverage	1	ft
	Right Soil Depth	9	ft
	Left soil depth	2	ft
	Thickness of Stem	1	ft
T	Thickness of Slab	1	ft
	Shear Key	0	ft
B	Base Length	6	ft
	Toe Length	2	ft
	Heel Length	3	ft
	Beta	18	degrees
10.97475909	Apparent soil height	10.97475909	ft

Loading

Surcharge	q		0	psf
	q factored		0	psf
	Ka	0.394805883		
		Unfactored	Factored	
	Live Load Horizontal Surcharge	0	0	lb
	Live Load Vertical Surcharge	0	0	lb
	Earth Surcharge Vertical	881.6723497	1322.509	lb
	Earth Surcharge Horizontal	2713.508476	4070.263	lb

Amount of Concrete
16

Fill Specs

Unit Weight	120	pcf
Friction angle	30	degrees
Cohesion	0	psf

Concrete Specs

f'c	3000	psi
fy	60	ksi
Req'd Cover	3	inches
Unit Weight	150	pcf

Bearing Specs

Unit Weight	118	pcf
Friction angle	0	degrees
Cohesion	2000	psf
Friction factor	0.45	

Rebar Specs

Stem bar diam	1	in
Slab Reinforcem	1	in

(LS) Live Load Factor	1.75	
(ES) Earth Surcharge Factor	1.5	
(DC) Dead Load Factor	1.25	
(EV) Earth Vertical Factor	1.35	
(EH) Passive Earth Factor	1.35	
Shear Phi	0.75	
Moment Phi	0.9	
Component Factors		
DL	1.4	

Required Steel in Stem	0.0018	in^2/linear foot
Required Steel in Slab	0.303841	in^2/linear foot
Min Long Steel in stem	2.592	in^2/linear foot
Min Long Steel in Slab	1.5552	in^2/linear foot

Bearing Capacity (Vesic')

	c	q	gamma		
N	5.1	1	0		
s	1	1	1	k	0.5
d	1.2	1	1		
i	1	1	1		
b	1	1	1		
g	1	1	1		

Sigma ZD	354	psf
gamma	118	psf
qult	12594	psf

Size Base

Resisting Moments

	Weight Unfactored	Factored	Arm (ft)	Moment (lb*ft)
Wc1	1500	1875	2.5	4687.5
Wc2	900	1125	3	3375
Ws3	3240	4374	4.5	19683
Ws4	480	648	1	648
WS5	175.456636	236.8664586	5	1184.332293
WC6	0	0	2.5	0
LL vertical	0	0	6	0
ES Vert	881.67235	1322.508525	6	7935.051148
q LL vertical	0	0	4.5	0
q LL Horiz	0	0	10.48738	0
SUM		9581.374983		37512.88344

Overturning FS	2.519325		1.5
Net Moment	22622.83	lb*ft	
Resultant Moment arm	2.361126		
eccentricity	0.638874		
e/B (ASD)	0.106479		0.166666667

qmax	2617.11123	psf		
qmin	576.680428	psf		
qult	12594	psf		
qult/qmax	4.81217605		2.5	ASD

	Phi			
LRFD	0.55	6926.7	Bearing	
LRFD	B/4	1.5	Eccentricity	

Overturning Moments

	Unfactored	Factored	Arm (ft)	Moment (lb*ft)
ES Horiz	2713.508	4070.262714	3.658253	14890.05
LL Horiz	0	0	5.48738	0
Sum of Mc		14890.0509	lb*ft	

LOAD FACTORS

	DC	EV	LS	ES	Passive	
Strength 1-a	0.9	1	1.75	1.5	1.35	Sliding and eccentricity
Strength 1-b	1.25	1.35	1.75	1.5	1.35	Bearing Capacity
Service	1	1	1	1	1	

Sliding Resistance

Passive Earth Kp	5.828427		
	Unfactored	Factored	
Soil at Toe Resisting	3094.895	2089.054	
Friction Resisting	4311.619	4311.619	
Sum of Forces Sliding	4070.263	lb	
Sum of Forces Resisting	6400.673	lb	
FS Sliding (ASD)	1.572545	1.5	ASD
	Phi		
LRFD	1	6400.673	Sliding Resistance

Design Slab Reinforcement

Toe

340.0718
1936.968

		Force (lb)	Factored	Arm (ft)	Moment	
	Passive Soil	3094.894803	4332.852725	0.5	2166.426	CW
1596.896	Ws4	480	672	1	672	CCW
	Bearing - Triangle	680.1436015	952.2010421	1.333333	1269.601	CW
	Bearing - Rectangle	3873.935263	5423.509368	1	5423.509	CW
	Sum of Moments				8187.537	lb*ft
					98.25045	k*in
	Sum of Forces				5703.71	lb
					5.70371	kip

Heel

	Force (lb)	Factored Ld	Arm (ft)	Moment	
q LL vert	0	0	1.5	0	CW
Ws3	3240	4536	1.5	6804	CW
Bearing - Tria	1530.323103	2142.452	1	2142.452	CCW
Bearing - Rec	1730.041285	2422.058	1.5	3633.087	CCW
WS5	175.456636	245.6393	2	491.2786	CW
LL Surcharge	0	0	3	0	CW
ES vert	881.6723497	1234.341	3	3703.024	CW
Sum of Moments				5222.763	lb*ft
				62.67316	k*in CW
Sum of Forces				1.45147	kip Down

Size Slab Reinforcement

d	8	inches
c	4.733727811	inches
a	4.023668639	inches
		25.57866
Mu	98.25044543	kip*in
As Req'd	0.303841408	in^2/linear foot
As min	0.0216	in^2/linear foot
Mn	109.1671616	kip*in
0.9Mn	98.25044543	kip*in

Slab Shear Check

Vuc	5.70371041	kip
Vnc	10.5162731	kip
0.75Vnc	7.887204828	kip

Flexural Reinforcement in Stem

Load		Factored Load		Moment arm	Moment	
q LL horiz	0		0	7.487379544	0	to right
ES horiz	2713.508476		3798.911866	0.65825303	2500.645	to left
LL horiz	0		0	2.487379544	0	to left
		Sum of Moment			30.00774	k-in.
		Sum of Shear			3.798912	kip

		Factored	Unfactored	
	Sum of Moments	2679.262763	1786.175	lb*ft
		2.679262763	1.786175	k*ft
		0.223271897	21.4341	k*in
	d	8		inches
	c	4.733727811		inches
	a	4.023668639		inches
	As	0.000690473	0.066285	in^2/linear foot
	Asmin	0.0018		
	Shear in base of stem			
	Phi Vc	7.887204828		kip
	Vu	3.798911866		kip

Retaining Wall Reinforcement Design: Connection from Footer to Stem

Stem height	10	ft			
Vu	4.1	kip			
Mu	164	k-in			
Slab Thickness	1	ft			
Footer Thickness	1	ft	fy	60	ksi
Cover	3	in			
d	9	in			
c	5.325444	in			
a	4.526627	in			
As reqd	0.450821	in^2			
As min	0.2592	in^2			
As	0.450821	in^2			
Bar Size	5				
Bar area	0.31	in^2			
# of bars	2	bars			
As	0.62	in^2			
Phi Mn	225.5443	k-in			
Phi Vc	8.873105	kip			

Retaining Wall Reinforcement Design: Stem Reinforcement

Mu	30	k-in
Vu	3.8	kip
Stem thick	1	ft
d	9	in
c	5.325444	in
a	4.526627	in
As	0.082467	in^2
As min	0.2592	in^2
As	0.2592	in^2
Bar Size	5	
Bar area	0.31	in^2
# of bars	1	bars
As	0.31	in^2
Phi Mn	112.7721	k-in
Phi Vc	8.873105	kip

Retaining Wall Reinforcement Design: Longitudinal Stem Reinforcement

Height of stem	10	ft
Stem thickness	1	ft
Ag	1440	in^2
As req	3.6	in^2
As/ ft	0.36	in^2
#	9	
Number	1	
As per bar	1	in^2
As	1	in^2

Retaining Wall Reinforcement Design: Longitudinal Footer Reinforcement

Slab Length	6	ft
Slab Thickness	1	ft
Ag	864	in^2
As req	1.5552	in^2/ ln ft
# of Bars	6	
#	5	bar
Bar Area	0.31	in^2
As	1.86	in^2/ foot of wall

Retaining Wall Reinforcement Design: Flexural Footer Reinforcement

Mu	74.6	k-in
Vu	3.85	kip
Slab thickness	1	ft
d	9	in
c	5.325444	in
a	4.526627	in
As req	0.205068	in^2
As min	0.2592	in^2
As	0.2592	in^2
# of bars	1	
#	5	
A bar	0.31	in^2
As	0.31	in^2
Phi Mn	112.7721	k-in
Phi Vc	8.873105	kip

Retaining Wall Settlement Evaluation

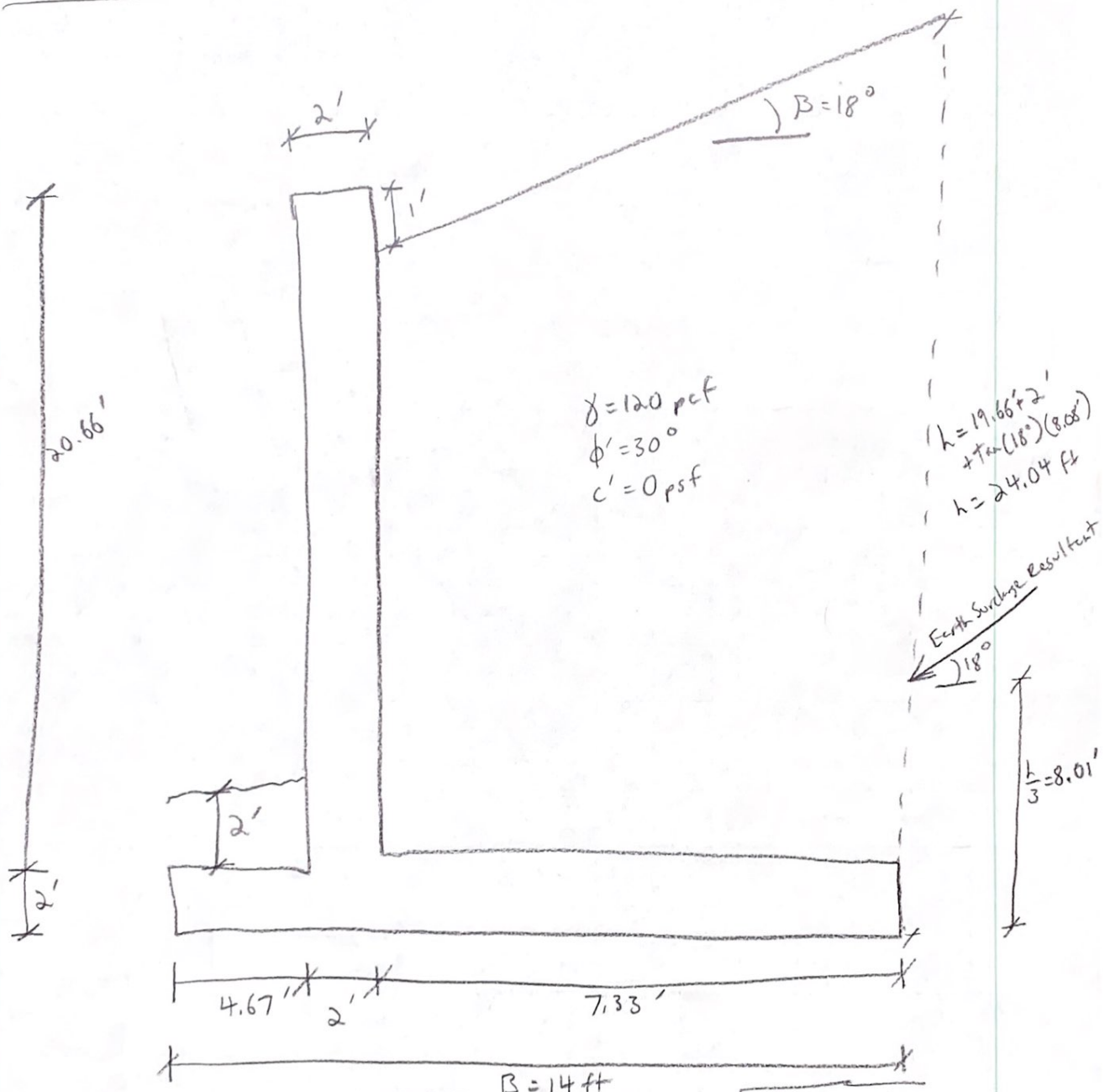
q	2617.111	psf
e0	0.443	
Cc	0.1057	
Cr	0.02457	
effective consolidation stress	10870	psf
Depth to bottom of footer	3	ft
Rigidity Factor	0.85	

H (ft)	D		Sigma' Z0	Sigma ' zf	Precons	Cr/1+e0	d	
1	26	29	2006	4623.111	10870	0.017027	0.160526	ft
							1.926306	in

q	3225	psf
e0	0.443	
Cc	0.1057	
Cr	0.02457	
effective consolidation stress	10870	psf
Depth to bottom of footer	4	ft
	0.85	

H (ft)	D		Sigma' Z0	Sigma ' zf	Precons	Cr/1+e0	d	
1	26	30	3894	7119	10870	0.017027	0.115998	ft
							1.39198	in

Worst-Case Section



$\beta = 18°$, $K_a = \cos\beta \times \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} = \cos(18°) \times \frac{\cos(18°) - \sqrt{\cos^2(18°) - \cos^2(30°)}}{\cos(18°) + \sqrt{\cos^2(18°) - \cos^2(30°)}}$

$K_a = 0.395$

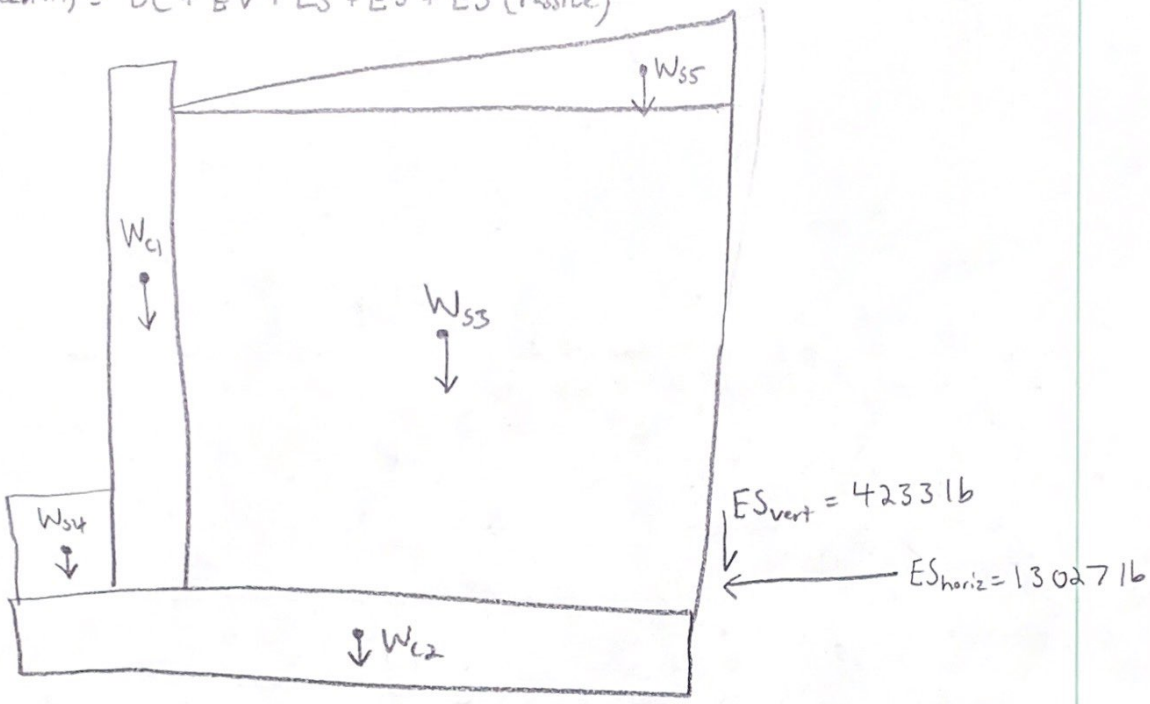
Earth Surcharge Resultant $= \frac{1}{2} K_a \gamma h^2 = \frac{1}{2}(0.395)(120\ pcf)(24.04')^2 = 13697\ lb$

ES horizontal $= (13697\ lb)\cos(18°) = 13027\ lb$

ES vertical $= (13697\ lb)\sin(18°) = 4233\ lb$

LRFD applicable LC's:

Strength Ia = 0.9DC + 1.0EV + 1.75LS + 1.5ES + 1.35ES (Passive)
Strength Ib = 1.25DC + 1.35EV + 1.75LS + 1.5ES + 1.35ES (Passive)
Serviceability = DC + EV + LS + ES + ES (Passive)



$W_{C1} = 150_{pcf}(2')(20.66') = 6198 lb$
$W_{C2} = 150_{pcf}(2')(14') = 4200 lb$
$W_{S3} = 120_{pcf}(7.33')(19.66') = 17301 lb$
$W_{S4} = 120_{pcf}(4.67')(2') = 1120 lb$
$W_{S5} = 150_{pcf}(7.33')(\tan(18°)7.33')\frac{1}{2} = 1048 lb$

Moment arms from toe

$W_{C1} = 4.67' + \frac{2'}{2} = 5.67'$
$W_{C2} = \frac{14'}{2} = 7'$
$W_{S3} = 4.67' + 2' + \frac{7.33'}{2} = 10.33'$
$W_{S4} = \frac{1}{2}(4.67') = 2.33'$
$W_{S5} = 4.67' + 2' + \frac{2}{3}(7.33') = 11.56'$
$ES_{vert} = 14'$
$ES_{horiz} = \frac{h}{3} = 8.01'$

Sum of moments about toe — Resisting

Sect	Unfactored Weight (lb)	Strength Ia (lb)	Strength Ib (lb)	Arm	Unfactored Moment	Strength Ia Moment	Strength Ib Moment
1	6198	5578	7748	5.67'	35122	31610	43903
2	4200	3780	5250	7'	29400	26460	36750
3	17301	17301	23356	10.33'	178775	178775	241346
4	1120	1120	1512	2.33'	2613	2613	3528
5	1048	1048	1415	11.56'	12115	12115	16355
ES_{vert}	4233	6347	6347	14'	59240	88860	88860
Σ	34099	35175	45628		317265	340433	430741

Sum of overturning Moments

Sect	Unfactored Weight (lb)	Strength Ia (lb)	Strength Ib (lb)	Arm	Unfactored Moment	Strength Ia Moment	Strength Ib Moment
ES_{horiz}	13027	19534	19534	8.01	104369	156554	156554

Bearing Capacity

Stiff, lean clay → undrained, $\phi = 0°$
$\gamma = 118$ psf
$c_u = 2000$ psf

Using Vesic' Equation:

$q_{ult} = c_u N_c s_c d_c i_c b_c g_c + \sigma_{z0} N_q s_q d_q i_q b_q g_q + \cancel{0.5 \gamma B N_\gamma s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma}$

Continuous footing: $s_c, s_q, s_\gamma = 1$
loading is not inclined: $i_c, i_q, i_\gamma = 1$
base is level: $b_c, b_q, b_\gamma = 1$
footing not located on slope: $g_c, g_q, g_\gamma = 1$

$N_c = 5.1 \quad N_q = 1.0 \quad N_\gamma = 0.0$

$q_{ult} = c_u N_c d_c + \sigma_{z0} N_q d_q$

$D = 2' + 2' = 4'$

$k = \frac{D}{B} = 0.286$

$\sigma_{z0} = (118\ pcf)(4') = 472\ psf$

$d_c = 1 + 0.4(0.286) = 1.114 \quad d_q = 1 + 2(0.286)\tan(0°)(1 - \sin(0))^2 = 1$

$q_{ult} = (2000\ psf)(5.1)(1.114) + (472\ psf)(1.0)(1) = \underline{11838\ psf}$

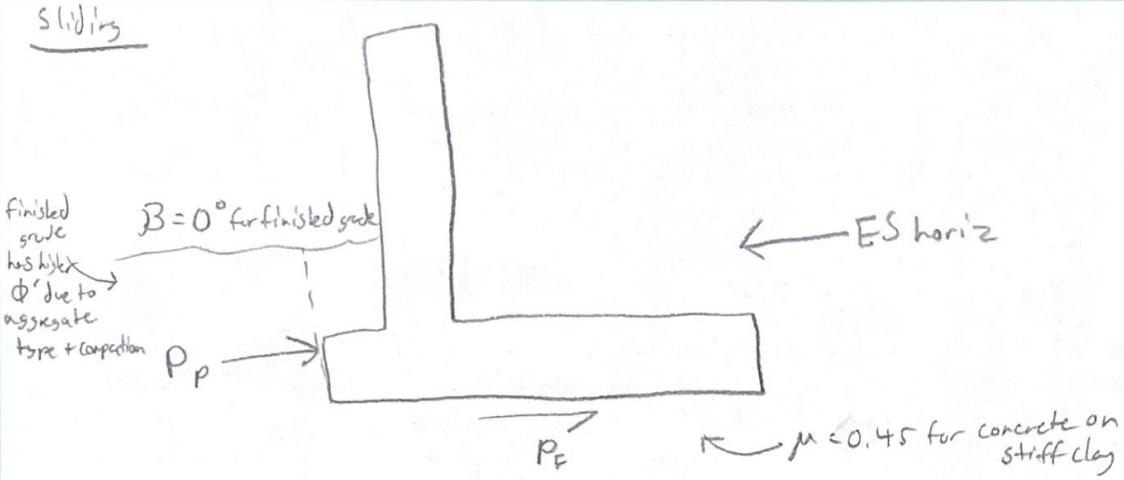
	Serviceability	Strength Ia	Strength Ib
Net moment $\Sigma M_R - \Sigma M_O$	212896 lb·ft	183879 lb·ft	274188 lb
Resultant Moment Arm $\frac{M_U}{P_U}$	6.244′	5.228′	6.009′
eccentricity $\frac{B}{2}$ − Moment arm	0.756′	1.77′	0.991′
$q_{max} = \frac{P_U}{BL}\left(1+\frac{6e}{B}\right)$	3225 psf	4421 psf	4643 psf
$q_{min} = \frac{P_U}{BL}\left(1-\frac{6e}{B}\right)$	1646	604 psf	1875 psf

LRFD Failure Checks: Bearing $q_{max} \leq \phi q_{ult}$ $\phi = 0.55$

Eccentricity $e \leq \frac{B}{4}$ $\frac{B}{4} = 3.5'$

	Serviceability	Strength Ia	Strength Ib
Bearing $\phi q_{ult} = 6510$ psf	3225 psf < 6510 psf ✓	4421 psf < 6510 psf ✓	4643 psf < 6510 psf ✓
eccentricity $\frac{B}{4} = 3.5'$	0.756′ < 3.5′ ✓	1.77′ < 3.5′ ✓	0.991′ < 3.5′ ✓

Sliding



$$K_p = \tan^2\left(45+\frac{\phi}{2}\right) = \tan^2\left(45+\frac{45}{2}\right) = 5.83$$

$$P_p = \frac{1}{2}\gamma h^2 K_p = \frac{1}{2}(120_{pcf})(4')^2(5.83) = 5502\ lb \rightarrow \text{serviceability}$$
$$= 7428\ lb \rightarrow \text{Strength Ia}$$
$$= 7428\ lb \rightarrow \text{Strength Ib}$$

$$P_{F\,service} = \mu(P_v) = (0.45)(340991b) = 15344\ lb$$

$$P_{F\,Str\,Ia} = 0.45(351751b) = 15829\ lb$$

$$P_{F\,Str\,Ib} = 0.45(456281b) = 20533\ lb$$

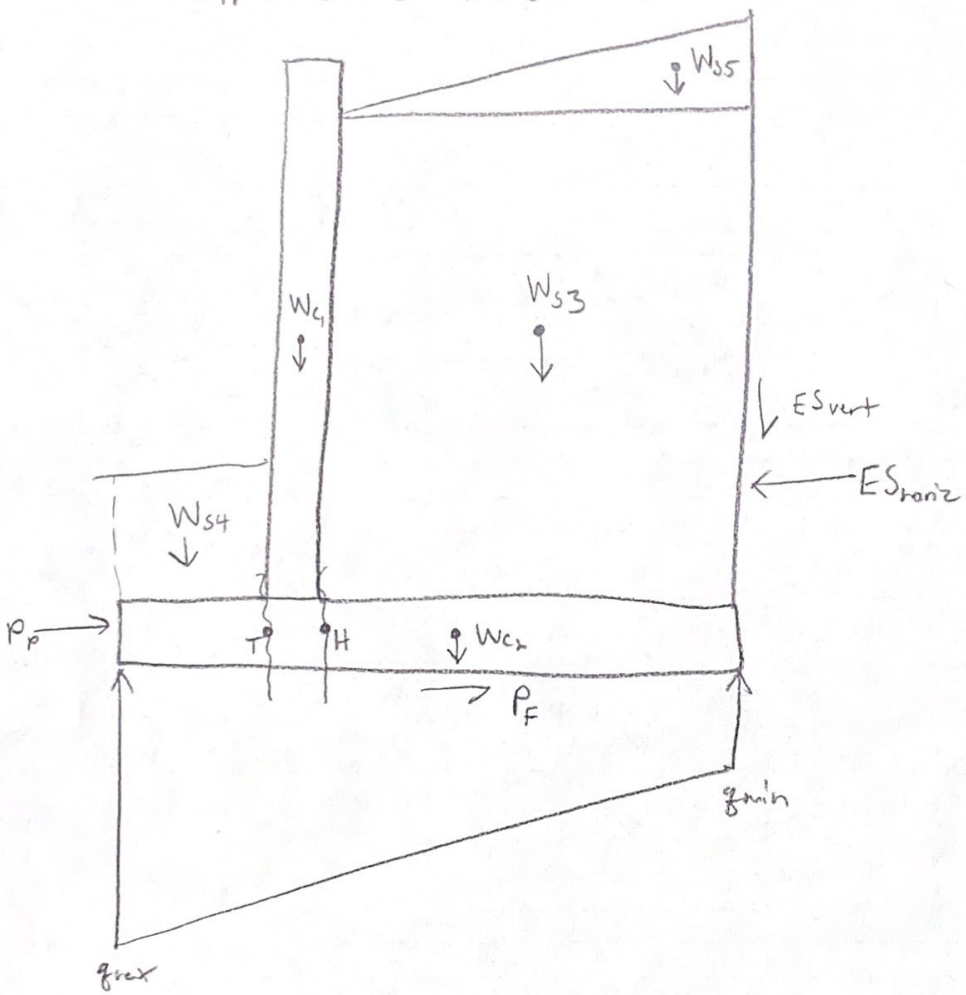
ES_{horiz} = 13027 lb for serviceability
= 19534 lb for Strength Ia
= 19534 lb for Strength Ib

$$F_S \leq \phi_v P_F + \phi_p P_p \qquad \phi_v = 1 \quad \phi_p = 0.5$$

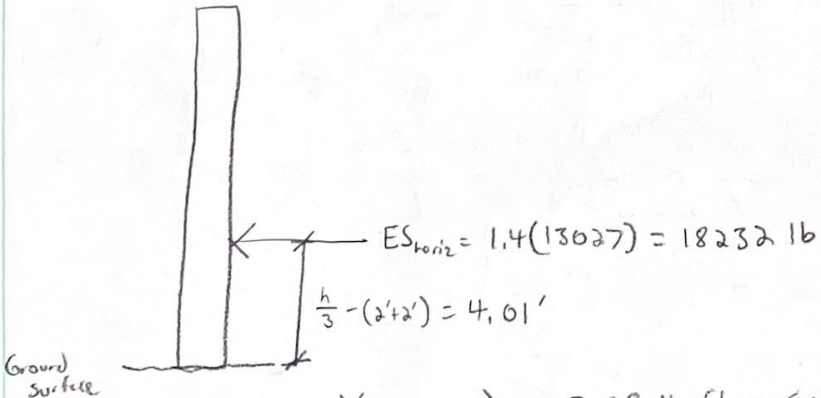
$130271b \leq (1)(15344\ lb) + (0.5)(5502\ lb) = 18095\ lb$ ✓
$195341b \leq (1)(15829\ lb) + (0.5)(7428\ lb) = 19542\ lb$ ✓
$195341b \leq (1)(20533\ lb) + (0.5)(7428\ lb) = 24247\ lb$ ✓

For concrete design, ACI-318 LC's must be used.

Applicable LC = 1.4 DL (NO LLs)



Maximum Moment & Shear in Stem



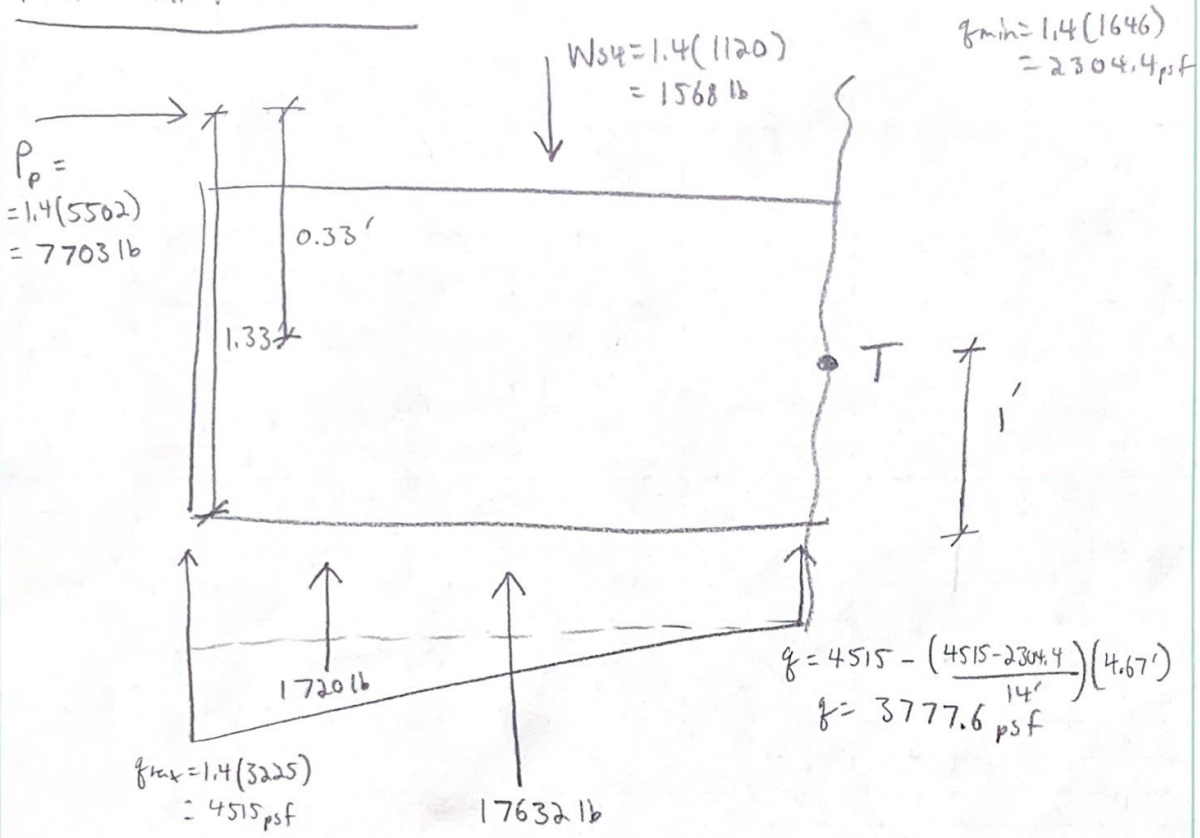
$\Sigma M = (4.01')(18232\ lb) = 73188\ lb \cdot ft = 878.3\ k \cdot in$

$\Sigma V = 18232\ lb = 18.2\ kip$

$M_u = 878.3\ k \cdot in$

$V_u = 18.2\ kip$

Max moment & shear in toe



$$\circlearrowleft \sum M_T = 1568\,lb\left(\frac{4.67'}{2}\right) - 7703\,lb(0.33') - 1720\,lb\left(\frac{2}{3}(4.67')\right) - 17632\,lb\left(\frac{4.67'}{2}\right)$$

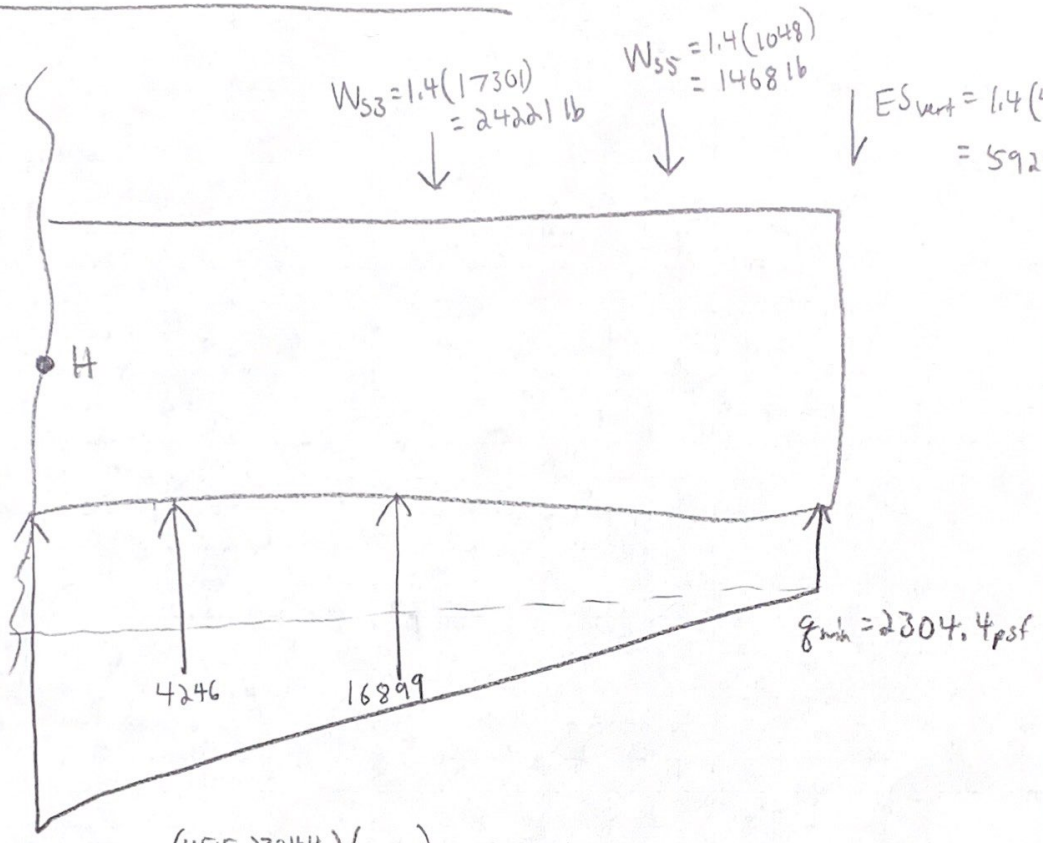
$$M_u = -45406.16\,ft = -544.9\ kip\cdot in$$

$$\boxed{M_u = 544.9\ k\cdot in\ \ CW}$$

$$+\uparrow \sum F_v = 17632\,lb + 1720\,lb - 1568\,lb = 17784\,lb$$

$$\boxed{V_u = 17.78\ kip\ \uparrow}$$

Max Moment & Sher in Heel



$$q = 2304.4 + \left(\frac{4515 - 2304.4}{14'}\right)(7.33') = 3462 \text{ psf}$$

$$\text{CCW}+ \quad \Sigma M_H = 16899\text{lb}\left(\frac{7.33'}{2}\right) + 4246\text{lb}\left(\frac{7.33'}{3}\right) - 24221\text{lb}\left(\frac{7.33'}{2}\right) - 1468\text{lb}\left(\frac{2}{3}(7.33')\right) - 5924\text{lb}(7.33')$$

$$M_U = -67086.5 \text{ lb·ft} = 805 \text{ k·in}$$

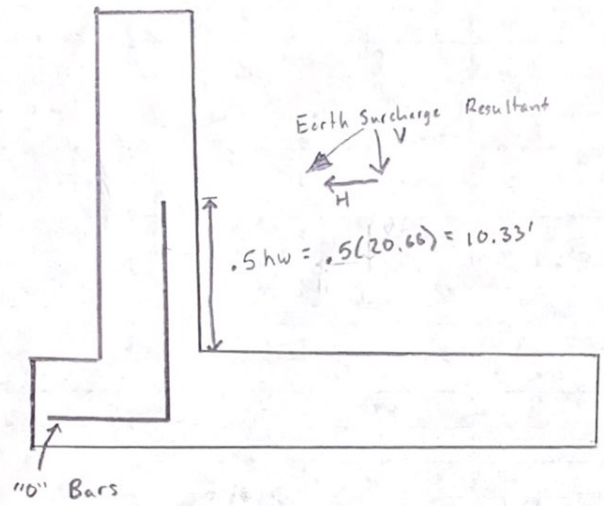
$$\boxed{M_U = 805 \text{ k·in CW}}$$

$$+\uparrow \Sigma F_V = 4246\text{lb} + 16899\text{lb} - 24221\text{lb} - 1468\text{lb} - 5924\text{lb}$$

$$V_U = -10468 \text{ lb}$$

$$\boxed{V_U = 10.47 \text{ kip} \downarrow}$$

Maximum Moment + Shear in footer + stem



Earth Surcharge Resultant = 13697 lb

V = 4233 lb
H = 13027 lb

$V_u = 1.6\,H = 1.6(13027) = 20{,}843.2 \text{ lb} = \boxed{20.84 \text{ k}}$

$M_u = V_u (h_w)/3 = 20.84 \text{ k}\,(20.66)/3 = 143.5 \text{ k-ft} = 1722 \text{ k-in}$

$\boxed{\begin{aligned} V_u &= 20.84 \text{ k} \\ M_u &= 1722 \text{ k-in} \end{aligned}}$

Reinforcement Connecting Footer + Stem

$d = 21''$

$c = \frac{21}{.00507}(.003) = 12.426''$

$a = \beta_1 c = .85(12.426) \Rightarrow a = 10.5621$

$M_u = .9(A_s)(f_y)(d - a/2) \Rightarrow 1722 = .9(A_s)(60)\left(21 - \frac{10.5621}{2}\right)$

→ $A_{s,\,req'd} = 2.029 \text{ in}^2$

$A_{s,\,min} = 1.8\%(12'' \times 24'') = .5184 \text{ in}^2$

* Use 5 #6 Bar/LF

$\rho_{min} = .003$

$\rho = \frac{A_s}{A_g} = \frac{(5)(.44)}{(12)(24)} = .0076 > \rho_{min}$ ✓

$\Phi M_n = .9(2.2)(60)\left(21 - \frac{10.5621}{2}\right) = 1867.4 \text{ k-in} > M_u$ ✓

$\Phi V_c = \Phi 2\sqrt{f'_c}\, b_w d = .85(2)\sqrt{3000}(12)(21) = 23.46 \text{ k} > V_u$ ✓

REINFORCEMENT (STEM) – "M" Bars

$M_u = 878.3$ k-in $\qquad V_u = 18.2$ k

$d = 21"$

$c = 12.426"$

$a = 10.5621$

$878.3 = .9(A_s)(60)\left(21 - \frac{10.5621}{2}\right) \rightarrow A_{s,reqd} = 1.035 \text{ in}^2$

$A_{s,min} = .5184 \text{ in}^2$

* Use 2 #7s

$\rho_{min} = .003$

$\rho = \frac{A_s}{A_g} = \frac{1.2}{(12)(21)} = .0047 > \rho_{min}$ ✓

$\Phi M_n = .9(1.2)(60)\left(21 - \frac{10.5621}{2}\right) = 1018.6 \text{ k-in} > M_u$ ✓

$\Phi V_n = \Phi 2\sqrt{f'_c}\, b_w d = .85(2)\sqrt{3000}(12)(21) = 23.46 \text{ k} > V_u$ ✓

"L_w" Bars in Stem

Use #9 @ 16"

$.0025(12)(24) = .72 \text{ in}^2/\text{ft}$

* #9 @ 16" $= 1.00(12/16) = .75 \text{ in}^2/\text{ft}$ ✓

"L_b" Bars in Slab

Use #5 -24

$.0018(12)(14)(24) = 7.258 \text{ in}^2/\text{ft}$

$24(.31) = 7.44 \text{ in}^2/\text{ft}$ ✓

• FLEXURAL REINFORCEMENT [SLAB]

$\Sigma M = M_u = 805\text{ K-in}$ ↻ $\Sigma V = V_u = 17.78\text{ K}$

TRY $d = 21''$: $\frac{21''}{.00507} = \frac{c}{.003}$: $c = 12.426''$

$a = \beta_1 c = .85(12.426'')$: $a = 10.5621''$

$M_u = .9(A_s)(f_y)(d - a/2)$: $805\text{ K-in} = .9A_s(60\text{ Ksi})(21'' - \frac{10.5621}{2})$

→ $A_{s,req} = .9484\text{ in}^2$

$A_{s,min} = 1.8\%(A_g) = .0018(24'' \times 12'') = .5184\text{ in}^2$

* * USE 5 #4 BAR / LINEAR FOOT

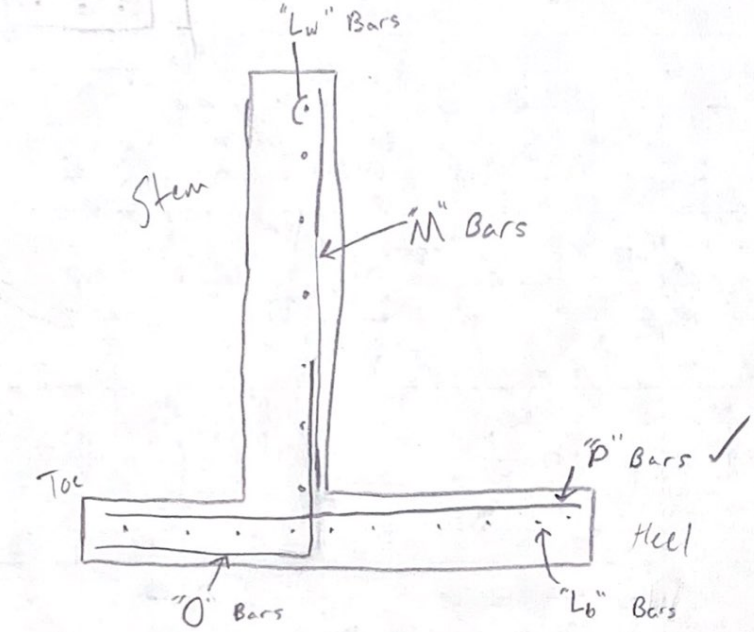
$\rho = \frac{A_s}{bd} = \frac{5(.2)}{(12'')(21'')} = .0039 > \rho_{min}$ ✓

$\rho_{min} = .0033$
$\rho_{MAX} = .0135$

$\phi M_n = .9(5 \times .2\text{ in}^2)(60\text{ Ksi})(21'' - \frac{10.561''}{2})$: $\phi M_n = 848.9\text{ K-in} > M_u$ ✓

$\phi V_c = \phi 2\sqrt{f'c}\, b_w d = .85(2)\sqrt{3{,}000\text{ psi}}(12'')(21'') = 23.46\text{ K}$

$\phi V_c \geq V_u$ ✓ OKAY



"M" Bars → 2 # 7 / LF

"P" Bars → 5 #4 / LF

"O" Bars → 5 #6 / LF

"L_b" Bars → 24 #5

"Lw" Bars → #9 @ 16"

Settlement Calculations

We need preconsolidation stress, initial void ratio, and compression + recompression indices to do LRFD settlement.

Preconsolidation Stress σ'_c

Using Skempton's relationship (1957) $\frac{C_u}{\sigma'_c} = 0.10 + 0.004\,PI$

$$\sigma'_c = \frac{C_u}{0.10 + 0.004\,PI}$$

$PI = LL - PL = 47 - 26 = 21$

$C_u = 2000$ psf

$$\sigma'_c = \frac{2000\,psf}{0.10 + 0.004(21)} = 10870\,psf$$

Initial void ratio e_o

$$e_o = \frac{G_s - \gamma_d}{G_s} \quad \text{if } \gamma_d \text{ is in } g/cm^3$$

*Assume $G_s = 2.70$ for clay

$\gamma_d = 94$ pcf $= 1.504\,g/cm^3$

$$e_o = \frac{2.70 - 1.504}{2.7} = 0.443$$

Compression Index C_c

Using Skempton model (1944) $C_c = 0.007(\omega - 10\%)$

$\omega = 25.1\%$

$$C_c = 0.007(25.1\% - 10\%) = 0.1057$$

↑ this is within the typical range for stiff clays

Recompression Index C_r

Using Javed parametric equation (2005) $C_r = 0.0007(LL)(e_o) + 0.01$

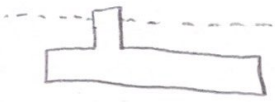
$$C_r = 0.0007(47)(0.443) + 0.01 = 0.0248$$

↑ this is $\sim \frac{1}{5} C_c$, which is typical

Settlement Calculations (cont.)

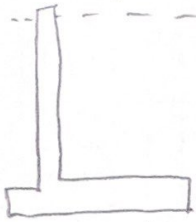
We essentially have 2 extremes of the retaining wall layout:

STA 3+40



One in which the existing grade is essentially at the level of the proposed grade (stem = 3')

STA 5+00



One in which existing grade is ~16' above proposed grade (stem = 18')

These two extremes will exhibit the largest difference in initial effective stress, and will likely exhibit the greatest differential settlement. From the boring logs there is at least a 30 ft layer of clay. Because D = 4', H_o = 26'. Assume stiff clay throughout. Assume Undrained behavior.

Sta 3+40

$q_{max} = 600\ psf$

σ_o @ mid-layer $= \left(\left(\frac{26'}{2}\right)+4'\right)(118\ pcf)$

$\sigma_o = 2006\ psf$

$OCR = \frac{\sigma_{pc}}{\sigma_o} = \frac{10870\ psf}{2006\ psf} = 5.42 > 1$

↳ this is an over-consolidated clay

$\sigma_f = \sigma_o + q_{max} = 2006\ psf + 600\ psf$

$\sigma_f = 2606\ psf$

$\sigma_f < \sigma_{pc}$

$\delta = H_o\left[\frac{C_r}{1+e_o}\log\left(\frac{\sigma_f}{\sigma_o}\right)\right]$

$\delta = \left(26' \times \frac{12''}{ft}\right)\left[\frac{0.0248}{1+0.443}\log\left(\frac{2606\ psf}{2006\ psf}\right)\right]$

$\delta = 0.61''$ ← less than the 2" max set by LRFD ✓

Sta 5+00

$q_{max} = 3225\ psf$

σ_o @ mid-layer $= \left(\left(\frac{26'}{2}\right)+2'+16'\right)(118\ pcf)$

$\sigma_o = 3658\ psf$

$OCR = \frac{\sigma_{pc}}{\sigma_o} = \frac{10870\ psf}{3658\ psf} = 2.97 > 1$

↳ This is an over-consolidated clay

$\sigma_f = \sigma_o + q_{max} = 3658\ psf + 3225\ psf$

$\sigma_f = 6883\ psf$

$\sigma_f < \sigma_{pc}$

$\delta = H_o\left[\frac{C_r}{1+e_o}\log\left(\frac{\sigma_f}{\sigma_o}\right)\right]$

$\delta = \left(26' \times \frac{12''}{ft}\right)\left[\frac{0.0248}{1+0.443}\log\left(\frac{6883\ psf}{3658\ psf}\right)\right]$

$\delta = 1.47''$ ← less than 2" max set by LRFD ✓

Length of Wall = 360 ft = 4320"

Allowable differential settlement = (1/500)·4320" = 8.64"

Ultimate differential settlement = 1.47" − 0.61" = 0.86" ← less than allowable total ✓

The proposed retaining wall meets the LRFD settlement serviceability requirements.

Darling, Wesley <wdarlin1@vols.utk.edu>

CE400 Section 3 Review Completed

7 messages

Retherford, Jenny <jretherf@utk.edu> Thu, Apr 2, 2020 at 2:21 PM
To: "Darling, Wes" <wdarlin1@vols.utk.edu>

Wes,

I've just completed review of your chapter. There were a few technical issues that I identified, along with a handful of primarily writing refinement improvements. I've placed the comments in the 100%DD ReviewedContent folder....a new file named "....2020VisionEngineering20200402...".

Please review and let me know if you have any questions. I think most are straight-forward. Some will require a bit of work related to the design of the wall, but I don't think it's an overwhelming amount (or, it may be done, and just needs to be articulated).

Schedule another meeting or drop in to the Zoom office hours as necessary.
Thanks,
Dr.J

THE UNIVERSITY OF TENNESSEE KNOXVILLE

Jennifer Retherford
Senior Lecturer

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851 Neyland Dr.
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(865) 974-2682
Big Orange. Big Ideas.

Darling, Wesley <wdarlin1@vols.utk.edu> Thu, Apr 2, 2020 at 3:59 PM
To: "Retherford, Jenny" <jretherf@utk.edu>

Dr. J,

Thank you for your comments on the retaining wall section of the report. I reviewed your comments and everything seems to make sense.

Regarding the design of the wall, several of your comments refer to the sizing of the wall, specifically related to the thickness of the stem and the uniformly-sized footing. When my team discussed the design of the wall with our mentors, they specifically instructed us to maintain a uniform thickness of the stem along the wall's length and to design the footing to the "worst-case cross-section" and to keep that uniform along the length as well. The reason for the stem's thickness was to resist shear failure caused by active earth pressure from the worst-case cross-section. They forwarded my design (which was the smallest I could get it to considering the 3 different load combinations) to a member of their structural department who reviewed my work and okayed it.

I am worried that redesigning the wall for other height ranges would severely increase the workload of my team, less related to developing the design itself (I have an Excel sheet that could help with that) but more so related to having to redraw all of the cross-sections in Microstation, transferring them to Civil3D, re-calculating the quantities for the cuts for the wall sections, the effects the new cross-sections have on drainage, right-of-way/easement, etc. Because so much of this additional work is computer-based and the virtual client, while suitable for minor refinements, is cumbersome for large amounts of CAD work, I fear this increases the work time exponentially, on top of the existing tasks which we are busy working to complete over the next 2.5 weeks.

Because we were following the direction of our mentors in the design process and the deadline for the entire project is so soon, I am not sure how we can tackle this work in time, especially virtually. Would it be possible for us to design the sizing of the wall for smaller cross-sections but leave this in some sort of tabulated form, either in the drawings or the report? And then include some sort of annotation that states that "drainage, easements, cross-sections, quantities of cuts, etc. were designed using the largest wall size" or something to that effect? Because changing the wall sizing at this point so fundamentally changes every other aspect of the project, we would essentially be restarting several major engineering tasks all over again to account for the changes...

Let me know if you have any thoughts or suggestions for how we could go about solving this problem. I would also be happy to have a Zoom meeting if that would be a better medium for discussion.

Thank you for your help and I look forward to receiving your response.

Kind regards,
Wesley
[Quoted text hidden]

Retherford, Jenny <jretherf@utk.edu> Thu, Apr 2, 2020 at 6:14 PM
To: "Darling, Wes" <wdarlin1@vols.utk.edu>

Do you happen to have a paper trail that tracks this guidance? Email conversations confirming this review and approval?

Get Outlook for iOS

From: Darling, Wesley <wdarlin1@vols.utk.edu>
Sent: Thursday, April 2, 2020 3:59:58 PM
To: Retherford, Jenny <jretherf@utk.edu>
Subject: Re: CE400 Section 3 Review Completed

[Quoted text hidden]

Darling, Wesley <wdarlin1@vols.utk.edu> Thu, Apr 2, 2020 at 8:47 PM
To: "Retherford, Jenny" <jretherf@utk.edu>

Dr. J,

I have looked through all of my notes, meeting minutes, and emails and I am not finding anywhere that I wrote this point down specifically (I see now why thorough note-taking is so important!). I did, however, find some correspondence that relates to our general discussion about the sizing of the wall:

I believe the conversation of footer sizing came up during my team's mid-semester Design Development meeting with our project mentors, which I am realizing now you were not present for. After the discussion resulting in us deciding not to design an MSE wall, I vividly remember asking about the procedure for wall sizing specifically because the wall height necessary varies so greatly (this was one of the reasons we strongly considered the MSE wall, because of the massive footer needed for a cantilever of ~21' tall). I remember our mentors saying to keep the footer and wall thickness uniform and only change the height because otherwise it would cause construction challenges when casting the concrete having the footer and wall sizes change abruptly. I remember saying something about having a massive footer for short parts of the wall and being concerned about it unnecessarily adding onto the property TDOT would need to purchase from the church and they said the "size of the footer doesn't matter." This was documented in the meeting minutes for this meeting. I have highlighted the statement in the first attached document. My team members also remember this conversation and this was the reason why I designed everything uniformly based on the worst case cross-section instead of designing cross-section by cross-section (which would have reduced the amount of additional property easement needed for the project).

I also remember being bothered by the size of the wall when I was designing the footer initially, so I emailed our mentors twice. In the first email (second attachment, towards the bottom), I asked if I had been designing for an incorrect load combination as the slab was so large and specifically reiterated how odd it seemed having a slab that size for sections when the wall was very short. Our mentors sent this email to one of their coworkers in the Nashville office and he answered my question about LC's but did not address the sizing issue. Our mentors did not address the sizing issue in their email responses either.

I sent an additional email directly to Don (the engineer in the Nashville office) but cc'd one of my mentors on the email and again reiterated the large necessary size of the slab. The general thrust of this email was to verify calculations, however, and it is likely that Don wouldn't have known the situation at hand hence why it was not remarked upon in his replies. This email is also attached, for reference.

I'm not sure if this is significant enough correspondence to constitute a paper trail for my decision-making, but I believe what my team took away from discussing this with our mentors was that ease of constructability was more important than cost and varying the wall thickness and footing size would have contributed to the complexity of the project so a uniform size was chosen. I apologize that there has been so much confusion about this point and I regret that I have not had the opportunity to discuss my team's most recent designs with our mentors to the extent that we had hoped to as, if it had been an issue with our design, we likely could have addressed it 3-4 weeks ago shortly after the 100% Design Development was shared with them and would not have run into this problem with such a limited time frame remaining.

Please let me know how you think my team should proceed and I apologize for the length of this email. I look forward to receiving your response and I hope you have a nice evening.

Kind regards,
Wesley
[Quoted text hidden]

---

3 attachments

1_DD Meeting Minutes.pdf
106K

2_RetainingWallSizingEmail1.pdf
98K

3_RetainingWallSizingEmail2.pdf
124K

Retherford, Jenny <jretherf@utk.edu> Fri, Apr 3, 2020 at 7:15 AM
To: "Darling, Wes" <wdarlin1@vols.utk.edu>

I'll say that if we were on campus, you would be making these adjustments. It's not a lot of work to modify the bar sizes - as you mentioned, you can do that in Excel quickly. It's not a lot of work to create the reinforcement schedule that would allow you to easily tabulate the bar sizes for different wall heights; all you do is create a cross-section that references bars as <A bars at B" on center>, and then you create a table that shows what "A" bars are and what "B" dimension/distance is. I would typically consider this a very minimal effort and we have a full month to complete that work. I'd encourage modifying some wall thicknesses; the formwork required to do this would not be insanely expensive. I don't have as much heartburn over the footing widths, but we'd talk to a contractor and get their genuine opinion and do a back of the napkin calculation to determine how much concrete we could reasonably save because it's not that complicated to change the width of the trench...

All that said, I want you to know that there's probably more to this story. I don't truly expect that Gresham would use the single wall design for the entire length. I know other firms that I have worked for wouldn't default to this extent; some uniformity, yes, but probably not to this scale.

Your documentation is ok for our course and I'll accept it.

I appreciate you recognizing to question this degree of simplification in future design work. I care more today about you taking this lesson forward than iterating the design during such a complicated time.

Please place this documentation in the Drive, in the appropriate Appendix.
If you can get an email to Don, I'd appreciate a direct question. One direction question about the dimensions. It'd be interesting and informative to hear his opinion about the single section being applied along the entire wall length. If he responds, please include that in the Appendix.

Thanks,
Dr.J

THE UNIVERSITY OF TENNESSEE KNOXVILLE

Jennifer Retherford
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851 Neyland Dr.
315 John D. Tickle Building
Knoxville, TN 37996
jqretherford@utk.edu
(865) 974-2682
Big Orange. Big Ideas.

From: Darling, Wesley <wdarlin1@vols.utk.edu>
Sent: Thursday, April 2, 2020 8:47 PM
[Quoted text hidden]

[Quoted text hidden]

Darling, Wesley <wdarlin1@vols.utk.edu> Fri, Apr 3, 2020 at 9:22 AM
To: "Retherford, Jenny" <jretherf@utk.edu>

Dr. J,

Thank you for your response and I appreciate you reviewing my documentation. I will place the documentation as well this thread of correspondence in the appropriate Appendix. I will also email Don about the dimension question and will place his response in the Appendix.

Thank you again for reviewing my work and again I apologize for all of the confusion and back-and-forth this has created. I hope you have a nice weekend and I will see you in virtual class on Monday.

Kind regards,
Wesley
[Quoted text hidden]

Retherford, Jenny <jretherf@utk.edu> Fri, Apr 3, 2020 at 9:25 AM
To: "Darling, Wes" <wdarlin1@vols.utk.edu>

No problem, Wes. The challenges of remote conversations and remote design efforts are more evident in this dialogue! Quite frustrating!...but hopefully didn't cause you too much stress!

Thanks for your professionalism through the communication process.
Dr. J

THE UNIVERSITY OF TENNESSEE KNOXVILLE

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851 Neyland Dr.
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Big Orange. Big Ideas.

From: Darling, Wesley <wdarlin1@vols.utk.edu>
Sent: Friday, April 3, 2020 9:22 AM
[Quoted text hidden]

[Quoted text hidden]

Darling, Wesley <wdarlin1@vols.utk.edu>

Faith Promise Senior Design Project: LRFD Factors for Retaining Wall Sizing

7 messages

Darling, Wesley <wdarlin1@vols.utk.edu> Mon, Mar 2, 2020 at 2:05 PM
To: donald.mccrary@greshamsmith.com
Cc: "Graves, Jake" <jake.graves@greshamsmith.com>

Hi Don,

My name is Wesley and I am working with Jake, Jason, and Patrick as part of my senior design project. My team is designing an on-ramp for a highway and, because the proposed alignments cut through an existing detention pond, we are designing a retaining wall so as not to alter the pond.

The most extreme cross section required a 20.66-foot tall wall holding back soil at an 18-degree angle. I have been using this cross-section as the basis for my design. I have found that given the height of the wall and the mass of soil we are holding back, a very large base size is necessary, particularly in order to meet the Sliding limit state.

Jake forwarded me your response regarding the load combinations and LRFD method.

The load factors I have been using are as follows:

	Live Load (LS)	Dead Load of Components (DC)	Vertical Earth Load (EV)	Horizontal Earth Surcharge (ES)	Passive Earth Surcharge (EH)
Strength 1a (Sliding + Eccentricity)	1.75	0.9	1	1.5	1.35
Strength 1b (Bearing)	1.75	1.25	1.35	1.5	1.35
Serviceability	1	1	1	1	1

The resistance factors I have been using are as follows:

	Resistance Factor Phi
Bearing	0.55
Eccentricity	< B/4
Sliding	1
Shear (Component Strength)	0.75
Moments (Component Strength)	0.9

When I check the sizing for the different load combinations, I find I require the following wall dimensions:

	Stem Thickness	Slab Thickness	Slab Size (B)	Shear Key?
Strength 1a	1.5’	4’	17’	Yes – 1’
Strength 1b	1.5’	3’	15’	No
Serviceability	1’	1.5’	14.5’	Yes – 1’

From this I would gather that I would need to use the largest size as this is the only one that meets all limit states, however this seems incredibly large...Does a 4'x17' slab seem reasonable for a ~21' wall?

I have attached to this email the spreadsheet I have been working with with the numbers I was checking filled in. I have also attached some calculations I did for a previous iteration. Though the number I used for my backfill friction angle and the factors used are incorrect, it will hopefully show you where the different load values I used are from. The Excel spreadsheet is correct and has certain conditional statements that check limit states.

I really appreciate your help with taking a look at this and I would be happy to provide you with additional information if you have any questions.

Kind regards,
Wesley Darling

---

2 attachments

Cantilever Workbook.xlsx
29K

Faith Promise Retaining Wall Calculations (1).pdf
3632K

McCrary, Donald <donald.mccrary@greshamsmith.com> Mon, Mar 2, 2020 at 3:06 PM
To: "Darling, Wesley" <wdarlin1@vols.utk.edu>
Cc: "Graves, Jake" <jake.graves@greshamsmith.com>

[External Email]

Hello Wesley,

Just wanted to confirm with you that I received your email. I will take a look at this and get back with you soon.

Thanks,

Donald McCrary, PE
Gresham Smith

D: 615.770.8495
M: 931.982.1885

[Quoted text hidden]

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McCrary, Donald <donald.mccrary@greshamsmith.com> Mon, Mar 2, 2020 at 4:50 PM
To: "Darling, Wesley" <wdarlin1@vols.utk.edu>
Cc: "Graves, Jake" <jake.graves@greshamsmith.com>

[External Email]

Wesley,

The load factors you are using are correct.

I believe you can ignore the Live Load surcharge. Since this wall is just a cut wall, there will be no traffic on top. With taking the LS load out, this should help make the footing smaller. Please let me know if you have any questions or comments.

Thanks,

Donald McCrary, PE
Gresham Smith

D: 615.770.8495
M: 931.982.1885

From: Darling, Wesley <wdarlin1@vols.utk.edu>
Sent: Monday, March 2, 2020 1:05 PM
To: McCrary, Donald <donald.mccrary@greshamsmith.com>
Cc: Graves, Jake <jake.graves@greshamsmith.com>
Subject: Faith Promise Senior Design Project: LRFD Factors for Retaining Wall Sizing

Hi Don,

[Quoted text hidden]

[Quoted text hidden]

Darling, Wesley <wdarlin1@vols.utk.edu> Mon, Mar 2, 2020 at 8:17 PM
To: "McCrary, Donald" <donald.mccrary@greshamsmith.com>
Cc: "Graves, Jake" <jake.graves@greshamsmith.com>

Don,

Thank you for your email. I took out the Live Load surcharge and was able to get the wall down to a 1.75'x14' slab with a 2' shear key. I have attached my calculations for this wall including the LRFD equations, with each load factored. If you're able to take a look at my calculations when you get the chance, I would greatly appreciate your feedback.

Thank you again for all of your help and I hope you have a nice evening.

Kind regards,
Wesley
[Quoted text hidden]

FaithPromise_RetainingWallCalculations.pdf
3357K

McCrary, Donald <donald.mccrary@greshamsmith.com> Tue, Mar 3, 2020 at 12:27 PM
To: "Darling, Wesley" <wdarlin1@vols.utk.edu>

Good afternoon Wesley,

I have looked over your calculations and they look good. I did notice one thing with your Ka equation. Attached is my comment on that.

Do you know if any Geotechical testing on this site has been done? This might affect some of your assumptions for the allowable bearing pressures.

One thing to take note also. When I check the shear strength for both the stem and footing, I am needing a thicker footing and stem. Also, when designing the reinforcement, TDOT uses f'c = 3000 psi for the concrete strength.

Please let me know if you have any questions.

[Quoted text hidden]
[Quoted text hidden]

FaithPromise_RetainingWallCalculations.pdf
4479K

Darling, Wesley <wdarlin1@vols.utk.edu> Tue, Mar 3, 2020 at 8:16 PM
To: "McCrary, Donald" <donald.mccrary@greshamsmith.com>

Don,

Thank you for all of your help! I have adjusted the Ka equation as you commented. I was provided with some geotechnical bore logs from projects nearby, however they did not provide very much information in terms of soil strength, beyond that it was a lean, stiff clay. I used reference materials to assume the values I used for my bearing pressure calculations. I have also changed the f'c to the 3000 psi that TDOT uses and found that I was able to get the wall down to a 2' thick stem, 2' thick slab, 14' base with no shear key necessary. I checked the shear strength of the new stem and slab using the following equation:

$$V_c = 2 \sqrt{f'_c} \; b_w d$$

I found each have a factored shear strength of ~19.7 kip (0.75*2*sqrt(3000)*12"*20") which is strong enough to resist the factored shear loads calculated.

I attached my updated LRFD wall calculations, but I used the same equations and approach as previous (with the adjusted Ka value) so they should be ready for me to move forward with required reinforcement design and drafting.

Again, I really appreciate you taking the time to help me through this part of the project and I hope you have a nice evening.

Kind regards,
Wesley

[Quoted text hidden]

FaithPromiseRWCalcs_3-3-20.pdf
2729K

McCrary, Donald <donald.mccrary@greshamsmith.com> Wed, Mar 4, 2020 at 2:11 PM
To: "Darling, Wesley" <wdarlin1@vols.utk.edu>

Wesley,

Thanks for the info. Everything looks good. I believe you can move forward with the reinforcement and detailing.

If you need anything, please let me know.

[Quoted text hidden]
[Quoted text hidden]

Darling, Wesley <wdarlin1@vols.utk.edu>

Faith Promise Project: Question about retaining wall sizing

2 messages

Darling, Wesley <wdarlin1@vols.utk.edu> Thu, Feb 27, 2020 at 6:04 PM
To: "Graves, Jake" <jake.graves@greshamsmith.com>, "Brady, Jason" <jason.brady@greshamsmith.com>, "Fiveash, Patrick" <patrick.fiveash@greshamsmith.com>

Good afternoon,

I am working on sizing the retaining wall and am finding that it varies pretty drastically based on the specific load combinations applied and the design method used (either LRFD or ASD).

Using the 20-foot cut cross-section as the design wall-height, I find that for the serviceability load combination (all load factors = 1), a 1-foot thick slab and 11-foot long base is needed.

When I examine more extreme load combinations, however, particularly those that increase the horizontal earth pressure (such as the Extreme Event Load combination, which increases it by 1.75 times and reduces the wall stability by a factor of 0.9), a 3-foot thick slab and 14-foot long base is needed. This slab essentially requires 4 times the amount of concrete as the general serviceability load combination one.

Alternatively, when I design the wall according to ASD factors of safety, a slab size of 2.5' by 12.5' meets the necessary requirements.

My two questions are:

- When you design something such as this, which load combination do you choose to design for? I am particularly unsure given that this is the cross-section with the tallest part of the wall, and all others would technically not require a slab this large (especially the cross-sections where the wall is only a few feet tall).

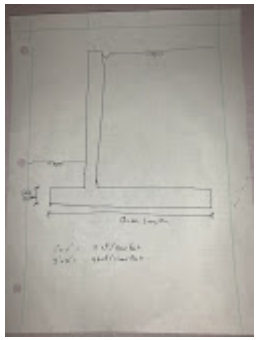
-Does your team design retaining walls according to LRFD or ASD limit states? I have set up my spreadsheet so that both can be calculated, but, as stated, differing results occur depending on the method chosen and load combinations. The concrete of the wall is being designed according to ACI-318.

I have attached a photo marking the specific dimensions I am referring to as well as a scan of the ASD calculations so you can see my train of thought. LRFD calculations have been done in Excel but I would be happy to write them down if you would like.

Kind regards,
Wesley Darling

2 attachments

RW sketch.jpg
963K



Retaining Wall Design.pdf
882K

Graves, Jake <jake.graves@greshamsmith.com> Mon, Mar 2, 2020 at 11:00 AM
To: "Darling, Wesley" <wdarlin1@vols.utk.edu>

[External Email]

See comments below.

Thanks,

Jake Graves, P.E.

D: 865.299.6145
M: 865.387.2035

From: McCrary, Donald <donald.mccrary@greshamsmith.com>
Sent: Monday, March 2, 2020 10:59 AM
To: Graves, Jake <jake.graves@greshamsmith.com>
Cc: Fiveash, Patrick <patrick.fiveash@greshamsmith.com>; Brady, Jason <jason.brady@greshamsmith.com>
Subject: RE: Faith Promise Project: Question about retaining wall sizing

Jake,

See below for comments.

Donald McCrary, PE
Gresham Smith

D: 615.770.8495
M: 931.982.1885

From: Graves, Jake <jake.graves@greshamsmith.com>
Sent: Friday, February 28, 2020 7:00 AM
To: McCrary, Donald <donald.mccrary@greshamsmith.com>
Cc: Fiveash, Patrick <patrick.fiveash@greshamsmith.com>; Brady, Jason <jason.brady@greshamsmith.com>
Subject: FW: Faith Promise Project: Question about retaining wall sizing

Don,

They have a few more questions for the retaining wall. Can you help with this?

Thanks,

Jake Graves, P.E.

D: 865.299.6145
M: 865.387.2035

From: Darling, Wesley <wdarlin1@vols.utk.edu>
Sent: Thursday, February 27, 2020 6:04 PM
To: Graves, Jake <jake.graves@greshamsmith.com>; Brady, Jason <jason.brady@greshamsmith.com>; Fiveash, Patrick <patrick.fiveash@greshamsmith.com>
Subject: Faith Promise Project: Question about retaining wall sizing

Good afternoon,

I am working on sizing the retaining wall and am finding that it varies pretty drastically based on the specific load combinations applied and the design method used (either LRFD or ASD).

Using the 20-foot cut cross-section as the design wall-height, I find that for the serviceability load combination (all load factors = 1), a 1-foot thick slab and 11-foot long base is needed.

When I examine more extreme load combinations, however, particularly those that increase the horizontal earth pressure (such as the Extreme Event Load combination, which increases it by 1.75 times and reduces the wall stability by a factor of 0.9), a 3-foot thick slab and 14-foot long base is needed. This slab essentially requires 4 times the amount of concrete as the general serviceability load combination one.

Alternatively, when I design the wall according to ASD factors of safety, a slab size of 2.5' by 12.5' meets the necessary requirements.

My two questions are:

- When you design something such as this, which load combination do you choose to design for? I am particularly unsure given that this is the cross-section with the tallest part of the wall, and all others would technically not require a slab this large (especially the cross-sections where the wall is only a few feet tall). You would design for Strength 1 and Service 1 load combination. The extreme event is not necessary unless this wall is located in a seismic zone.

-Does your team design retaining walls according to LRFD or ASD limit states? I have set up my spreadsheet so that both can be calculated, but, as stated, differing results occur depending on the method chosen and load combinations. The concrete of the wall is being designed according to ACI-318. We design retaining walls according to AASHTO LRFD. I can give you a copy of the load factors that are associated with the load combinations if you need this.

I have attached a photo marking the specific dimensions I am referring to as well as a scan of the ASD calculations so you can see my train of thought. LRFD calculations have been done in Excel but I would be happy to write them down if you would like. Can I also see the calculations for the LRFD method? I would like to see what load factors you are using.

Kind regards,

Wesley Darling

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2020 VISION ENGINEERING

MEETING MINUTES

Date: 02/24/2020
Time: 9:00AM – 10:00AM

Location: Gresham Smith Office

FAITH PROMISE PROJECT DESIGN DEVELOPMENT MEETING

Status updates and 100% DD expectations for scope of work

Roadway Design

-Driveway profiles complete
-ROW to be completed in the next couple of weeks
-Check labeling of easements

Retaining Wall Design

-Send to Gresham Smith by March 6th
- Changes made to cross sections required postponing RW work; will be drawn in Microstation
-Wall type discussed; between MSE and Cast-in-place cantilever; Cast-in-place cantilever selected
-Footer size should not be an issue, 2 feet of ROW will be purchased to accommodate wall footer

Drainage Plan

-Finished ditches on Faith Promise Lane, ditches on entrance ramp will be done after retaining wall: can begin work, but won't have alignment (will have depth, slope, etc.)
-Inlet spacing on Faith Promise Lane to be completed, don't exceed 300ft for maintenance
-After inlet spacing work is completed, can begin work on pipes

Erosion Control

-Waiting for drainage to be done, but will finish by March 11
-Shouldn't be too difficult: Follow the TDOT standard for erosion control

Traffic Control/Signage & Striping

-Send to Gresham when final TCDs are added to check (QA/QC) (All three sheets (9A, 10, 10A)
-Add arrow board, type 3 barricade, legend; see TDOT standard drawings to find these
-Add signage to the striping and signage sheet once TC is done

Construction Scheduling & Quantities

-Work on schedule using RSMeans for construction timing

-Drainage plan soon to be incorporated into quantities

-Wall is priced based off of square footage of face: top of footer to top of wall, footer is included in cost

Miscellaneous

-15ft construction easement past the slope easement which is past the ROW

Looking ahead to the semester

Points of Contact

Decided that there is no need for additional points of contact as Patrick, Jason, and Jake are responsive. Retaining wall plans will be sent to Gresham Structural department in Nashville for review.

Final Client Meeting

Date selected: ***April 17 @ 8:15AM at UTK***

INDIVIDUAL ACTION ITEMS

Wesley:

-Determine with team if an additional QA/QC date is needed before the 100% DD deadline; Create Outlook event for Final Client Meeting and book room; Complete retaining wall work

Logan:

-Work on R.O.W. to be completed by 100% DD turn-in

Collis:

-Assist with R.O.W.; develop quantities as available; assist with EPSC

Tyler:

-Complete retaining wall structural work

Brady:

-Send traffic control drawings; add signage to striping plan and send for review

Ben:

-Add items to construction scheduling and send for review; complete pavement schedule on typical sections; assist with retaining wall work

Zachary:

-Complete drainage plan and send for review; work on EPSC plan

MAJOR DEADLINES

100% Design Development Package due Wednesday, March 11th at 3pm

95% Contract Documents Package due Wednesday, April 22nd at 3pm

Darling, Wesley <wdarlin1@vols.utk.edu>

UTK Senior Design Project: Question about uniform retaining wall dimensions

4 messages

Darling, Wesley <wdarlin1@vols.utk.edu> Fri, Apr 3, 2020 at 9:51 AM
To: "McCrary, Donald" <donald.mccrary@greshamsmith.com>

Hi Don,

I'm not sure if you remember me but we corresponded briefly about a month ago about my senior design project at UT Knoxville that I am working on with Jason, Patrick, and Jake and you provided your expertise in reviewing my calculations for the retaining wall I was designing.

Though the course is winding down, the course instructor was reviewing my section of the report and asked about why I selected a uniform footer thickness and width and a uniform wall thickness for the entire length of the wall (with wall height being the only variance from section to section). For context, the retaining wall supports the berm of a detention pond which, because of the change in elevation along the layout of the wall, results in sections of the wall varying in height from 3 feet all the way up to ~21 feet.

Jason, Patrick, and Jake instructed us to design the footer of the wall and wall thickness to be uniform along its entire length as this would simplify the construction. Because the footer was designed for the worst-case cross-section (21 feet) this results in some sections of the wall that are only 3 feet tall having the uniform 2'x14' footer and a wall thickness of 2'. My course instructor, who previously worked as a structural engineer, said that while some uniformity along the cross-section is necessary, the extent to which we applied it was not reasonable and would result in unnecessarily high concrete costs.

My course instructor asked me to reach back out to you and see if you had any specific thoughts on how a retaining wall footer/wall thickness should be sized: **should a retaining wall footer/wall thickness be designed for the worst-case cross-section and then applied uniformly across all other cross-sections, or should there be a range of footer/thickness sizes for the parts of the wall that are shorter in height?**

I really appreciate your continued help with my team's project and I look forward to receiving your response. Please let me know if you would like any additional context about the design and I hope you have a nice weekend.

Kind regards,
Wesley Darling

McCrary, Donald <donald.mccrary@greshamsmith.com> Mon, Apr 6, 2020 at 12:19 PM
To: "Darling, Wesley" <wdarlin1@vols.utk.edu>

[External Email]

Hey Wes. Typically, we usually design for the worst case and keep this same design constant just so to simplify construction. Usually, if the majority of the wall is 21', it will probably be cheaper as far as construction goes to keep this same uniformity rather than have many different designs. Can you tell me how much of the wall is around the 21' and how much of the wall is 3-5'?

Thanks,

Donald McCrary, PE
Gresham Smith

D: 615.770.8495
M: 931.982.1885

[Quoted text hidden]

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Darling, Wesley <wdarlin1@vols.utk.edu> Tue, Apr 7, 2020 at 12:54 PM
To: "McCrary, Donald" <donald.mccrary@greshamsmith.com>

Don,

Thank you for your reply. The typical situation makes sense and was how Jake, Jason, and Patrick directed me to design the wall.

Only about 60 feet of the wall is 3'-5', whereas 160 feet of the wall length is around 21' (or has a change in height over the segment length where at least one end is around 21', which would still require the larger slab size). About 100 feet of the wall length is also 5'-10'. As the majority of the wall length requires the largest slab size, would this still govern the overall design, or would a subsequent iteration of the wall design require two different designs, one for 10' and below and one for the 21' segments?

Thank you again for your help with this and I really appreciate your response.

Kind regards,
Wes
[Quoted text hidden]

McCrary, Donald <donald.mccrary@greshamsmith.com> Tue, Apr 7, 2020 at 1:06 PM
To: "Darling, Wesley" <wdarlin1@vols.utk.edu>

I think it would be good to have two different designs for this wall since it sounds like there is an equal amount of wall length for both scenarios. Yes, I would do a design a wall for 10’ and then use this design for the 10’ and below portions. Then use the 21’ design for the portions that are greater than 10’. This should economize the wall some.

Hope this helps. Let me know if you need anything.

[Quoted text hidden]
[Quoted text hidden]

	Serviceability	Strength Ia	Strength Ib
Net Moment	17750.62 lb·ft	14787.28 lb·ft	22622 lb·ft
Resultant Moment Arm	2.473 ft	2.004 ft	2.36 ft
Eccentricity	0.527 ft	0.996 ft	0.639 ft
q_{max}	1826 psf	2454 psf	2617 psf
q_{min}	566 psf	5 psf	577 psf
Bearing Capacity	12594 psf	12594 psf	12594 psf
Bearing Check $q_{max} \leq 0.55 q_{ult}$	1826 psf $\leq$ 6926.7 psf	2454 $\leq$ 6926.7	2617 $\leq$ 6926.7
eccentricity check $e \leq \frac{B}{4}$	0.527 ft $\leq$ 1.5 ft	0.996 $\leq$ 1.5	0.639 $\leq$ 1.5
Sliding check $F_s \leq (1)P_F + (0.5)P_p$	27141 lb $\leq$ 4777.2 lb	4070 $\leq$ 5409	4070 $\leq$ 6400

1' stem thickness
1' slab thickness meets all LCs
6' slab length

Scanned with CamScanner

Stem-Slab interface

$M_u = 151.96$ k-in
$V_u = 3.798$ kip

"O" bar → use (2)#5 /ln ft @ 6" O.C.

$A_s = 0.62 in^2/ft$

$\phi M_n = 225.5$ k-in > 151.96 k-in
$\phi V_c = 8.87$ kip > 3.8 kip

Stem

$M_u = 30.0$ k-in
$V_u = 3.8$ kip

"M" bar → use (1)#5 /ln ft @ 12" O.C.

$A_s = 0.31 in^2/ft$ ← min steel must be met

$\phi M_n = 112.8$ k-in > 30 k-in
$\phi V_c = 8.87$ kip > 3.8 kip

Slab

$M_u = 74.6$ k-in
$V_u = 3.85$ kip

"P" bar → use (1) #5/ln ft @ 12" O.C.

$A_s = 0.31 in^2/ft$ ← min steel req

$\phi M_n = 112.8$ k-in > 74.6 k-in
$\phi V_c = 8.87$ kip > 3.8 kip

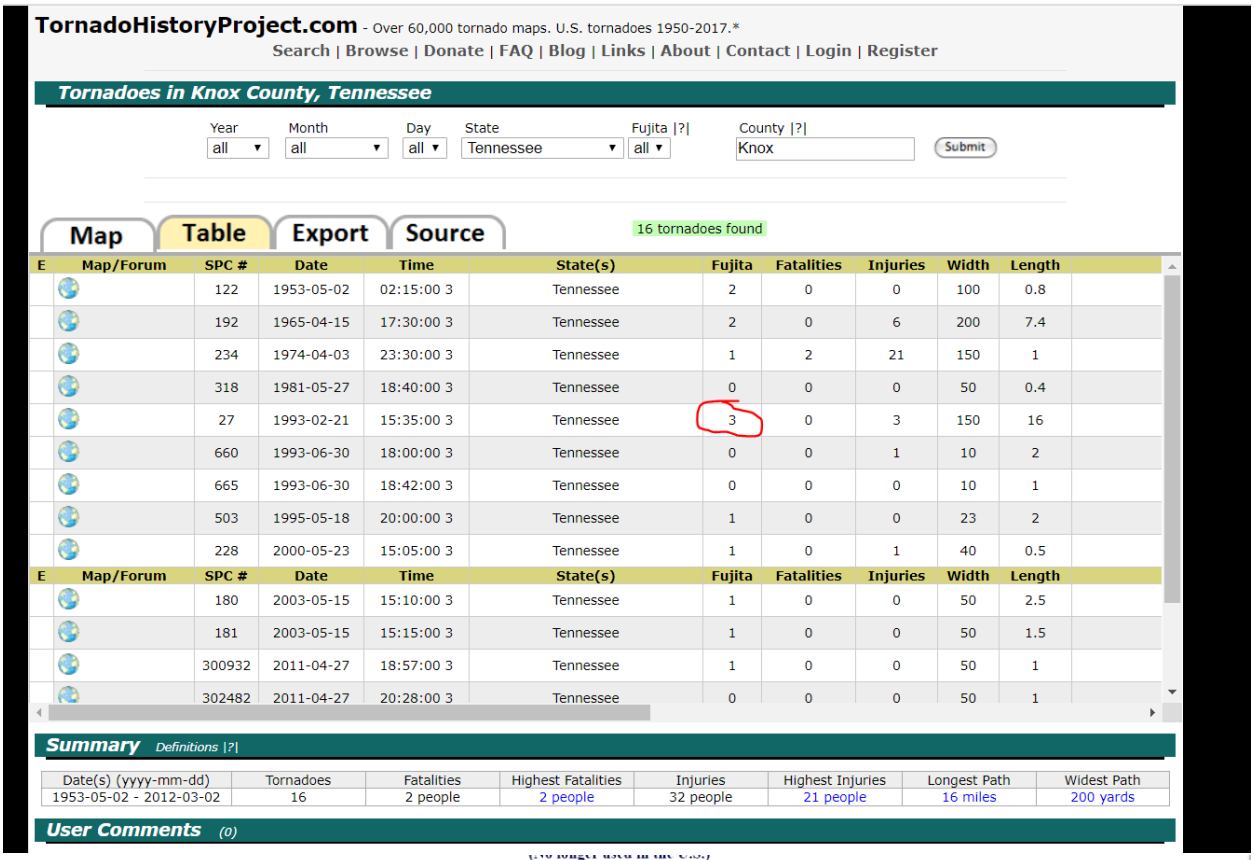
Longitudinal Bars

→ In Stem: "L_w" bars $A_{s\,req} = 3.6 in^2$ for 10' stem or $0.36 in^2/ft$ = (1)#7@16" O.C.
$A_s = 0.45 in^2/ft$

→ In Slab: "L_b" bars $A_{s\,req} = 1.6 in^2$ = (6)#5@12" O.C.
$A_s = 1.86 in^2/ft$

Scanned with CamScanner

Wind Loads Evaluation



TornadoHistoryProject.com - Over 60,000 tornado maps. U.S. tornadoes 1950-2017.*

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Tornadoes in Knox County, Tennessee

Year: all | Month: all | Day: all | State: Tennessee | Fujita |?|: all | County |?|: Knox | Submit

Map | Table | Export | Source

16 tornadoes found

E	Map/Forum	SPC #	Date	Time	State(s)	Fujita	Fatalities	Injuries	Width	Length
		122	1953-05-02	02:15:00 3	Tennessee	2	0	0	100	0.8
		192	1965-04-15	17:30:00 3	Tennessee	2	0	6	200	7.4
		234	1974-04-03	23:30:00 3	Tennessee	1	2	21	150	1
		318	1981-05-27	18:40:00 3	Tennessee	0	0	0	50	0.4
		27	1993-02-21	15:35:00 3	Tennessee	3	0	3	150	16
		660	1993-06-30	18:00:00 3	Tennessee	0	0	1	10	2
		665	1993-06-30	18:42:00 3	Tennessee	0	0	0	10	1
		503	1995-05-18	20:00:00 3	Tennessee	1	0	0	23	2
		228	2000-05-23	15:05:00 3	Tennessee	1	0	1	40	0.5
E	Map/Forum	SPC #	Date	Time	State(s)	Fujita	Fatalities	Injuries	Width	Length
		180	2003-05-15	15:10:00 3	Tennessee	1	0	0	50	2.5
		181	2003-05-15	15:15:00 3	Tennessee	1	0	0	50	1.5
		300932	2011-04-27	18:57:00 3	Tennessee	1	0	0	50	1
		302482	2011-04-27	20:28:00 3	Tennessee	0	0	0	50	1

Summary Definitions |?|

Date(s) (yyyy-mm-dd)	Tornadoes	Fatalities	Highest Fatalities	Injuries	Highest Injuries	Longest Path	Widest Path
1953-05-02 - 2012-03-02	16	2 people	2 people	32 people	21 people	16 miles	200 yards

User Comments (0)

SCALE	WIND ESTIMATE *** (MPH)	TYPICAL DAMAGE
F0	< 73	**Light damage.** Some damage to chimneys; branches broken off trees; shallow-rooted trees pushed over; sign boards damaged.
F1	73-112	**Moderate damage.** Peels surface off roofs; mobile homes pushed off foundations or overturned; moving autos blown off roads.
F2	113-157	**Considerable damage.** Roofs torn off frame houses; mobile homes demolished; boxcars overturned; large trees snapped or uprooted; light-object missiles generated; cars lifted off ground.
F3	158-206	**Severe damage.** Roofs and some walls torn off well-constructed houses; trains overturned; most trees in forest uprooted; heavy cars lifted off the ground and thrown.
F4	207-260	**Devastating damage.** Well-constructed houses leveled; structures with weak foundations blown away some distance; cars thrown and large missiles generated.
F5	261-318	**Incredible damage.** Strong frame houses leveled off foundations and swept away; automobile-sized missiles fly through the air in excess of 100 meters (109 yds); trees debarked; incredible phenomena will occur.

*** IMPORTANT NOTE ABOUT F-SCALE WINDS: F-scale winds were not meant to be used literally. These precise windspeed numbers were guesses and have not been verified in science or engineering. Different wind speeds may cause similar-looking damage from place to place -- even from building to building. *Without a thorough engineering analysis of tornado damage in any event, the actual wind speeds needed to cause that damage are unknown.* The Enhanced F-scale was implemented in February 2007.

Table 3.8.1.2.1-1—Base Pressures, P_B, Corresponding to V_B = 100 mph

Superstructure Component	Windward Load, ksf	Leeward Load, ksf
Trusses, Columns, and Arches	0.050	0.025
Beams	0.050	NA
Large Flat Surfaces	0.040	NA

$$P_D = P_B \left(\frac{V_{DZ}}{V_B} \right)^2 = P_B \frac{V_{DZ}^{\,2}}{10{,}000} \qquad (3.8.1.2.1\text{-}1)$$

P_B = base wind pressure specified in Table 3.8.1.2.1-1 (ksf)

The wind force on the structure shall be calculated by multiplying the design wind pressure, P_D, calculated using Eq. 3.8.1.2.1-1, by the exposed area, including the area of sound barriers, if existing, regardless of the design wind pressure used in designing the sound barriers themselves.

Worst tornado in Knox County → 3 on Fujita Scale

↓

206 MPH max

↓

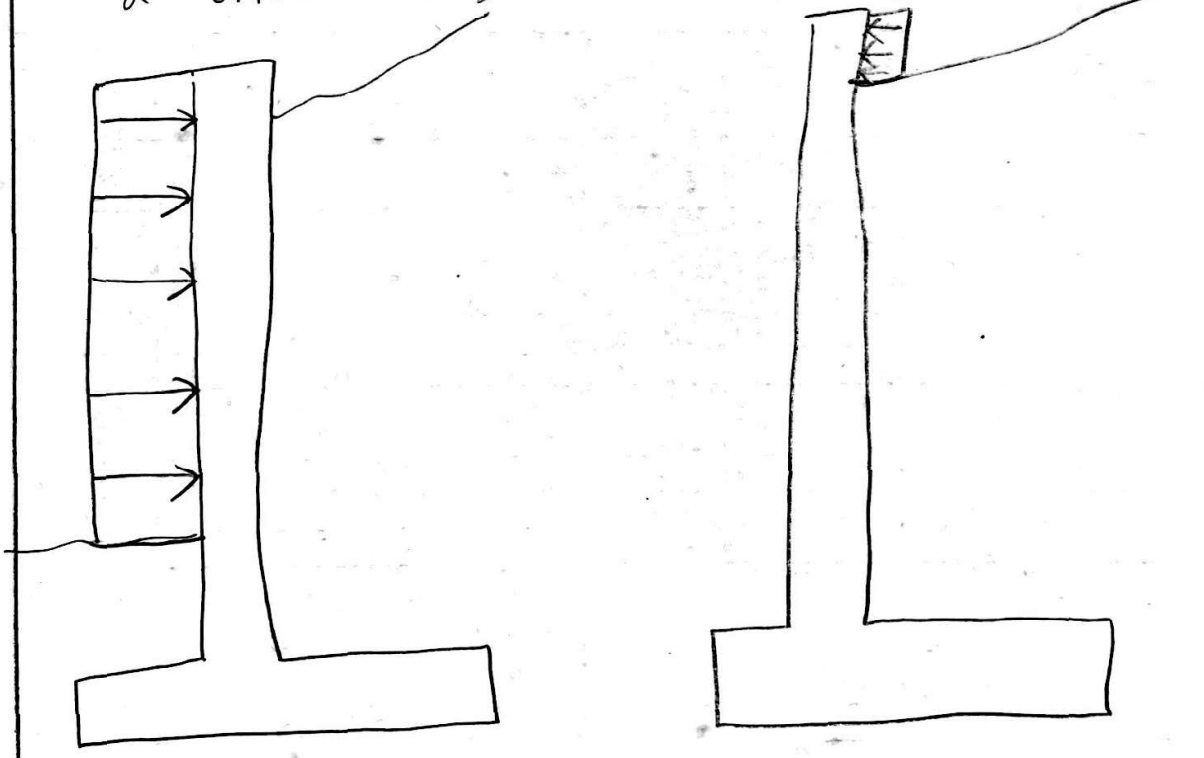
$V_{DZ} = 206$ MPH

$P_B = 0.040$ ksf for large flat surfaces

$$P_D = (0.040 \text{ ksf})\left(\frac{(206 \text{ mph})^2}{10000}\right)$$

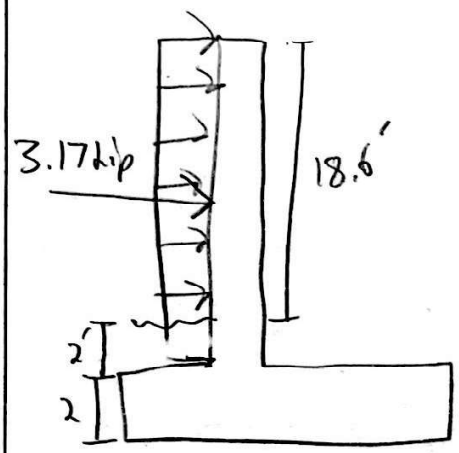
$$P_D = 0.1697 \text{ ksf}$$

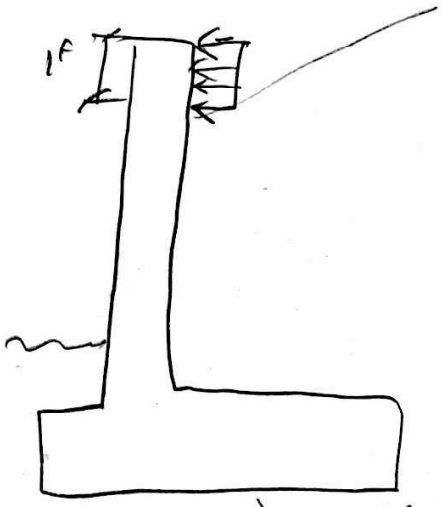
2 different ways wind can hit the wall:



Scanned with CamScanner

Design 1 : 20.66' tall wall





$(18.66)(0.1697\ ksf) = 3.17\ kip$

$(1')(0.1697) = 0.1697\ kip$

Str III = $1.4(3.17) = 4.44$ kip

Service = $0.3(3.17) = 0.951$ kip

Arm = $2' + 2' + \frac{18.66}{2} = 13.33'$

Str III moment = 59.19 kip-ft

Service moment = 12.68 kip-ft

Str II = 0.238 kip

Service = 0.051 kip

Arm = $2 + 2 + (18.66 - 0.5) = 22.16'$

Str II moment = 5.27 k-ft

Service = 1.13 k-ft

	Str III	Service	Str III	Service
Bearing:	2611 psf < 6510 psf	2837 psf < 6510 psf	4582 psf < 6510 psf	3260 psf < 6510 psf
Eccentricity:	0.091 ft < 3.5 ft	0.385 ft < 3.5'	1.92 < 3.5'	0.79 ft < 3.5 ft
Sliding:	19534 lb < 23977 lb	13023 lb < 19046	19534 < 19542	13023 lb < 18095 lb

LC = 0.9DL + 1.6WL

Stem-Slab
$M_U = 1056$ k-in < 1867 k-in
$V_U = 13.2$ kip < 23.46 k

Stem:
$M_U = 767.62$ k-in < 1018.6 k-in
$V_U = 13.2$ kip < 23.46 k

Slab:
$M_U = 509.6$ k-in < 848.9 k-in
$V_U = 7.97$ k < 23.46 k

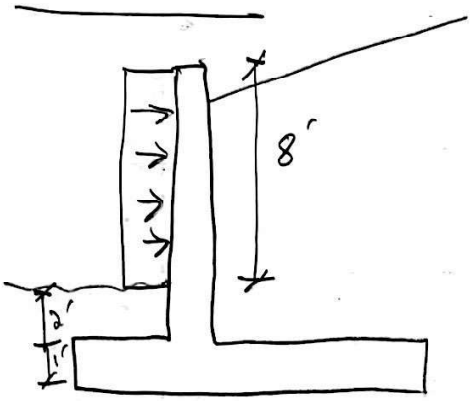
Stem-Slab
$M_U = 1480$ k-in < 1867 k-in
$V_U = 18.5$ kip < 23.46 kip

Stem:
$M_U = 950.5$ k-in < 1018.6 k-in
$V_U = 18.5$ kip < 23.46 kip

Slab:
$M_U = 740$ k-in < 848.9 k-in
$V_U = 18.98$ kip < 23.46 kip

Scanned with CamScanner

Design 2: 10' tall wall



$(8')(0.1697\ ksf) = 1.36\ k/ft$

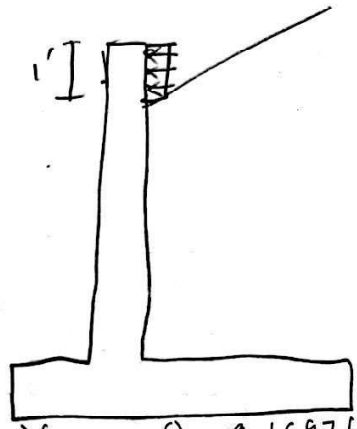
Str III = 1.9 k/ft

Service = 0.41 k/ft

$Arm = 2' + 1' + \frac{8'}{2} = 7'$

Str III moment = 13.3 k-ft

Serv moment = 2.87 k-ft



$(1')(0.1697\ ksf) = 0.1697\ k/ft$

Str III = 0.238 k/ft

Service = 0.051 k/ft

$Arm = 2' + 1' + (8 - 0.5) = 10.5'$

Str III moment = 2.5 k-ft

Serv moment = 0.54 k-ft

	Str III	Service	Str III	Service
Bearing	2223 psf < 6928 psf	1351 psf < 6928 psf	2870 psf < 6928 psf	1915 psf < 6928 psf
Eccentricity	0.81' < 1.5'	0.13' < 1.5'	1.33' < 1.5'	0.60' < 1.5'
Sliding	40701b < 73101b	27131b < 51851b	40701b < 54101b	27141b < 47771b

$LC = 0.9DL + 1.6WL$

Stem-Slab:

$M_U = 152.5\ k\text{-in} < 225.5\ k\text{-in}$

$V_U = 1.63\ k < 8.87\ k/ft$

Stem:

$M_U = 65.2\ k\text{-in} < 112.8\ k\text{-in}$

$V_U = 1.63\ k < 8.87\ k/ft$

Slab:

$M_U = 33.3\ k\text{-in} < 112.8\ k\text{-in}$

$V_U = 2.5\ k < 8.87\ k/ft$

Stem-Slab:

$M_U = 164\ k\text{-in} < 225.5\ k\text{-in}$

$V_U = 4.1\ k/ft < 8.87\ k/ft$

Stem:

$M_U = 64.3\ k\text{-in} < 112.8\ k\text{-in}$

$V_U = 4.1\ k/ft < 8.87\ k/ft$

Slab:

$M_U = 98.7\ k\text{-in} < 112\ k\text{-in}$

$V_U = 4.64\ k/ft < 8.87\ k/ft$

Scanned with CamScanner

Seismic Analysis

11.5.4.2—Extreme Event I, No Analysis

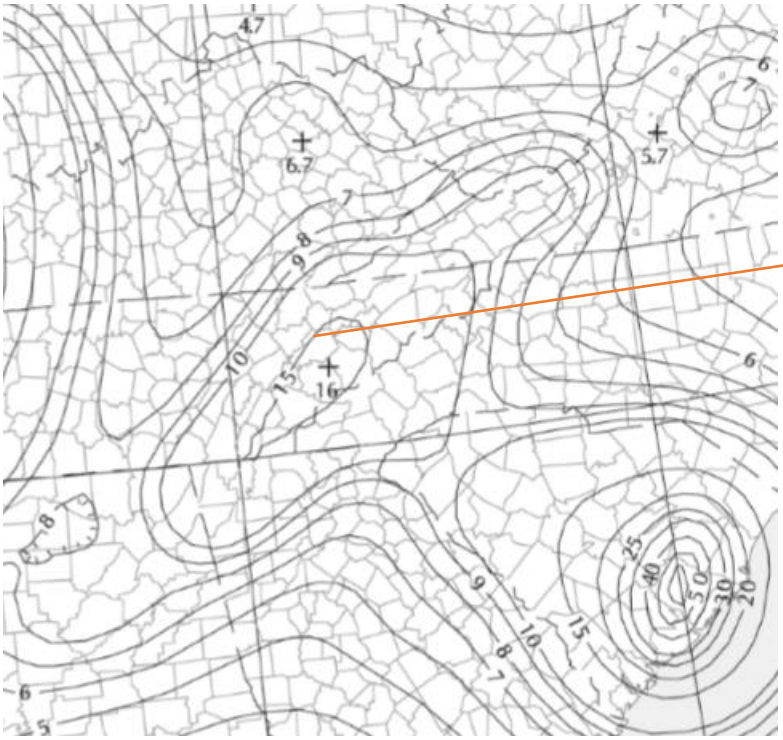
A seismic design shall not be considered mandatory for walls located in Seismic Zones 1 through 3, or for walls at sites where the site adjusted peak ground acceleration, A_s, is less than or equal to $0.4g$, unless one or more of the following is true:

- Liquefaction induced lateral spreading or slope failure, or seismically induced slope failure, due to the presence of sensitive clays that lose strength during the seismic shaking, may impact the stability of the wall for the design earthquake.
- The wall supports another structure that is required, based on the applicable design code or specification for the supported structure, to be designed for seismic loading and poor seismic performance of the wall could impact the seismic performance of that structure.

The no-seismic-analysis option should be limited to internal and external seismic stability design of the wall. If the wall is part of a bigger slope, overall seismic stability of the wall and slope combination should still be evaluated.

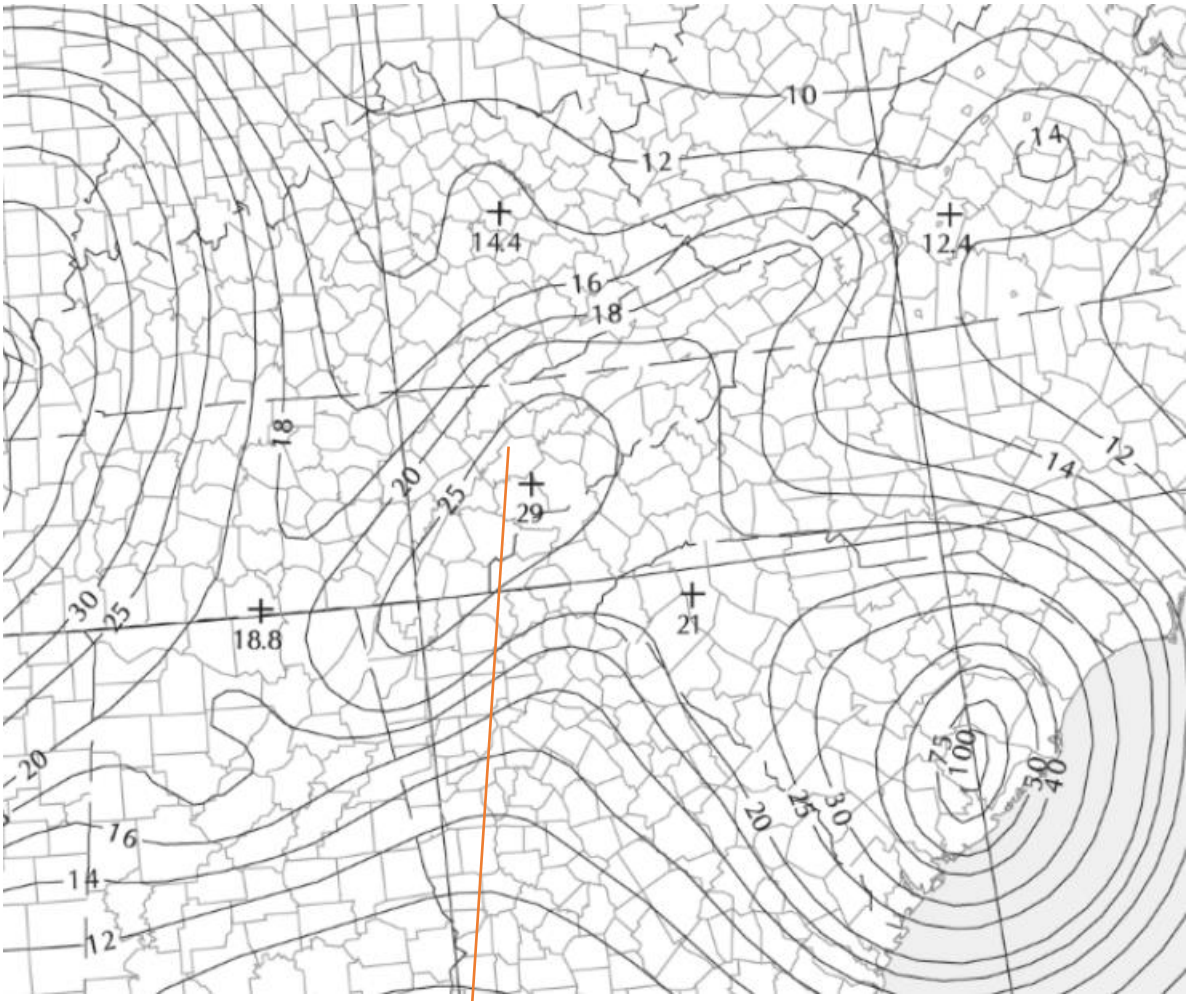
These no-seismic-analysis provisions shall not be considered applicable to walls functioning as support piers for bridges.

PGA Map



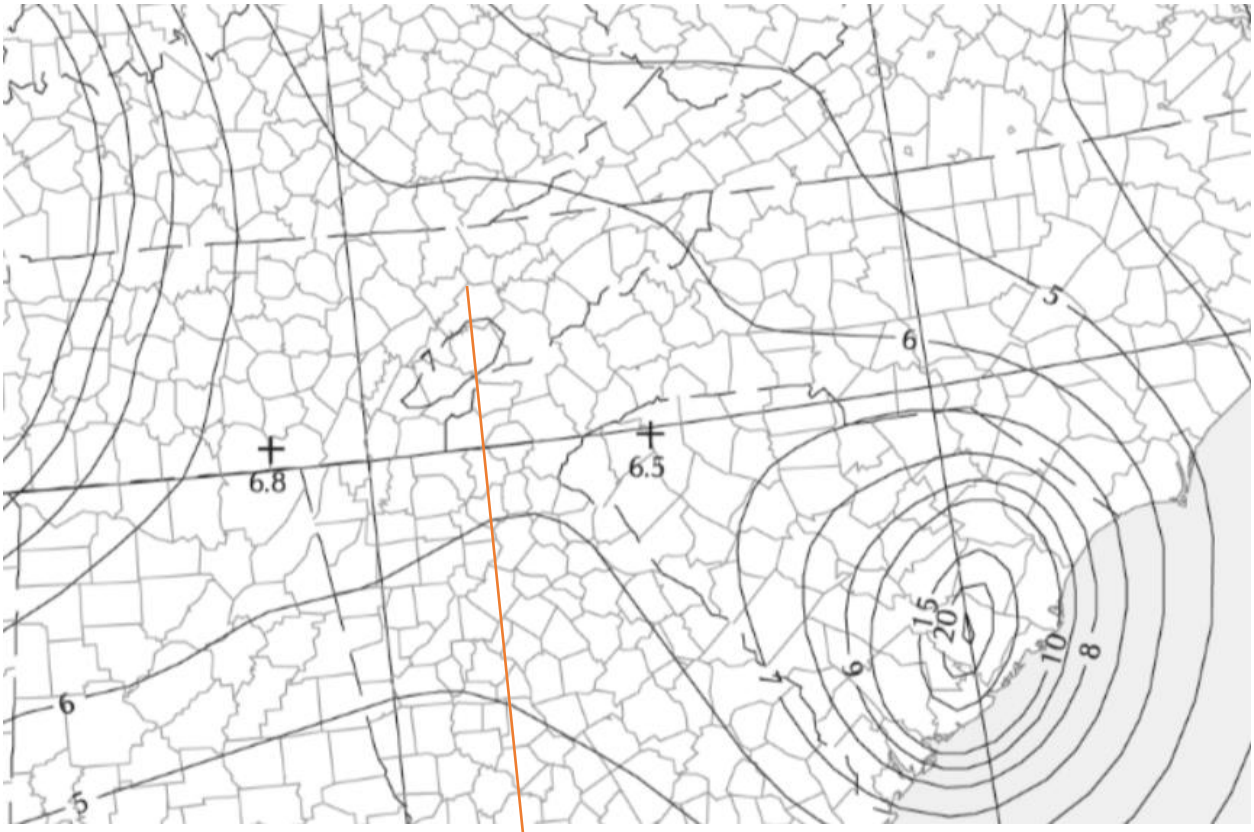
PGA = 15% g

Ss Map


10
12
14
14.4
12.4
16
18
18
20
25
29
14
12
30
25
18.8
21
20
16
14
12
75
100
30
40
20
25
30

Ss = 26% g

S1 Map



S1 = 6.75% g

3.10.6—Seismic Performance Zones

Each bridge shall be assigned to one of the four seismic zones in accordance with Table 3.10.6-1 using the value of S_{D1} given by Eq. 3.10.4.2-6.

Table 3.10.6-1—Seismic Zones

Acceleration Coefficient, S_{D1}	Seismic Zone
$S_{D1} \leq 0.15$	1
$0.15 < S_{D1} \leq 0.30$	2
$0.30 < S_{D1} \leq 0.50$	3
$0.50 < S_{D1}$	4

$$S_{D1} = F_v S_1 \qquad (3.10.4.2\text{-}6)$$

where:

S_1 = horizontal response spectral acceleration coefficient at 1.0 sec period on rock (Site Class B)

Table 3.10.3.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\overline{v}_s > 5{,}000$ ft/s
B	Rock with 2,500 ft/sec $< \overline{v}_s < 5{,}000$ ft/s
C	Very dense soil and soil rock with 1,200 ft/sec $< \overline{v}_s < 2{,}500$ ft/s, or with either $\overline{N} > 50$ blows/ft, or $\overline{s}_u > 2.0$ ksf
D	Stiff soil with 600 ft/s $< \overline{v}_s < 1{,}200$ ft/s, or with either $15 < \overline{N} < 50$ blows/ft, or $1.0 < \overline{s}_u < 2.0$ ksf
E	Soil profile with $\overline{v}_s < 600$ ft/s or with either $\overline{N} < 15$ blows/ft or $\overline{s}_u < 1.0$ ksf, or any profile with more than 10 ft of soft clay defined as soil with $PI > 20$, $w > 40$ percent and $\overline{s}_u < 0.5$ ksf
F	Soils requiring site-specific evaluations, such as: • Peats or highly organic clays ($H > 10$ ft of peat or highly organic clay where H = thickness of soil) • Very high plasticity clays ($H > 25$ ft with $PI > 75$) • Very thick soft/medium stiff clays ($H > 120$ ft)

Table 3.10.3.2-3—Values of Site Factor, F_v, for Long-Period Range of Acceleration Spectrum

Site Class	Spectral Acceleration Coefficient at Period 1.0 sec (S_1)[1]				
	$S_1 <$ 0.1	$S_1 =$ 0.2	$S_1 =$ 0.3	$S_1 =$ 0.4	$S_1 >$ 0.5
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F[2]	*	*	*	*	*

Notes:

[1]Use straight-line interpolation for intermediate values of S_1.

[2]Site-specific geotechnical investigation and dynamic site response analysis should be performed for all sites in Site Class F.

Table 3.10.3.2-1—Values of Site Factor, F_{pga}, at Zero-Period on Acceleration Spectrum

Site Class	Peak Ground Acceleration Coefficient (PGA)[1]				
	$PGA <$ 0.10	$PGA =$ 0.20	$PGA =$ 0.30	$PGA =$ 0.40	$PGA >$ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F[2]	*	*	*	*	*

Notes:

[1]Use straight-line interpolation for intermediate values of PGA.

[2]Site-specific geotechnical investigation and dynamic site response analysis should be performed for all sites in Site Class F.

Interpolation Required:

Fpga = 2.1

Table 3.10.3.2-2—Values of Site Factor, F_a, for Short-Period Range of Acceleration Spectrum

Site Class	Spectral Acceleration Coefficient at Period 0.2 sec (S_S)[1]				
	$S_S <$ 0.25	$S_S =$ 0.50	$S_S =$ 0.75	$S_S =$ 1.00	$S_S >$ 1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F[2]	*	*	*	*	*

Notes:

[1]Use straight-line interpolation for intermediate values of S_S.

[2]Site-specific geotechnical investigation and dynamic site response analysis should be performed for all sites in Site Class F.

Interpolation Required:

Fa = 2.468

- PGA = 15%

Short S_S = 26%

Long S_1 = 6.75%

Class E soil

Interpolation

$$F_{pga} = 2.5 + \left(\frac{2.5-1.7}{0.10-0.20}\right)(0.15-0.10) = 2.1$$

$$F_a = 2.5 + \left(\frac{2.5-1.7}{0.25-0.5}\right)(0.26-0.25) = 2.468$$

$$F_v = 3.5$$

$$A_S = F_{pga} PGA = (2.1)(0.15) = 0.315 < 0.4 \rightarrow \text{No Analysis}$$

$$S_{DS} = F_a S_S = (2.468)(0.26) = 0.642$$

$$S_{D1} = F_v S_1 = (3.5)(0.0675) = 0.236$$

↓

Seismic Zone 2

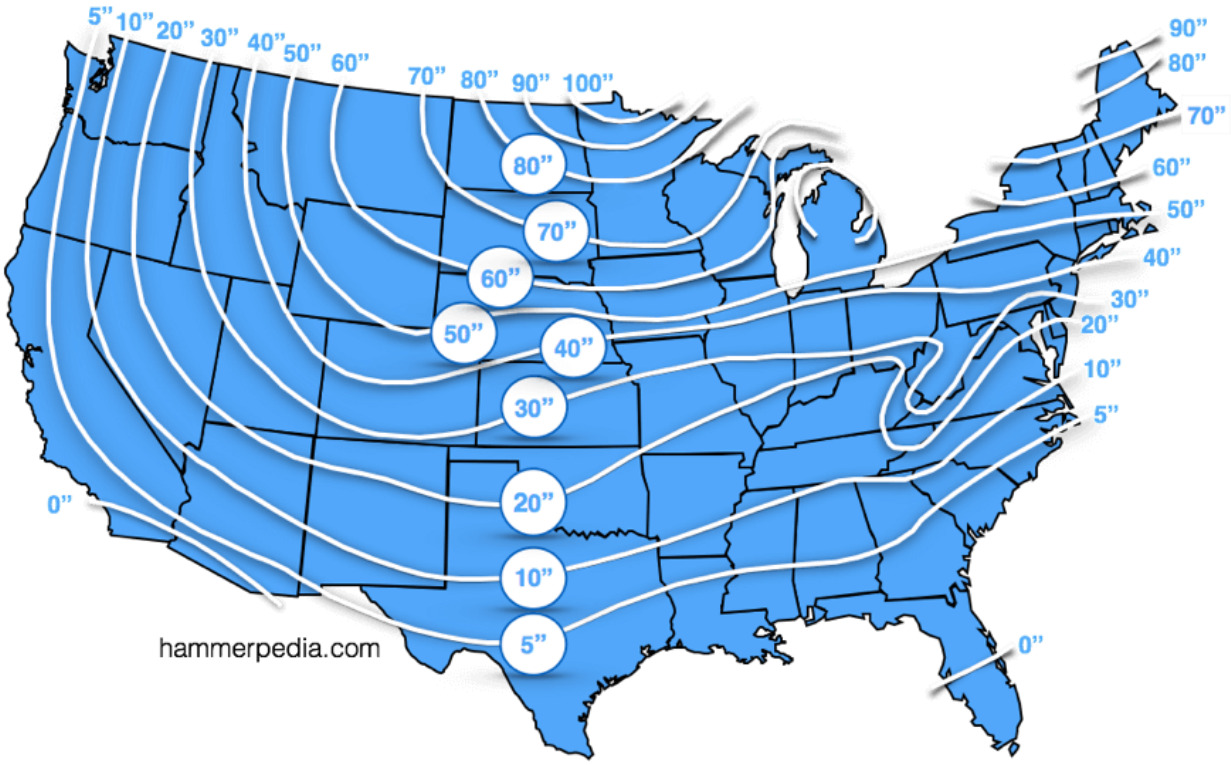
↓

No Analysis

No seismic analysis required

Scanned with CamScanner

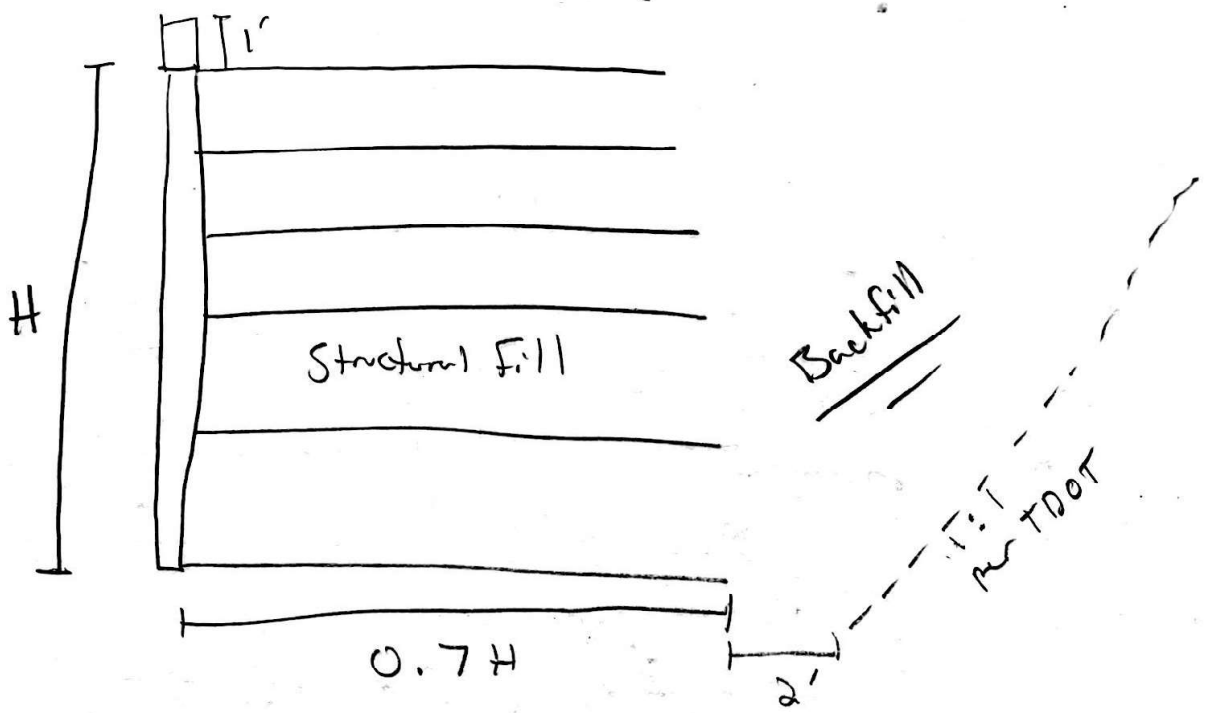
Frost Line



https://www.hammerpedia.com/frost-line-map/

Knoxville is approximately at the 20” line → design footer at 24” depth

MSE Backfill/R.O.W. | Wesley Darling



Backfill needed for MSE = 3443 c.y.

Backfill needed for C-I-P = 3124 c.y.

R.O.W.

For MSE = $(0.7H + 2 + (H-1))20'$ per section
from edge of wall

For C-I-P = $(3' + 2' + (H-1))20'$ per section
from edge of wall
For 0-10' tall

$(7.33' + 2' + (H-1))20'$ per section
For 10' tall

Additional R.O.W. req'd past edge of wall

For MSE = 6904 ft^2

For C-I-P = 6156 ft^2

Scanned with CamScanner

MSE Backfill Calculations

Station	Distance (ft)	Backfill Area (ft^2)	Average End Area (ft^2)	Volume (ft^3)
3+20		46.58		
	20		28.99	579.8
3+40		11.4		
	20		14.39	287.8
3+60		17.38		
	20		17.92	358.4
3+80		18.46		
	20		25.79	515.8
4+00		33.12		
	20		41.9	838
4+20		50.68		
	20		78.78	1575.6
4+40		106.88		
	20		193.7	3874
4+60		280.52		
	20		395.15	7903
4+80		509.78		
	20		593.97	11879.4
5+00		678.16		
	20		624.625	12492.5
5+20		571.09		
	20		585.11	11702.2
5+40		599.13		
	20		583.215	11664.3
5+60		567.3		
	20		559.195	11183.9
5+80		551.09		
	20		412.47	8249.4
6+00		273.85		
	20		232.165	4643.3
6+20		190.48		
	20		139.795	2795.9
6+40		89.11		
	20		62.58	1251.6
6+60		36.05		
	20		31.29	625.8
6+80		26.53		
	20		26.53	530.6
			TOTAL	92951.3 ft^3
				3442.640741 cy

Cast-In-Place Concrete Cantilever Right of Way Requirements

Station	Height of Wall (ft)	R.O.W. Length (ft)	Average R.O.W. Length (ft)	R.O.W. Area (ft^2)
3+20	6.42	10.42		
			8.71	174.2
3+40	3	7		
			7.24	144.8
3+60	3.48	7.48		
			7.705	154.1
3+80	3.93	7.93		
			8.655	173.1
4+00	5.38	9.38		
			10.21	204.2
4+20	7.04	11.04		
			14.5	290
4+40	9.63	17.96		
			21.55	431
4+60	16.81	25.14		
			26.835	536.7
4+80	20.2	28.53		
			28.76	575.2
5+00	20.66	28.99		
			27.14	542.8
5+20	16.96	25.29		
			25.485	509.7
5+40	17.35	25.68		
			25.405	508.1
5+60	16.8	25.13		
			24.685	493.7
5+80	15.91	24.24		
			22.02	440.4
6+00	11.47	19.8		
			18.44	368.8
6+20	8.75	17.08		
			13.675	273.5
6+40	6.27	10.27		
			9.2	184
6+60	4.13	8.13		
			7.565	151.3
6+80	3	7		
			TOTAL	6155.6 ft^2

MSE Wall Right of Way Requirements

Station	Height of Wall (ft)	R.O.W. Length (ft)	Average R.O.W. Length (ft)	R.O.W. Area (ft^2)
3+20	6.42	11.914		
			9.007	180.14
3+40	3	6.1		
			6.508	130.16
3+60	3.48	6.916		
			7.2985	145.97
3+80	3.93	7.681		
			8.9135	178.27
4+00	5.38	10.146		
			11.557	231.14
4+20	7.04	12.968		
			15.1695	303.39
4+40	9.63	17.371		
			23.474	469.48
4+60	16.81	29.577		
			32.4585	649.17
4+80	20.2	35.34		
			35.731	714.62
5+00	20.66	36.122		
			32.977	659.54
5+20	16.96	29.832		
			30.1635	603.27
5+40	17.35	30.495		
			30.0275	600.55
5+60	16.8	29.56		
			28.8035	576.07
5+80	15.91	28.047		
			24.273	485.46
6+00	11.47	20.499		
			18.187	363.74
6+20	8.75	15.875		
			13.767	275.34
6+40	6.27	11.659		
			9.84	196.8
6+60	4.13	8.021		
			7.0605	141.21
6+80	3	6.1		
			TOTAL	6904.32 ft^2

Appendix C: Stormwater & Drainage Design

	Interstate System and Arterial With Full Access Control	Arterial Without Full Access Control	Collector	Local Road
Inlet Design Frequency	50-yr	10-yr [1]	10-yr [1]	10-yr
Sewer Design Frequency	50-yr	10-yr [1]	10-yr [1]	10-yr
Culvert Design Frequency	50-yr Check for 100-yr	50-yr Check for 100-yr	50-yr Check for 100-yr	50-yr Check for 100-yr
Roadway Freeboard [2]	50-yr	50-yr	50-yr	50-yr
Ditch Design Frequency	50-yr	10-yr [1]	10-yr [1]	10-yr

[1] 50-year in Roadway Sag Sections

[2] The design high water elevation should be at or below the bottom of the roadway subgrade.

Table 4-1
Hydrologic Design Criteria

	A	B	C	D	E	F	G
1							
2		**OB 8**					
3		**Time of Concentration *Figure 4A-13, TDOT Drainage Manual***					
4			Scenario 1	Scenario 2	Scenario 3	Scenario 4	
5		**Sheet Flow**					
6		Surface Description (Table 4-3)					
7		Manning's roughness coefficient, n (Table 4-3)	0.35				
8		Flow length (ft)	205.07				
9		Rainfall Intensity (in/hr) (Figure 4A-7)	4.5				
10		Land Slope (ft/ft)	0.01341				
11		Compute $t_{t(sheet)}$ (Kinematic Wave Equation)	24.337239	#DIV/0!	#DIV/0!	#DIV/0!	
12		**Shallow Concentrated Flow**					
13		Surface Description (paved or unpaved)			u		
14		Flow length, L (ft)	0		466	614.12	
15		Watercourse Slope, S (ft/ft)	0		0.08798	0.06839	
16		Average velocity, V (ft/s) (Figure 4A-12)	0		4.7857223	5.3161255	
17		Compute $t_{t(shallow)}$	0	#DIV/0!	0.027048	0.032089	
18		**Channel Flow**					
19		Cross Sectional flow area, a (ft^s)	10.125	13.4076	3.1415	13.78	
20		Wetted perimeter, p_w (ft)	9.48	5.46	6.28	1.067997	
21		Hydraulic radius, r (ft)	1.068038	2.4556044	0.5002389	12.902658	
22		Channel Slope, S	0.063055	0.0104	0.03647	0.08	
23		Mannings coefficient, n (Table 5A-1)	0.35	0.012	0.012	0.35	
24		Velocity (ft/s)	1.1169553	23.047684	14.942543	6.6239342	
25		Flow Length, L (ft)	218.58	24	548.26	556.6	
26		Compute $t_{t(channel)}$	0.0543591	0.0002893	0.010192	0.0233413	24.42542
27		Time of Concentration	24.391598	#DIV/0!	#DIV/0!	#DIV/0!	
28							
29							
30							
31		Calculated Values					
32		Inputs					
33		Guess (Interative)					

OB 2

Time of Concentration *Figure 4A-13, TDOT Drainage Manual*				
	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Sheet Flow				
Surface Description (Table 4-3)				
Manning's roughness coefficient, n (Table 4-3)	0.35	0.35	0.35	0.35
Flow length (ft)	210.72	300	284.33	284.33
Rainfall Intensity (in/hr) (Figure 4A-7)	5.3	7.8	5.8	5.8
Land Slope (ft/ft)	0.0533	0.1708	0.18288	0.18288
Compute $t_{t(sheet)}$ (Kinematic Wave Equation)	15.315762	11.43752923	12.2156397	12.2156397
Shallow Concentrated Flow				
Surface Description (paved or unpaved)			p	
Flow length, L (ft)	0	16.8	614.12	614.12
Watercourse Slope, S (ft/ft)	0	0.595	0.06839	0.06839
Average velocity, V (ft/s) (Figure 4A-12)	0	12.44554714	5.316125522	5.316125522
Compute $t_{t(shallow)}$	0	0.000374967	0.032088951	0.032088951
Channel Flow				
Cross Sectional flow area, a (ft^s)	13.78	13.78	3	3.16228
Wetted perimeter, p_w (ft)	11.0676	11.0676	6.32	0.237171
Hydraulic radius, r (ft)	1.245075717	1.245075717	0.474683544	13.33333333
Channel Slope, S	0.0755	0.0755	0.08365	
Mannings coefficient, n (Table 5A-1)	0.35	0.35	0.35	
Velocity (ft/s)	1.353803217	1.353803217	0.749239469	#DIV/0!
Flow Length, L (ft)	89.5	437	119.54	
Compute $t_{t(channel)}$	0.018363903	0.089665091	0.044319015	#DIV/0!
Time of Concentration	15.3341259	11.52756928	12.29204766	#DIV/0!

	O	P	Q	R	S	T	U
1							
2		**Outfall 2**					
3		**Time of Concentration *Figure 4A-13, TDOT Drainage Manual***					
4			Scenario 1	Scenario 2	Scenario 3	Scenario 4	
5		**Sheet Flow**					
6		Surface Description (Table 4-3)					
7		Manning's roughness coefficient, n (Table 4-3)	0.35				
8		Flow length (ft)	300				
9		Rainfall Intensity (in/hr) (Figure 4A-7)	3.9				
10		Land Slope (ft/ft)	0.0175				
11		Compute $t_{t(sheet)}$ (Kinematic Wave Equation)	29.8936648	#DIV/0!	#DIV/0!	#DIV/0!	
12		**Shallow Concentrated Flow**					
13		Surface Description (paved or unpaved)	u		u		
14		Flow length, L (ft)	586.05		466	614.12	
15		Watercourse Slope, S (ft/ft)	0.0921		0.08798	0.06839	
16		Average velocity, V (ft/s) (Figure 4A-12)	4.896495125		4.785722255	5.316125522	
17		Compute $t_{t(shallow)}$	0.03324657	#DIV/0!	0.027048048	0.032088951	
18		**Channel Flow**					
19		Cross Sectional flow area, a (ft^s)	13.4076	13.4076	3.1415	13.78	
20		Wetted perimeter, p_w (ft)	5.46	5.46	6.28	1.067997	
21		Hydraulic radius, r (ft)	2.455604396	2.455604396	0.500238854	12.90265797	
22		Channel Slope, S	0.08	0.0104	0.03647	0.08	
23		Mannings coefficient, n (Table 5A-1)	0.35	0.012	0.012	0.35	
24		Velocity (ft/s)	2.191637985	23.04768414	14.94254311	6.623934166	
25		Flow Length, L (ft)	623.99	24	548.26	556.6	
26		Compute $t_{t(channel)}$	0.07908722	0.000289255	0.010192003	0.023341281	
27		Time of Concentration	30.00599859	#DIV/0!	#DIV/0!	#DIV/0!	
28							
29							
30							
31							
32							
33							

	V	W	X	Y	Z	AA	AB
1							
2		**OB 6**					
3		**Time of Concentration *Figure 4A-13, TDOT Drainage Manual***					
4			Scenario 1	Scenario 2	Scenario 3	Scenario 4	
5		**Sheet Flow**					
6		Surface Description (Table 4-3)					
7		Manning's roughness coefficient, n (Table 4-3)	0.35				
8		Flow length (ft)	125.46				
9		Rainfall Intensity (in/hr) (Figure 4A-7)	5.9				
10		Land Slope (ft/ft)	0.04646				
11		Compute $t_{t(sheet)}$ (Kinematic Wave Equation)	11.20165696	#DIV/0!	#DIV/0!	#DIV/0!	
12		**Shallow Concentrated Flow**					
13		Surface Description (paved or unpaved)			u		
14		Flow length, L (ft)	0		466	614.12	
15		Watercourse Slope, S (ft/ft)	0		0.08798	0.06839	
16		Average velocity, V (ft/s) (Figure 4A-12)	0		4.785722255	5.316125522	
17		Compute $t_{t(shallow)}$	0	#DIV/0!	0.027048048	0.032088951	
18		**Channel Flow**					
19		Cross Sectional flow area, a (ft^s)	10.83	13.4076	3.1415	13.78	
20		Wetted perimeter, p_w (ft)	12.02	5.46	6.28	1.067997	
21		Hydraulic radius, r (ft)	0.900998336	2.455604396	0.500238854	12.90265797	
22		Channel Slope, S	0.0117	0.0104	0.03647	0.08	
23		Mannings coefficient, n (Table 5A-1)	0.35	0.012	0.012	0.35	
24		Velocity (ft/s)	0.429563278	23.04768414	14.94254311	6.623934166	
25		Flow Length, L (ft)	232.22	24	548.26	556.6	
26		Compute $t_{t(channel)}$	0.150165433	0.000289255	0.010192003	0.023341281	
27		Time of Concentration	11.35182239	#DIV/0!	#DIV/0!	#DIV/0!	
28							
29							
30							
31							
32							
33							

OB 7

Time of Concentration *Figure 4A-13, TDOT Drainage Manual*				
	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Sheet Flow				
Surface Description (Table 4-3)				
Manning's roughness coefficient, n (Table 4-3)	0.35			
Flow length (ft)	127.86			
Rainfall Intensity (in/hr) (Figure 4A-7)	6			
Land Slope (ft/ft)	0.0481			
Compute $t_{t(sheet)}$ (Kinematic Wave Equation)	11.13731395	#DIV/0!	#DIV/0!	#DIV/0!
Shallow Concentrated Flow				
Surface Description (paved or unpaved)			u	
Flow length, L (ft)	0		466	614.12
Watercourse Slope, S (ft/ft)	0		0.08798	0.06839
Average velocity, V (ft/s) (Figure 4A-12)	0		4.785722255	5.316125522
Compute $t_{t(shallow)}$	0	#DIV/0!	0.027048048	0.032088951
Channel Flow				
Cross Sectional flow area, a (ft^s)	3.76	13.4076	3.1415	13.78
Wetted perimeter, p_w (ft)	7.08	5.46	6.28	1.067997
Hydraulic radius, r (ft)	0.531073446	2.455604396	0.500238854	12.90265797
Channel Slope, S	0.005	0.0104	0.03647	0.08
Mannings coefficient, n (Table 5A-1)	0.35	0.012	0.012	0.35
Velocity (ft/s)	0.197411728	23.04768414	14.94254311	6.623934166
Flow Length, L (ft)	33.53	24	548.26	556.6
Compute $t_{t(channel)}$	0.047180018	0.000289255	0.010192003	0.023341281
Time of Concentration	11.18449396	#DIV/0!	#DIV/0!	#DIV/0!

OB 2 and SB 0

i	5.2
t	15.42 minutes

Surface Type and Condition (Table 4-2)	Runoff Coefficent	Acres	C X A	Q = CiA
Forested Areas	0.3	1.6193	0.48579	
Asphalt Pavement	0.9	1.8607	1.67463	
Turf meadows	0.4	4.972	1.9888	
		8.452	4.14922	
			0.490916	21.57594

Outfall 2

i	3.9
t	30 minutes

Surface Type and Condition (Table 4-2)	Runoff Coefficent	Acres	C X A	Q = CiA	
	0.4		13.6688	21.32333	pond
				42.89927	

OB 1

i	3.9
i	5.3
t	15 minutes

Surface Type and Condition (Table 4-2)	Runoff Coefficent	Acres	C X A	Q = CiA
Impervious	0.9	2.3549	2.11941	
Grass	0.4	5.7536	2.30144	
Forest	0.3	2.6285	0.78855	
		10.737	5.2094	
			0.485182	27.60982

OB 2 + OB 1 + Outfall 2 70.50909

Retardance Class	Concrete
C	
slope	0.04846
max velocity	10

Trial Depth	Flow Area	Wetted Perim	Radius	n	Q	V	max shear stress	T	F
1.05	4.41	8.734772033	0.5049	0.013	70.34472	15.95118	3.1750992	8.4	3.879585
1.06	4.4944	8.817960338	0.5097	0.013	72.14569	16.05235	3.20533824	8.48	3.885731
2.11	17.8084	17.55273237	1.0146	0.013	452.4608	25.40715	6.38043744	16.88	4.359151
2.17	18.8356	18.0518622	1.0434	0.013	487.5933	25.88679	6.56187168	17.36	4.37961
2.18	19.0096	18.13505051	1.0482	0.013	493.609	25.9663	6.59211072	17.44	4.382974
2.03	16.4836	16.88722593	0.9761	0.013	408.1423	24.7605	6.13852512	16.24	4.331103

1.06 feet plus 1 foot of freeboard (Concrete) 1:2 slope and 1:6 slope

SB 2 and EW-1

i	7.8
t	5 minutes

Surface Type and Condition (Table 4-2)	Runoff Coefficent	Acres	C X A	Q = CiA
Turn meadows	0.4	0.2176	0.08704	
Asphalt Pavement	0.9	0.2561	0.23049	
		0.4737	0.31753	
			0.670319	2.476734

Q from EW-1 13.93583
Total Q 16.41256

Retardance Class	Class C rip rap
C	23
slope	0.08
allowable shear	6.7

Trial Depth	Flow Area	Wetted Perim	Radius	n	Q	V	max shear stress
1	4.5	9.24504019	0.4867	0.072	16.25082	3.611294	4.992
1.1	5.445	10.16954421	0.5354	0.072	20.95413	3.848326	5.4912
1.2	6.48	11.09404823	0.5841	0.072	26.42724	4.078277	5.9904
1.15	5.95125	10.63179622	0.5598	0.072	23.59155	3.964134	5.7408
1.12	5.6448	10.35444501	0.5452	0.072	21.98568	3.894855	5.59104
1.11	5.54445	10.26199461	0.5403	0.072	21.46603	3.871625	5.54112

1.12 feet plus 6 inches of freeboard (Riprap Class A-1) 1:3 slope and 1:6 slope

SB 0
Sta 2+90 to Sta 5+90

i	7.8
TOC	5 minutes

Surface Type and Condition (Table 4-2)	Acres	Runoff Coefficient	C X A	Q = CiA
Turf meadows	0.1853	0.4	0.07412	
	0.1853		0.07412	
			0.4	0.578136

Retardance Class	Class A rip rap
C	
slope	0.08
allowable shear	3

Trial Depth	Flow Area	Wetted Perim	Radius	n	Q	V	max shear stress
1	5	10.20586816	0.4899	0.124	10.52985	2.10597	4.992
0.5	1.25	5.102934078	0.245	0.124	1.657964	1.326372	2.496
0.4	0.8	4.082347262	0.196	0.124	0.914358	1.142947	1.9968
0.41	0.8405	4.184405944	0.2009	0.124	0.9766	1.161927	2.04672
0.39	0.7605	3.980288581	0.1911	0.124	0.854656	1.123808	1.94688
0.68	2.312	6.939990346	0.3331	0.124	3.76464	1.628305	3.39456

0.39 feet (rip rap Class A) plus .39 feet of freeboard 1:4 slope and 1:6 slope

OB 9
Sta. 0+00 to 2+00 South Side
Example Problem 1: 5.08.2.1 TDOT Drainage Manual

i	7.8	
TOC	5 minutes	

Surface Type and Condition (Table 4-2)	Acres	Runoff Coefficient	C X A	Q = CiA
Turf meadows	0.16	0.4	0.064	
Total	0.16		0.064	
			0.4	0.4992

Trial Depth	Flow Area	Wetted Perim	Radius	n	Q	V	max shear stress
1	3	6.32455532	0.4743	0.568858748	0.798807	0.266269	1.75344
0.99	2.9403	6.261309767	0.4696	0.616356498	0.71775	0.244108	1.7359056
0.98	2.8812	6.198064214	0.4649	0.673342547	0.639455	0.221941	1.7183712
0.97	2.8227	6.134818661	0.4601	0.742978805	0.563884	0.199768	1.7008368
0.96	2.7648	6.071573108	0.4554	0.830007742	0.491	0.17759	1.6833024
0.95	2.7075	6.008327554	0.4506	0.941881043	0.420764	0.155407	1.665768

Retardance Class	B
C	23
slope	0.0281
allowable shear	2.1

0.96 feet plus 6 inches of freeboard 1:3 slopes on each side

SB 7
Sta. 0+00 to 4+25 North Side
Example Problem 1: 5.08.2.1 TDOT Drainage Manual

i	7.8	
TOC	5 minutes	

Surface Type and Condition (Table 4-2)	Acres	Runoff Coefficient	C X A	Q = CiA
Turf meadows	0.281	0.4	0.1124	
Total	0.281		0.1124	
			0.4	0.87672

Trial Depth	Flow Area	Wetted Perim	Radius	n	Q	V	max shear stress
1	3	6.32455532	0.4743	0.568858748	0.798807	0.266269	1.75344
1.1	3.63	6.957010852	0.5218	0.331127792	1.769476	0.487459	1.928784
1.02	3.1212	6.451046427	0.4838	0.494196867	0.969355	0.310571	1.7885088
1.01	3.0603	6.387800874	0.4791	0.528660131	0.882662	0.288423	1.7709744

Retardance Class	B
C	23
slope	0.0281
max velocity	2.1

1.02 feet plus 6 inches of freeboard 1:3 slopes on each side

OB 7
Sta. 4+25 to 4+75 North Side and Driveway A
Example Problem 1: 5.08.2.1 TDOT Drainage Manual

i	6	
TOC	11.2 minutes	

Surface Type and Condition (Table 4-2)	Acres	Runoff Coefficient	C X A	Q = CiA
Turf meadows	0.1496	0.4	0.05984	
Asphalt Pavement	0.2461	0.9	0.22149	
	0.3957		0.28133	
			0.710968	1.049815

Trial Depth	Flow Area	Wetted Perim	Radius	n	Q	V	max shear stress
1	3	6.32455532	0.4743	0.932554396	0.446555	0.148852	1.47264
1.5	6.75	9.486832981	0.7115	0.160959013	7.629025	1.130226	2.20896
1.2	4.32	7.589466384	0.5692	0.288021373	2.35126	0.544273	1.767168
1	3	6.32455532	0.4743	0.932554396	0.446555	0.148852	1.47264
1.07	3.4347	6.767274193	0.5075	0.505020919	0.987657	0.287553	1.5757248
1.08	3.4992	6.830519746	0.5123	0.475439252	1.075462	0.307345	1.5904512

Retardance Class	B
C	23
slope	0.0236
max velocity	2.1

1.08 feet plus 6 inches of freeboard 1:3 slopes on each side 1:3 slopes on each side

OB 6
Sta. 5+00 to 8+00 North Side
Example Problem 1: 5.08.2.1 TDOT Drainage Manual

i	5.9	
TOC	11.4 minutes	

Surface Type and Condition (Table 4-2)	Acres	Runoff Coefficient	C X A	Q = CiA
Turn meadows	0.972	0.4	0.3888	
Asphalt Pavement	0.9497	0.9	0.85473	
Asphalt Pavement	0.0633	0.9	0.05697	
	1.985		1.3005	
			0.655164	7.67295

Trial Depth	Flow Area	Wetted Perim	Radius	n	Q	V	max shear stress
2	12	12.64911064	0.9487	0.143058862	13.01722	1.084768	1.46016
1.5	6.75	9.486832981	0.7115	0.274987009	3.144192	0.465806	1.09512
1.7	8.67	10.75174404	0.8064	0.194677916	6.20121	0.715249	1.241136
1.8	9.72	11.38419958	0.8538	0.172398075	8.155775	0.839071	1.314144
1.78	9.5052	11.25770847	0.8443	0.176311062	7.740633	0.814358	1.2995424
1.77	9.3987	11.19446292	0.8396	0.178358246	7.537674	0.801991	1.2922416

Retardance Class	B
C	23
slope	0.0117
max velocity	2.1

1.78 feet plus 6 inches of freeboard 1:3 slopes on each side

OB 5

Driveway B

Example Problem 1: 5.08.2.1 TDOT Drainage Manual

i	7.8
TOC	5 minutes

Surface Type and Condition (Table 4-2)	Acres	Runoff Coefficient	C X A	Q = CiA
Turn meadows	0.0934	0.4	0.03736	
Asphalt Pavement	0.5045	0.9	0.45405	
	0.5979		0.49141	
			0.821893	3.832998

Retardance Class	B
C	23
slope	0.0236
max velocity	2.1
max shear stress	

Trial Depth	Flow Area	Wetted Perim	Radius	n	Q	V	
1	3	6.32455532	0.4743	0.932554396	0.446555	0.148852	1.47264
1.5	6.75	9.486832981	0.7115	0.160959013	7.629025	1.130226	2.20896
1.2	4.32	7.589466384	0.5692	0.288021373	2.35126	0.544273	1.767168
1.25	4.6875	7.90569415	0.5929	0.250676885	3.012268	0.642617	1.8408
1.3	5.07	8.221921916	0.6166	0.223244865	3.755384	0.740707	1.914432
1.31	5.1483	8.28516747	0.6214	0.218608357	3.914215	0.760293	1.9291584

1.31 feet plus 6 inches of freeboard

0

#DIV/0!

Item	Value	Reference
Gutter Manning's n-value	0.013	TDOT Table 5A-1

Gutter cross slope (ft/ft)	0.085	Typical Sections
Maximum inlet spacing	400	Page 7-21
Maximum Allowable Spread	8	Page 7-21

i	7.8	TDOT Figure 4A-7
Runoff Coefficient	0.853	Typical Runoff Coefficient

Typical Runoff Coefficient

Soil	Cover Description	TR-55 CN	Area (ac)	CN X A
B	Turf Meadows	0.9	1950	1755
B	Asphalt Pavement	0.4	200	80
B				0
	CN	0.853	2150	1835

Assume TOC of less than 5 Minutes for all Inlets

Drainage Area	Station	Length	Area	Q	Qadjusted	Slope	Conveyance Value	Depth	Spread
SB 9	2+25.00	225	0.1015	0.676	0.68	0.0281	4.0309	0.1815	2.5725
SB 6	4+25.00	425	0.1115	0.742	0.74	0.0274	4.4843	0.1954	3.2705
SB 3	7+31.94	731.94	0.3041	2.024	2.02	0.0225	13.4964	0.3604	6.7887
				0	0	0	0	0.156396	1.8186

Drainage Area	Station	Length	Area	Q	Qadjusted	Slope	Conveyance Value	Depth	Spread
SB 8	2+25.00	225	0.1015	0.676	0.68	0.0281	4.0309	0.1815	2.0903
SB 5	4+25.00	425	0.1113	0.741	0.74	0.0274	4.4762	0.1921	3.1065
SB 4	8+09.34	809.34	0.2867	1.909	1.91	0.0141	16.0735	0.2784	7.42
		883.01	0.0869	0.578511	0.5785115		397.88%		4.1414
				0	0		0	0.156396	1.8186

		3.360387	
1.96	3.55	0.880747	4.81 10.25
0.10	0.14	0.03523	0.20 0.25
		0.1352	
		3.360387	
1.96	3.55	0.880747	4.81 10.25
1.60	2.00	0.352299	3.50 6.00
		1.9523	
		3.360387	
1.96	3.55	0.880747	4.81 10.25
0.11	0.17	0.052845	0.25 0.49
		0.1628	

	Q	R	S	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG
1																	
2																	
3																	
4																	
5																	
6																	
7																	
8																	
9																	
10																	
11																	
12																	
13																	
14																	
15																	
16		**Adapted from Figure 7A-1 in TDOT Drainage Manual**															
17				**North Side**													
18	Area	Velocity	Width	Vo	Af	Pf	Ku	Kf	Qf	Vf	Qs	As	Vs	Rs	Qsi	Qi	Qb
19	0.2005	3.37	1.75	6.07	0.59	2.16	28.17	23.94	4.01	6.83	0.00	-0.39	0.00	1.00	0.00	4.01	0.00
20	0.2378	3.12	1.75	6.07	0.59	2.16	28.17	23.94	3.96	6.75	0.00	-0.35	0.00	1.00	0.00	0.74	0.00
21	0.6036	3.35	1.75	6.07	0.59	2.16	28.17	23.94	3.59	6.11	0.00	0.02	0.00	1.00	0.00	3.59	0.00
22	0.14279	0	1.75	6.0733201	0.5873438	2.16	28.1668548	23.9418266	#DIV/0!	#DIV/0!	#DIV/0!	-0.4445538	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
23																	
24				**South Side**													
25	Area	Velocity	Width	Vo	Af	Pf	Ku	Kf	Qf	Vf	Qs	As	Vs	Rs	Qsi	Qi	Qb
26	0.2005	3.37	1.75	6.07	0.59	2.16	28.17	23.94	4.01	6.83	0.00	-0.39	0.00	1.00	0.00	4.01	0.00
27	0.229	3.24	1.75	6.07	0.59	2.16	28.17	23.94	3.96	6.75	0.00	-0.36	0.00	1.00	0.00	3.96	0.00
28	0.6945	2.75	1.75	6.07	0.59	2.16	28.17	23.94	2.84	4.84	0.00	0.11	0.00	1.00	0.00	2.84	-0.93
29	0.3116	1.8565837	1.75	6.0733201	0.5873438	2.16	28.1668548	23.9418266	3.4811284	5.9269013	-2.9026169	-0.2757438	10.526501	0.0479493	-0.1391784	3.3419501	-2.7634386
30	0.14279	0	1.75		0.5873438	2.16	28.1668548	23.9418266	#DIV/0!	#DIV/0!	#DIV/0!	-0.4445538	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
31																	
32																	
33																	
34																	

Table 7A-4

Depth	Spread	Area	Conveyance
0.03	0.40	0.01	0.05
0.07	0.80	0.03	0.31
0.10	1.20	0.06	0.91
0.14	1.60	0.11	1.96
0.17	2.00	0.17	3.55
0.20	3.50	0.25	4.81
0.25	6.00	0.49	10.25
0.30	8.50	0.85	20.54
0.35	11.00	1.34	36.84
0.40	13.50	1.95	60.28
0.45	16.00	2.69	91.91
0.50	18.50	3.55	132.73
0.55	21.00	4.54	183.68
0.60	23.50	5.65	245.66

35																	
36																	
37																	
38	6.205748																
39	0.2565713											16.0963					
40	0.0128286									10.25	20.54	0.56815355					
41	0.2128									0.25	0.30	0.02840768					
42												0.2784					
43	6.205748																
44	0.2565713											16.0963					
45	0.6414283									10.25	20.54	0.56815355					
46	4.1414									6.00	8.50	1.42038387					
47												7.4204					
48	6.205748																
49	0.2565713											16.0963					
50	0.0615771									10.25	20.54	0.56815355					
51	0.3116									0.49	0.85	0.20453528					
52												0.6945					

Item	Value	Reference
mannings n	0.013	TDOT Table 5A-1
minimum depth (Type #12) for 18" pipes	3.9 feet	D-CB-12P
minimum depth (Type #42) for 18" pipes	3.82 feet	D-CB-42SB
maximum velocity	12 fps	TDOT 7.03.4.3

Adapted from Figure 7A-4 TDOT Drainage Manual

Inlets

Name	Drainage Area(s)	Elevation	CN	Area	CN X A	Inlet Collects	CN X A to that Inlet	TOC (minutes)	i (rainfall)	Q	V	Depth (Elevation - Lowest Outlet Elevation)
CB-1	SB 1	994.9	0.85	0.0986	0.08381	CB-1, CB-2, CB-3, CB-4	2.362005	11.35	5.9	13.93583	7.886073	6.43
CB-2	OB 5, OB 6	994.85	0.69	2.5829	1.79191	CB-2, CB-3, CB-4	2.050395	11.35	5.9	12.09733	6.845694	3.92
CB-3	SB 3	997.17	0.85	0.3041	0.258485	CB-3	0.258485	5.00	7.8	2.016183	1.140927	5.35
CB-4	SB 4	995.71	0.85	0.2680	0.2278	CB-4	0.2278	5.00	7.8	1.77684	1.005487	3.90
CB-5	SB 5	1000.25	0.85	0.1113	0.094605	CB-5, CB-6, CB-7	2.856626	24.39	4.5	12.85482	7.274345	6.57
CB-6	SB 6	1000.25	0.85	0.1115	0.094775	CB-6, CB-7	2.589471	24.39	4.5	11.65262	6.594039	5.71
CB-7	OB 7, OB 8	998.67	0.58	4.3012	2.494696	CB-7	2.494696	24.39	4.5	11.22613	6.352697	3.82
CB-8	SB 8	1005.85	0.85	0.1015	0.086275	CB-8, CB-9	0.17255	5.00	7.8	1.34589	0.761619	4.10
CB-9	SB 9	1005.85	0.85	0.1015	0.086275	CB-9	0.086275	5.00	7.8	0.672945	0.380809	3.90

Adapted from Figure 7A-5 TDOT Drainage Manual

Pipes

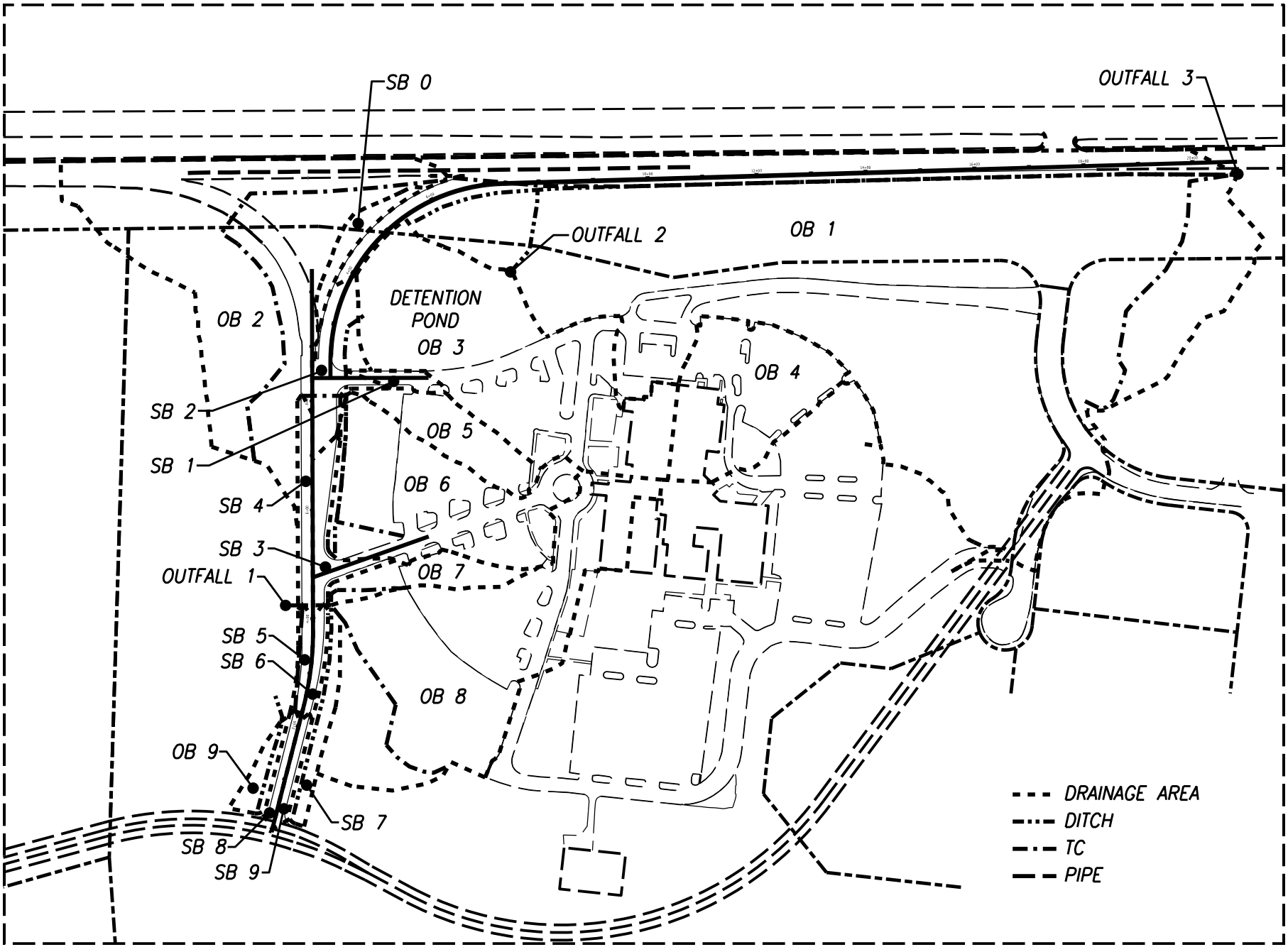
Outlet	Elevation	Inlet	Elevation	Length	Slope	Diameter	Q capacity	V capacity
CB-1	988.47	EW-1	987.90	28.46	0.0200	1.5000	14.85210546	8.404579543
CB-2	990.93	CB-1	990.28	43.60	0.0150	1.5000	12.86230063	7.278579392
CB-3	991.82	CB-2	991.03	79.20	0.0100	1.5000	10.50202449	5.942935188
CB-4	991.81	CB-2	991.03	80.97	0.0096	1.5000	10.2898405	5.822863514
CB-5	993.68	EW-2	993.21	31.41	0.0150	1.5000	12.86230063	7.278579392
CB-6	994.54	CB-5	993.97	38.00	0.0150	1.5000	12.86230063	7.278579392
CB-7	994.85	CB-6	994.64	14.00	0.0150	1.5000	12.86230063	7.278579392
CB-8	1001.75	CB-5	996.35	195.36	0.0275	1.5000	17.41563738	9.855243086
CB-9	1001.95	CB-8	1001.85	26.00	0.0050	1.5000	7.426052732	4.202289771

Time of Concentration *Figure 4A-13, TDOT Drainage Manual*

	CB-1	CB-2	CB-5	CB-6	CB-7
Sheet Flow					
Surface Description (Table 4-3)					
Manning's roughness coefficient, n (Table 4-3)	0.35	0.35	0.35	0.35	0.35
Flow length (ft)	125.46	125.46	205.07	205.07	205.07
Rainfall Intensity (in/hr) (Figure 4A-7)	5.9	5.9	4.5	4.5	4.5
Land Slope (ft/ft)	0.04646	0.04646	0.01341	0.01341	0.01341
Compute $t_{(sheet)}$ (Kinematic Wave Equation)	11.20165696	11.20165696	24.33723888	24.337239	24.33723888
Shallow Concentrated Flow					
Surface Description (paved or unpaved)		P	u		
Flow length, L (ft)	0		0		
Watercourse Slope, S (ft/ft)	0		0		
Average velocity, V (ft/s) (Figure 4A-12)	0	0	0		
Compute $t_{(shallow)}$	0	0	0	0	0
Channel Flow					
Cross Sectional flow area, a (ft^2)	10.83	10.83	10.125	10.125	10.125
Wetted perimeter, p_w (ft)	12.02	12.02	9.48	9.48	9.48
Hydraulic radius, r (ft)	0.900998336	0.900998336	1.068037975	1.068038	1.068037975
Channel Slope, S	0.0117	0.0117	0.063055	0.063055	0.063055
Mannings coefficient, n (Table 5A-1)	0.35	0.35	0.35	0.35	0.35
Velocity (ft/s)	0.429563278	0.429563278	1.11695528	1.1169553	1.11695528
Flow Length, L (ft)	232.22	232.22	218.58	218.58	218.58
Compute $t_{(channel)}$	0.150165433	0.150165433	0.054359085	0.0543591	0.054359085
Time of Concentration	11.35182239	11.35182239	24.39159797	24.391598	24.39159797
Channel Flow					
Cross Sectional flow area, a (ft^2)	1.77		1.77	1.77	
Wetted perimeter, p_w (ft)	4.71		4.71	4.71	
Hydraulic radius, r (ft)	0.375796178	#DIV/0!	0.375796178	0.3757962	#DIV/0!
Channel Slope, S	0.015		0.015	0.015	
Mannings coefficient, n (Table 5A-1)	0.013		0.013	0.013	
Velocity (ft/s)	7.310102252	#DIV/0!	7.310102252	7.3101023	#DIV/0!

	A	B	C	D	E	F	G	H	I	J	K	L	M
67	Flow Length, L (ft)	43.6		52	38								
68	Compute $t_{(channel)}$	0.001656764	0	0.001975957	0.001444	0							
69	Time of Concentration	11.35347915	11.35182239	24.39357393	24.393042	24.39159797							

DRAINAGE SUB BASINS



1" = 250'

Surface Type and Condition[1,2]	Runoff Coefficient (C)
Rural Areas	
Concrete or sheet asphalt pavement	0.8 - 0.9
Asphalt macadam pavement	0.6 - 0.8
Gravel roadways or shoulders	0.4 - 0.6
Bare earth	0.2 - 0.9
Steep grassed areas (2H:1V)	0.5 - 0.7
Turf meadows	0.1 - 0.4
Forested areas	0.1 - 0.3
Cultivated fields	0.2 - 0.4
Urban Areas	
Flat residential, with about 30 percent of area impervious	0.40
Flat residential, with about 60 percent of area impervious	0.55
Moderately steep residential, with about 50 percent of area impervious	0.65
Moderately steep developed area, with about 70 percent of area impervious	0.80
Flat commercial/industrial, with about 90 percent of area impervious	0.80

[1]For flat slopes and/or permeable soil, use the lower values. For steep slopes and/or impermeable soil, use the higher values.
[2]For areas where there is a shallow bedrock surface, use the higher values.

Table 4-2
Runoff Coefficients (C) for Use in the Rational Method
Reference: USDOT, FHWA, HDS-4 (2001)

4.04.1.2 INTENSITY

Rainfall intensity (I) is the average rainfall rate (in/hr) for a duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design, and the time of concentration calculated for the drainage area, the rainfall intensity can be determined from Rainfall Intensity Duration Frequency (IDF) Curves. The designer should select the appropriate curve based on the location of the project (refer to Figure 4A-1 in the chapter Appendix for an IDF-zone map). The curves are identified on ten charts and designated by the selected gauged Tennessee city located in each IDF-zone. The intensity-duration-frequency curves for Chattanooga, Clarksville, Cookeville, Jackson, Johnson City, Knoxville, Lawrenceburg, Lebanon, Manchester, and Shelbyville, are provided in the Appendix as Figures 4A-2 through 4A-11.

Curve Number Computation Worksheet							
Project			By			Date	
Location			Checked			Date	
Check one: ☐ Present ☐ Developed							
Soil Name and Hydrologic Group (see Table 4A-4)	Cover Description: (cover type, treatment and hydrologic condition; % impervious; unconnected or connected impervious area ratio)	Curve Number[1] Table 4A-1	Table 4A-2	Table 4A-3	Area ☐ ac. ☐ mi^2 ☐ %	Product of CN x Area	
[1] Use only one CN source per line				**TOTALS:**			
$CN\ (weighted) = \frac{total\ product}{total\ area} =$ _____ = _____				**USE CN =**			

Figure 4A-14
Curve Number Worksheet
Reference: USDA, Urban Hydrology for Small Watersheds (1986)

Table integrated into "Drainage Calculation" excel file.

Table integrated into "Drainage Calculations"

Time of Concentration (T_c) Worksheet

Project	By	Date
Location	Checked	Date

Check one: ☐ Present ☐ Developed

Notes: As many as two segments of each flow type can be analyzed on this worksheet.
Include a map, schematic or description of the flow segments.

Sheet Flow

	Segment ID				
1. Surface Description (Table 4-3) ·····					
2. Manning's roughness coefficient, n (Table 4-3)					
3. Flow length (total ≤ 300 feet) ····· ft					
or 4. 2-year, 24-hour rainfall, P_2 ·····in					
or 4. Rainfall intensity ····· in/hr					
5. Land slope, S ····· ft/ft					
6. Compute $t_{t(sheet)}$ ····· hr		+		=	

Check one: ☐ NRCS $t_t = \frac{0.007(nL)^{0.8}}{P_2^{0.5} S^{0.4}}$ ☐ Kinematic Wave $t_t = 0.938 \frac{n^{0.6} L^{0.6}}{i^{0.4} S^{0.3}}$

Shallow Concentrated Flow

	Segment ID				
7. Surface description (paved or unpaved) ·····					
8. Flow length, L ····· ft					
9. Watercourse slope, S····· ft/ft					
10. Average velocity, V (Figure 4A-10) ····· ft/sec					
11. Compute $t_{t(shallow)}$ $t_t = L / (3600V)$ ·····hr		+		=	

Channel Flow

	Segment ID				
12. Cross sectional flow area, a ····· ft^2					
13. Wetted perimeter, p_w ····· ft					
14. Hydraulic radius $r = a/p_w$ ····· ft					
15. Channel slope, S ····· ft/ft					
16. Mannings coefficient, n (Table 5A-1) ·····					
17. Velocity $V=1.49\ r^{2/3}\ S^{1/2} / n$·····ft/s					
18. Flow length, L ····· ft					
19. $t_t = L / (3600V)$ Compute $t_{t(channel)}$ ·····hr		+		=	

20. Time of concentration (T_c): $t_{t(sheet)} + t_{t(shallow)} + t_{t(channel)}$ ····· hr =

Figure 4A-13
NRCS Time of Concentration Worksheet
Reference: USDA, Urban Hydrology for Small Watersheds (1986)

land, the designer will need to use engineering judgment to select an average condition based on mowing practices and channel lining material for when the greatest likelihood of heavy rainfall would occur. For areas that are or will be infrequently mowed, tall grass should be considered.

Surface Type	Minimum	Normal	Maximum
Concrete	0.010	0.011	0.013
Asphalt	0.010	0.012	0.015
Bare Soil	0.010	0.011	0.030
Bare Sand	0.010	0.010	0.016
Graveled Surface	0.011	0.012	0.030
Bare Clay-Loam (eroded)	0.012	0.020	0.033
Packed Clay		0.030	
Fallow (no residue)	0.006	0.050	0.160
Cultivated (till) (residue $\leq$ 20%)	0.006	0.060	0.120
Cultivated (till) (residue > 20%)	0.070	0.170	0.470
No Till (no residue)	0.030	0.040	0.100
No Till (20% - 40% residue)	0.010	0.070	0.170
No Till (>60% residue)	0.016	0.300	0.470
Plow (fall)	0.020	0.020	0.100
Range (natural)	0.100	0.130	0.320
Pasture	0.300	0.350	0.400
Pasture (sparse vegetation)	0.053	0.070	0.130
Grass (bluegrass sod)	0.390	0.450	0.630
Grass (Bermuda)	0.300	0.410	0.480
Lawns	0.200	0.250	0.300
Woods and Shrubbery	0.400	0.400	0.800

Table 4-3
Manning's n Values for Overland Flow

Table References:

American Society of Civil Engineers and Water Environment Federation. *Design and Construction of Urban Stormwater Management Systems*. ASCE Manuals and Reports of Engineering Practice No. 77 and WEF Manual of Practice FD-20. New York, New York and Alexandria, Virginia. 1992.

Indiana Department of Transportation. Indiana Design Manual Part IV Volume I. Indianapolis, IN. 1999.

Kentucky Transportation Cabinet, Drainage Guidance Manual - Proposed Revisions. Frankfort, KY. September 29, 2000.

Metropolitan Government of Nashville and Davidson County Department of Public Works Engineering Division. *Stormwater Management Manual.* Nashville, Tennessee. Sept. 1999.

United States Department of Agriculture. Soil Conservation Service. Engineering Division. *Urban Hydrology for Small Watersheds - Technical Release 55.* June 1996.

Virginia Department of Transportation. *Drainage Manual*. Richmond, Virginia. February 1989.

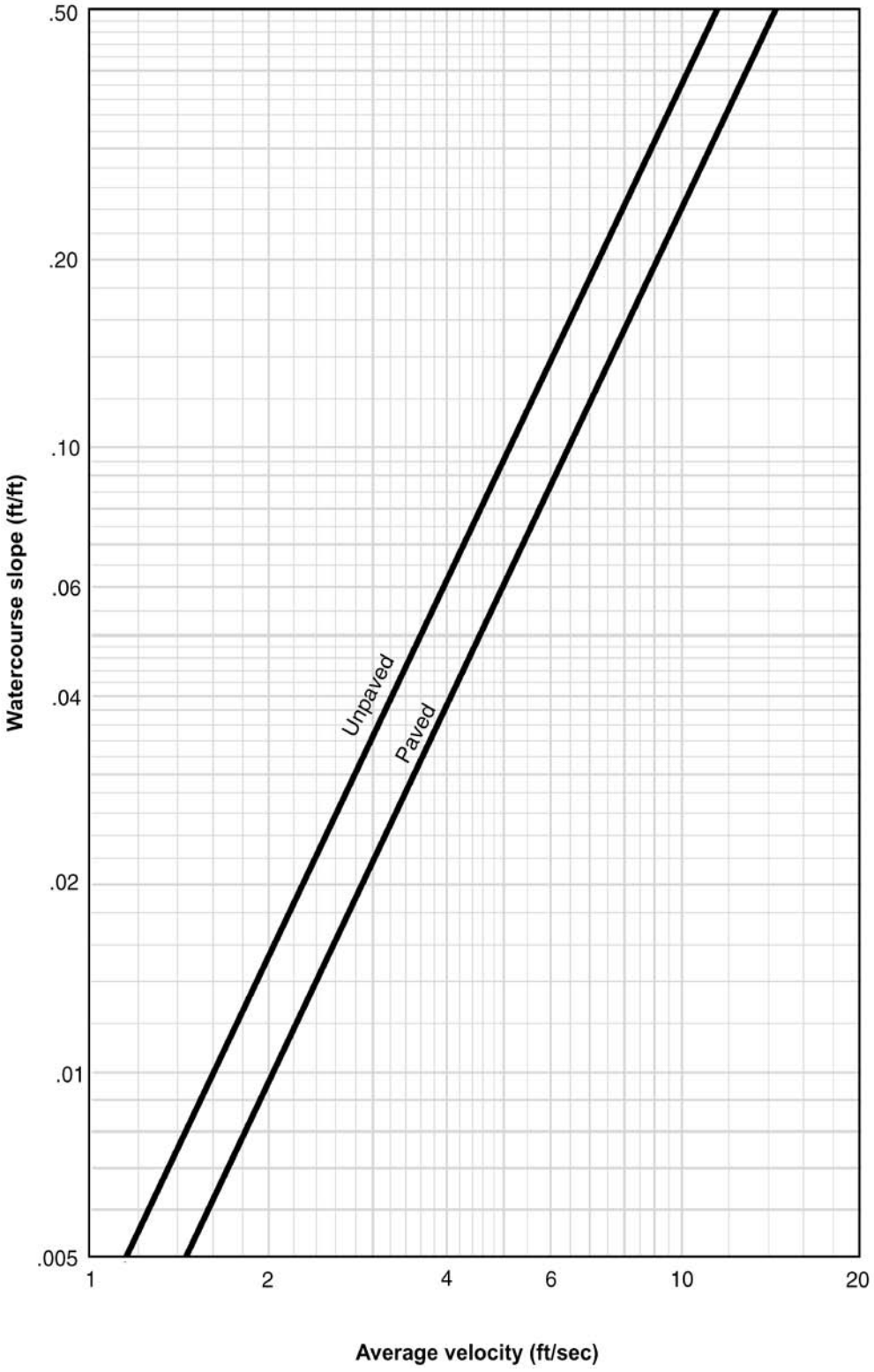


Figure 4A-12
Shallow Concentrated Flow Average Velocities
Reference: USDA, Urban Hydrology for Small Watersheds (1986)

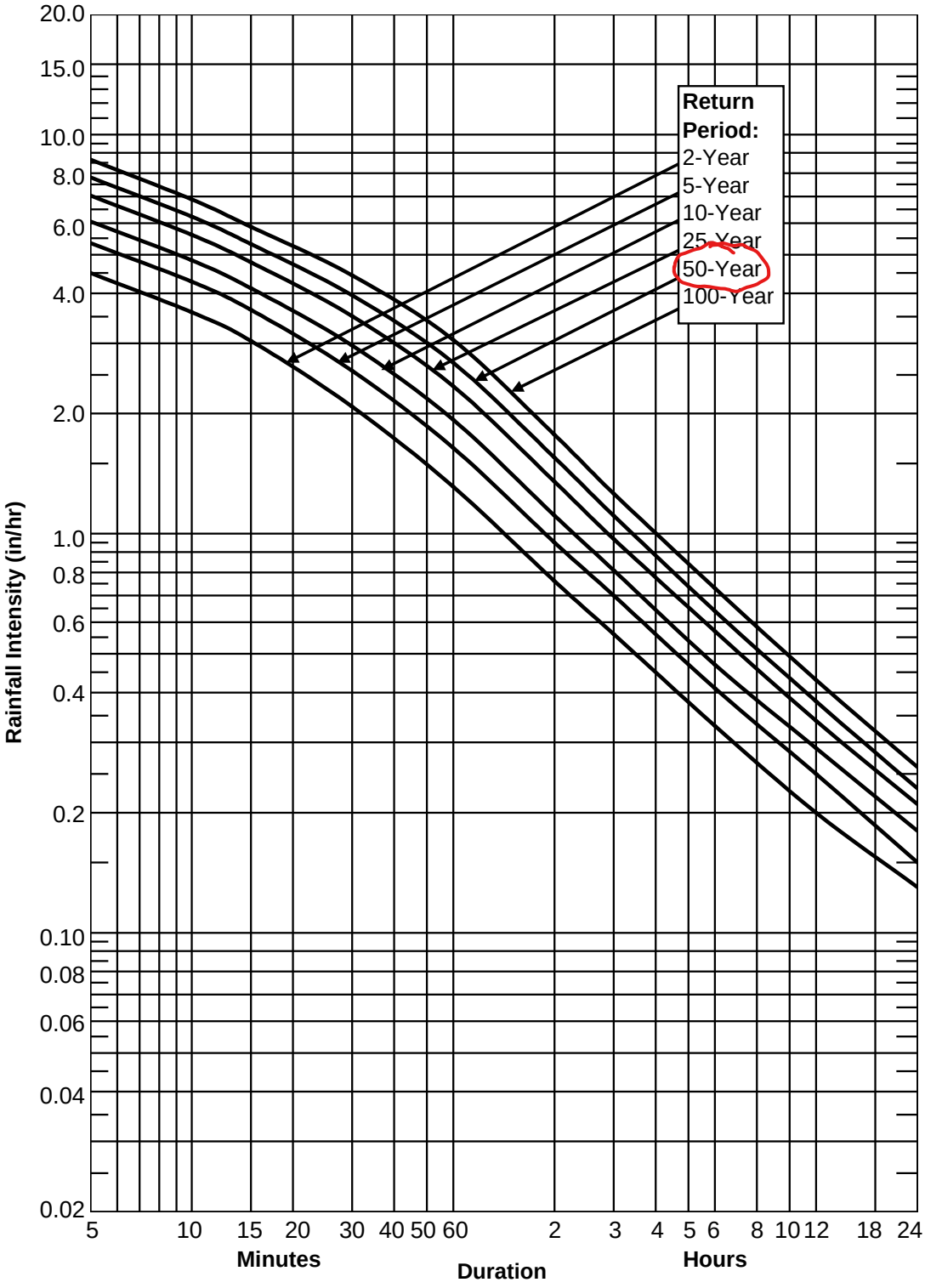


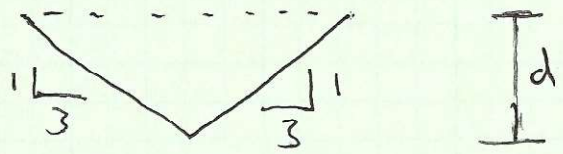
Figure 4A-7
Knoxville IDF Curve
NOTE: T_c = 5 minutes is a minimum value to use in all cases
Reference: National Weather Service, NOAA Atlas 14, Volume 2 (2004)

Drainage Area OB 9

Stations 0+00 to 2+00 South Side

$Q = 0.4992$ (see Excel File)

Ditch Design



$Area = 3 \times d^2$

TDOT Drainage Manual Chapter 5 Figure 5A-4

$Wetted\ Perimeter = 2d\sqrt{3^2+1}$

$Hydraulic\ Radius = \frac{Area}{Wetted\ Perimeter}$

n-value → Equation 5-14

$$n = \frac{R^{1/6}}{Cf + 19.97 \log(R^{1.4} S^{0.4})}$$

Retardance class B → Table 5-1

$C_f = 23$

Slope = 2.81% → see Vertical Profile

$$Q = \frac{1.486}{n} A R^{0.667} S^{0.5}$$

Trial depth (d) = 1.00

$Q = 0.798807$

See Excel File for depth choice of **0.96 feet**

Follow Example Problem #1 5.08.2.1 TDOT Drainage Manual

Check for maximum shear stress, τ_{max}

Allowable (Table 5A-7)

2.10

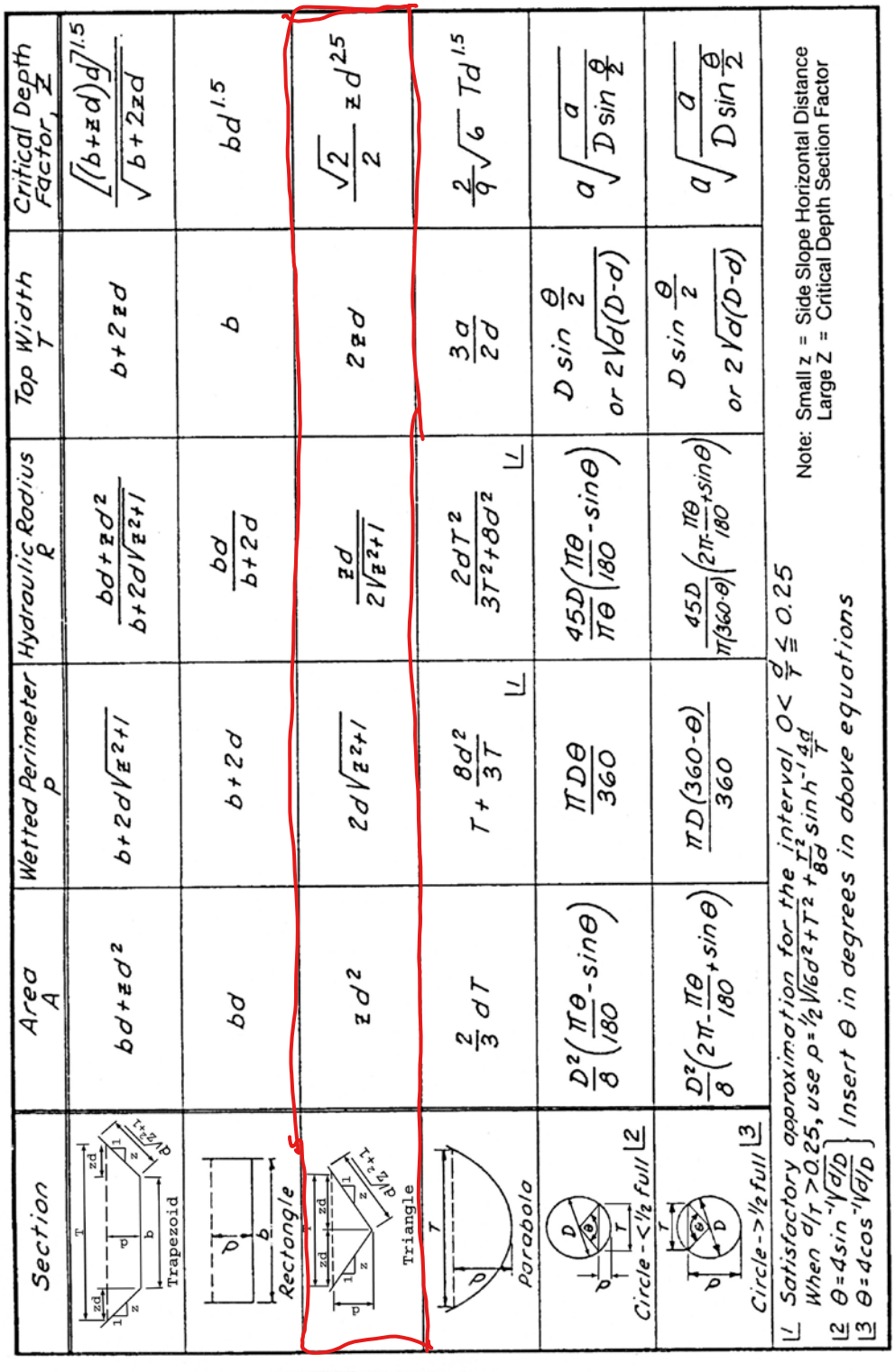
$$V = \frac{Q}{A} = \frac{0.491}{2.7648} = 0.17759 \text{ Ft/s}$$

$$\tau_{max} = \gamma ds = (0.0281)(62.4)(0.96) = 1.68$$ ✓

6" Freeboard

0.96' + 6" = 1.46'

Zachary Jerome



Section	Area A	Wetted Perimeter p	Hydraulic Radius R	Top Width T	Critical Depth Factor, $\frac{Q}{\sqrt{g}}$
Trapezoid	$bd+zd^2$	$b+2d\sqrt{z^2+1}$	$\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$	$b+2zd$	$\frac{\sqrt{(b+zd)d}\,d^{1.5}}{\sqrt{b+2zd}}$
Rectangle	bd	$b+2d$	$\frac{bd}{b+2d}$	b	$bd^{1.5}$
Triangle	zd^2	$2d\sqrt{z^2+1}$	$\frac{zd}{2\sqrt{z^2+1}}$	$2zd$	$\frac{\sqrt{2}}{2}zd^{2.5}$
Parabola	$\frac{2}{3}dT$	$T+\frac{8d^2}{3T}$ [1]	$\frac{2dT^2}{3T^2+8d^2}$ [1]	$\frac{3a}{2d}$	$\frac{2}{9}\sqrt{6}\,Td^{1.5}$
Circle - <½ full [2]	$\frac{D^2}{8}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$\frac{\pi D\theta}{360}$	$\frac{45D}{\pi\theta}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$
Circle - >½ full [3]	$\frac{D^2}{8}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$\frac{\pi D(360-\theta)}{360}$	$\frac{45D}{\pi(360-\theta)}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$

[1] Satisfactory approximation for the interval $0<\frac{d}{T}\leq 0.25$ When $d/T>0.25$, use $\rho=\frac{1}{2}\sqrt{16d^2+T^2}+\frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

[2] $\theta=4\sin^{-1}\sqrt{d/D}$ } Insert θ in degrees in above equations

[3] $\theta=4\cos^{-1}\sqrt{d/D}$ }

Note: Small z = Side Slope Horizontal Distance
Large Z = Critical Depth Section Factor

Figure 5A-4
Open Channel Geometric Relationships for Various Cross Sections
Reference: USDA, SCS, NEH-5 (1956)

Retardance Class	C_{rf}
A	15.8
B	23.0
C	30.2
D	34.6
E	37.7

Use to determine Manning's N Value

Table 5-1
Retardance Factor Coefficients

Step 2: Using the hydraulic radius computed in Step 1, determine a value for Manning's n from either Equation 5-14 or from Tables 5A-8 through 5A-12 in the Appendix.

Step 3: Compute the flow rate in the ditch using Manning's equation (Equation 5-5), based on the trial depth assumed in Step 1 and the Manning's n-value determined in Step 2.

Step 4: Compare the flow rate computed in Step 3 with the actual design discharge at that location on the ditch. If the computed flow rate does not match the design flow rate, the designer should assume a different trial depth and return to Step 1. Generally, if the computed flow rate is greater than the design flow rate, a lower trial depth should be assumed. Conversely, if the computed flow rate is too low, a greater depth should be assumed. The assumed depth should be adjusted until the computed flow rate is reasonably close to the design discharge. Usually, the elevation can be determined to within 0.01 feet with only a few trials. This level of precision should be more than adequate.

5.06.1.3.2 CAPACITY COMPUTATIONS FOR DITCHES WITH GEOTEXTILE LININGS

The method used to determine the hydraulic capacity of ditches lined with erosion control blankets or turf reinforcement mats will usually depend on whether or not the vegetation has been fully established. Where the grass lining has been well established, the capacity of the ditch should be determined based on a vegetated lining, as described in Section 5.06.1.3.1. On the other hand, while the lining is still in the "unvegetated" state (that is, little or no grass has grown), the capacity would typically be determined based on the slope-conveyance method described in Section 5.06.1.3.4.

The slope conveyance method is based on the use of Manning's equation with the assumption that the roughness coefficient "n" is constant. However, when evaluating the resistance of a geotextile lining, it is generally assumed that the value of "n" can vary with depth. Generally, a relatively large n-value should be used for depths up to 0.5 feet, and a lower value should be used for depths greater than 2 feet. Between the two depths, the effective n-value may be assumed to vary linearly with depth. Table 5A-6 provides n-values that may be used to evaluate the roughness of unvegetated turf reinforcement mats for depth of 0.5 to 2 feet.

Where the design discharge, slope and cross section of the ditch are known, the depth may be determined as follows:

Quality of Stand	Average Length of Vegetation	Retardance Class
Good	Longer than 30 inches	A
	11 to 24 inches	B
	6 to 10 inches	C
	2 to 6 inches	D
	Less than 2 inches	E
Fair	Longer than 30 inches	B
	11 to 24 inches	C
	6 to 10 inches	D
	2 to 6 inches	D
	Less than 2 inches	E

Table 5A-4
Guide to Selection of Vegetal Retardance
Reference: USDA, SCS, TP-61 (March, 1947)

		Depth Ranges [a]		
Lining Category	**Lining Type**	**0 - 0.5 ft**	**0.5 - 2.0 ft**	**> 2.0 ft**
Rigid	Concrete (Broom or Float Finish)	0.015	0.013	0.013
	Gunite	0.022	0.02	0.02
	Grouted Riprap	0.04	0.03	0.028
	Stone Masonry	0.042	0.032	0.03
	Soil Cement	0.025	0.022	0.02
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.02	0.02
	Rock Cut	0.035	0.035	0.025
Erosion Control Blankets [b]	Type I	0.055	0.055 - 0.021	0.021
	Type II	0.055	0.055 - 0.021	0.021
	Type III	0.055	0.055 - 0.021	0.021
	Type IV	0.022	0.022 - 0.014	0.014
Turf Reinforcement Mats [b]	Unvegetated	0.04	0.04 - 0.015	0.015
Machined Riprap [c,d]	Class A1	0.124	0.072	0.038
	Class B	0.153	0.086	0.041
	Class C	0.181	0.095	0.042

[a] Values listed are representative values for the respective depth ranges. Manning's roughness coefficients vary with the flow depth

[b] General values based on vendor information. Consult with individual vendors for more specific information

[c] Values interpolated from data provided in HEC-15

[d] In general, $n = 0.0395(d_{50})^{0.167}$, where d_{50} = median stone diameter

Table 5A-6
Recommended Manning's n-Values for Artificial Channels
References: USDOT, FHWA, HEC-15 (1988) &
North American Green, Evansville, Indiana

Type of Channel and Description	Minimum	Normal	Maximum
LINED CHANNELS (Selected Linings)			
a. Concrete			
Trowel Finish	0.011	0.013	0.015
Float Finish	0.013	0.015	0.016
Gunite, good section	0.016	0.019	0.023
b. Asphalt			
Smooth	0.013	0.013	-
Rough	0.016	0.016	-
EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
Clean, recently completed	0.016	0.018	0.020
Clean, after weathering	0.018	0.022	0.025
Gravel, uniform section, clean	0.022	0.025	0.030
With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
No vegetation	0.023	0.025	0.030
Grass, some weeds	0.025	0.030	0.033
Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
Earth bottom and rubble sides	0.025	0.030	0.035
Stony bottom and weedy sides	0.025	0.035	0.045
Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline excavated or dredged			
No vegetation	0.025	0.028	0.033
Light brush on banks	0.035	0.050	0.060
d. Rock Cuts			
Smooth and uniform	0.025	0.035	0.040
Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, uncut weeds and brush			
Dense weeds as high as flow depth	0.050	0.080	0.120
Clean bottom, brush on sides	0.040	0.050	0.080
Same, highest stage of flow	0.045	0.070	0.110
Dense brush, high stage	0.800	0.100	0.140
NATURAL STREAMS			
1. Minor streams (top width at flood stage<100 ft)			
a. Streams on Plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055

Table 5A-1
Values of Roughness
Coefficient 'n' (Uniform Flow)
Reference: Chow, Ven T., *Open Channel Hydraulics* (1959)

Continue on following page

6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
2. Floodplains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated area			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
b. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as 2, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as 4, but with flood stage reaching branches	0.100	0.120	0.160
3. Major Streams (top width at flood stage > 100 ft) The n-value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	-	0.060
b. Irregular and rough section	0.035	-	0.100

Table 5A-1 (continued)
Values of Roughness
Coefficient ‘n’ (Uniform Flow)
Reference: Chow, Ven T., *Open Channel Hydraulics* (1959)

		Permissible Unit Shear Stress	
Lining Category	**Lining Type**	**(lb/ft^2)**	**(Pa)**
Erosion Control Blanket [a]	Type I	1.5	72
	Type II	1.75	84
	Type III	2.00	96
	Type IV	2.25	108
Turf Reinforcement Mat [a]	Unvegetated	3.0	143.6
	Class I	6.0	288
	Class II	8.0	384
	Class III	10.0	480
Grass [b]	Class A	3.70	177.2
	Class B	2.10	100.5
	Class C	1.00	47.9
	Class D	0.60	28.7
	Class E	0.35	16.8
Rock Riprap	Class A1	3.00	143.6
	Class B	5.00	239.4
	Class C	6.70	320.8
Bare Soil	Non-cohesive	(See Hydraulic Engineering Circular No. 15)	
	Cohesive		

FPL

Entrance Ramp

[a] General values based on vendor information, assuming a vegetated condition. Maximum permissible shear stress for an unvegetated mat is 3.0 lb/ft^2)
Consult with individual vendors for more specific information.

[b] Grassed linings are classified into 5 vegetal retardance classifications
See Section 5.04.6.1 and Table 5A-4

Table 5A-7
Permissible Shear Stresses for Lining Materials
Reference: USDOT, FHWA, HDS-4 (2001) &
Erosion Control Technology Council, St. Paul, Minnesota

3. Where the computed shear on a vegetated lining exceeds 2 lb/sf, a turf reinforcement mat should be provided. The heaviest classes of these materials are able to withstand a shear stress of up to 10 lb/sf after vegetation has established in the ditch.

4. Where permanent turf reinforcement mats are used, they should be checked to ensure that the shear stress imposed by the 2-year storm event will not exceed the strength of the unvegetated lining. Where this is exceeded, it is likely that high flows in the ditch will cause damage to the lining before its fully vegetated shear strength can be developed. In this case, riprap should be used on the bottom of the channel up to a height equal to the depth of flow in the 2-year storm event on the class of riprap selected. Permanent turf reinforcement mat may then be placed above the riprap. It should be noted that the class of riprap and turf reinforcement mat should be selected based on the design flow rate.

5. Riprap should also be used where perennial flows or frequent ponding could drown the grass in a vegetated lining. In this case, the height of the riprap above the channel bottom only needs to be sufficient to accommodate the low flows. Permanent turf reinforcement mat, sod, or seeded grasses with erosion control blanket may be placed above the riprap, depending upon the shear stress imposed by the design discharge. It should be noted that this riprap may not be required where a ditch has a rock bottom.

6. Hard armor, such a concrete lining or grouted riprap may be used where the shear stress imposed by the design flow exceeds 10 lb/sf. However, in these cases the designer should consider the use of ditch checks or other means of reducing the velocity of the flow.

7. When the slope of the proposed ditch is 10 percent or greater, the designer should refer to Section 5.04.7.1.2.1 for special criteria regarding the use of riprap ditches on steep slopes.

Table 5A-13 through 5A-18 in the Appendix can serve as guide to selecting a lining material.

5.04.7.2.1 TRACTIVE FORCE ANALYSIS

The hydrodynamic force of water flowing in a channel is known as the tractive force. The ditch will be stable when the flow-induced tractive force does not exceed the permissible shear stress of the lining materials. In a uniform flow, the tractive force is equal to the effective component of the gravitational force acting on the body of water, parallel to the channel bottom. Thus, the maximum shear stress for a straight channel occurs on the ditch bottom and is less than or equal to the shear stress at maximum depth. This is expressed as:

$$\tau_{max} = \gamma d S \qquad (5\text{-}10)$$

Where:
τ_{max} = maximum shear stress, (lb/ft^2)
γ = unit weight of water, (62.4 lb/ft^3)
d = maximum depth of flow, (ft)
S= average bed slope or energy slope, expressed as a decimal, (ft/ft)

5.04.7.1.2.1 RIPRAP LININGS ON STEEP SLOPES

In general, riprap stability on a ditch side slope depends on the forces acting on the individual stones in the riprap layer. These forces include the weight of a stone, the lift, and drag forces induced on it by the ditch flow. On a steep slope, the weight of a stone has a significant component in the direction of flow. Because of this, a stone within the riprap will tend to be moved by the flow more easily than the same stone on a mild gradient. Hence, for a given discharge, ditches on steep slopes require larger stones to compensate for the greater shear forces in the flow direction.

Stone size is less critical for wire enclosed riprap (gabions) on steep slopes because the stones are bound by a wire mesh which allows the material to act as a single unit. However, the stability of wire enclosed riprap depends on the integrity of the wire mesh. In ditches carrying high concentrations of sediment or rocks, the wire mesh may be abraded and could potentially fail. Thus, the use of these structures should be avoided where such conditions exist.

The actual vector analysis of riprap stability on steep slopes can be very complex and involves a number of assumptions. Tables 5A-22 through 5A-25 are provided in the Appendix as a guide for selecting riprap for ditches on steep gradients. For a given ditch slope and cross section, these tables specify the greatest design discharge allowed for different classes of riprap linings. It should be noted that the tables are based on a safety factor of 1.5. A detailed procedure for the use of these tables is presented in Section 5.06.1.3.6.

To the extent possible, steep ditches should be on straight alignments. Freeboard should be equal to the average depth of flow, since the wave heights in the flow may reach approximately twice the mean depth. In addition, an energy dissipation structure such as a riprap basin may be required where the steep ditch transitions to a mild gradient ditch. More information on the use and design of these structures may be obtained in Chapter 9 of this Manual.

5.04.7.1.3 GROUTED RIPRAP

This material consists of hand-placed rubble-stone riprap with grout poured into the voids to form a rigid lining. The principal advantage of this material is that it can withstand velocities up to 15 feet per second without the need to transport large stones to the construction site with the resulting wear and tear on equipment.

However, before recommending the use of grouted riprap, the designer should be aware of its limitations. First, construction of grouted riprap is labor-intensive. The higher construction cost may offset the cost of transporting larger sized riprap to the project site. Second, even though the material is rigid, it is fairly weak structurally. Thus, it may fail if large gaps form in the underlying material. The designer should make an effort to check whether undermining may occur before recommending grouted riprap at a site.

5.04.7.1.4 CAST-IN-PLACE CONCRETE

A concrete paved side or median ditch may be used where it would be more economical than a riprap-lined ditch or where flow velocities exceed 12 feet per second. Where the use of a concrete-lined ditch is not preferable, it may be possible to line the ditch with riprap and use rock check dams to control the velocity. However, these situations tend to occur where the

slope of the ditch is steep, and the designer should refer to Section 5.04.7.1.2.1 for criteria to be applied to riprap ditches at slopes of 10 percent or greater.

Because concrete lining is rigid, it is subject to failure from undermining or from any other change in grade that may occur in the underlying materials. Thus, the design of a paved ditch should include considerations to minimize the potential for failure. Concrete-lined ditches should be anchored to the subgrade by cut-off walls (or lugs) at the intervals specified in the TDOT Standard Drawings. These cut-off walls anchor the ditch into the subgrade, help to minimize erosion beneath the lining and defend against erosion at the ditch outlet.

Paved ditches should be provided with a minimum freeboard of 1 foot for the design flow. This will allow for splashing and bulking effects from air entrainment due to high flow velocities. Typically, above the paved section, sod will be used on the ditch slopes to provide additional protection.

The effect of high velocity flows at the channel exit must be considered and some provision should be made to dissipate the excess energy; otherwise, erosion at the outlet likely will result in undermining, and potentially, a progressive failure of the upstream paved ditch sections. Design of stilling basins and energy dissipators is discussed in Chapter 9 of this Manual.

5.04.7.1.5 WIRE-ENCLOSED STONE (GABIONS)

Wire-enclosed stone refers to rectangular containers made of galvanized woven (twisted) steel wire mesh or welded steel wire mesh in a uniform pattern which are filled with stone. Typically, wire-enclosed stone, or gabions, will be either rectangular boxes or thin mattresses. The individual stone-filled units are tied together and anchored to the ditch side slope. Because of their flexibility, gabion structures can yield to earth movement and remain structurally sound while maintaining their intended purpose. Gabions absorb energy contained in a storm discharge by forcing water to pass through the void spaces in the structure; whereby reducing velocity in the ditch or channel. Because this material is permeable, it should be placed on a geotextile fabric to prevent any piping of the underlying materials.

Gabions may be used where machined riprap is either not available, impractical, or not economical. Mattress units may be placed on a geotextile on standard ditch side slopes. In addition, gabions may be stacked to stabilize extremely steep slopes. Thus, they may be used where machined riprap would not provide sufficient slope stability. The use of wire-enclosed stone should be avoided where flows may carry high quantities of abrasive materials such as sand, gravel, or a heavy debris load. The coating, which protects the wire from rust, can be worn off, resulting in the potential failure of the structure. Due to safety considerations, gabions should not be used within the clear zone of any roadway.

5.04.7.1.6 COMPOSITE DITCH LININGS

As shown in the Standard Roadway Drawings, the bottom of a roadside ditch may be lined with concrete, riprap or other material while the sides adjacent to the bottom are provided with a vegetated strip of sod or erosion control blanket. The vegetated strip should be at least two feet wide. This type of composite lining is useful in situations where frequent low flows or sustained flows could drown the grass in a sodded ditch lining.

Calculation 4/13/2020

Drainage Area → SB9 Is 225' enough length?

C = 0.853 (From Drainge Calculation Excel File)

i = 7.8 (Figure 4A-7)

A = 0.1015 (Drainage Area Map)

$Q = CiA \rightarrow$ ~~0.68~~ 0.676 cfs

Adjusted Q from previous inlet spill over

- This Area located at high point → no run over from previous area

Slope = 0.0281 (Vertical Profiles)

$\text{Conveyance} = K = \frac{Q}{\text{Slope}^{0.5}}$ → Equation 7-2 TDOT Drainage Manual

$K = \frac{\text{~~0.68~~ } 0.676}{0.0281^{0.5}} = 4.0309$

Depth Spread Area → Interpolated From TABLE 7A-4

Depth = 0.18 ft

Spread = 2.57 ft → under 8' ✓

Area = 0.20 sf

$Q = AV$

$V = \frac{0.676}{0.20} = 3.37 \text{ f/s}$

Width = 1.75 (See TDOT Standard Drawing D-CB-12P)

Splash Over Velocity V_o

$$V_o = 2.407 (L)^{0.794}$$ Equation 7-10

$L = 3.208$ (D-CB-12P)

$V_o = 6.07$ ft/s

Flow in front of the grate area (Af)

$$A_f = w \frac{2y - w S_g}{2}$$

S_g = cross slope of gutter = 0.085 (Typical Sections)

$y = 0.41'$ (Typical sections)

$$A_f = 1.75 \frac{2(0.41) - 1.75(0.085)}{2} = 0.59 \text{ ft}^2$$

P_f = wetted perimeter = $y + w$ Equation 7-12

$P_f = 2.16$ ft

K_u = Conveyance of the subsection

$$K_u = \frac{1.486 A_f \left(\frac{A_f}{P_f}\right)^{0.667}}{n}$$ Equation 7-13

$$= \frac{1.486 (0.59) \left(\frac{0.59}{2.16}\right)^{0.667}}{0.013} = 28.17$$

Adjusted frontal flow subsection conveyance

$$K_f = 0.85 \times K_u \qquad \text{7-14 equation}$$

$$K_f = 0.85 \times 28.17 = 23.94$$

Compute frontal flow

$$Q_f = Q \frac{K_f}{K} \qquad \text{equation 7-15}$$

$$= 0.676 \frac{23.94}{4.03} = 4.01 \text{ cfs}$$

Gutter Inlet Interception

$$V_f = \frac{Q_f}{A_f} \qquad \text{Equation 7-16}$$

$$V_f = \frac{4.01 \text{ cfs}}{0.59} = 6.83 \text{ f/s}$$

Compute side flow Q_s

$$Q_s = Q - Q_f \qquad \text{Equation 7-19}$$

Q_s is negative so no flow passes the frontal flow. Hence, 225' is enough for this inlet.

Zachary Jerome

7.03.3.7 DESIGN STORM FREQUENCY AND SPREAD FOR INLET SPACING

As discussed in Section 7.03.3.1, the spread of flow on the roadway is the criterion used to select the location of inlets on a continuous slope. As shown in Table 4-1 of this Manual, the storm frequency selected for the design of pavement drainage should be consistent with the storm frequency used to design the other elements of the storm drainage system. The allowable spread on roadway pavements, for both curb inlets and median barrier wall inlets, should be based on the following criteria:

- In general, the greatest allowable spread on any facility should be 8 feet, including the gutter width.
- The designer may choose to specify a smaller spread where the roadway cross section includes a bicycle lane. This would serve to limit the depth of flow in the bicycle lane.
- The curb height should also be considered in determining the allowable spread. On some roadway cross sections, a spread of 8 feet may result in a water depth greater than the curb height. In such cases, the allowable spread should be reduced.
- Where inlets are used along a freeway, the allowable spread should not exceed the shoulder width.
- The spacing between two adjacent catch basins should not be greater than 400 feet, as provided in Section 7.03.5.6.

7.03.4 STORM SEWER PIPES

A typical storm drainage system is made up of pipes, catch basins and manholes. Storm sewer pipes serve to convey storm water from the drainage system inlets to the ultimate outlet point of the drainage system. They should be provided with adequate capacity to accommodate the design flow and should be designed to minimize maintenance concerns. The size and grade of a storm sewer pipe are closely related to the type of structures connected to the upstream and downstream ends of the pipe. The design software GEOPAK Drainage (see Section 7.05.2) as implemented by TDOT, contains features which allow the designer to ensure that the proposed pipe design is compatible with the proposed structures on either end. This section discusses the various criteria that should apply to the selection and design of storm sewer pipes. The design of catch basins and manholes is discussed in Section 7.03.5.

7.03.4.1 PIPE TYPE AND COVER CRITERIA

Storm sewer pipes for all classifications of roadway, except local roads, should be reinforced concrete pipe. As shown on Table 6A-1, the use of alternate pipe types is permitted for local roads only. The selection of concrete pipe class is governed by the maximum depth of cover that will be placed on the pipe at any point along the pipe run. Typically, the maximum allowable cover will be determined by the maximum depths allowed for the catch basins or manholes to which the pipe will be connected.

The minimum allowable depth of cover for all pipes under design loads will be 12 inches, measured from the bottom of the subgrade to the outside surface of the pipe. For construction loads, polyvinyl chloride or high-density polyethylene pipe should be provided with a minimum cover of 24 inches. The designer should insure that the minimum cover is maintained at all points where a pipe is beneath travel lanes or shoulders. In particular, this may become an issue when designing a pipe to connect to the catch basin at the sag point of a steep grade. The

WORKSHEET FOR INLET SPACING ON CONTINUOUS SLOPES

PROJECT: _____ From Station: _____ (high point)
To Station: _____ (sag point)

ADT: _____ vpd 5-minute, 10-year rainfall intensity: _____ in/hr

Designer: ____ Date: _____ Sheet ____ of ____ (See comp's for sag inlet on Sheet ____)

See Drainage Design Manual Section 7.04.4.2

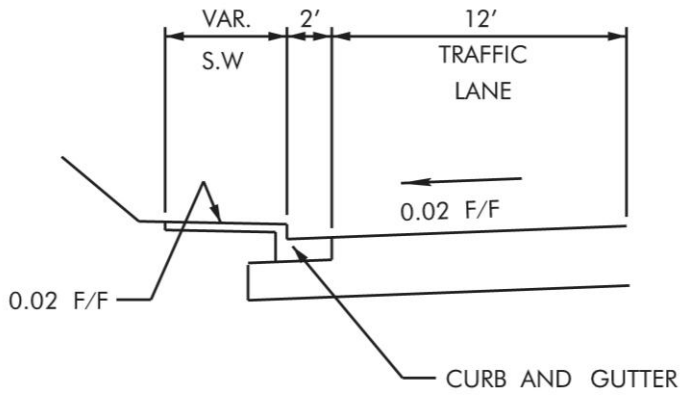
Sta.	Road Area (ac.)[1]	Road C	Road t_c (min.)	Off-Site Area (ac.)	Off-Site C	Off-Site t_c (min.)	i (in/hr)[2]	Q (cfs)[3]	T (feet)	Allow-able T (feet)	See Interception Worksheet Q_i (cfs)	See Interception Worksheet Q_b (cfs)	Inlet Type
1	2	3	4	5	6	7	8	9	10	11	12	13	14

[1] Determine as A = L w /43560, where A = area; L = Length from upstream inlet; and w = average roadway width.
[2] Determine i based upon the maximum of the roadway and off-site times of concentration. See Section 4.04.1.2.
[3] $Q = [C_rA_r + C_oA_o] i + Q_{u/s}$, where C_r and A_r are the runoff coefficient and area of the roadway; C_o and A_o are the off-site runoff coefficient and area; i = rainfall intensity; and $Q_{u/s}$ = any bypass flows from upstream.

Figure 7A-1
Inlet Spacing on Continuous Slopes

Table integrated into "Drainage Calculations" excel file

SECTION WITH NO SHOULDER OR SHOULDER AT 2%



Depth (feet)	Spread (feet)	Area (sf)	Conveyance, K_t
0.03	0.40	0.01	0.05
0.07	0.80	0.03	0.31
0.10	1.20	0.06	0.91
0.14	1.60	0.11	1.96
0.17	2.00	0.17	3.55
0.20	3.50	0.25	4.81
0.25	6.00	0.49	10.25
0.30	8.50	0.85	20.54
0.35	11.00	1.34	36.84
0.40	13.50	1.95	60.28
0.45	16.00	2.69	91.91
0.50	18.50	3.55	132.73
0.55	21.00	4.54	183.68
0.60	23.50	5.65	245.66

Table 7A-4
Conveyance, Spread, and Flow Area for Standard Gutter Sections

2020 Vision Eng | Pipe Capacity Sample Calculation | Zachary Jerome

CB-1 → EW-1

Compute Sum of CA → Equation 7-32

$\sum CA = C_r + A_r + C_o A_o + \ldots$

TDOT Drainage Manual

Incoming Inlets and Drainage Areas

see Design Drawing

Inlet	Drainage Area
CB-2	OB 5, OB-6
CB-3	SB 3
CB-4	SB 4

$\sum CA = 0.85 \times 0.0986 + 0.69 \times 2.5829 + 0.85 \times 0.3041 + 0.85 \times 0.2680 = 2.362005$

Find $Q = CiA$

TOC = 11.35 min (see excel file)

i = 5.9 (Figure 4A-7)

$Q = 2.362005 \times 5.9 = 13.94$ cfs

$V = \frac{Q}{A}$

Assume diameter = 1.5'

$V = \frac{13.94}{(\frac{1.5'}{2})^2 \pi} = 7.89$ f/s

Calculation

Flow demand = 13.94 cfs

Velocity demand = 7.89 f/s

Design a pipe that meets these demands

$$Q_{capacity} = \frac{0.463}{n} D^{2.667} SL_p^{0.5}$$ Equation 7-36

n = 0.013

D = 1.5'

SL = 0.005

$Q_{capacity}$ = 7.43 cfs

not enough

Increase slope

n = 0.013

D = 1.5'

SL = 0.02

$Q_{capacity}$ = 14.85 ✓

V = 8.40 f/s < 12 f/s ✓

Use a 1.5' diameter pipe at a 2% slope

Check inlet elevations

Elevation of EW-1 (per ditch design)

987.90

$$SL_p = \frac{Invus - Invds}{L_p}$$ Equation 7-35

2020 Vision Eng Pipe Capacity Sample Zachary Jerome

Calculation

$$0.02 = \frac{Inv_{US} - Inv_{DS}}{L_P}$$

Length between CB-1 and EW-1

$= 28.46'$ (Design Drawing)

$$0.02 = \frac{Inv_{US} - 987.90}{28.46}$$

Inv_{US} (CB-1) $= 988.47'$

Check the depth of CB-1

TC Elevation $= 994.9$

~~Q~~ Outlet Elevation $= 988.47'$

Depth $= 6.43'$

> 3.9 feet (D-CB-12P)

✓

This pipe size works

Zachary Jerome

7.03.4.3 MINIMUM AND MAXIMUM VELOCITY

It is important that flow velocities in storm sewer pipes be sufficient to prevent sediment deposition and subsequent capacity loss. Thus, all storm sewer pipes should be designed such that the velocity at the design flow rate will be 3 feet per second or greater. For most design situations, the flow velocity at the actual design discharge will be approximately equal to the velocity at full flow. Thus, the full flow velocity may be used to check this criterion.

It is preferable to maintain a pipe slope of at least 0.5% at all points in the storm sewer system. However, a slope of 0.4% should be considered the minimum constructible grade. The TDOT implementation of GEOPAK Drainage contains a check to ensure that this criteria is met.

Slopes that incur uniform flow velocities in excess of 12 feet per second should be avoided due to the potential for abrasion. In steeper terrain, large elevation differences can be accommodated by the use of drop structures. A figure illustrating the use of drop structures and a discussion of their use for velocity control in culverts is provided in Section 6.04.1.1.1.5.

7.03.5 CATCH BASINS AND MANHOLES

Catch basins and manholes serve a number of purposes in the storm sewer system, including collecting inflow, providing access for maintenance, and allowing for changes in direction and grade of the trunk line. By definition, catch basins serve as inlet points for discharges along curbs or ditches. They may also be used to connect trunk line storm sewer pipes. Manholes are often used to connect trunk line storm sewer pipes.

Specific information on allowable depths, maximum pipe connection sizes and structural details are provided in the TDOT Standard Drawings. In addition, this information has been entered into the TDOT implementation of GEOPAK Drainage, thus allowing the designer to ensure that the proposed pipe connections will be consistent with the current standard drawings. This section of the Manual discusses the types of catch basins and manhole available for design, as well as providing criteria for pipe connections to these structures.

7.03.5.1 CATCH BASINS

Catch basins consist of a metal inlet casting placed on an inlet structure. Information on the available inlet castings is provided in Section 7.03.3.5 and in the TDOT Standard Drawings. Inlet structures may be round, square or rectangular and may be constructed of precast concrete, cast-in-place concrete or brick. Table 7-4 summarizes the available types of standard inlet structures.

The designer should check the current TDOT Standard Drawings for any changes or additions to this list. If required, catch basins of sizes other than those shown may be used.

DESIGN DISCHARGE DETERMINATION FOR STORM SEWER PIPE SIZING

See Drainage Design Manual Section 7.04.5.2

PROJECT: _____

DESCRIPTION OF LATERAL: _____ Start Station: _____ End Station: _____

Designer: ____ Date: _____ Sheet ____ of ____

Str #	Station	Added Road Area (ac.)	Road C	Added Off-Site Area (ac.)	Off-Site C	Total Area (ac.)[1]	Σ "CA"[2]	Off-Site t_c (min.)	t_c (min.)[3]	i (in/hr)	Design Q (cfs)
1	2	3	4	5	6	7	8	9	10	11	12

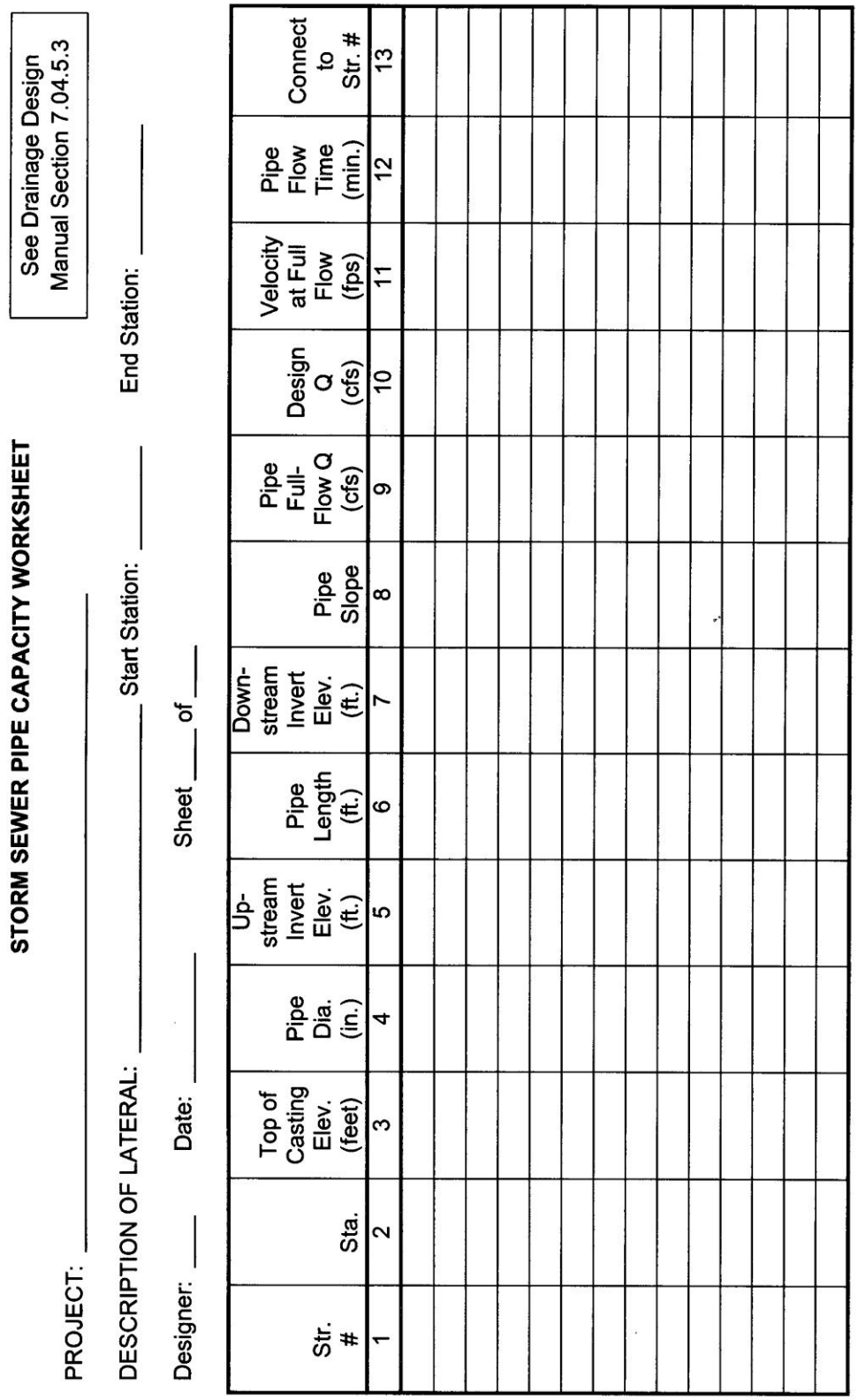
[1] Added road area + added off-site area + total area on previous row

[2] [Added Road Area * Road C] + [Added Off-Site Area * Off-Site C] + ΣCA on previous row

[3] Greater of Off-Site t_c or [t_c + Pipe Flow Time] on previous row

See "Drainage Calculations" Excel file

Figure 7A-4
Design Discharge for Storm Sewer Pipe Sizing



STORM SEWER PIPE CAPACITY WORKSHEET

See Drainage Design Manual Section 7.04.5.3

PROJECT: _____

DESCRIPTION OF LATERAL: _____ Start Station: _____ End Station: _____

Designer: ____ Date: _____ Sheet ___ of ___

Str. #	Sta.	Top of Casting Elev. (feet)	Pipe Dia. (in.)	Up-stream Invert Elev. (ft.)	Pipe Length (ft.)	Down-stream Invert Elev. (ft.)	Pipe Slope	Pipe Full-Flow Q (cfs)	Design Q (cfs)	Velocity at Full Flow (fps)	Pipe Flow Time (min.)	Connect to Str. #
1	2	3	4	5	6	7	8	9	10	11	12	13

See "drainage calculations" excel file

Figure 7A-5
Storm Sewer Pipe Capacity

L_p = pipe length, (ft)
V_{full} = pipe full flow velocity, (ft/s)

This procedure is repeated for each pipe in the system until the outfall is reached. A sample computation form and an example problem are included in the Appendix.

7.04.6 ENERGY GRADE LINE AND HYDRAULIC GRADE LINE COMPUTATIONS

As described in Section 7.03.4, hydraulic grade line computations should be performed to ensure that the HGL computed for the 50-year storm event will be below each of the castings in the storm sewer system. However, where the entire system has been designed for the 50-year storm event, the HGL check will not be needed. These computations can become quite complicated due to the variety of flow conditions that may be encountered in the storm sewer system. A number of the pipes in the system may be entirely under pressure flow, other may be entirely under gravity flow and still others may flow full for only a portion of their length. In order to account for these complexities, the energy equation (see Section 5.03.2) may be written as:

$$inv_{u/s} + d_{u/s} + \frac{P_{u/s}}{\gamma} + \frac{V_{u/s}^2}{2g} = inv_{d/s} + d_{d/s} + \frac{P_{d/s}}{\gamma} + \frac{V_{d/s}^2}{2g} + h_f + h_m \qquad (7\text{-}39)$$

Where:
$inv_{u/s}$ = upstream invert elevation of the pipe or structure, (ft)
$d_{u/s}$ = depth in the upstream structure, (ft)
$P_{u/s}$ = pressure in the upstream structure, (lb/ft^2)
γ = specific weight of water, (lb/ft^3)
$V_{u/s}$ = flow velocity in the upstream structure, (ft/s)
g = acceleration of gravity, (32.2 ft/s^2)
$inv_{d/s}$ = upstream invert elevation of the pipe or structure, (ft)
$d_{d/s}$ = depth in the upstream structure, (ft)
$P_{d/s}$ = pressure in the upstream structure, (lb/ft^2)
$V_{d/s}$ = flow velocity in the downstream structure, (ft/s)
h_f = friction head loss, (ft)
h_m = summation of minor head losses at junctions, bends etc., (ft)

Major head losses result from friction within the pipe. Minor head losses are those resulting from turbulence at:

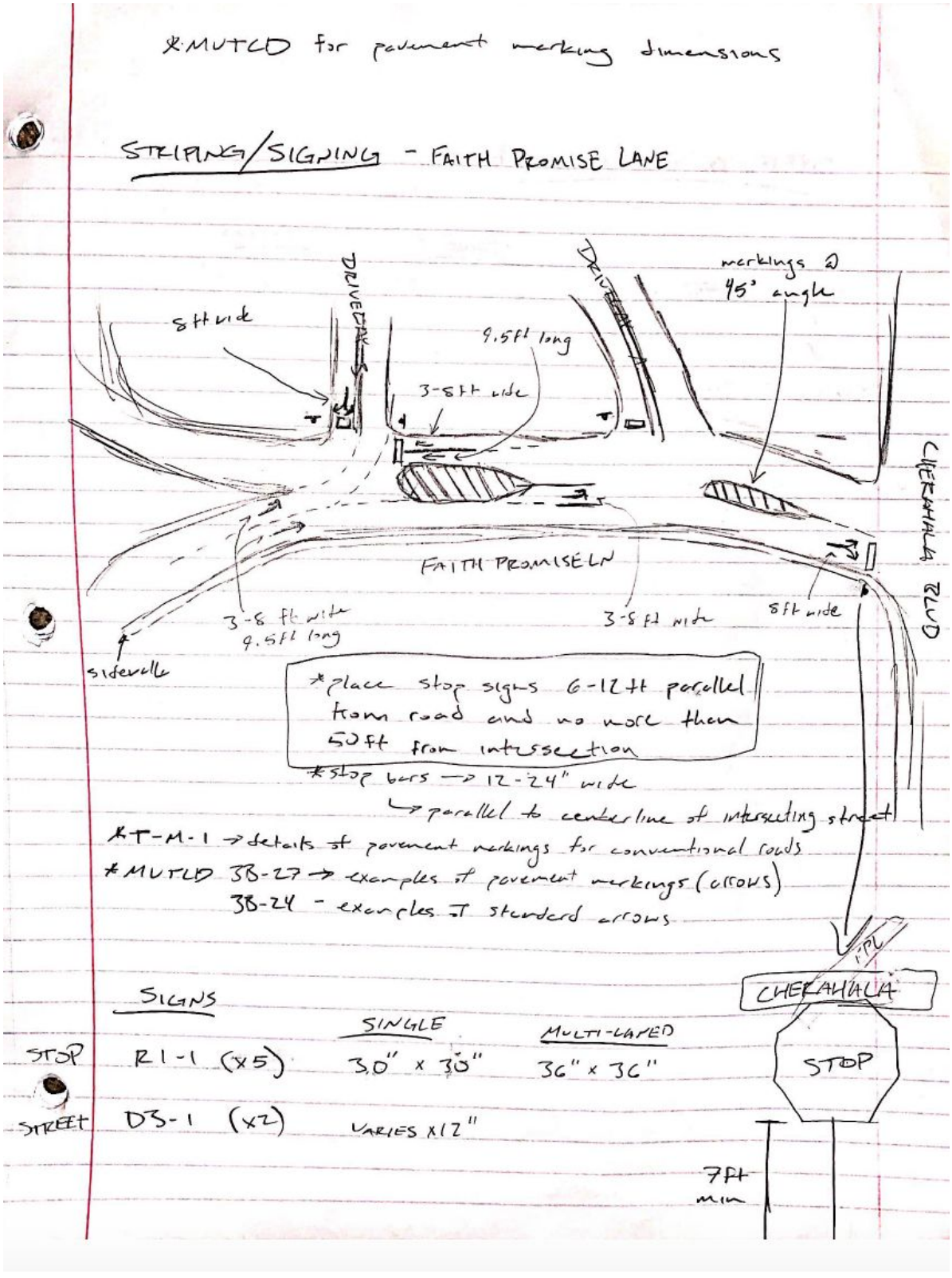
- catch basins
- pipe outfalls
- pipe inlets
- bends in pipes
- plunging flow
- expansions or contractions

Minor losses at a specific point in a storm drain system are often insignificant. In a large system, however, the combined effect of a large number of minor losses may be significant. Minor losses may be minimized to some extent by careful design. For example, most TDOT standard catch basins include some form of benching in order to provide for a smooth flow transition across the structure.

Appendix D: Striping & Signage

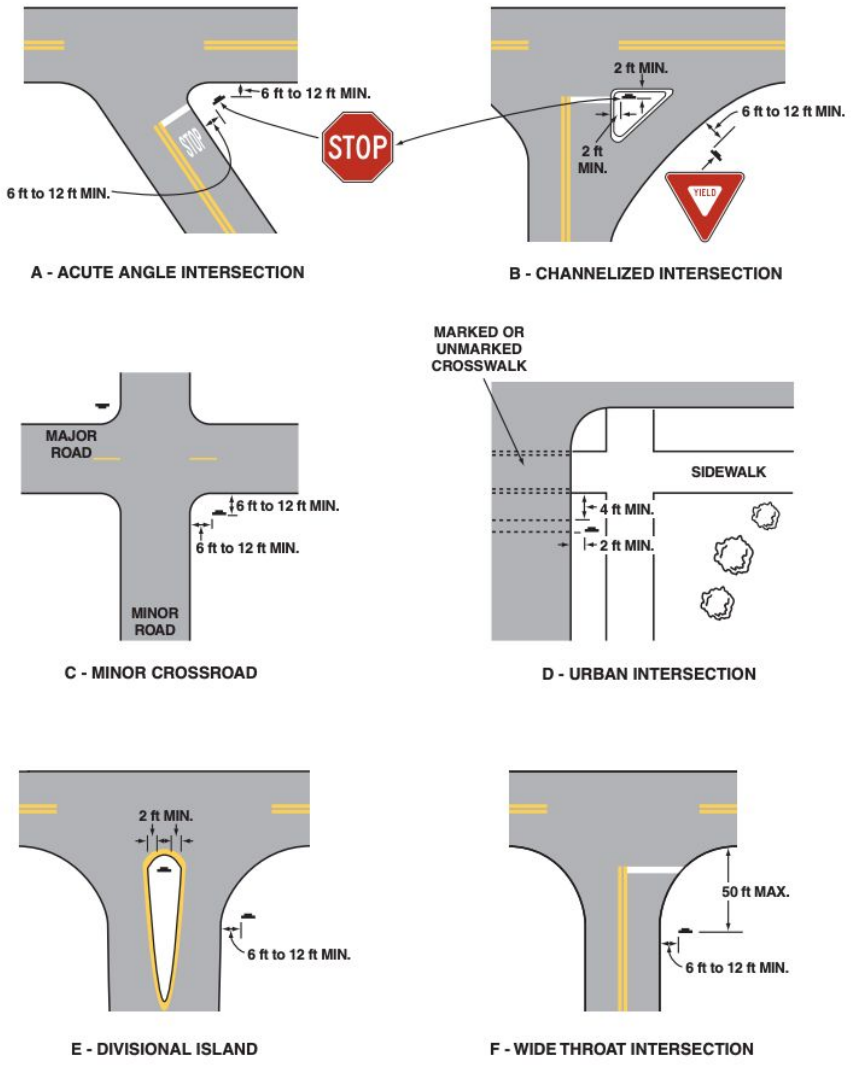
Appendix D: Striping and Signing

Engineering Design Work for Faith Promise Lane (first iteration)



MUTCD: Figure 2A-3 (see F – Wide Throat Intersection for intersection of Cherahala Blvd. and Faith Promise Lane)

Figure 2A-3. Examples of Locations for Some Typical Signs at Intersections



A - ACUTE ANGLE INTERSECTION

B - CHANNELIZED INTERSECTION

C - MINOR CROSSROAD

D - URBAN INTERSECTION

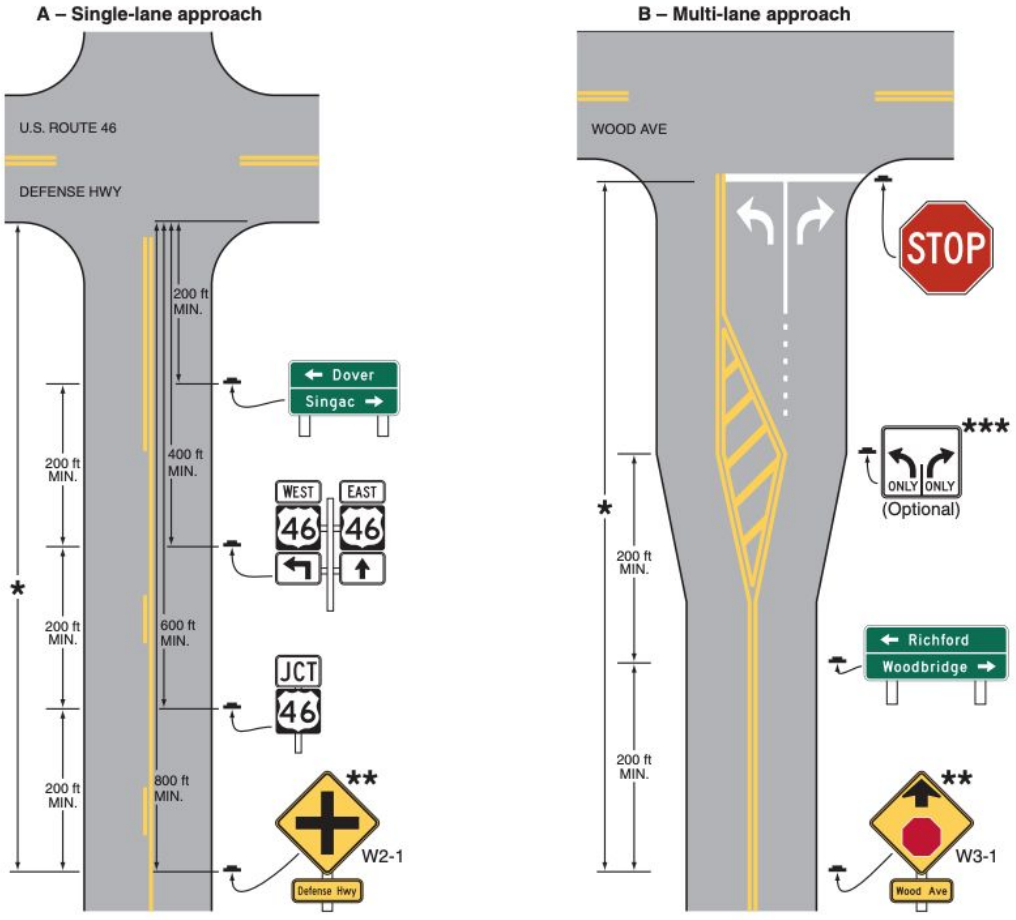
E - DIVISIONAL ISLAND

F - WIDE THROAT INTERSECTION

Note: Lateral offset is a minimum of 6 feet measured from the edge of the shoulder, or 12 feet measured from the edge of the traveled way. See Section 2A.19 for lower minimums that may be used in urban areas, or where lateral offset space is limited.

MUTCD: Figure 2A-4 (see B – Multi-lane approach for all-stop intersection heading southbound)

Figure 2A-4. Relative Locations of Regulatory, Warning, and Guide Signs on an Intersection Approach



Note: See Chapter 2D for information on guide signs and Part 3 for information on pavement markings

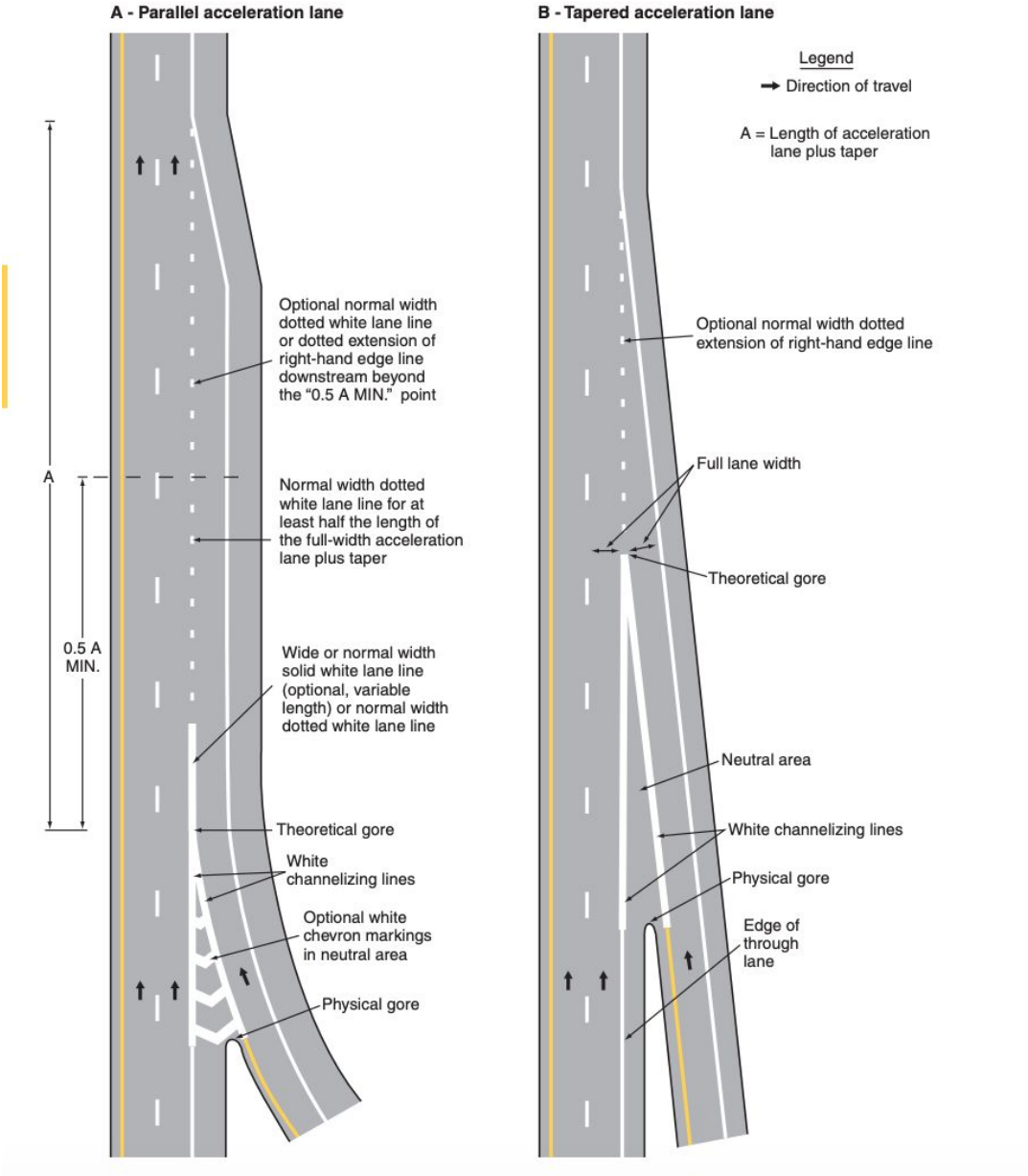
★ See Table 2C-4 for the recommended minimum distance

★★ See Section 2C.46 for the application of the W2-1 sign and Section 2C.36 for the application of the W3-1 sign

★★★ See Section 2B.22 for the application of Intersection Lane Control signs

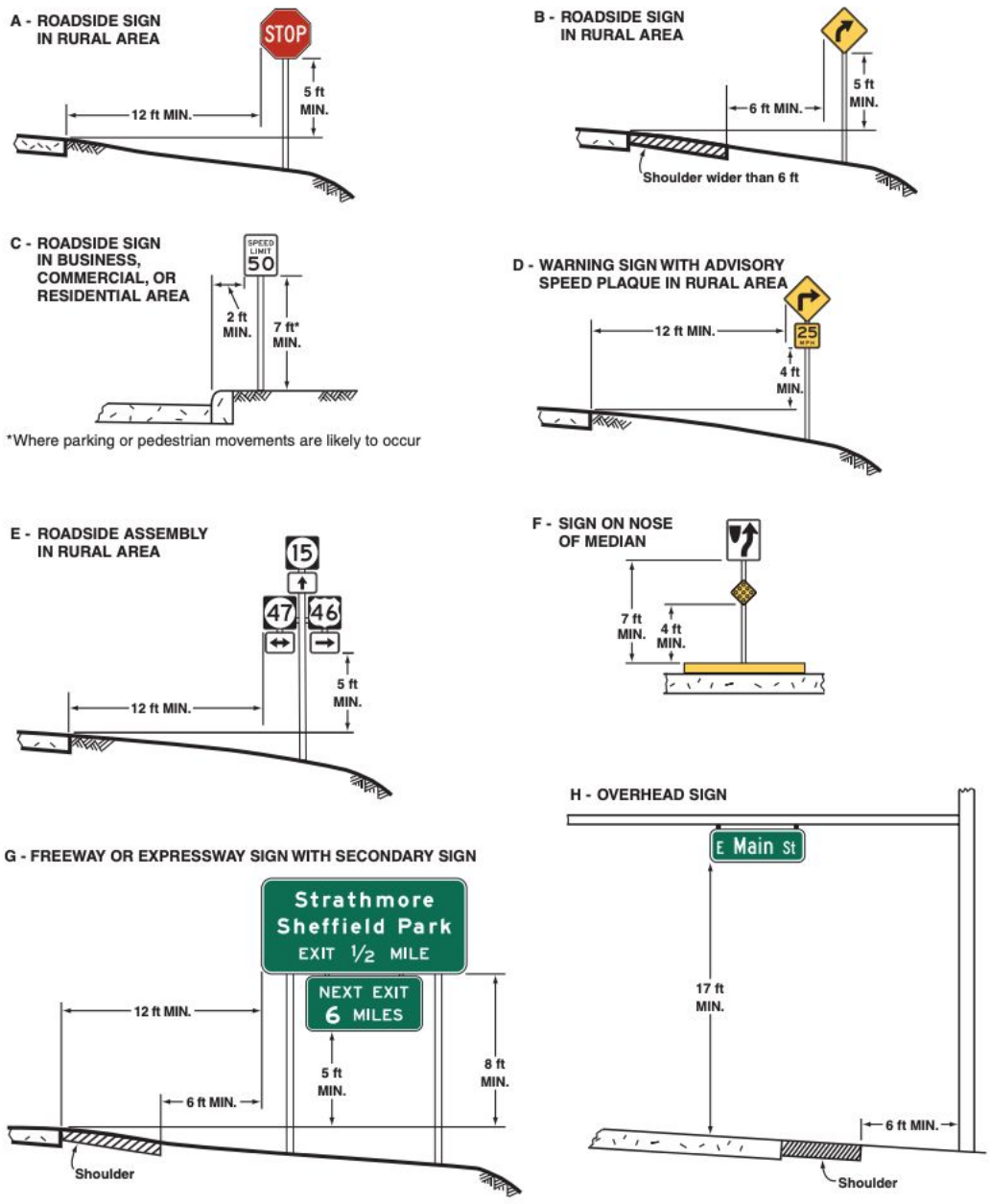
MUTCD: Figure 3B-9 (see B – Tapered acceleration lane to be used for the entrance ramp)

Figure 3B-9. Examples of Dotted Line and Channelizing Line Applications for Entrance Ramp Markings (Sheet 1 of 2)



MUTCD: Figure 2A-2 (see A and B)

Figure 2A-2. Examples of Heights and Lateral Locations of Sign Installations



Note:
See Section 2A.19 for reduced lateral offset distances that may be used in areas where lateral offsets are limited, and in business, commercial, or residential areas where sidewalk width is limited or where existing poles are close to the curb.

MUTCD: Section 3B-16 (Guidance/Sizing for Stop Lines)

Guidance:

08 *Stop lines should be 12 to 24 inches wide.*

09 *The individual triangles comprising the yield line should have a base of 12 to 24 inches wide and a height equal to 1.5 times the base. The space between the triangles should be 3 to 12 inches.*

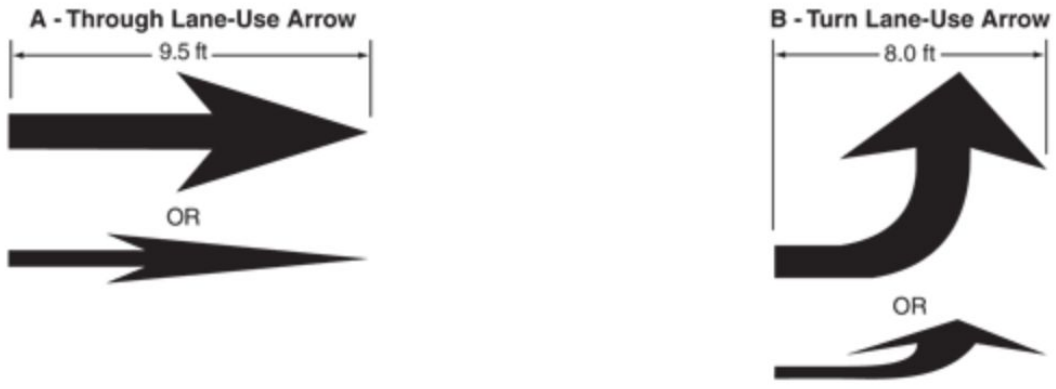
10 *If used, stop and yield lines should be placed a minimum of 4 feet in advance of the nearest crosswalk line at controlled intersections, except for yield lines at roundabouts as provided for in Section 3C.04 and at midblock crosswalks. In the absence of a marked crosswalk, the stop line or yield line should be placed at the desired stopping or yielding point, but should not be placed more than 30 feet or less than 4 feet from the nearest edge of the intersecting traveled way.*

11 *Stop lines at midblock signalized locations should be placed at least 40 feet in advance of the nearest signal indication (see Section 4D.14).*

12 *If yield or stop lines are used at a crosswalk that crosses an uncontrolled multi-lane approach, the yield lines or stop lines should be placed 20 to 50 feet in advance of the nearest crosswalk line, and parking should be prohibited in the area between the yield or stop line and the crosswalk (see Figure 3B-17).*

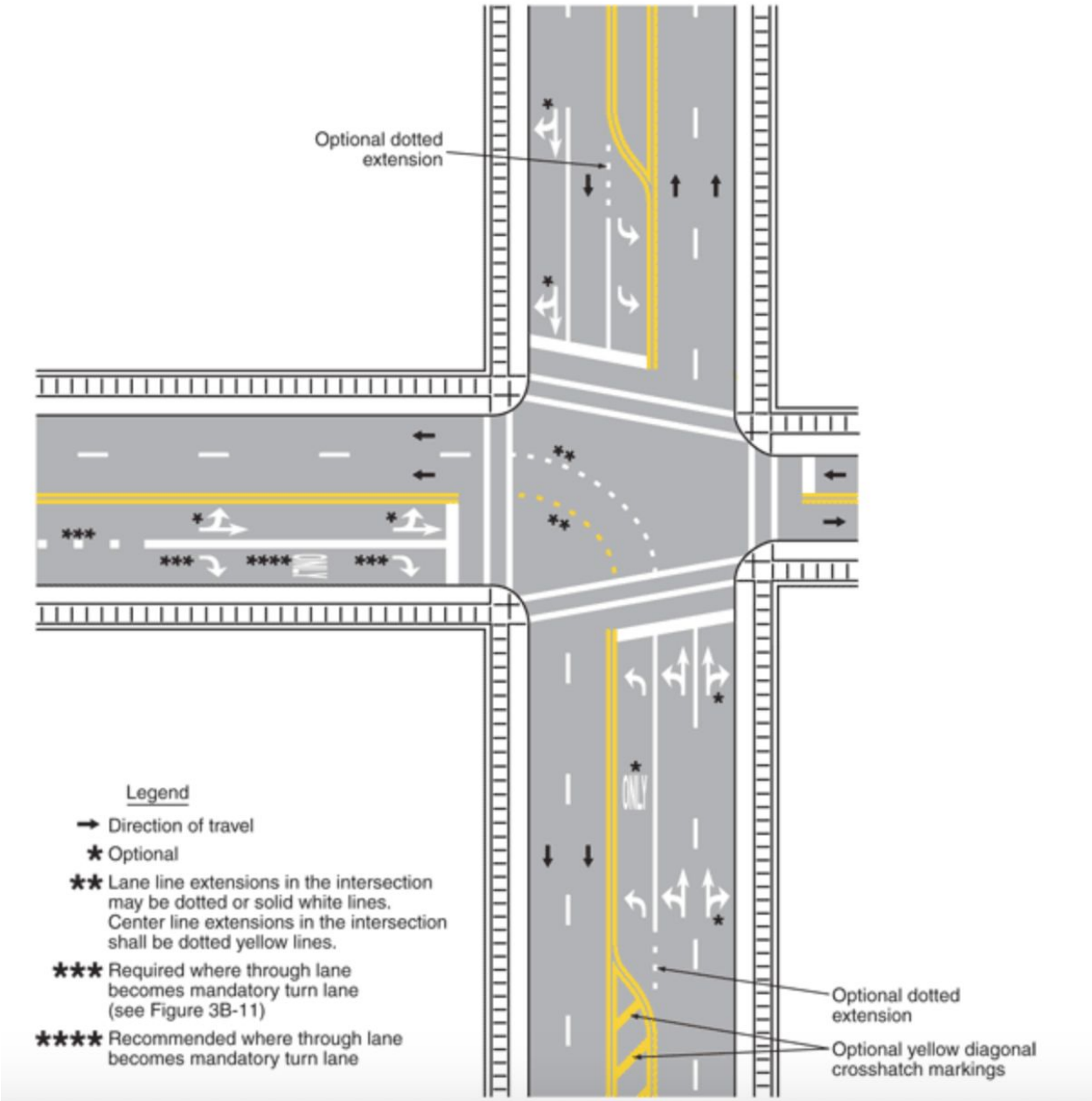
MUTCD: Figure 3B-24 (Pavement Marking Sizing for Arrows--See A and B)

Figure 3B-24. Examples of Standard Arrows for Pavement Markings



MUTCD: Figure 3B-27 (Crosshatch markings for channelizing median and other pavement markings)

Figure 3B-27. Examples of Lane-Use Control Word and Arrow Pavement Markings



MUTCD: Applicable Signs and Sizing of Chosen Signage

Sign or Plaque	Sign Designation	Section	Conventional Road		Expressway	Freeway	Minimum	Oversized
			Single Lane	Multi-Lane				
Stop	R1-1	2B.05	30 x 30*	36 x 36	36 x 36	—	30 x 30*	48 x 48

Figure 2B-1. STOP and YIELD Signs and Plaques



R1-1 R1-3P R1-2 R1-2aP R1-10P

MUTCD: MUTCD: Section 4C.04 and Figure (Warrant for Signage)

Section 4C.04 Warrant 3, Peak Hour

Support:
01 The Peak Hour signal warrant is intended for use at a location where traffic conditions are such that for a minimum of 1 hour of an average day, the minor-street traffic suffers undue delay when entering or crossing the major street.

Standard:
02 **This signal warrant shall be applied only in unusual cases, such as office complexes, manufacturing plants, industrial complexes, or high-occupancy vehicle facilities that attract or discharge large numbers of vehicles over a short time.**

03 **The need for a traffic control signal shall be considered if an engineering study finds that the criteria in either of the following two categories are met:**

A. If all three of the following conditions exist for the same 1 hour (any four consecutive 15-minute periods) of an average day:

1. **The total stopped time delay experienced by the traffic on one minor-street approach (one direction only) controlled by a STOP sign equals or exceeds: 4 vehicle-hours for a one-lane approach or 5 vehicle-hours for a two-lane approach; and**
2. **The volume on the same minor-street approach (one direction only) equals or exceeds 100 vehicles per hour for one moving lane of traffic or 150 vehicles per hour for two moving lanes; and**
3. **The total entering volume serviced during the hour equals or exceeds 650 vehicles per hour for intersections with three approaches or 800 vehicles per hour for intersections with four or more approaches.**

B. The plotted point representing the vehicles per hour on the major street (total of both approaches) and the corresponding vehicles per hour on the higher-volume minor-street approach (one direction only) for 1 hour (any four consecutive 15-minute periods) of an average day falls above the applicable curve in Figure 4C-3 for the existing combination of approach lanes.

TDOT Standards: MM-SW-1 (Standard Sidewalk Drawing)

Appendix E: Construction Management

Sample Calculation for Taper Length on Pellissippi

Table 6C-4 → Formulas for Taper Length (MUTCD)

Table 6C-4. Formulas for Determining Taper Length

Speed (S)	Taper Length (L) in feet
40 mph or less	$L = WS^2 / 60$
45 mph or more	$L = WS$

Where:

- L = taper length in feet
- W = width of offset in feet
- S = posted speed limit, or off-peak 85th-percentile speed prior to work starting, or the anticipated operating speed in mph

Speed Limit = 65mph
Width = 12ft

L = WS = 65mph * 12ft = **780ft taper**

Sample Calculations for Channelizing Drum Quantities

Section 6C.08 → Channelizing Drums (MUTCD)

Guidance:
13 *When used, a downstream taper should have a length of approximately 100 feet per lane with devices placed at a spacing of approximately 20 feet.*

Total Quantity Required for Channelizing Drums

Pellissippi

Taper length = 780ft
Activity Area length = 1520ft

Total length = 780ft + 1520ft = 2300ft

Total Quantity of Drums = 2300ft / 20ft/drum = **115 drums**

Faith Promise Lane

Activity Area length = 340ft

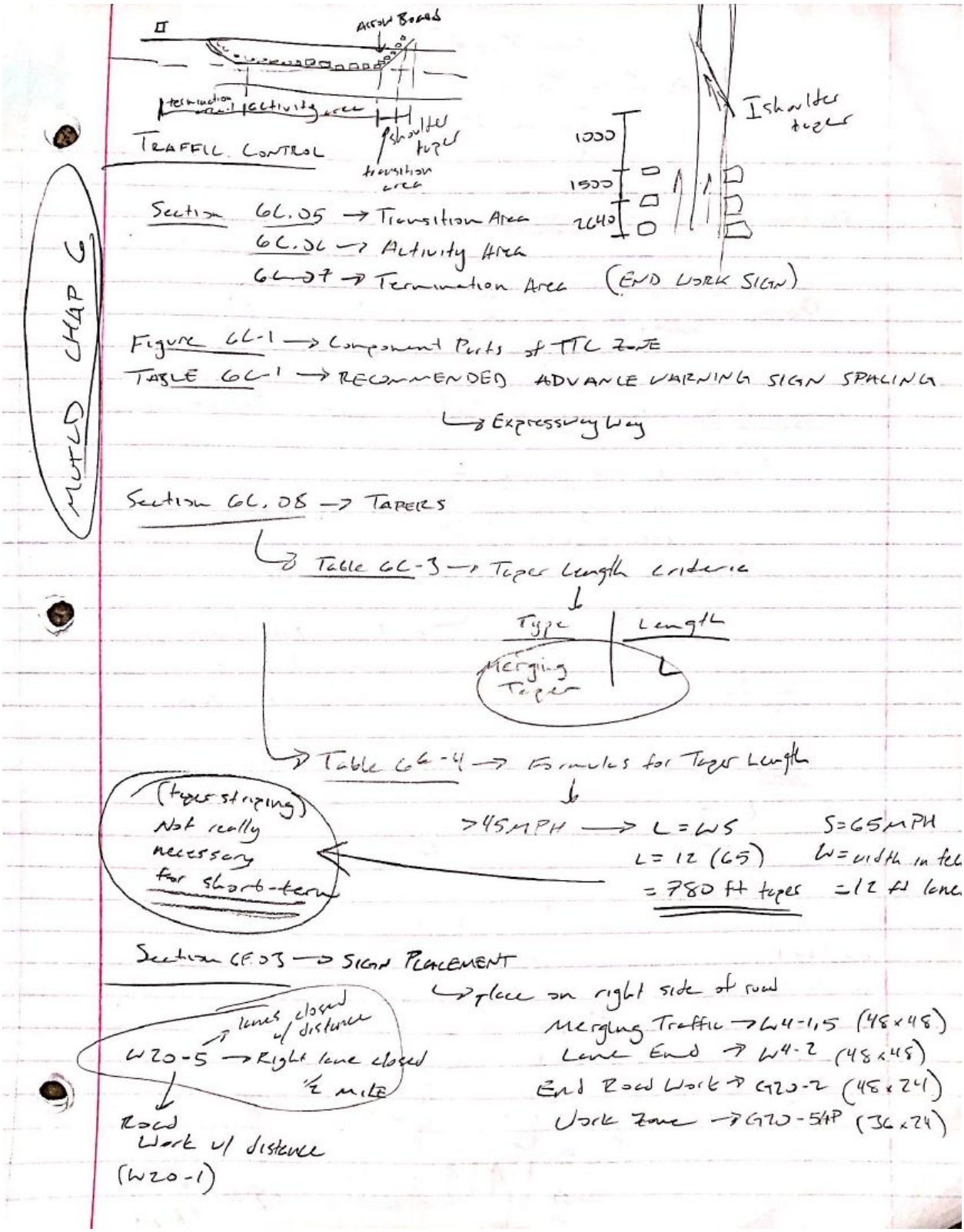
Total Quantity of Drums = 340ft / 20ft/dum = **17 drums**

Total Drum Quantity

Quantity for Pellissippi = 115 drums
Quantity for Faith Promise Lane = 17 drums

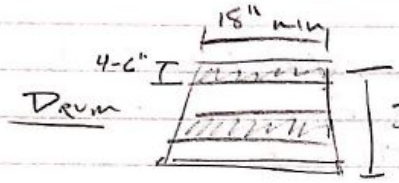
Total Quantity of Channelizing Drums = **132 drums**

Engineering Design Process Work for Pellissippi



Arrow Board?
Temp Speed Limit?
Barricades?

Section 6F.63 → Channelizing Devices



(Section 6F.67)

TABLE 6H-2 → Meaning of Symbols on Typical Diagrams

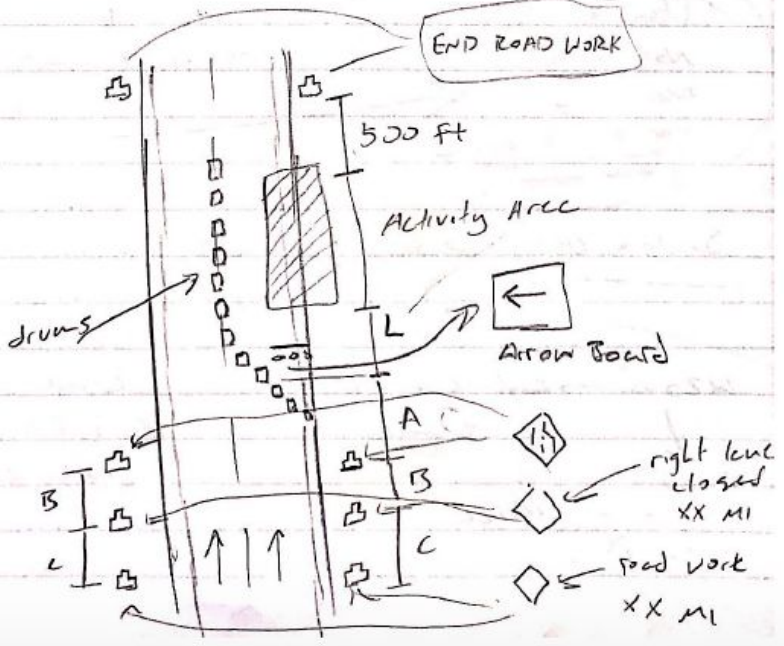
↳ direction of traffic →
channelizing device
sign
work space

↳ closest ROAD WORK AHEAD sign is A distance from work space

Figure 6H-33 → Lane Closure on a Divided Highway

↳ (B)
↓
short term

L = 780 ft
A = 1000 ft
B = 1500 ft
C = 2640 ft
Activity Area ≈ 1520 ft



TRAFFIC CONTROL SHEET. DWG

Section 6F.63 → Channelizing Devices

*Spacing should not exceed a distance in feet equal to 1.0 x Speed Limit (mph) when used for taper channelization

↓

AKA, spacing cannot exceed 65 ft

length needed for drums

↳ 760 + 1520 = 2300 ft

Spacing = 20 ft

$\frac{2300\ ft}{20\ ft} = 115$ drums

Section 6C.08 (TAPERS)

↓

13 → devices spaced approximately 20 ft apart

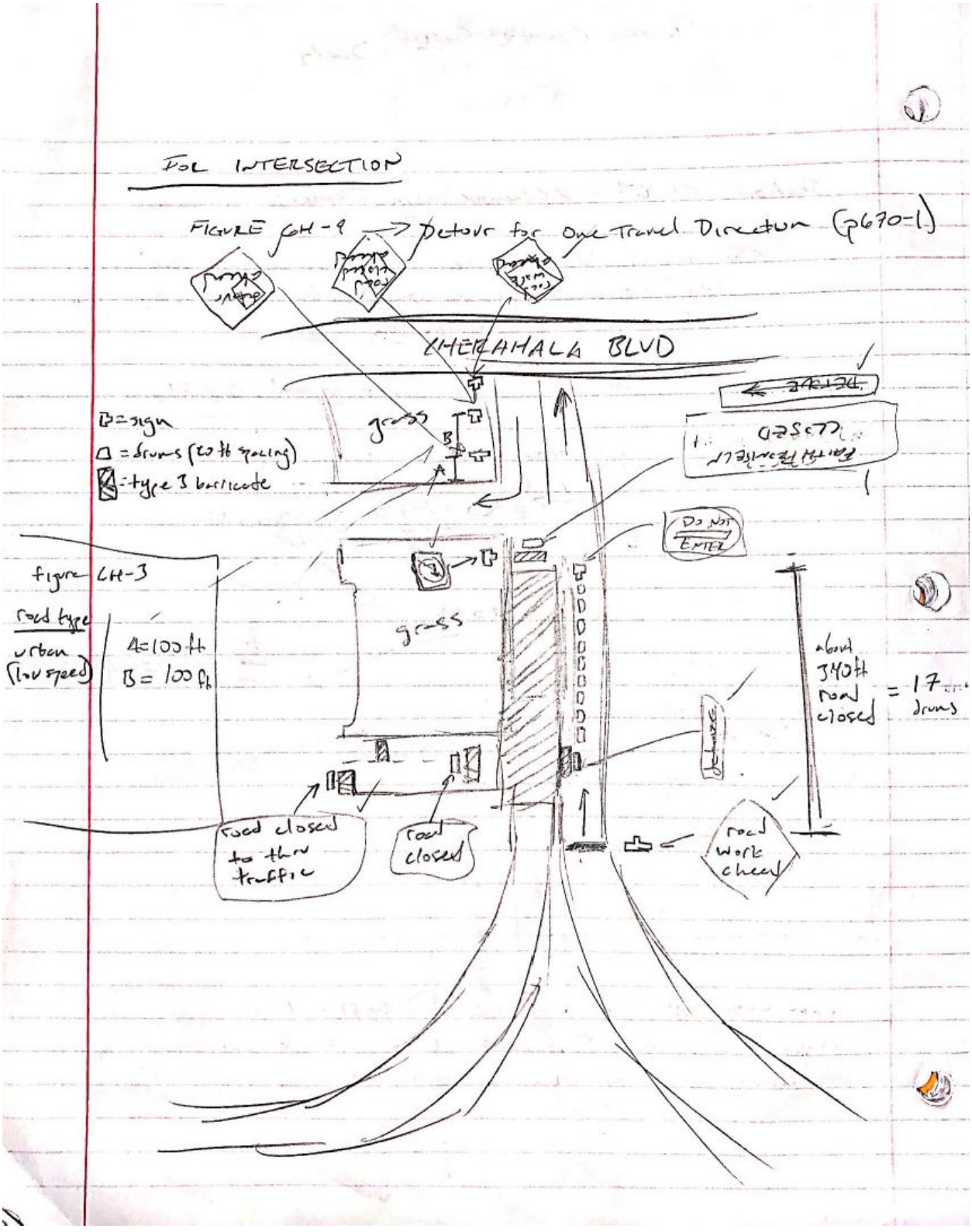
road work sign → A + B + C = 5140 ft ≈ 1 mi

right lane closed → A + B = 2500 ft = ½ mi

merge → A = 1000 ft

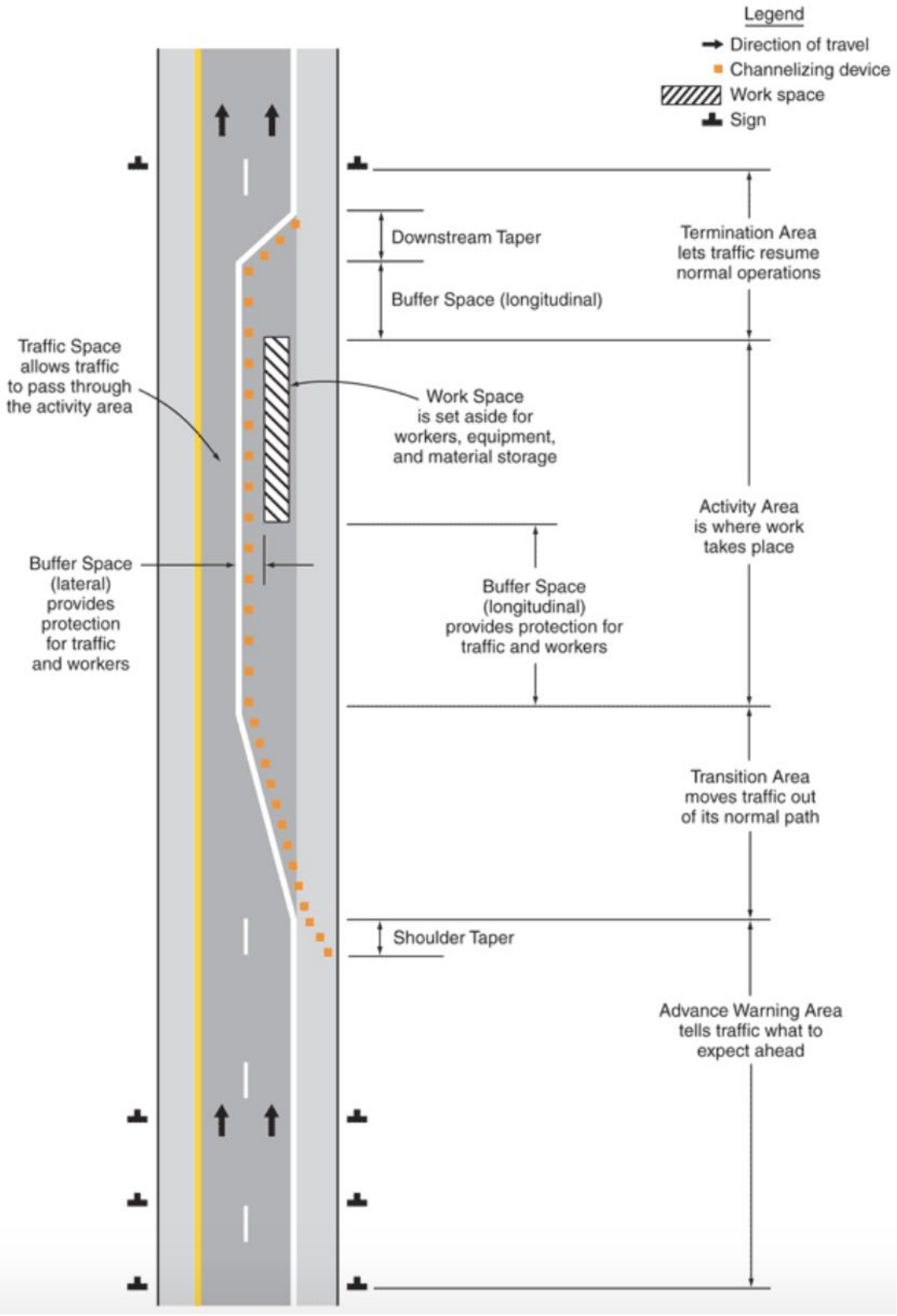
FOR INTERSECTION

FIGURE CH-9 → Detour for One Travel Direction (G670-1)



MUTCD: Figure 6C-1

Figure 6C-1. Component Parts of a Temporary Traffic Control Zone

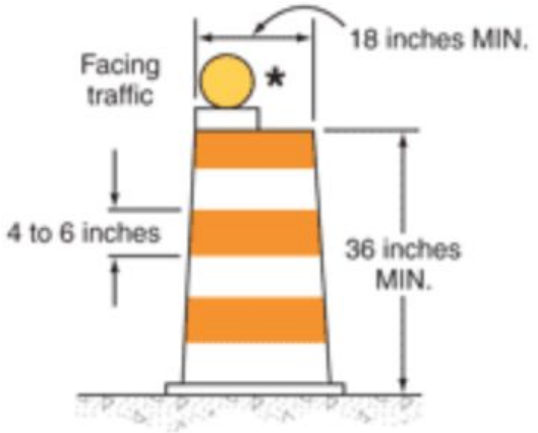


MUTCD: Table 6C-1

Table 6C-1. Recommended Advance Warning Sign Minimum Spacing

Road Type	Distance Between Signs**		
	A	B	C
Urban (low speed)*	100 feet	100 feet	100 feet
Urban (high speed)*	350 feet	350 feet	350 feet
Rural	500 feet	500 feet	500 feet
Expressway / Freeway	1,000 feet	1,500 feet	2,640 feet

MUTCD: Figure 6F-7: Channelizing Drum Dimensions



DRUM

MUTCD: Section 6F-63: Channelizing Devices

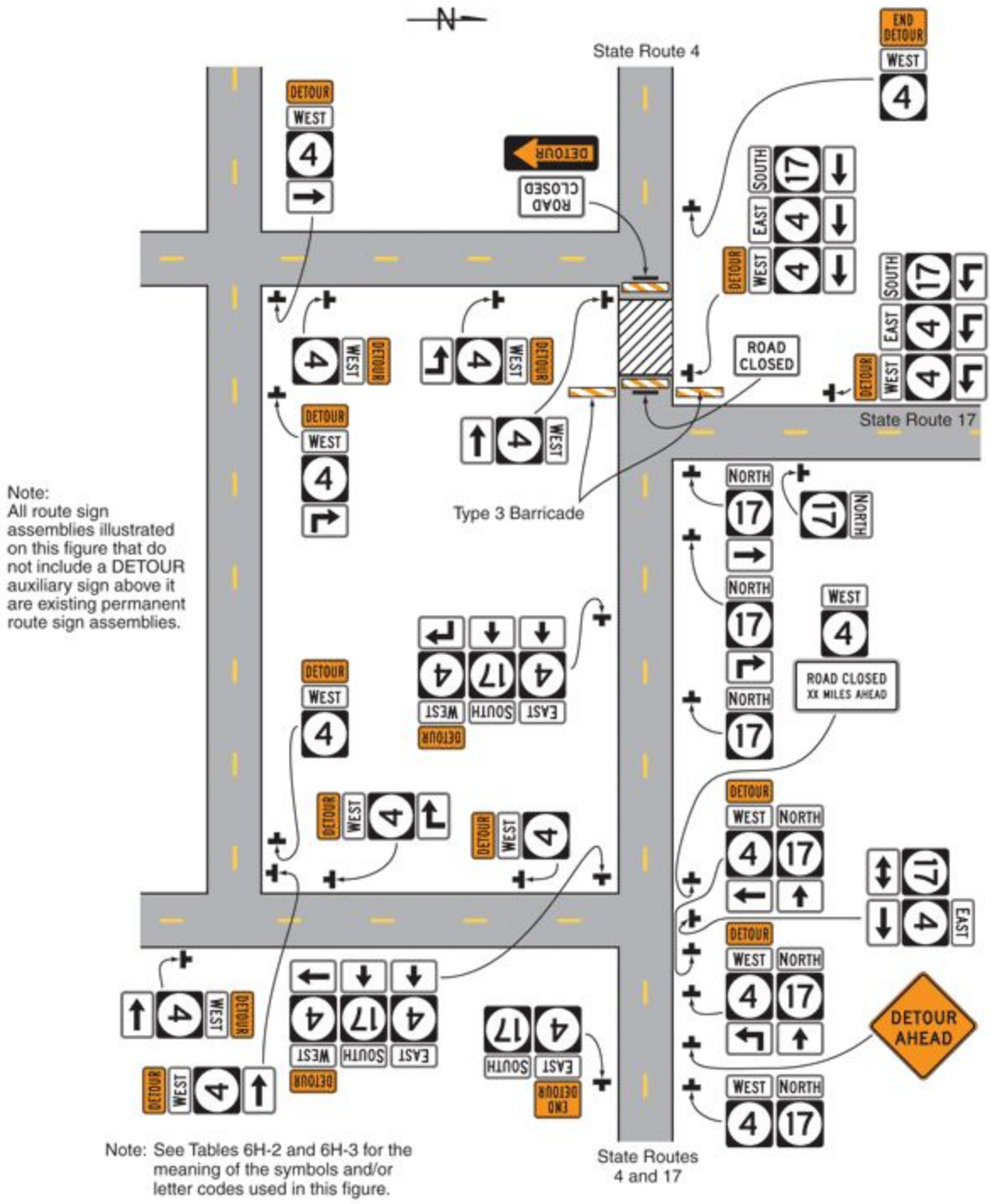
Support:

02 The function of channelizing devices is to warn road users of conditions created by work activities in or near the roadway and to guide road users. Channelizing devices include cones, tubular markers, vertical panels, drums, barricades, and longitudinal channelizing devices.

03 Channelizing devices provide for smooth and gradual vehicular traffic flow from one lane to another, onto a bypass or detour, or into a narrower traveled way. They are also used to channelize vehicular traffic away from the work space, pavement drop-offs, pedestrian or shared-use paths, or opposing directions of vehicular traffic.

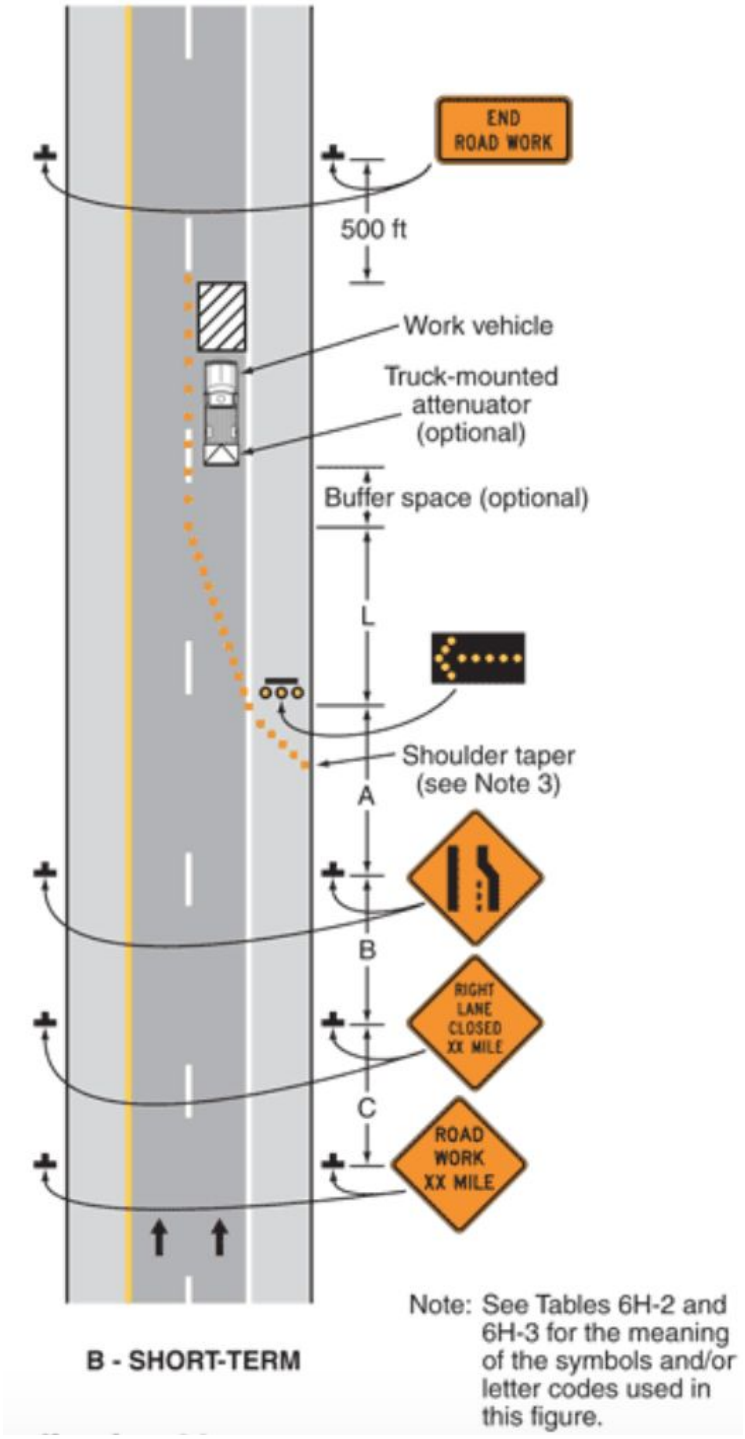
MUTCD: Figure 6H-09: Overlapping Routes with a Detour

Figure 6H-9. Overlapping Routes with a Detour (TA-9)



Typical Application 9

MUTCD: Figure 6H-33: Stationary Lane Closure on a Divided Highway



Description	Unit of Measure	Average Unit Price	Total Cost	Total quantity	Item Number
CONSTRUCTION STAKE, LINES AND GRADES	SF	$ 30,294.00	$ 30,294.00	1	105-01
CLEARING AND GRUBBING	LS	$ 52,643.79	$ 52,643.79	1	201-01
REMOVAL OF CURB AND GUTTER	LF	$ 10.00	$ 7,120.00	712	202-08.15
WATER	MG	$ 22.33	$ 22.33	1	203-06
REMOVAL & DISPOSAL EXISTING ASPHALT PVMT	SY	$ 33.92	$ 112,343.04	3312	202-03.01
MINERAL AGGREGATE (SIZE 57)	TON	$ 43.92	$ 30,499.80	694.44	303-10.01
MINERAL AGGREGATE, TY A BASE, GRADING D GRAVEL	TON	$ 31.05	$ 187,883.55	6051	303-01
ASPHALT CONC MX (PG64-22) (BPMB-HM) GR B-M2	TON	$ 125.61	$ 58,534.26	466	307-01.08
ASP. CONC. MIX (PG76-22) (BPMB-HM) GR A-S	TON	$ 88.60	$ 46,160.60	521	307-01.22
TREATED PERMEABLE BASE	TON	$ 23.63	$ 1,110.61	47	313-03
BITUMINOUS MATERIAL FOR PRIME COAT (PC)	TON	$ 514.68	$ 3,602.76	7	402-01
AGGREGATE FOR COVER MATERIAL	TON	$ 26.94	$ 511.86	19	402-02
BITUMINOUS MATERIAL FOR TACK COAT (TC)	TON	$ 618.68	$ 2,474.72	4	403-01
ACS MIX (PG64-22) GRADING D	TON	$ 155.70	$ 40,014.90	257	411-01.10
PORTLAND CEM CONCRETE PVMT (PLAIN) 10"	SY	$ 92.74	$ 24,019.66	259	501-01.03
TEMPORARY SILT FENCE (WITH BACKING)	LF	$ 5.63	$ 13,018.98	2312.43	209-08.02
GRANULAR BACKFILL (RETAINING WALLS)	TON	$ 50.00	$ 17,750.00	355	303-01.03
SAW CUTTING ASPHAULT PAVEMENT	LF	$ 2.35	$ 164.50	70	407-20.05
MACHINED RIP-RAP (CLASS A-1)	TON	$ 45.85	$ 9,903.60	216	709-05.06
ARROW BOARD (TYPE C)	EACH	$ 1,061.66	$ 1,061.66	1	712-08.03
FLEXIBLE DRUMS (CHANNELIZING)	EACH	$ 29.43	$ 3,884.76	132	712-04.01
WARNING LIGHTS TYPE A	EACH	$ 22.40	$ 156.80	7	712-05.01
SIGNS (CONSTRUCTION)	SF	$ 8.99	$ 224.75	25	712-06
GEOTEXTILE (TYPE III) (EROSION CONTROL)	SY	$ 5.42	$ 357.72	66	740-10.03
EROSION CONTROL BLANKET (TYPE I)	SY	$ 1.27	$ 18,440.40	14,520	805-12.01
TEMPORARY BARRICADES (TYPE III)	LF	$ 17.56	$ 70.24	4	712-07.03
SIGNS (STOP)	EACH	$ 68.75	$ 275.00	4	713-16.31
REMOVAL OF SIGNS, POSTS, AND FOOTINGS	LS	$ 1,534.44	$ 1,534.44	1	713-15
PAINTED PAVEMENT MARKINGS (4" LINE)	LM	$ 561.07	$ 561.07	1	716-05.01
PRINTED PAVEMENT MKG (8" BARRIER LINE)	LF	$ 0.36	$ 828.00	2300	716-05.02
ARROW PAINTING	EACH	$ 116.94	$ 233.88	2	716-05.09
STOP LINE	LF	$ 2.72	$ 65.28	24	717-05.05
ROCK CHECK DAM PER	EACH	$ 511.43	$ 12,785.75	25	209-08.07
CATCH BASIN, TYPE 12 0'-4'	EACH	$ 4,466.48	$ 8,932.96	2	611-12.01
CATCH BASIN, TYPE 12 4'-8'	EACH	$ 4,396.12	$ 21,980.60	5	611-12.02
CATCH BASIN, TYPE 42 0'-4'	EACH	$ 4,341.45	$ 8,682.90	2	611-42.01
CATCH BASIN PROTECTION (TYPE A)	EACH	$ 223.38	$ 1,563.66	7	209-40.30
CONSTRUCTION OF HAUL ROAD	LS	$ 60,000.00	$ 60,000.00	1	203-50
18' CONCRETE CULVERT (CLASS III)	LF	$ 69.24	$ 37,389.60	540	607-03.02
CLASS A CONCRETE	CY	$ 144.00	$ 22,032.00	153	604-01.01
SELECT SAND BACKFILL	TON	$ 19.00	$ 72,409.00	3811	303-01.04
ROAD & DRAINAGE EXCAVATION (UNCLIFIED)	C.Y.	$ 7.40	$ 176,068.20	23793	203-01
RETAINING WALL	S.F.	$ 137.66	$ 529,991.00	3850	604-07.02

Site Preparation	$	151,614.56
ESPC	$	41,720.70
Earthwork	$	266,227.20
Traffic Control	$	5,398.21
Roadway Developme	$	507,320.26
Retaining Wall	$	529,991.00
Drainage	$	113,367.47
Signage & Striping	$	1,963.23
TOTAL	$	1,617,602.64

Task	Week 1	Week 2	Week 3	Week 4	Week 5	Week 6	Week 7	Week 8	Week 9	Week 10	Week 11	Week 12	Week 13	Week 14	Week 15	Week 16	Week 17	Week 18
Phase 1																		
Clearing and Grubbing																		
Cut and Fill																		
Grading and Compaction																		
Catch Basin Removal																		
Drainage Removal																		
Catch Basin Instalation																		
Area Drain Instalation																		
Drainage Instalation																		
Lay subbase																		
Lay the pavement																		
Headwall Removal																		
Tree Removal																		
Existing Faith Promise Lane and Driveways Removal																		
Phase 2																		
Clearing and Grubbing																		
Cut and Fill																		
Construct Retaining Wall																		
Grading and Compaction																		
Lay Subbase																		
Lay the Pavement																		
Headwall Removal																		
Tree Removal																		
Phase 3																		
Closing Horseshoe Bend																		
Sidewalk Construction																		
Signage																		
Striping																		
Mulching																		

Phase 1	1 Crew Output per Day	Units	Project Measurement	Total Days	Crew
Clearing and Grubbing	0.47	Acres	0.95	2.02	B30
Cut and Fill	5280	LCY	9888.03	1.87	B14J
Grading and Compaction	400	SY	3847	9.62	B18
Catch Basin Removal	4	Ea.	3	0.75	B6
Drainage Removal	175	Lf.	314	1.79	B6
Catch Basin Instalation	2	Ea.	7	3.50	B24
Area Drain Instalation	4	Ea.	2	0.50	Q1
Drainage Instalation	190	Lf.	540	2.84	B14
Lay subbase	25875	SF	34620	1.34	B36
Lay the pavement	1800	SY	3847	2.14	B26
Headwall Removal	6	Ea.	3	0.50	B6
Tree Removal	15	Ea.	15	1.00	1 Clab
Existing Faith Promise Lane and Driveways Removal	640	SY	3312	5.18	B38
Phase 2	1 Crew Output per Day	Units	Project Measurement	Total Days	
Clearing and Grubbing	0.47	Acres	1.66	3.53	B30
Cut and Fill	5280	LCY	18066.08	3.42	B14J
Construct Retaining Wall	7.5	LF	200	26.67	C17C
Grading and Compaction	400	SY	2324	5.81	B18
Catch Basin Instalation	2	Ea.	0	0.00	B24
Area Drain Instalation	4	Ea.	0	0.00	Q1
Drainage Instalation	190	Lf.	0	0.00	B14
Lay Subbase	25875	SF	20913	0.81	B36
Lay the Pavement	1800	SY	2324	1.29	B26
Headwall Removal	6	Ea.	1	0.17	B6
Tree Removal	15	Ea.	7	0.47	1 Clab
Phase 3	1 Crew Output per Day	Units	Project Measurement	Total Days	
Closing Horseshoe Bend	640	Acres	1257	1.96	B38
Sidewalk Construction	720	SY	666	0.93	B37
Signage	70	Ea.	4	0.06	B80
Striping	11000	LF	4459	0.41	B78
Mulching	475	SY	6461.4	13.60	1 Clab

Equipment
Tractor
Bulldozer
Grader
Shovel
Roller
Dumper
Crusher
Tractor
Hot Mix Asphalt Plant
Asphalt Mixer
Bitumen Boiler
Bitumen Sprayer
Aggregate Spreader
Roller

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
104-03.71	ADDITIONAL WORK	S.F.	2	$1.50	$11,775.00	7850.00
			STATE	$1.50	$11,775.00	7850.00
105-01	CONSTRUCTION STAKES, LINES AND GRADES	LS	1	$30,294.72	$545,305.01	18.00
			2	$28,181.53	$648,175.23	23.00
			3	$17,663.50	$141,308.00	8.00
			4	$154,063.33	$462,190.00	3.00
			STATE	$34,557.27	$1,796,978.24	52.00
109-04.20	FORCE ACCOUNT	DOLL	1	$1.00	$1,000,000.00	1000000.00
			2	$1.00	$95,000.00	95000.00
			3	$1.00	$5,000,000.00	5000000.00
			STATE	$1.00	$6,095,000.00	6095000.00
109-10.01	TRAINEE	HOUR	1	$0.80	$6,656.00	8320.00
			2	$0.80	$5,392.00	6740.00
			4	$0.80	$5,744.00	7180.00
			STATE	$0.80	$17,792.00	22240.00
201-01	CLEARING AND GRUBBING	LS	1	$52,643.79	$1,579,313.60	30.00
			2	$383,007.34	$4,213,080.69	11.00
			3	$75,400.00	$377,000.00	5.00
			4	$801,749.33	$2,405,248.00	3.00
			STATE	$174,992.70	$8,574,642.29	49.00
201-05.31	VEGETATION REMOVAL	LS	1	$5,000.00	$5,000.00	1.00
			2	$6,031.68	$24,126.72	4.00
			3	$12,000.00	$12,000.00	1.00
			STATE	$6,854.45	$41,126.72	6.00
201-07.01	REMOVAL AND DISPOSAL OF BRUSH & TREES	LS	1	$10,000.00	$20,000.00	2.00
			2	$6,000.00	$6,000.00	1.00
			3	$58,392.17	$58,392.17	1.00
			STATE	$21,098.04	$84,392.17	4.00
202-01	REMOVAL OF STRUCTURES AND OBSTRUCTIONS	LS	1	$60,700.00	$60,700.00	1.00
			2	$25,770.70	$77,312.10	3.00
			STATE	$34,503.03	$138,012.10	4.00
202-01.02	REMOVAL OF ASBESTOS	LS	1	$31,166.67	$93,500.00	3.00
			2	$20,200.00	$20,200.00	1.00
			3	$10,000.00	$10,000.00	1.00
			4	$45,000.00	$45,000.00	1.00
			STATE	$28,116.67	$168,700.00	6.00
202-01.03	REMOVAL OF TRASH AND DEBRIS	LS	2	$36,037.20	$72,074.39	2.00
			STATE	$36,037.20	$72,074.39	2.00
202-01.13	REMOVAL OF PIPE	L.F.	1	$13.08	$5,834.00	446.00
			STATE	$13.08	$5,834.00	446.00
202-01.50	REMOVAL OF STRUCTURES AND OBSTRUCTIONS	EACH	2	$2,138.40	$6,415.20	3.00
			STATE	$2,138.40	$6,415.20	3.00
202-01.56	REMOVAL OF STRUCTURES & OBSTRUCTIONS	LS	1	$4,500.00	$4,500.00	1.00
			STATE	$4,500.00	$4,500.00	1.00
202-02.01	REMOVAL OF PIPE	L.F.	2	$12.83	$1,513.86	118.00
			3	$57.94	$7,590.00	131.00
			4	$23.34	$3,758.00	161.00
			STATE	$31.37	$12,861.86	410.00
202-02.02	REMOVAL OF PIPE	L.F.	2	$14.07	$1,970.08	140.00
			3	$100.00	$3,700.00	37.00
			4	$50.00	$2,100.00	42.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
			STATE	$38.00	$202,350.00	5325.00
203-03.10	SELECT GRANULAR MATERIAL	TON	4	$6.25	$1,794,831.25	287173.00
			STATE	$6.25	$1,794,831.25	287173.00
203-04	PLACING AND SPREADING TOPSOIL	C.Y.	1	$14.57	$102,726.50	7049.00
			2	$3.64	$209,719.15	57621.00
			3	$8.49	$82,022.98	9659.00
			4	$0.66	$11,785.00	17980.00
			STATE	$4.40	$406,253.63	92309.00
203-05	UNDERCUTTING	C.Y.	1	$31.66	$5,636.00	178.00
			4	$1.00	$162,833.00	162833.00
			STATE	$1.03	$168,469.00	163011.00
203-06	WATER	M.G.	1	$22.33	$16,214.32	726.00
			2	$7.32	$23,444.22	3202.00
			3	$9.47	$2,755.00	291.00
			4	$14.46	$123,141.90	8517.50
			STATE	$13.00	$165,555.44	12736.50
203-07	FURNISHING & SPREADING TOPSOIL	C.Y.	1	$31.47	$138,316.00	4395.00
			2	$30.49	$42,436.70	1392.00
			3	$52.50	$525.00	10.00
			STATE	$31.27	$181,277.70	5797.00
203-08	CHANNEL EXCAVATION (UNCLASSIFIED)	C.Y.	2	$24.03	$14,752.86	614.00
			STATE	$24.03	$14,752.86	614.00
203-10	EMBANKMENT (COMPACTED IN PLACE)	C.Y.	4	$35.00	$22,610.00	646.00
			STATE	$35.00	$22,610.00	646.00
203-10.15	WASTE MATERIAL	C.Y.	1	$140.00	$280,000.00	2000.00
			2	$10.75	$78,141.75	7269.00
			3	$9.69	$68,250.60	7047.00
			STATE	$26.13	$426,392.35	16316.00
203-11	SCALING AND TRIMMING	S.Y.	1	$110.60	$215,552.68	1949.00
			2	$25.00	$32,625.00	1305.00
			STATE	$76.27	$248,177.68	3254.00
203-15.03	COMPACTED CLAY	C.Y.	1	$165.00	$4,125.00	25.00
			2	$25.00	$4,500.00	180.00
			STATE	$42.07	$8,625.00	205.00
203-20.01	CHANNEL SUBSTRATE	C.Y.	2	$85.00	$4,080.00	48.00
			STATE	$85.00	$4,080.00	48.00
203-30.01	ROADWAY APPROACHES	LS	3	$5,000.00	$5,000.00	1.00
			STATE	$5,000.00	$5,000.00	1.00
203-40.01	ROCK ANCHORS	EACH	1	$17,250.00	$258,750.00	15.00
			STATE	$17,250.00	$258,750.00	15.00
203-40.03	ROCK ANCHORS	L.F.	1	$195.00	$118,950.00	610.00
			STATE	$195.00	$118,950.00	610.00
203-40.05	DRILLING	L.F.	1	$0.01	$21.70	2170.00
			STATE	$0.01	$21.70	2170.00
203-40.13	TIE-BACK ANCHORS (4-STRAND UNCLASSIFIED)	L.F.	1	$240.00	$524,880.00	2187.00
			STATE	$240.00	$524,880.00	2187.00
203-40.14	TIE-BACK ANCHORS (5-STRAND UNCLASSIFIED)	L.F.	1	$0.01	$21.50	2150.00
			STATE	$0.01	$21.50	2150.00
203-50	CONSTRUCTION OF HAUL ROAD	LS	1	$60,000.00	$60,000.00	1.00
			4	$1,000,000.00	$1,000,000.00	1.00
			STATE	$530,000.00	$1,060,000.00	2.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
204-01	CULVERT EXCAVATION (UNCLASSIFIED)	C.Y.	2	$23.54	$8,474.40	360.00
			3	$30.00	$13,110.00	437.00
			STATE	$27.08	$21,584.40	797.00
204-02.01	DRY EXCAVATION (BRIDGES)	C.Y.	1	$15.37	$21,931.00	1427.00
			2	$25.00	$21,575.00	863.00
			3	$14.00	$2,772.00	198.00
			4	$9.42	$35,593.00	3777.00
			STATE	$13.07	$81,871.00	6265.00
204-03.01	WET EXCAVATION	C.Y.	2	$50.00	$7,250.00	145.00
			STATE	$50.00	$7,250.00	145.00
204-05.01	CORE DRILLING AND SAMPLING	L.F.	1	$83.00	$93,541.00	1127.00
			3	$85.00	$4,930.00	58.00
			STATE	$83.10	$98,471.00	1185.00
204-06.01	FLOWABLE FILL (GENERAL)	C.Y.	3	$442.00	$1,326.00	3.00
			4	$350.00	$19,250.00	55.00
			STATE	$354.76	$20,576.00	58.00
204-08	FOUNDATION FILL MATERIAL	C.Y.	1	$57.00	$5,415.00	95.00
			2	$91.88	$45,295.01	493.00
			3	$400.00	$18,800.00	47.00
			4	$43.00	$34,615.00	805.00
			STATE	$72.31	$104,125.01	1440.00
204-08.01	BACKFILL MATERIAL (FLOWABLE FILL)	C.Y.	1	$364.00	$18,200.00	50.00
			2	$273.67	$30,651.05	112.00
			4	$229.02	$65,270.00	285.00
			STATE	$255.30	$114,121.05	447.00
204-10.01	FOUNDATION PREPARATION	LS	2	$50,000.00	$50,000.00	1.00
			STATE	$50,000.00	$50,000.00	1.00
204-10.02	FOUNDATION PREPARATION	LS	2	$50,000.00	$50,000.00	1.00
			STATE	$50,000.00	$50,000.00	1.00
204-10.07	FOUNDATION PREPARATION	LS	2	$50,000.00	$50,000.00	1.00
			STATE	$50,000.00	$50,000.00	1.00
204-11	BRIDGE EXCAVATION (UNCLASSIFIED)	C.Y.	4	$30.00	$840.00	28.00
			STATE	$30.00	$840.00	28.00
204-14	CORE DRILLING FOR PILES(ROCK)	L.F.	3	$100.00	$3,600.00	36.00
			STATE	$100.00	$3,600.00	36.00
204-15	CORE DRILLING FOR PILES(SOIL)	L.F.	3	$90.00	$6,660.00	74.00
			STATE	$90.00	$6,660.00	74.00
208-01.02	SHOULDER PREPARATION	L.M.	3	$1,933.57	$1,239.42	0.64
			4	$4,658.14	$8,012.00	1.72
			STATE	$3,918.43	$9,251.42	2.36
208-01.05	BROOMING & DEGRASSING SHOULDERS	L.M.	1	$2,412.24	$2,412.24	1.00
			3	$975.14	$543,736.44	557.60
			4	$1,137.41	$302,243.18	265.73
			STATE	$1,029.19	$848,391.86	824.33
209-02.03	8" TEMPORARY SLOPE DRAIN	L.F.	2	$8.99	$80,714.97	8979.00
			STATE	$8.99	$80,714.97	8979.00
209-02.04	10" TEMPORARY SLOPE DRAIN	L.F.	1	$21.00	$1,050.00	50.00
			STATE	$21.00	$1,050.00	50.00
209-02.05	12" TEMPORARY SLOPE DRAIN	L.F.	1	$18.72	$27,800.00	1485.00
			2	$12.00	$9,720.00	810.00
			STATE	$16.35	$37,520.00	2295.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
209-02.06	15" TEMPORARY SLOPE DRAIN	L.F.	2	$15.00	$6,825.00	455.00
			STATE	$15.00	$6,825.00	455.00
209-02.07	18" TEMPORARY SLOPE DRAIN	L.F.	1	$34.60	$31,936.00	923.00
			2	$26.71	$24,837.45	930.00
			3	$10.00	$3,290.00	329.00
			4	$20.00	$3,420.00	171.00
			STATE	$26.98	$63,483.45	2353.00
209-03.20	FILTER SOCK (8 INCH)	L.F.	2	$30.16	$6,032.00	200.00
			STATE	$30.16	$6,032.00	200.00
209-03.21	FILTER SOCK (12 INCH)	L.F.	1	$4.72	$99,423.85	21083.00
			3	$3.00	$11,250.00	3750.00
			STATE	$4.46	$110,673.85	24833.00
209-03.22	FILTER SOCK (18 INCH)	L.F.	1	$35.00	$9,800.00	280.00
			2	$13.30	$7,182.00	540.00
			STATE	$20.71	$16,982.00	820.00
209-03.23	FILTER SOCK (24 INCH)	L.F.	1	$18.50	$6,882.00	372.00
			2	$12.27	$9,938.70	810.00
			4	$15.00	$4,905.00	327.00
			STATE	$14.40	$21,725.70	1509.00
209-03.36	STREAM MITIGATION-STEP POOL	EACH	2	$750.00	$15,750.00	21.00
			STATE	$750.00	$15,750.00	21.00
209-03.37	STREAM MITIGATION CROSS VANE STRUCTURE	EACH	2	$700.00	$2,100.00	3.00
			STATE	$700.00	$2,100.00	3.00
209-03.59	STRM MITIG - BRUSH MATTRESS	S.Y.	2	$50.00	$2,050.00	41.00
			STATE	$50.00	$2,050.00	41.00
209-03.67	Strm Mtigation - Wood Toe W/Reinfo Earth	L.F.	2	$50.00	$12,300.00	246.00
			STATE	$50.00	$12,300.00	246.00
209-05	SEDIMENT REMOVAL	C.Y.	1	$24.39	$56,171.20	2303.00
			2	$4.72	$53,470.62	11318.00
			3	$5.77	$13,585.00	2355.00
			4	$3.45	$30,444.00	8831.00
			STATE	$6.19	$153,670.82	24807.00
209-06.02	12" DIA COIR LOG (DESCRIPTION)	L.F.	4	$12.00	$2,304.00	192.00
			STATE	$12.00	$2,304.00	192.00
209-08.02	TEMPORARY SILT FENCE (WITH BACKING)	L.F.	1	$5.63	$53,592.97	9525.00
			2	$3.81	$268,609.96	70430.00
			3	$4.41	$59,134.00	13413.00
			4	$3.79	$364,534.80	96171.00
			STATE	$3.94	$745,871.73	189539.00
209-08.03	TEMPORARY SILT FENCE (WITHOUT BACKING)	L.F.	1	$2.49	$22,871.70	9193.00
			2	$1.60	$67,851.20	42407.00
			3	$1.85	$11,148.10	6026.00
			4	$1.53	$117,414.00	76776.00
			STATE	$1.63	$219,285.00	134402.00
209-08.04	TEMPORARY ENHANCED SILT FENCE	L.F.	2	$11.58	$8,337.60	720.00
			STATE	$11.58	$8,337.60	720.00
209-08.06	ENHANCED SILT FENCE CHECK (TRAPEZOIDAL)	EACH	1	$727.00	$727.00	1.00
			STATE	$727.00	$727.00	1.00
209-08.07	ROCK CHECK DAM PER	EACH	1	$511.43	$41,425.70	81.00
			2	$217.65	$246,811.51	1134.00
			3	$320.76	$14,755.00	46.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
			3	$250.00	$750.00	3.00
			4	$450.00	$11,250.00	25.00
			STATE	$473.74	$19,423.29	41.00
209-40.42	CATCH BASIN FILTER ASSEMBLY(TYPE 2)	EACH	1	$670.00	$8,040.00	12.00
			4	$550.00	$4,950.00	9.00
			STATE	$618.57	$12,990.00	21.00
209-40.43	CATCH BASIN FILTER ASSEMBLY(TYPE 3)	EACH	1	$533.00	$2,132.00	4.00
			4	$550.00	$2,750.00	5.00
			STATE	$542.44	$4,882.00	9.00
209-40.44	CATCH BASIN FILTER ASSEMBLY(TYPE 4)	EACH	1	$537.33	$3,224.00	6.00
			4	$550.00	$1,100.00	2.00
			STATE	$540.50	$4,324.00	8.00
209-40.45	CATCH BASIN FILTER ASSEMBLY(TYPE 5)	EACH	1	$651.00	$3,906.00	6.00
			STATE	$651.00	$3,906.00	6.00
209-40.46	CATCH BASIN FILTER ASSEMBLY(TYPE 6)	EACH	1	$630.00	$7,560.00	12.00
			STATE	$630.00	$7,560.00	12.00
209-40.47	CATCH BASIN FILTER ASSEMBLY(TYPE 7)	EACH	1	$751.50	$4,509.00	6.00
			STATE	$751.50	$4,509.00	6.00
209-40.48	CATCH BASIN FILTER ASSEMBLY(TYPE 8)	EACH	1	$843.00	$843.00	1.00
			2	$1,626.38	$13,011.04	8.00
			STATE	$1,539.34	$13,854.04	9.00
209-65.03	TEMPORARY DIVERSION CHANNEL	L.F.	1	$59.50	$11,602.50	195.00
			2	$52.02	$66,380.70	1276.00
			3	$9.25	$6,737.50	728.00
			4	$37.00	$94,720.00	2560.00
			STATE	$37.71	$179,440.70	4759.00
209-65.04	TEMPORARY IN STREAM DIVERSION	L.F.	2	$49.74	$17,856.66	359.00
			4	$20.00	$4,200.00	210.00
			STATE	$38.76	$22,056.66	569.00
209-70.01	VACUUM TRUCK/SWEEPER FOR ROADWAY DEBRIS	HOUR	2	$203.80	$88,652.65	435.00
			STATE	$203.80	$88,652.65	435.00
303-01	MINERAL AGGREGATE, TY A BASE, GRADING D	TON	1	$31.04	$2,120,552.08	68327.00
			2	$21.32	$7,047,598.54	330639.00
			3	$20.53	$2,045,772.48	99652.00
			4	$17.91	$4,190,101.00	233962.00
			STATE	$21.03	$15,404,024.10	732580.00
303-01.01	GRANULAR BACKFILL (ROADWAY)	TON	1	$32.10	$39,163.51	1220.00
			2	$30.67	$404,759.40	13196.00
			3	$50.00	$5,900.00	118.00
			4	$25.00	$346,925.00	13877.00
			STATE	$28.04	$796,747.91	28411.00
303-01.02	GRANULAR BACKFILL (BRIDGES)	TON	1	$38.09	$291,935.00	7665.00
			2	$47.77	$25,315.68	530.00
			3	$74.00	$3,700.00	50.00
			4	$36.98	$13,425.00	363.00
			STATE	$38.84	$334,375.68	8608.00
303-01.03	GRANULAR BACKFILL (RETAINING WALLS)	TON	1	$50.00	$1,750.00	35.00
			STATE	$50.00	$1,750.00	35.00
303-01.04	SELECT SAND BACKFILL	TON	2	$19.00	$99,883.00	5257.00
			STATE	$19.00	$99,883.00	5257.00
303-01.08	MINERAL AGGR,TY A BASE,GRADING D GRAVEL	TON	4	$80.00	$720.00	9.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
			STATE	$80.00	$720.00	9.00
303-01.09	MINERAL AGGR,TY A BS,GRADING D LIMESTONE	TON	4	$48.95	$9,564.83	195.40
			STATE	$48.95	$9,564.83	195.40
303-02	MINERAL AGGR,TY B BASE, GR	TON	4	$33.36	$2,316,715.73	69444.00
			STATE	$33.36	$2,316,715.73	69444.00
303-10.01	MINERAL AGGREGATE (SIZE 57)	TON	1	$43.92	$417,281.57	9502.00
			2	$36.16	$42,893.79	1186.20
			3	$32.71	$31,400.00	960.00
			4	$39.03	$11,670.00	299.00
			STATE	$42.12	$503,245.36	11947.20
303-10.02	MINERAL AGGREGATE (SIZE 2)	TON	1	$45.00	$11,790.00	262.00
			STATE	$45.00	$11,790.00	262.00
303-10.05	MINERAL AGGREGATE	TON	1	$56.00	$392.00	7.00
			STATE	$56.00	$392.00	7.00
307-01.01	ASP. CONC. MIX(PG64-22) (BPMB-HM) GR. A	TON	1	$102.11	$251,285.24	2461.00
			2	$67.94	$2,686,774.67	39545.00
			3	$78.11	$455,676.24	5834.00
			4	$128.44	$2,073,333.85	16143.00
			STATE	$85.45	$5,467,070.00	63983.00
307-01.02	ASPHALT CEM. (PG64-22)(BPMB-HM) GR. A-S	TON	2	$898.67	$35,048.13	39.00
			STATE	$898.67	$35,048.13	39.00
307-01.03	AGGREGATE (BPMB-HM) GRADING A-S	TON	2	$71.29	$82,767.69	1161.00
			STATE	$71.29	$82,767.69	1161.00
307-01.07	ASPHALT CONC MIX(PG64-22)(BPMB-HM)GR B-M	TON	3	$86.83	$161,935.75	1865.00
			STATE	$86.83	$161,935.75	1865.00
307-01.08	ASPHALT CONC MX(PG64-22)(BPMB-HM)GR B-M2	TON	1	$125.61	$462,119.64	3679.00
			2	$84.20	$2,499,042.54	29679.00
			3	$79.75	$1,293,132.12	16215.00
			4	$107.26	$1,613,634.65	15044.00
			STATE	$90.81	$5,867,928.95	64617.00
307-01.09	ASPHALT CONC MIX (PG64-22)(BPMB-HM)GR C	TON	1	$159.00	$9,381.00	59.00
			STATE	$159.00	$9,381.00	59.00
307-01.10	ASPHALT CONC MIX(PG64-22)(BPMB-HM)GR C-W	TON	2	$94.44	$881,231.39	9331.00
			4	$101.35	$2,796,374.85	27591.00
			STATE	$99.60	$3,677,606.24	36922.00
307-01.15	ASC MIX (PG64-22) (BPMLC-HM) GR. CS	TON	1	$115.77	$1,528,814.27	13206.00
			2	$117.39	$717,961.80	6116.00
			3	$90.15	$829,335.00	9200.00
			STATE	$107.85	$3,076,111.07	28522.00
307-01.20	ASP. CONC. MIX(PG64-22) (BPMB-HM) GR A-S	TON	2	$68.00	$1,260,448.00	18536.00
			3	$76.00	$11,172.00	147.00
			4	$99.80	$59,979.80	601.00
			STATE	$69.05	$1,331,599.80	19284.00
307-01.21	ASP. CONC. MIX(PG70-22) (BPMB-HM) GR A-S	TON	1	$104.43	$293,854.70	2814.00
			2	$99.80	$56,586.60	567.00
			3	$77.98	$99,034.60	1270.00
			4	$102.48	$89,870.75	877.00
			STATE	$97.57	$539,346.65	5528.00
307-01.22	ASP. CONC. MIX(PG76-22) (BPMB-HM) GR A-S	TON	1	$88.60	$111,990.40	1264.00
			4	$80.10	$1,786,710.60	22306.00
			STATE	$80.56	$1,898,701.00	23570.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
307-02.01	ASPHALT CONC. MIX (PG70-22)(BPMB-HM)GR A	TON	1	$101.64	$497,614.84	4896.00
			2	$99.80	$174,550.20	1749.00
			3	$85.05	$193,924.12	2280.00
			4	$104.70	$419,209.25	4004.00
			STATE	$99.41	$1,285,298.41	12929.00
307-02.07	ASPHALT CONC MIX(PG70-22)(BPMB-HM)GR B-M	TON	2	$92.80	$123,424.00	1330.00
			STATE	$92.80	$123,424.00	1330.00
307-02.08	ASPHALT CONC MX(PG70-22)(BPMB-HM)GR B-M2	TON	1	$117.79	$374,444.90	3179.00
			2	$121.14	$238,396.48	1968.00
			3	$83.88	$662,029.88	7893.00
			4	$115.95	$169,984.50	1466.00
			STATE	$99.60	$1,444,855.76	14506.00
307-03.01	ASPHALT CONC MIX (PG76-22)(BPMB-HM) GR A	TON	1	$78.72	$183,662.75	2333.00
			3	$83.65	$34,129.20	408.00
			4	$85.90	$5,189,820.30	60417.00
			STATE	$85.62	$5,407,612.25	63158.00
307-03.02	PERF. GRADE ASPH. CEMENT(PG76-22)GR AS	TON	1	$995.00	$5,970.00	6.00
			STATE	$995.00	$5,970.00	6.00
307-03.03	AGGREGATE(BPMB-HM) GRADING AS	TON	1	$124.00	$20,088.00	162.00
			STATE	$124.00	$20,088.00	162.00
307-03.06	ASPHALT CONC MIX (PG76-22)(BPMB-HM) GR B	TON	1	$129.00	$30,960.00	240.00
			STATE	$129.00	$30,960.00	240.00
307-03.08	ASPHALT CONC MX(PG76-22)(BPMB-HM)GR B-M2	TON	1	$89.41	$143,415.60	1604.00
			2	$304.78	$54,250.84	178.00
			3	$87.57	$243,890.90	2785.00
			4	$89.30	$1,731,794.90	19393.00
			STATE	$90.71	$2,173,352.24	23960.00
307-03.10	ASPHALT CONC MIX (PG76-22)(BPMB-HM)GR CS	TON	3	$124.00	$53,444.00	431.00
			STATE	$124.00	$53,444.00	431.00
307-04.08	ASPHALT CONC MX(PG82-22)(BPMB-HM)GR B-M2	TON	3	$107.41	$28,034.01	261.00
			STATE	$107.41	$28,034.01	261.00
307-50.12	/*SPH CONC. MIX (PG70-22) GRD A (PLACED)	TON	2	$87.50	$177,537.50	2029.00
			STATE	$87.50	$177,537.50	2029.00
309-01.01	MINERAL AGGREGATE (A-CBC)	TON	4	$15.00	$927,240.00	61816.00
			STATE	$15.00	$927,240.00	61816.00
309-01.02	PORTLAND CEMENT (A-CBC)	TON	4	$245.00	$584,815.00	2387.00
			STATE	$245.00	$584,815.00	2387.00
309-02	BITUMINOUS MATERIAL (A-CBC)	TON	4	$1,200.00	$205,200.00	171.00
			STATE	$1,200.00	$205,200.00	171.00
311-03.04	HOT IN PLACE RECYCLING OF ASPHALT PAVEME	S.Y.	4	$4.13	$3,060,865.75	740687.00
			STATE	$4.13	$3,060,865.75	740687.00
311-03.10	Asphalt Rejuvenating Agent	GAL.	4	$3.78	$1,539,166.45	407380.00
			STATE	$3.78	$1,539,166.45	407380.00
313-03	TREATED PERMEABLE BASE	S.Y.	1	$23.63	$304,748.00	12898.00
			3	$29.90	$90,960.18	3042.00
			4	$18.00	$1,155,456.00	64192.00
			STATE	$19.36	$1,551,164.18	80132.00
402-01	BITUMINOUS MATERIAL FOR PRIME COAT (PC)	TON	1	$514.68	$51,334.00	99.74
			2	$553.98	$284,745.24	514.00
			3	$855.61	$59,122.50	69.10
			4	$485.80	$263,790.00	543.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
			STATE	$537.58	$658,991.74	1225.84
402-02	AGGREGATE FOR COVER MATERIAL (PC)	TON	1	$26.94	$5,658.00	210.00
			2	$29.58	$55,136.39	1864.00
			3	$58.68	$14,000.00	238.60
			4	$6.38	$13,659.00	2140.00
			STATE	$19.87	$88,453.39	4452.60
403-01	BITUMINOUS MATERIAL FOR TACK COAT (TC)	TON	1	$618.68	$1,323,644.69	2139.48
			2	$566.45	$784,420.71	1384.80
			3	$656.63	$1,660,548.10	2528.90
			4	$692.67	$940,639.20	1357.99
			STATE	$635.43	$4,709,252.70	7411.17
403-01.01	BITUMINOUS MATERIAL FOR TACK COAT (MICRO	TON	2	$615.45	$107,088.86	174.00
			STATE	$615.45	$107,088.86	174.00
403-01.10	High Performance Fog Seals	S.Y.	2	$0.39	$291,433.60	742167.00
			STATE	$0.39	$291,433.60	742167.00
403-02	ASPHALT CEMENT FOR TACK COAT (TC)	TON	1	$1,350.00	$14,850.00	11.00
			2	$752.63	$301.05	0.40
			3	$300.00	$300.00	1.00
			STATE	$1,246.05	$15,451.05	12.40
403-02.01	TRACKLESS TACK COAT	TON	2	$575.55	$130,075.20	226.00
			4	$944.38	$215,319.00	228.00
			STATE	$760.78	$345,394.20	454.00
403-05	BIT MATERIAL(TC/FOG SEAL)(UNDILUTED)	TON	1	$2,500.00	$7,500.00	3.00
			STATE	$2,500.00	$7,500.00	3.00
403-05.01	BITUMINOUS MATERIAL (FOG SEAL) SHOULDER	TON	2	$1,228.51	$6,756.81	5.50
			3	$889.60	$18,681.60	21.00
			STATE	$959.94	$25,438.41	26.50
404-01.01	BITUMINOUS MATERIAL (DBST)	TON	2	$1,272.04	$10,837.78	8.52
			STATE	$1,272.04	$10,837.78	8.52
404-01.02	MINERAL AGGREGATE (DBST)	TON	2	$201.41	$13,615.32	67.60
			STATE	$201.41	$13,615.32	67.60
407-02.11	REMOVAL & DISPOSAL EXISTING ASPHALT PVMT	C.Y.	3	$25.00	$5,625.00	225.00
			STATE	$25.00	$5,625.00	225.00
407-02.13	REMOVAL & DISPOSAL EXISTING ASPHALT PVMT	S.Y.	3	$10.00	$3,230.00	323.00
			STATE	$10.00	$3,230.00	323.00
407-07.01	INTELLIGENT COMPACTION EQUIPMENT	LS	1	$54,398.86	$380,792.00	7.00
			2	$19,540.07	$156,320.58	8.00
			3	$20,571.43	$144,000.00	7.00
			4	$51,785.71	$362,500.00	7.00
			STATE	$35,986.64	$1,043,612.58	29.00
407-20.05	SAW CUTTING ASPHALT PAVEMENT	L.F.	1	$2.35	$2,540.00	1080.00
			2	$2.83	$13,032.00	4600.00
			3	$9.43	$12,529.00	1328.00
			STATE	$4.01	$28,101.00	7008.00
411-01.07	ACS MIX (PG64-22) GRADING E SHOULDER	TON	1	$92.68	$2,362,182.73	25488.00
			2	$88.04	$1,352,536.19	15363.00
			3	$119.52	$115,219.40	964.00
			4	$101.65	$1,881,274.64	18508.00
			STATE	$94.68	$5,711,212.96	60323.00
411-01.10	ACS MIX(PG64-22) GRADING D	TON	1	$155.70	$852,780.10	5477.00
			2	$106.84	$7,947,186.04	74383.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
			STATE	$521.21	$2,879,700.00	5525.00
604-03.02	STEEL BAR REINFORCEMENT (BRIDGES)	LB.	1	$1.00	$298,721.00	298721.00
			2	$1.09	$115,224.12	105840.00
			3	$1.30	$27,535.30	21181.00
			4	$1.09	$502,390.74	460869.00
			STATE	$1.06	$943,871.16	886611.00
604-03.04	PAVEMENT @ BRIDGE ENDS	S.Y.	1	$225.00	$108,225.00	481.00
			2	$328.34	$324,400.00	988.00
			3	$300.00	$48,600.00	162.00
			4	$263.04	$497,140.00	1890.00
			STATE	$277.87	$978,365.00	3521.00
604-03.07	CLASS A CONCRETE	C.Y.	4	$250.00	$2,250.00	9.00
			STATE	$250.00	$2,250.00	9.00
604-03.09	CLASS D CONCRETE (BRIDGE DECK)	C.Y.	2	$822.32	$597,830.00	727.00
			3	$680.00	$176,800.00	260.00
			4	$604.92	$1,051,957.00	1739.00
			STATE	$670.06	$1,826,587.00	2726.00
604-03.20	BRIDGE JOINT MODIFICATION	L.F.	1	$220.00	$16,500.00	75.00
			2	$112.47	$36,102.87	321.00
			STATE	$132.84	$52,602.87	396.00
604-03.32	CLASS DS CONCRETE	C.Y.	1	$525.00	$873,075.00	1663.00
			2	$850.00	$289,000.00	340.00
			4	$590.00	$549,880.00	932.00
			STATE	$583.29	$1,711,955.00	2935.00
604-03.74	CLASS X CONCRETE	C.Y.	3	$9,400.00	$47,000.00	5.00
			4	$550.00	$207,350.00	377.00
			STATE	$665.84	$254,350.00	382.00
604-04.01	APPLIED TEXTURE FINISH (NEW STRUCTURES)	S.Y.	1	$8.83	$48,196.00	5458.00
			2	$7.34	$26,450.90	3602.00
			3	$7.00	$19,915.00	2845.00
			4	$7.00	$154,395.40	22060.00
			STATE	$7.33	$248,957.30	33965.00
604-04.02	APPLIED TEXTURE FINISH (EX STRUCTURES)	S.Y.	1	$12.00	$30,612.00	2551.00
			2	$10.25	$309,796.51	30228.00
			3	$28.00	$7,560.00	270.00
			4	$9.11	$38,613.75	4238.00
			STATE	$10.37	$386,582.26	37287.00
604-04.03	BRIDGE END DRAINS(2'X8')	EACH	2	$2,000.00	$8,000.00	4.00
			4	$5,000.00	$20,000.00	4.00
			STATE	$3,500.00	$28,000.00	8.00
604-04.41	THREE STAR STATE EMBLEM	EACH	2	$1,625.00	$13,000.00	8.00
			3	$1,500.00	$3,000.00	2.00
			4	$1,500.00	$36,000.00	24.00
			STATE	$1,529.41	$52,000.00	34.00
604-05.31	BRIDGE DECK GROOVING (MECHANICAL)	S.Y.	1	$3.90	$22,226.10	5699.00
			2	$5.64	$56,240.01	9963.00
			4	$2.73	$28,109.40	10302.00
			STATE	$4.10	$106,575.51	25964.00
604-07.01	RETAINING WALL	S.F.	1	$177.28	$2,030,584.50	11454.00
			STATE	$177.28	$2,030,584.50	11454.00
604-07.02	RETAINING WALL	S.F.	1	$137.66	$68,280.00	496.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
			STATE	$137.66	$68,280.00	496.00
604-07.03	RETAINING WALL	S.F.	1	$135.00	$391,905.00	2903.00
			STATE	$135.00	$391,905.00	2903.00
604-07.04	RETAINING WALL	S.F.	1	$170.00	$742,560.00	4368.00
			STATE	$170.00	$742,560.00	4368.00
604-07.05	RETAINING WALL	S.F.	1	$147.00	$220,500.00	1500.00
			STATE	$147.00	$220,500.00	1500.00
604-07.06	RETAINING WALL	S.F.	1	$168.00	$43,680.00	260.00
			STATE	$168.00	$43,680.00	260.00
604-10.01	CONCRETE REPAIRS	LS	4	$11,115.00	$11,115.00	1.00
			STATE	$11,115.00	$11,115.00	1.00
604-10.02	CONCRETE REPAIRS	C.Y.	1	$2,000.00	$106,000.00	53.00
			2	$2,820.00	$16,920.00	6.00
			4	$2,100.00	$90,300.00	43.00
			STATE	$2,090.39	$213,220.00	102.00
604-10.04	DEMOLITION WORK	LS	4	$41,240.00	$41,240.00	1.00
			STATE	$41,240.00	$41,240.00	1.00
604-10.05	CONCRETE	S.F.	1	$145.06	$83,845.00	578.00
			2	$172.62	$188,157.32	1090.00
			3	$268.25	$44,261.25	165.00
			4	$135.00	$3,375.00	25.00
			STATE	$172.03	$319,638.57	1858.00
604-10.09	CONCRETE	C.Y.	1	$2,300.00	$29,900.00	13.00
			STATE	$2,300.00	$29,900.00	13.00
604-10.13	CONCRETE SLAB REMOVAL	LS	4	$70,000.00	$70,000.00	1.00
			STATE	$70,000.00	$70,000.00	1.00
604-10.14	REMOVE EXISTING WEARING SURFACE	LS	1	$24,883.33	$74,650.00	3.00
			2	$18,964.76	$94,823.81	5.00
			4	$27,250.00	$54,500.00	2.00
			STATE	$22,397.38	$223,973.81	10.00
604-10.16	EXPANSION JOINT REPAIRS	LS	2	$2,358.42	$2,358.42	1.00
			STATE	$2,358.42	$2,358.42	1.00
604-10.17	NON-PENETRATING CONCRETE SEAL	S.Y.	1	$100.00	$44,600.00	446.00
			2	$22.00	$8,800.00	400.00
			STATE	$63.12	$53,400.00	846.00
604-10.18	REINFORCING STEEL (REPAIRS)	LB.	2	$1.88	$3,731.80	1985.00
			3	$25.00	$1,575.00	63.00
			4	$2.82	$74,354.75	26388.00
			STATE	$2.80	$79,661.55	28436.00
604-10.20	HYDRODEMOLITION	S.Y.	2	$32.21	$178,041.35	5528.00
			STATE	$32.21	$178,041.35	5528.00
604-10.22	CONCRETE PARAPET REPAIR	L.F.	2	$315.32	$945.96	3.00
			3	$157.50	$33,862.50	215.00
			STATE	$159.67	$34,808.46	218.00
604-10.26	BRACING	EACH	2	$25,000.00	$25,000.00	1.00
			STATE	$25,000.00	$25,000.00	1.00
604-10.30	BRIDGE DECK REPAIRS (FULL DEPTH OF SLAB)	S.Y.	1	$500.00	$5,000.00	10.00
			2	$560.30	$30,816.70	55.00
			STATE	$551.03	$35,816.70	65.00
604-10.32	EXPANSION JOINT REPAIRS (TYPE A)	L.F.	1	$600.00	$43,800.00	73.00
			2	$1,200.00	$117,600.00	98.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
			2	$58.28	$23,720.55	407.00
			3	$65.00	$15,600.00	240.00
			4	$68.68	$31,526.00	459.00
			STATE	$62.79	$80,746.55	1286.00
705-02.01	SINGLE GUARDRAIL, WITH RUB-RAIL (TYPE 2)	L.F.	1	$19.75	$2,468.75	125.00
			2	$24.54	$20,715.50	844.00
			3	$15.00	$1,125.00	75.00
			4	$14.00	$2,100.00	150.00
			STATE	$22.12	$26,409.25	1194.00
705-02.02	SINGLE GUARDRAIL (TYPE 2)	L.F.	1	$18.37	$473,807.65	25795.00
			2	$18.59	$565,867.86	30436.00
			3	$17.02	$511,400.00	30050.00
			4	$15.00	$225,000.00	15000.00
			STATE	$17.54	$1,776,075.51	101281.00
705-02.03	SINGLE GUARDRAIL (TYPE 2) LONG POST	L.F.	1	$30.00	$12,000.00	400.00
			STATE	$30.00	$12,000.00	400.00
705-02.10	GUARDRAIL TRANSITION 27IN TO 31IN	EACH	1	$45.83	$29,195.60	637.00
			2	$52.63	$27,631.61	525.00
			3	$51.80	$35,018.00	676.00
			4	$29.02	$32,449.80	1118.00
			STATE	$42.05	$124,295.01	2956.00
705-02.20	SING GRDRL (TY 2) LONG POST (6 FT-9 FT)	L.F.	1	$23.58	$566,770.05	24033.00
			2	$26.00	$76,700.00	2950.00
			3	$23.50	$8,225.00	350.00
			4	$13.88	$64,879.50	4676.00
			STATE	$22.39	$716,574.55	32009.00
705-02.25	SING GRDRL (TY 2) LONG POST (9 FT-12 FT)	L.F.	1	$27.00	$12,825.00	475.00
			2	$30.00	$8,250.00	275.00
			3	$25.00	$3,125.00	125.00
			4	$13.88	$58,275.00	4200.00
			STATE	$16.25	$82,475.00	5075.00
705-03.01	MEDIAN DV GUARDRAIL,WITH RUB-RAIL (TY 2)	L.F.	1	$20.00	$1,000.00	50.00
			2	$25.00	$1,250.00	50.00
			3	$15.00	$750.00	50.00
			4	$16.00	$416.00	26.00
			STATE	$19.41	$3,416.00	176.00
705-03.02	MEDIAN DIVIDER GUARDRAIL (TYPE 2)	L.F.	1	$21.00	$525.00	25.00
			2	$25.00	$1,250.00	50.00
			3	$15.00	$750.00	50.00
			4	$17.00	$2,550.00	150.00
			STATE	$18.45	$5,075.00	275.00
705-03.03	MEDIAN DV GUARDRAIL (TY 2) (27"MT HT)	L.F.	1	$21.50	$11,825.00	550.00
			2	$10.00	$500.00	50.00
			3	$15.00	$3,375.00	225.00
			4	$14.00	$700.00	50.00
			STATE	$18.74	$16,400.00	875.00
705-03.04	MEDIAN DIVIDER GUARDRAIL AT CONC. BARR	L.F.	1	$15.00	$375.00	25.00
			2	$10.00	$180.00	18.00
			3	$15.00	$750.00	50.00
			4	$14.00	$700.00	50.00
			STATE	$14.02	$2,005.00	143.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
			2	$939.19	$161,540.57	172.00
			3	$830.00	$19,920.00	24.00
			4	$800.00	$171,200.00	214.00
			STATE	$856.00	$368,935.57	431.00
710-09.01	6"PERFORATED PIPE WITH VERTIC DRN SYSTEM	L.F.	1	$41.16	$17,577.00	427.00
			2	$27.96	$12,108.70	433.00
			3	$55.00	$7,480.00	136.00
			4	$17.55	$18,460.00	1052.00
			STATE	$27.16	$55,625.70	2048.00
710-09.02	6" PIPE UNDERDRAIN	L.F.	1	$9.50	$826.50	87.00
			2	$11.00	$759.00	69.00
			3	$15.00	$720.00	48.00
			4	$12.00	$3,768.00	314.00
			STATE	$11.72	$6,073.50	518.00
710-09.07	6"PERFORATED PLASTIC PIPE (FRENCH DRAIN)	L.F.	4	$5.00	$380.00	76.00
			STATE	$5.00	$380.00	76.00
710-10.01	4" PERFORATED PLASTIC PIPE	L.F.	1	$15.00	$1,290.00	86.00
			STATE	$15.00	$1,290.00	86.00
710-10.08	18" PERFORATED PLASTIC PIPE	L.F.	3	$45.00	$24,300.00	540.00
			STATE	$45.00	$24,300.00	540.00
711-05.70	32IN SINGLE SLOPE CONCRETE BARRIER WALL	L.F.	1	$79.00	$39,895.00	505.00
			STATE	$79.00	$39,895.00	505.00
711-05.71	51IN SINGLE SLOPE CONCRETE BARRIER WALL	L.F.	4	$350.00	$29,050.00	83.00
			STATE	$350.00	$29,050.00	83.00
711-05.72	SINGLE SLOPE HALF CONCRETE BARRIER WALL	L.F.	1	$230.00	$58,190.00	253.00
			STATE	$230.00	$58,190.00	253.00
711-05.78	GRADE SEPARATED SINGE SLOPE MEDIAN WALL	L.F.	1	$187.25	$385,370.00	2058.00
			STATE	$187.25	$385,370.00	2058.00
712-01	TRAFFIC CONTROL	LS	1	$35,477.41	$2,376,986.76	67.00
			2	$29,564.56	$2,690,374.85	91.00
			3	$26,694.92	$2,028,813.87	76.00
			4	$52,167.43	$1,930,194.73	37.00
			STATE	$33,307.64	$9,026,370.21	271.00
712-01.02	LANE CLOSURE	EACH	1	$471.46	$347,940.00	738.00
			2	$610.90	$444,125.00	727.00
			3	$720.68	$908,782.80	1261.00
			4	$746.28	$280,600.00	376.00
			STATE	$638.76	$1,981,447.80	3102.00
712-01.03	LANE CLOSURE(NIGHT WORK)	EACH	1	$1,250.00	$6,250.00	5.00
			2	$1,100.00	$4,400.00	4.00
			3	$700.00	$3,500.00	5.00
			4	$925.64	$36,100.00	39.00
			STATE	$948.11	$50,250.00	53.00
712-01.05	FLAGMAN	HOUR	3	$65.00	$3,900.00	60.00
			STATE	$65.00	$3,900.00	60.00
712-01.50	MAINTENANCE OF TRAFFIC	EACH	1	$915.54	$33,875.08	37.00
			2	$1,276.05	$40,833.60	32.00
			3	$1,180.78	$49,592.64	42.00
			4	$876.81	$36,825.84	42.00
			STATE	$1,053.12	$161,127.16	153.00
712-01.52	MAINTENANCE OF TRAFFIC	EACH	1	$805.12	$9,661.44	12.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
			2	$1,939.34	$34,908.06	18.00
			3	$750.00	$9,000.00	12.00
			4	$473.77	$2,842.62	6.00
			STATE	$1,175.25	$56,412.12	48.00
712-01.53	MAINTENANCE OF TRAFFIC	EACH	2	$1,575.00	$18,900.00	12.00
			STATE	$1,575.00	$18,900.00	12.00
712-01.54	MAINTENANCE OF TRAFFIC	EACH	2	$1,575.00	$18,900.00	12.00
			STATE	$1,575.00	$18,900.00	12.00
712-01.56	MAINTENANCE OF TRAFFIC	EACH	1	$1,575.00	$18,900.00	12.00
			3	$2,104.21	$39,980.08	19.00
			STATE	$1,899.36	$58,880.08	31.00
712-01.60	MAINTENANCE OF TRAFFIC (HENLEY STREET)	EACH	1	$1,575.00	$18,900.00	12.00
			STATE	$1,575.00	$18,900.00	12.00
712-02.02	INTERCONNECTED PORTABLE BARRIER RAIL	L.F.	1	$29.74	$878,042.60	29523.00
			2	$33.33	$896,963.51	26908.00
			3	$30.63	$570,336.00	18619.00
			4	$32.29	$296,625.30	9187.00
			STATE	$31.36	$2,641,967.41	84237.00
712-02.36	REMOVE & RELOCATE PORTABLE BARRIER RAIL	L.F.	1	$16.50	$51,975.00	3150.00
			STATE	$16.50	$51,975.00	3150.00
712-02.47	BRIDGE MOUNTED INTERCON PORT BAR RAIL	L.F.	2	$75.00	$30,225.00	403.00
			4	$150.00	$34,800.00	232.00
			STATE	$102.40	$65,025.00	635.00
712-04.01	FLEXIBLE DRUMS (CHANNELIZING)	EACH	1	$29.43	$205,139.86	6971.00
			2	$17.77	$73,850.33	4156.00
			3	$23.69	$119,791.25	5056.00
			4	$25.70	$75,793.00	2949.00
			STATE	$24.81	$474,574.44	19132.00
712-04.02	FLEXIBLE DRUMS (CHANNELIZATION)	EACH	2	$10.59	$17,343.10	1638.00
			STATE	$10.59	$17,343.10	1638.00
712-04.50	BARRIER RAIL DELINEATOR	EACH	1	$11.49	$6,114.16	532.00
			2	$14.07	$4,895.24	348.00
			3	$11.13	$5,289.00	475.00
			4	$12.75	$828.75	65.00
			STATE	$12.06	$17,127.15	1420.00
712-05.01	WARNING LIGHTS (TYPE A)	EACH	1	$22.40	$43,849.67	1958.00
			2	$27.69	$11,546.92	417.00
			3	$21.00	$14,196.24	676.00
			4	$19.70	$2,404.00	122.00
			STATE	$22.69	$71,996.83	3173.00
712-05.03	WARNING LIGHTS (TYPE C)	EACH	1	$37.72	$11,430.60	303.00
			2	$10.59	$9,238.00	872.00
			3	$2.00	$300.00	150.00
			4	$22.86	$6,400.00	280.00
			STATE	$17.05	$27,368.60	1605.00
712-06	SIGNS (CONSTRUCTION)	S.F.	1	$8.99	$379,140.64	42175.90
			2	$8.70	$400,710.38	46040.50
			3	$7.26	$614,996.27	84717.67
			4	$7.92	$407,529.95	51486.00
			STATE	$8.03	$1,802,377.24	224420.07
712-06.01	VERTICAL PANELS	S.F.	2	$16.00	$9,056.00	566.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
			STATE	$16.00	$9,056.00	566.00
712-06.16	SIGNS (CNSTR)(REDUCED SPEED WARNING)	EACH	2	$544.50	$2,178.00	4.00
			3	$722.50	$2,890.00	4.00
			STATE	$633.50	$5,068.00	8.00
712-07.03	TEMPORARY BARRICADES (TYPE III)	L.F.	1	$17.56	$24,507.64	1396.00
			2	$16.90	$31,496.08	1864.00
			3	$30.37	$31,344.00	1032.00
			4	$11.52	$14,582.00	1266.00
			STATE	$18.34	$101,929.72	5558.00
712-08.01	UNIFORMED POLICE OFFICER	DOLL	3	$1.00	$140,000.00	140000.00
			STATE	$1.00	$140,000.00	140000.00
712-08.03	ARROW BOARD (TYPE C)	EACH	1	$1,061.66	$74,315.99	70.00
			2	$1,287.55	$59,227.27	46.00
			3	$1,000.94	$73,068.54	73.00
			4	$1,299.43	$102,655.00	79.00
			STATE	$1,153.98	$309,266.80	268.00
712-08.10	MOBILE MESSAGE SIGN UNIT W/ATTENUATOR	HOUR	2	$107.40	$85,922.50	800.00
			3	$105.42	$329,950.00	3130.00
			4	$104.06	$105,936.00	1018.00
			STATE	$105.46	$521,808.50	4948.00
712-08.14	PORTABLE QUEUE WARNING SYSTEM	DAY	3	$1,750.00	$24,500.00	14.00
			STATE	$1,750.00	$24,500.00	14.00
712-08.20	TRUCK MOUNTED ATTENUATOR W/MESSAGE BOARD	DAY	1	$657.95	$75,664.60	115.00
			2	$952.42	$180,959.60	190.00
			3	$786.71	$267,480.20	340.00
			4	$667.02	$125,400.00	188.00
			STATE	$779.72	$649,504.40	833.00
712-09.01	REMOVABLE PAVEMENT MARKING LINE	L.F.	1	$3.18	$152,434.65	47875.00
			2	$2.43	$34,586.00	14230.00
			3	$2.97	$35,551.00	11955.00
			4	$3.10	$72,546.20	23402.00
			STATE	$3.03	$295,117.85	97462.00
712-09.02	REMOVABLE PAVEMENT MKG (8" BARRIER LINE)	L.F.	1	$3.80	$88,424.50	23265.00
			2	$3.16	$220,986.50	70008.00
			3	$2.96	$54,770.56	18479.00
			4	$4.07	$44,709.00	10990.00
			STATE	$3.33	$408,890.56	122742.00
712-09.03	REMOVABLE PVMT MKG(CHNZIZING STRIPING)	S.Y.	2	$53.00	$2,650.00	50.00
			STATE	$53.00	$2,650.00	50.00
712-09.04	REMOVABLE PAVEMENT MARKING (STOP LINE)	L.F.	1	$22.15	$9,923.20	448.00
			2	$31.22	$8,523.40	273.00
			3	$15.92	$1,353.00	85.00
			4	$20.00	$480.00	24.00
			STATE	$24.43	$20,279.60	830.00
712-09.08	REMOVABLE PAVEMENT MARKING (6" line)	L.F.	2	$2.00	$20,000.00	10000.00
			STATE	$2.00	$20,000.00	10000.00
712-09.09	REMOVABLE PAVEMENT MARKING (4" line)	L.F.	3	$3.00	$5,190.00	1730.00
			STATE	$3.00	$5,190.00	1730.00
712-09.20	TEMPORARY PLASTIC PVMT MKG(NOISE STRIP)	L.F.	2	$16.00	$768.00	48.00
			STATE	$16.00	$768.00	48.00
712-09.33	MODIFY EXISTING OVERHEAD SIGNS	EACH	2	$6,871.46	$41,228.76	6.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
			4	$4.08	$2,361.00	579.00
			STATE	$4.19	$2,993.25	714.00
713-11.21	P POST SLIP BASE	EACH	1	$410.20	$13,126.50	32.00
			2	$407.08	$3,256.60	8.00
			3	$273.93	$3,835.00	14.00
			4	$321.73	$34,425.00	107.00
			STATE	$339.40	$54,643.10	161.00
713-11.23	ROUND POST SLIP BASE	EACH	1	$427.34	$10,683.44	25.00
			STATE	$427.34	$10,683.44	25.00
713-13.02	FLAT SHEET ALUMINUM SIGNS (0.080" THICK)	S.F.	1	$13.88	$13,282.13	957.00
			2	$13.37	$16,303.26	1219.25
			3	$13.51	$5,716.44	422.99
			4	$13.17	$19,021.50	1444.00
			STATE	$13.44	$54,323.33	4043.24
713-13.03	FLAT SHEET ALUMINUM SIGNS (0.100" THICK)	S.F.	1	$15.31	$23,346.46	1525.00
			2	$13.86	$42,365.85	3057.56
			3	$15.08	$8,457.65	561.00
			4	$16.16	$86,661.20	5364.00
			STATE	$15.31	$160,831.16	10507.56
713-13.12	FLAT SHEET ALUM SIGNS OVLY (0.080"THICK)	S.F.	1	$15.00	$2,250.00	150.00
			2	$18.50	$2,775.00	150.00
			3	$17.50	$3,150.00	180.00
			4	$18.50	$4,162.50	225.00
			STATE	$17.50	$12,337.50	705.00
713-13.14	FLORESCENT YELLOW SIGN SHEETING	S.F.	1	$4.25	$425.00	100.00
			2	$4.35	$870.00	200.00
			3	$3.25	$325.00	100.00
			4	$4.35	$326.25	75.00
			STATE	$4.10	$1,946.25	475.00
713-14	EXTRUDED ALUMINUM PANEL SIGNS	S.F.	1	$25.34	$146,809.00	5793.00
			2	$22.57	$97,948.40	4340.00
			3	$21.68	$98,617.25	4548.50
			4	$22.11	$159,683.00	7223.00
			STATE	$22.97	$503,057.65	21904.50
713-14.21	STREET NAME SIGN (RIGID 0.100IN THICK	S.F.	1	$40.34	$4,477.92	111.00
			3	$23.00	$644.00	28.00
			STATE	$36.85	$5,121.92	139.00
713-14.22	STREET NAME SGN (SUSPENDED 0.100IN THICK	S.F.	2	$22.03	$1,784.69	81.00
			3	$24.44	$2,640.00	108.00
			STATE	$23.41	$4,424.69	189.00
713-15	REMOVAL OF SIGNS, POSTS AND FOOTINGS	LS	1	$1,534.44	$7,672.18	5.00
			2	$590.74	$3,544.43	6.00
			3	$375.00	$750.00	2.00
			4	$800.00	$1,600.00	2.00
			STATE	$904.44	$13,566.61	15.00
713-15.01	REMOVAL OF EXISTING SIGN POST	EACH	1	$80.00	$400.00	5.00
			2	$65.00	$195.00	3.00
			3	$25.00	$200.00	8.00
			4	$65.00	$325.00	5.00
			STATE	$53.33	$1,120.00	21.00
713-15.02	REMOVAL & RELOCATION OF SIGN & SUPPORT	EACH	3	$287.81	$2,302.50	8.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
			STATE	$287.81	$2,302.50	8.00
713-15.03	RMVL OF LG BARRIER MOUNT SGNS W/SDL SUP	EACH	1	$60.00	$1,800.00	30.00
			2	$75.00	$2,250.00	30.00
			3	$70.00	$2,100.00	30.00
			4	$75.00	$2,250.00	30.00
			STATE	$70.00	$8,400.00	120.00
713-15.06	MILE POST SIGN	EACH	2	$201.14	$1,407.98	7.00
			STATE	$201.14	$1,407.98	7.00
713-15.07	SUSPENDED FLAT SHT ALM SGN (0.080"THICK)	EACH	1	$362.75	$7,980.50	22.00
			2	$85.00	$170.00	2.00
			STATE	$339.60	$8,150.50	24.00
713-15.10	RMVL OF SM BARRIER MOUNT SGN W/FL PL SP	EACH	1	$50.00	$1,500.00	30.00
			2	$65.00	$1,950.00	30.00
			3	$45.00	$1,350.00	30.00
			4	$65.00	$1,950.00	30.00
			STATE	$56.25	$6,750.00	120.00
713-15.36	REMOVE SIGN, SUPPORT & FOOTING	EACH	1	$106.00	$2,650.00	25.00
			2	$39.41	$2,680.00	68.00
			3	$50.90	$3,359.50	66.00
			4	$26.63	$4,420.00	166.00
			STATE	$40.34	$13,109.50	325.00
713-15.37	INSTALLATION OF SIGN MESSAGE(LETTER,#,AR	S.F.	1	$55.00	$1,320.00	24.00
			2	$55.00	$825.00	15.00
			3	$40.00	$1,800.00	45.00
			4	$55.00	$1,650.00	30.00
			STATE	$49.08	$5,595.00	114.00
713-15.38	EXTRD ALM PANELS FOR OVERLAYING BKGROUND	S.F.	1	$6.50	$123.50	19.00
			2	$7.00	$210.00	30.00
			3	$3.75	$168.75	45.00
			4	$7.00	$133.00	19.00
			STATE	$5.62	$635.25	113.00
713-15.41	SIGN REMOVAL (DESCRIPTION)	LS	2	$105.00	$105.00	1.00
			STATE	$105.00	$105.00	1.00
713-16.01	CHANGEABLE MESSAGE SIGN UNIT	EACH	1	$7,687.28	$115,309.26	15.00
			2	$5,572.26	$317,618.78	57.00
			3	$5,773.87	$288,693.48	50.00
			4	$6,331.54	$164,620.00	26.00
			STATE	$5,988.12	$886,241.52	148.00
713-16.06	DEAD END SIGN AND SUPPORT	EACH	4	$222.50	$890.00	4.00
			STATE	$222.50	$890.00	4.00
713-16.07	END OF ROADWAY SIGN AND SUPPORT	EACH	2	$210.00	$420.00	2.00
			4	$220.00	$880.00	4.00
			STATE	$216.67	$1,300.00	6.00
713-16.09	RAILROAD ADVANCE WARNING SIGN AND SUPORT	EACH	3	$300.00	$1,200.00	4.00
			STATE	$300.00	$1,200.00	4.00
713-16.20	SIGNS	EACH	1	$266.03	$4,256.50	16.00
			2	$189.68	$3,224.49	17.00
			3	$163.90	$19,832.00	121.00
			4	$289.50	$2,316.00	8.00
			STATE	$182.90	$29,628.99	162.00
713-16.21	SIGNS	EACH	1	$300.00	$300.00	1.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
			STATE	$500.00	$1,500.00	3.00
716-04.01	PLASTIC PVMT MKG (STRAIGHT-TURN ARROW)	EACH	1	$203.91	$4,690.00	23.00
			2	$188.86	$3,399.45	18.00
			3	$234.70	$12,908.75	55.00
			4	$236.30	$5,435.00	23.00
			STATE	$222.13	$26,433.20	119.00
716-04.02	PLASTIC PVMT MKG(DOUBLE TURNING ARROW)	EACH	1	$154.29	$1,080.00	7.00
			2	$184.75	$1,108.47	6.00
			3	$184.62	$2,400.00	13.00
			4	$120.00	$1,440.00	12.00
			STATE	$158.64	$6,028.47	38.00
716-04.03	PLASTIC PAVEMENT MARKING(4" DOTTED LINE)	L.F.	1	$1.25	$431.25	345.00
			STATE	$1.25	$431.25	345.00
716-04.04	PLASTIC PVMT MKG (TRANSVERSE SHOULDER)	L.F.	1	$4.78	$317,599.35	66436.00
			2	$5.29	$9,512.11	1799.00
			3	$7.64	$9,682.00	1267.00
			4	$9.25	$12,527.50	1355.00
			STATE	$4.93	$349,320.96	70857.00
716-04.05	PLASTIC PAVEMENT MKG (STRAIGHT ARROW)	EACH	1	$116.94	$7,952.20	68.00
			2	$112.00	$2,687.94	24.00
			3	$144.03	$7,921.40	55.00
			4	$161.18	$2,740.00	17.00
			STATE	$129.89	$21,301.54	164.00
716-04.06	PLASTIC PAVEMENT MKG (WRONG WAY ARROW)	EACH	1	$111.33	$1,670.00	15.00
			2	$75.00	$1,125.00	15.00
			3	$125.00	$1,250.00	10.00
			4	$10.00	$100.00	10.00
			STATE	$82.90	$4,145.00	50.00
716-04.07	PLASTIC PAVEMENT MKG (EXIT ONLY ARROW)	EACH	1	$99.23	$1,290.00	13.00
			2	$179.70	$4,133.00	23.00
			3	$150.00	$1,500.00	10.00
			4	$220.00	$1,980.00	9.00
			STATE	$161.87	$8,903.00	55.00
716-04.08	PLASTIC PAVEMENT MKG (OPTION LANE ARROW)	EACH	1	$111.00	$1,443.00	13.00
			2	$206.83	$2,482.00	12.00
			3	$175.00	$2,625.00	15.00
			4	$225.00	$2,250.00	10.00
			STATE	$176.00	$8,800.00	50.00
716-04.09	PLASTIC PAVEMENT MKG (H.O.V. DIAMOND)	EACH	1	$75.00	$1,500.00	20.00
			2	$100.00	$500.00	5.00
			3	$250.00	$3,750.00	15.00
			4	$50.00	$1,000.00	20.00
			STATE	$112.50	$6,750.00	60.00
716-04.10	PLASTIC PAVEMENT MKG (HANDICAP SYMBOL)	EACH	1	$148.50	$1,188.00	8.00
			2	$150.00	$750.00	5.00
			3	$245.00	$2,450.00	10.00
			4	$129.44	$2,330.00	18.00
			STATE	$163.85	$6,718.00	41.00
716-04.11	PLAS PVMNT MKG(BICYCLE SYMBOL W/RIDER)	EACH	2	$400.00	$800.00	2.00
			STATE	$400.00	$800.00	2.00
716-04.12	PLASTIC PAVEMENT MARKING (YIELD LINE)	S.F.	1	$9.28	$58,866.96	6342.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
			STATE	$0.54	$177,866.25	328593.70
716-08.02	REMOVAL OF PVMT MKG (8" BARRIER LINE)	L.F.	1	$0.45	$450.00	1000.00
			2	$1.00	$1,000.00	1000.00
			3	$0.75	$750.00	1000.00
			4	$0.40	$400.00	1000.00
			STATE	$0.65	$2,600.00	4000.00
716-08.03	REMOVAL OF PAVEMENT MARKING (CROSS-WALK)	L.F.	1	$3.23	$485.00	150.00
			2	$3.97	$588.00	148.00
			3	$1.50	$150.00	100.00
			4	$1.00	$100.00	100.00
			STATE	$2.66	$1,323.00	498.00
716-08.04	REMOVAL PAVEMENT MKG (CHNZ STRIPING)	S.Y.	1	$6.00	$600.00	100.00
			2	$14.31	$26,858.24	1877.00
			3	$8.40	$6,268.78	746.00
			4	$14.58	$2,274.00	156.00
			STATE	$12.50	$36,001.02	2879.00
716-08.05	REMOVAL OF PAVEMENT MARKING (STOP LINE)	L.F.	1	$2.72	$1,148.00	422.00
			2	$3.33	$1,440.00	432.00
			3	$6.23	$3,744.40	601.00
			4	$7.14	$6,329.70	886.00
			STATE	$5.41	$12,662.10	2341.00
716-08.06	REMOVAL OF PVMT MKG (TURN LANE ARROW)	EACH	1	$47.61	$2,666.00	56.00
			2	$61.82	$9,396.40	152.00
			3	$50.42	$1,210.00	24.00
			4	$22.05	$463.00	21.00
			STATE	$54.29	$13,735.40	253.00
716-08.07	REMOVAL PVMT MKG (STRAIGHT-TURN ARROW)	EACH	1	$80.71	$1,130.00	14.00
			2	$150.00	$600.00	4.00
			3	$65.36	$457.50	7.00
			4	$20.00	$80.00	4.00
			STATE	$78.19	$2,267.50	29.00
716-08.08	REMOVAL PVMT MKG (DOUBLE TURNING ARROW)	EACH	1	$25.00	$50.00	2.00
			2	$159.74	$479.23	3.00
			3	$75.00	$150.00	2.00
			4	$20.00	$40.00	2.00
			STATE	$79.91	$719.23	9.00
716-08.09	REMOVAL OF PAVEMENT MKG (DOTTED LINE)	L.F.	1	$0.35	$35.00	100.00
			2	$1.00	$100.00	100.00
			3	$1.02	$182.95	179.00
			4	$0.50	$50.00	100.00
			STATE	$0.77	$367.95	479.00
716-08.10	REMOVAL PVMT MKG (TRANSVERSE SHOULDER)	L.F.	1	$2.00	$200.00	100.00
			2	$3.76	$1,360.88	362.00
			3	$3.00	$300.00	100.00
			4	$2.00	$200.00	100.00
			STATE	$3.11	$2,060.88	662.00
716-08.11	REMOVAL OF WORD PVMT MKG	EACH	1	$50.00	$250.00	5.00
			2	$95.00	$570.00	6.00
			3	$101.88	$815.00	8.00
			4	$81.43	$570.00	7.00
			STATE	$84.81	$2,205.00	26.00

Item Number	Description	Unit of Measure	Region	Average Unit Price	Total Cost	Total Quantity
			2	$50.00	$10,426,375.00	208525.00
			STATE	$46.24	$25,836,875.00	558700.00
805-05.03	REINFORCED SHOTCRETE FACING	S.F.	1	$77.65	$3,902,692.50	50260.00
			2	$72.58	$3,931,831.00	54175.00
			STATE	$75.02	$7,834,523.50	104435.00
805-05.04	SECONDARY WALL (DESCRIPTION)	S.F.	1	$74.38	$2,357,900.00	31700.00
			2	$127.50	$4,656,156.00	36520.00
			STATE	$102.82	$7,014,056.00	68220.00
805-12.01	EROSION CONTROL BLANKET (TYPE I)	S.Y.	1	$1.27	$20,315.30	15986.00
			2	$0.85	$130,627.15	153679.00
			3	$3.00	$2,550.00	850.00
			4	$1.30	$35,408.10	27237.00
			STATE	$0.96	$188,900.55	197752.00
805-12.02	EROSION CONTROL BLANKET (TYPE II)	S.Y.	1	$1.16	$38,783.39	33570.00
			2	$0.92	$70,349.81	76312.00
			3	$0.95	$39,235.00	41300.00
			4	$1.30	$242,673.60	186672.00
			STATE	$1.16	$391,041.80	337854.00
805-12.03	EROSION CONTROL BLANKET (TYPE III)	S.Y.	1	$1.87	$31,113.50	16603.00
			4	$2.00	$364.00	182.00
			STATE	$1.88	$31,477.50	16785.00
805-12.04	EROSION CONTROL BLANKET (TYPE IV)	S.Y.	1	$2.50	$38,197.50	15279.00
			STATE	$2.50	$38,197.50	15279.00
805-12.08	700 GRAM COIR FIBER EROSION BLANKET	S.Y.	2	$9.00	$2,079.00	231.00
			STATE	$9.00	$2,079.00	231.00
806-01	MOWING	ACRE	3	$73.18	$339,900.00	4645.00
			STATE	$73.18	$339,900.00	4645.00
806-02.03	PROJECT MOWING	CYCL	1	$1,280.00	$7,680.00	6.00
			2	$1,714.70	$17,147.02	10.00
			3	$1,000.00	$4,000.00	4.00
			4	$6,500.00	$39,000.00	6.00
			STATE	$2,608.73	$67,827.02	26.00
806-02.13	SWATH MOWING	ACRE	3	$66.84	$451,556.00	6756.00
			STATE	$66.84	$451,556.00	6756.00
908-21.01	BEARINGS	EACH	1	$1,100.00	$12,100.00	11.00
			STATE	$1,100.00	$12,100.00	11.00
920-08.22	TEMPORARY SHORING	S.F.	2	$40.00	$53,760.00	1344.00
			STATE	$40.00	$53,760.00	1344.00

Wall Concrete Quantities											
		Station	Height of Wall (ft)	Average Wall Height (ft)	Stem Thickness (ft)	Slab Thickness (ft)	Slab Length (ft)	Stem Area (ft^2)	Slab Area (ft^2)	Distance Between (ft)	Volume (ft^3)
		3+20	6.42								
				4.71	1	1	6	4.71	6	20	214.2
		3+40	3								
				3.24	1	1	6	3.24	6	20	184.8
		3+60	3.48								
				3.705	1	1	6	3.705	6	20	194.1
		3+80	3.93								
				4.655	1	1	6	4.655	6	20	213.1
		4+00	5.38								
				6.21	1	1	6	6.21	6	20	244.2
		4+20	7.04								
				8.335	1	1	6	8.335	6	20	286.7
		4+40	9.63								
				13.22	2	2	14	26.44	28	20	1088.8
		4+60	16.81								
				18.505	2	2	14	37.01	28	20	1300.2
		4+80	20.2								
				20.43	2	2	14	40.86	28	20	1377.2
		5+00	20.66								
				18.81	2	2	14	37.62	28	20	1312.4
		5+20	16.96								
				17.155	2	2	14	34.31	28	20	1246.2
		5+40	17.35								
				17.075	2	2	14	34.15	28	20	1243
		5+60	16.8								
				16.355	2	2	14	32.71	28	20	1214.2
		5+80	15.91								
				13.69	2	2	14	27.38	28	20	1107.6
		6+00	11.47								
				10.11	2	2	14	20.22	28	20	964.4
		6+20	8.75								
				7.51	1	1	6	7.51	6	20	270.2
		6+40	6.27								
				5.2	1	1	6	5.2	6	20	224
		6+60	4.13								
				3.565	1	1	6	3.565	6	20	191.3
		6+80	3								
									Total Volume	ft^3	12876.6
										cy	476.9111
									COST ($)		68675.2

Wall Backfill Quantities											
		Station	Granular Backfill area (ft^2)	Sand Backfill Area (ft^2)	Granular Average End Area (ft^2)	Sand Average End Area (ft^2)	Distance Between (ft)	Granular Volume (ft^3)	Sand Volume (ft^3)		
		3+20	10.73	39.62							
					7.295	28.51	20	145.9	570.2		
		3+40	3.86	17.4							
					4.35	18.485	20	87	369.7		
		3+60	4.84	19.57							
					5.345	18.955	20	106.9	379.1		
		3+80	5.85	18.34							
					7.355	22.86	20	147.1	457.2		
		4+00	8.86	27.38							
					10.515	34.27	20	210.3	685.4		
		4+20	12.17	41.16							
					14.715	78.325	20	294.3	1566.5		
		4+40	17.26	115.49							
					24.44	170.795	20	488.8	3415.9		
		4+60	31.62	226.1							
					35.195	311.735	20	703.9	6234.7		
		4+80	38.77	397.37							
					39.47	460.56	20	789.4	9211.2		
		5+00	40.17	523.75							
					36.795	496.035	20	735.9	9920.7		
		5+20	33.42	468.32							
					33.81	475.795	20	676.2	9515.9		
		5+40	34.2	483.27							
					33.645	474.625	20	672.9	9492.5		
		5+60	33.09	465.98							
					31.695	445.655	20	633.9	8913.1		
		5+80	30.3	425.33							
					26.23	352.37	20	524.6	7047.4		
		6+00	22.16	279.41							
					19.14	241.825	20	382.8	4836.5		
		6+20	16.12	204.24							
					14.08	138.38	20	281.6	2767.6		
		6+40	12.04	72.52							
					9.105	55.15	20	182.1	1103		
		6+60	6.17	37.78							
					5.745	34.005	20	114.9	680.1		
		6+80	5.32	30.23							
								Granular	Sand		
							Total Volume (ft^3)	7178.5	77166.7		84345.2
							cy	265.8704	2858.026		3123.896
							ton	354.4938	3810.701		
							COST ($)	11308.35	72403.32		

Wall Rebar Quantities

Station	Height of Wall (ft)	Average Wall Height (ft)	Stem Thickness (ft)	Slab Thickness (ft)	Slab Length (ft)	Distance Between (ft)	"O" bar slab length (in.)	"O" bar stem length (in.)	Weight of single "O" bar (lb/ft)	# of "O" bars	Weight of "O" bar for this section	Weight of single "M" bar (lb/ft)	# of "M" bars/ft	Weight of "M" bar for this section	Weight of "P" bar (lb/ft)	# of "P" bars/ft	Length of "P" bar (ft)	Weight of "P" bar for this section	Weight of "Lb" bar (lb/ft)	# of "Lb" bars in slab	Weight of "Lb" bar for this section	Weight of "Lw" bar (lb/ft)	# of "Lw" bars placed 16"	Weight of "Lw" bar for this section
3+20	6.42																							
		4.71	1	1	6	20	17	17	1.043	2	118.2067	1.043	1	98.2506	1.043	1	6	125.16	1.043	6	125.16	2.044	4	163.52
3+40	3																							
		3.24	1	1	6	20	17	17	1.043	2	118.2067	1.043	1	67.5864	1.043	1	6	125.16	1.043	6	125.16	2.044	3	122.64
3+60	3.48																							
		3.705	1	1	6	20	17	17	1.043	2	118.2067	1.043	1	77.2863	1.043	1	6	125.16	1.043	6	125.16	2.044	3	122.64
3+80	3.93																							
		4.655	1	1	6	20	17	17	1.043	2	118.2067	1.043	1	97.1033	1.043	1	6	125.16	1.043	6	125.16	2.044	4	163.52
4+00	5.38																							
		6.21	1	1	6	20	17	17	1.043	2	118.2067	1.043	1	129.5406	1.043	1	6	125.16	1.043	6	125.16	2.044	5	204.4
4+20	7.04																							
		8.335	1	1	6	20	17	17	1.043	2	118.2067	1.043	1	173.8681	1.043	1	6	125.16	1.043	6	125.16	2.044	7	286.16
4+40	9.63																							
		13.22	2	2	14	20	26	26	1.502	5	650.8667	2.044	2	1080.867	1.502	5	14	2102.8	1.043	24	500.64	3.4	10	680
4+60	16.81																							
		18.505	2	2	14	20	26	26	1.502	5	650.8667	2.044	2	1512.969	1.502	5	14	2102.8	1.043	24	500.64	3.4	14	952
4+80	20.2																							
		20.43	2	2	14	20	26	26	1.502	5	650.8667	2.044	2	1670.357	1.502	5	14	2102.8	1.043	24	500.64	3.4	16	1088
5+00	20.66																							
		18.81	2	2	14	20	26	26	1.502	5	650.8667	2.044	2	1537.906	1.502	5	14	2102.8	1.043	24	500.64	3.4	15	1020
5+20	16.96																							
		17.155	2	2	14	20	26	26	1.502	5	650.8667	2.044	2	1402.593	1.502	5	14	2102.8	1.043	24	500.64	3.4	13	884
5+40	17.35																							
		17.075	2	2	14	20	26	26	1.502	5	650.8667	2.044	2	1396.052	1.502	5	14	2102.8	1.043	24	500.64	3.4	13	884
5+60	16.8																							
		16.355	2	2	14	20	26	26	1.502	5	650.8667	2.044	2	1337.185	1.502	5	14	2102.8	1.043	24	500.64	3.4	13	884
5+80	15.91																							
		13.69	2	2	14	20	26	26	1.502	5	650.8667	2.044	2	1119.294	1.502	5	14	2102.8	1.043	24	500.64	3.4	11	748
6+00	11.47																							
		10.11	2	2	14	20	26	26	1.502	5	650.8667	2.044	2	826.5936	1.502	5	14	2102.8	1.043	24	500.64	3.4	8	544
6+20	8.75																							
		7.51	1	1	6	20	17	17	1.043	2	118.2067	1.043	1	156.6586	1.043	1	6	125.16	1.043	6	125.16	2.044	6	245.28
6+40	6.27																							
		5.2	1	1	6	20	17	17	1.043	2	118.2067	1.043	1	108.472	1.043	1	6	125.16	1.043	6	125.16	2.044	4	163.52
6+60	4.13																							
		3.565	1	1	6	20	17	17	1.043	2	118.2067	1.043	1	74.3659	1.043	1	6	125.16	1.043	6	125.16	2.044	3	122.64
6+80	3																							
												Reinforcement on # sides		2		Reinforcement on # sides		2						
		Total Steel	87669.3556	lb					Total "O" bar weight (lb)		6921.66	Total "M" bar weight (lb)		25733.9		Total "P" bar weight		40103.28		Total "Lb"		5632.2		Total "Lw"	9278.32
		Cost	78902.42004	$																				

MSE Wall Quantities

Station	Distance (ft)	Height of Stem (ft)	Height of Exposed Wall (ft)	Beta (deg)	h1 (ft)	h2 (ft)	Average (ft)	Exposed Area of Wall Face (ft^2)						
3+20	20	6.42	4.42	0	4.42	1	2.71	54.2						
3+40	20	3	1	23.3	1	1.48	1.24	24.8						
3+60	20	3.48	1.48	6.4	1.48	1.93	1.705	34.1						
3+80	20	3.93	1.93	0	1.93	3.38	2.655	53.1						
4+00	20	5.38	3.38	-4	3.38	5.04	4.21	84.2						
4+20	20	7.04	5.04	-4.5	5.04	7.63	6.335	126.7						
4+40	20	9.63	7.63	-2.5	7.63	14.81	11.22	224.4						
4+60	20	16.81	14.81	-10	14.81	18.2	16.505	330.1						
4+80	20	20.2	18.2	18	18.2	18.66	18.43	368.6						
5+00	20	20.66	18.66	18	18.66	14.96	16.81	336.2						
5+20	20	16.96	14.96	18	14.96	15.35	15.155	303.1						
5+40	20	17.35	15.35	18	15.35	14.8	15.075	301.5						
5+60	20	16.8	14.8	18	14.8	13.91	14.355	287.1						
5+80	20	15.91	13.91	18	13.91	9.47	11.69	233.8						
6+00	20	11.47	9.47	18	9.47	6.75	8.11	162.2						
6+20	20	8.75	6.75	18	6.75	4.27	5.51	110.2						
6+40	20	6.27	4.27	18	4.27	2.13	3.2	64						
6+60	20	4.13	2.13	18	2.13	1	1.565	31.3						
6+80	20	3	1	18	1	0								
								3129.6	sf					0
360	Total Length													
							Cost	312960	$					

Faith Promise Lane				
Station	Cut Area (sf)	Fill Area (sf)	Cut Volume (cy)	Fill Volume (cy)
0+00	0.00	0.00	-	-
0+50	1247.85	0.00	1155.42	0.00
1+00	881.67	0.00	1971.78	0.00
1+50	487.51	0.00	1267.76	0.00
2+00	99.38	0.00	543.42	0.00
2+50	0.00	133.80	92.02	123.89
3+00	0.00	471.96	0.00	560.89
3+50	10.16	578.52	9.41	972.67
4+00	23.37	410.29	31.05	915.56
4+50	20.20	322.60	40.34	678.60
5+00	9.52	143.72	27.52	431.78
5+50	23.05	21.40	30.16	152.89
6+00	40.50	3.24	58.84	22.81
6+50	60.28	22.74	93.31	24.06
7+00	78.86	41.32	128.83	59.31
7+50	103.29	37.42	168.66	72.91
8+00	37.72	96.05	130.56	123.58

Sum	5749.08	4138.95
Shrink	4790.90	
Excess cut material	782.33	c.y.

Ramp				
Station	Cut Area (sf)	Fill Area (sf)	Cut Volume (ccy)	Fill Volume (ccy)
0+00	0	0.00		
0+50	0	0.00	0.00	0.00
1+00	0	0.00	0.00	0.00
1+50	0	0.00	0.00	0.00
2+00	0	0.00	0.00	0.00
2+20	78.83	13.72	29.20	5.08
2+40	177.42	0.00	94.91	5.08
2+60	200.56	0.40	139.99	0.15
2+80	215.86	0.00	154.23	0.15
3+00	241.85	0.00	169.52	0.00
3+20	111.22	0.41	130.77	0.15
3+40	78.06	2.37	70.10	1.03
3+60	82.39	0.48	59.43	1.06
3+80	110.36	0.36	71.39	0.31
4+00	177.97	0.25	106.79	0.23
4+20	343.42	0.23	193.11	0.18
4+40	536.73	0.37	325.98	0.22
4+60	896.42	0.35	530.80	0.27
4+80	1190.89	0.00	773.08	0.13
5+00	1180.44	0.00	878.27	0.00
5+20	1063.8	0.00	831.20	0.00
5+40	795.9	0.00	688.78	0.00
5+60	512.7	0.00	484.67	0.00
5+80	231.74	0.32	275.72	0.12
6+00	158.98	0.28	144.71	0.22
6+20	86.63	1.07	90.97	0.50
6+40	39.55	8.75	46.73	3.64
6+60	0.23	1.11	14.73	3.65
6+80	0	1.04	0.09	0.80
7+00	147.7	0.26	54.70	0.48
7+20	115.48	0.00	97.47	0.10
7+40	110.9	0.11	83.84	0.04
7+60	210.05	0.00	118.87	0.04
7+80	263.2	0.00	175.28	0.00
8+00	316.77	0.00	214.80	0.00
8+20	388.26	0.00	261.12	0.00
8+40	482.46	0.00	322.49	0.00
8+60	570.81	0.00	390.10	0.00
8+80	570.08	0.00	422.55	0.00
9+00	584.78	0.00	427.73	0.00
9+50	550.54	0.00	1051.22	0.00
10+00	525.96	0.00	996.76	0.00
10+50	519.51	0.00	968.03	0.00

Ramp				
Station	Cut Area (sf)	Fill Area (sf)	Cut Volume (ccy)	Fill Volume (ccy)
11+00	493.43	0.00	937.91	0.00
11+50	394.01	0.00	821.70	0.00
12+00	350.96	0.00	689.79	0.00
12+50	360.05	0.00	658.34	0.00
13+00	259.03	0.00	573.22	0.00
13+50	89.91	0.00	323.09	0.00
14+00	58.4	0.00	137.32	0.00
14+50	137.03	0.00	180.95	0.00
15+00	147.31	0.00	263.28	0.00
15+50	124.86	0.00	252.01	0.00
16+00	170.95	0.00	273.90	0.00
16+50	106.43	0.00	256.83	0.00
17+00	50.68	0.00	145.47	0.00
17+50	56.74	0.00	99.46	0.00
18+00	82.67	0.00	129.08	0.00
18+50	33.07	0.00	107.17	0.00
19+00	25.07	0.00	53.83	0.00
19+50	23.59	0.00	45.06	0.00
20+00	13.29	0.00	34.15	0.00
20+50	170.06	0.00	169.77	0.00

Sum	18042.46	23.61
Shrink	15035.39	
Excess cut material	18014.13	c.y.

	4	2	3		5a	5b	6	1	
Location	303-01	307-01.08	307-01.22	313-03 (cy)	402-01	402-02	403-01	411-01.10	501-01.03
Faith Promise Lane	2346	373	445		5	17	2	219	
Entrance Ramp	3366			47					259
Driveway A	101	18	22		1	1	1	11	
Driveway B	238	45	54		1	2	1	27	
	1			2					3

Faith Promise Lane	29635	ft2	Phase 1 (sf)	34620	sy	3847
Entrance Ramp	20913	ft2	Phase 2 (sf)	20913	sy	2324
Driveway A	1427	ft2				
Driveway B	3558	ft2				

yd2	3292.77	158.58	395.36		yd2	2323.65

	Faith Promise Ln	Driveway A	Driveway B			Entrance Ramp	
1	218.15	10.51	26.19	tons	1	3366.00	tons
2	372.08	17.92	44.68	tons	2	258.18	cy
3	444.52	21.41	53.37	tons	3	46.80	tons
4	2345.56	100.45	237.49	tons			
5a	4.63	0.22	0.56	tons			
5b	16.46	0.79	1.98	tons			
6	1.07	0.05	0.13	tons			