November 28, 2018

901 Design

Herff College Department of Civil Engineering

Engineering Sciences Suite 313

Dear Dr. Arellano,

Enclosed is the final report prepared by 901 Design for the I-69 Rest Area since the submission of the interim report. The report contains all engineering design methods, calculations and drawings that support the design. The civil engineering disciplines included in this report are wastewater treatment, structural design, geotechnical design, transportation design, water resources and cost estimating. 901 Design is privileged to have had the opportunity to conduct this design for your firm.

Regards,

901 Design

(enclosed)

THE UNIVERSITY OF MEMPHIS

CIVL 4199 – CIVIL ENGINEERING SENIOR DESIGN

REST AREA ADJACENT TO PROPOSED I-69

Final Design Report

Senior Design Report

Fall 2018

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Disclaimer: This report is student work. The contents of this report reflect the views of the students who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the views of the University of Memphis. The recommendations, drawings and specifications in this report should not be used without consulting a professional engineer.

TABLE OF CONTENTS

ACKNOWLEDGEMENTS

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901 Design would also like to recognize all Civil Engineering faculty members at the University of Memphis for their dedication to equipping students with the necessary skills to complete this project.

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1. INTRODUCTION

The introduction is intended to provide an overview of the project as well as a summary of services that are to be provided throughout the project. Prior work done for this final report will also be discussed.

1.1 Project overview

The scope of the project is to design a rest area along I-69 for TDOT. Design aspects include the following: water resource, geotechnical, structural, environmental, transportation, and more. Water and sewage facilities make up the environmental design. The structural engineer chooses the site location and designs the restroom building. Transportation design focuses on exits, entrances, parking, and pavement. Drainage and storm water management are included in the water resources design. Subsurface exploration of the site is handled by the geotechnical engineer, who is also responsible for retaining walls, foundations and building slabs. Several remaining facets left to the design group include but are not limited to: picnic tables, shelters, sidewalks, benches, trash collectors, onsite and imported fill requirements. All items must meet Self Sustainability Building (SSB) goals by minimizing carbon footprint and maximizing its LEED rating. In addition, rest area design must consider: minimizing land use, compliance with Americans with Disabilities Act (ADA), and the reduction of operational, maintenance, and construction costs. Feasible examples of attaining LEED ratings and SSB goals could include: grey or rain water recycling, recyclable pavement materials, alternative energy sources, LEED certified building materials, landscaping, and vegetation. Facility aging must be considered in design choices for the rest area to mitigate rising O&M costs over time. Finally, the rest area must be well lit, adequately secured, and include all relevant emergency response technologies.

1.2 Summary of scope of services

The scope of services is the official description of the work that is to be completed during the contract. This section is to clarify all work that will be performed from the beginning through the completion of the project for the design of a rest area adjacent to proposed I-69.

The following is the list of services that 901 Design will perform to complete the project:

- Site Selection: 901 Design has selected a location from the given project criteria. 901 Design made use of an alternative analysis (reported in the interim report) to determine the best location for the site. Refer to drawing S.1.

- Structural Design: Building plans with a full structural analysis of the building's structural frame. A detailed plan of the building dimensions. Included in the structural analysis will be the various load case combinations that the building will be subjected to. A thorough assessment will be conducted for the structural frame as well as the major connections for the structure. Refer to drawings S.B.1 through S.B.9 for structural plans.
- Transportation Design: The transportation section will provide the following services: overall site layout design, car and truck parking lot design, and entrance and exit ramp design. In addition, 901 Design will also perform Level of Service analysis for the section of the proposed I-69 Highway associated with the rest area. Finally, as an effort of achieving the criteria of self-sustaining, the Smart Park technology is introduced.
- Water Resources Design: Drainage analysis of the existing site and post development will be done, so that it can be compensated for during and after development. The storm water analysis will be done, so designs can be made per the TDOT requirements.
- Geotechnical Design: A bore plan was submitted to the owner for the required subsurface soil investigation. A foundation design was chosen based on existing soil parameters obtained by the soil investigation. The foundation will be analyzed for settlement and bearing capacity. The settlement analysis will only include primary consolidation due to the preliminary earthwork. The bearing capacity analysis will examine the total stress and effective stress of the foundation site soil. The structural design of the foundation included is based off Welded Wire Institute design guide.
- Other Design Considerations: In addition to the services listed above, 901 Design will consider design methods that will allow the facility and site to meet the client's self-sustaining building goal as well as implementing design methods to minimize the carbon footprint and maximize the LEED rating.

1.3 Prior work and reports

An interim report was submitted October 22nd, 2018 which provided several alternative analysis decisions made for the design of this project. The interim report also provided preliminary design work that had been completed up through October 22nd, 2018. A summary of the accomplished services through October 22nd, 2018 was also reported at that time.

1.4 Organization of report

This report consists of nine chapters which will cover the design process that 901 Design has performed. Listed below is an overview of the content of each presented chapter:

- Chapter One: *Introduction* – this chapter introduces the project and gives an overview of the services to be provided for the duration of the project.
- Chapter Two: *Wastewater Treatment* – this chapter details the design process and provides the results of the design calculations for the potable water supply as well as the recirculating sand filter.
- Chapter Three: *Structural* – this chapter will discuss the methods and procedures for the structural component of the report. Provided at the end of the chapter will be a summary of the overall design work that has been completed for this project.
- Chapter Four: *Geotechnical* – this chapter will give an overview of the sub surface soil investigation, the sizing of the foundation by bearing capacity and settlement analysis, and the structural design of the foundation.
- Chapter Five: *Transportation* – this chapter discuss the design of entrance and exit ramp, car parking lot, truck parking lot, level of service analysis, and an introduction to the Smart Park technology as a solution to achieve the owner's goal of self-sustaining building.
- Chapter Six: *Water Resources* – this chapter will discuss the various aspects for the water resources section of the report. It will discuss the overall design work completed for this project.
- Chapter Seven: *Opinion of Most Probable Cost* – this chapter will discuss the methods used in determining the most probable cost of the project.
- Chapter Eight: *Summary* – this chapter will provide a summary of the design decisions for the overall project, as well as a summary of final cost estimates.

CHAPTER 2. WASTEWATER TREATMENT

2.1 Introduction

Wastewater Treatment for the rest area will be provided by a Recirculating Sand Filter (RSF). The RSF was determined to be needed after the submission of the interim report and was not a part of that report. All deviations made during the final design of the project had to submitted to and approved by Dr. Arellano prior to proceeding with design changes. The design change was implemented upon learning that the Tennessee Department of Environmental Conservation would not approve 901 Design's original proposal. Dr. Arellano approved the decision to design the RSF for wastewater treatment. The I-69 Rest Area is located in area that doesn't have any nearby sewer municipalities. This report focuses on the details as to how the RSF was designed and how it treats waste water as opposed to how waste water is treated in full sized treatment plants. RSF's do not share the same design parameters that a full-size plant facility has since they are treating small buildings in rural areas where sanitary sewers are not feasible to obtain.

2.2 RSF Overview

The RSF provides treatment to wastewater through a multi-step process. Wastewater effluent is received by gravity into a septic tank. Suspended Solids are allowed to settle into the septic tank before moving forward in the system. The effluent is discharged by gravity and is then received into a recirculation tank. The effluent is diluted in the recirculation tank with water that has already made a pass through the entire system. The wastewater is then pumped from the recirculation tank to the sand filter bed. The sand filter removes the suspended solids that were too small to settle in the septic tank and provides microbiological treatment as the effluent percolates through. Effluent from the sand filter is then sent back to the recirculation tank to mix with the septic tank effluent according the recirculation ratio. Water in access needed for recirculation is then discharged back to the environment.

2.3 Design Loading

AASHTO's *Guide for Development of Rest Areas on Major Arterials and Freeways* was used to determine the amount of building effluent. The water usage of the building was determined to be 3,455 gpd. This is assuming that each user uses 3.5 gallons and that all the water used will be treated. Refer to Figure 1.

Restroom Stalls	T_1=A*UV*B*PF*P*UHF	T=Total Toilets	33			
		A= 1 way Design Year ADT	17575			
		UV= 1.3 Restroom users per vehicle	1.3			
		B= .15= Ratio of Design hourly volume to ADT	0.15	T_2	33	
	or	PF= 1.8= Peak Factor	1.8			
		P= Total % of traffic stopping at rest area	0.16	T_3= A*P*.0117	33	
		UHF= 30= Restroom users per hour per fixture	30			
		based on 2 min cycle				
	T_2=(S*1.3*1.5*1.8*P)/30					
	W=T * .6	W= Number of women's toilets		W=	20	
	M=T * .4	M= Total number of men's toilets & urinals		M=	13	
Water Usage	PHD= Peak hour demand					
	[ADT*B*PF*P*UV*(13.25 liter/user)]					
	or					
	[ADT*B*PF*P*UV*(3.5 gallon/user)]					
				flow=	3455	gpd
	The PHD rate in liters per minute or gallons per minute can be computed by					
	dividing the product obtained in the above formulas by 60 minutes per hour.					

Figure 1. Building Water Usage

TDEC requires that the design flow to be 1.5 times the amount of average daily flow. The design flow of the RSF will be 5,183 gpd as indicated by Table 1.

Table 1. Design Flows

AVG. Daily Flow	3455	gpd
Design Flow	5183	gpd

The design loading strength of the influent entering the system is listed in Table 2.

Table 2. Strength of TN Rest Area Influent

Strength (mg/L): Provided by James E. Etzel						
	BOD5	COD	SS	N	P	pH
max	223	885	310	173	41	8.7
min	65	160	16	60	9.5	7.1
avg	158	362	124	96	24	7.7

These wastewater loadings were obtained from James E. Etzel's research on "Treatment of Sanitary Wastes at Interstate Rest Areas." These values represent the average wastewater strength of samples of all rest areas in the state of Tennessee.

2.4 TDEC Preliminary Treatment Requirements

Preliminary treatment of the building wastewater effluent will be supplied by a Septic Tank Effluent Gravity system. The wastewater will flow into the septic tank by gravity. At a minimum, TDEC requires that the septic tank be sized to accommodate 2.5 times the design daily sewage flow anticipated to flow through the tank.

2.5 TDEC Secondary Treatment Requirements

Secondary treatment is provided in the recirculation tank. TDEC requires that the recirculation tank volume should equal the daily design flow. A minimum of 2 recirculation pumps are required so that the system can still operate during the failure of a pump. The recirculation pumps shall have a control panel with timed switches so that the number of doses and recirculation ratios can be adjusted. Float switches are also required to regulate fluctuating flows throughout the seasons of the year. The system shall also be equipped with a computerized process flow splitter that allows the effluent to be split between the recycle stream and discharge. The flow splitter shall be a device can be programmed to different return ratios.

2.6 TDEC Sand Filter Requirements

Effluent from the recirculation tank is received into the filter bed. The sand filter provides the primary treatment for the system. Design considerations include the media type and size, surface area, depth, dose volumes, and dosing frequencies. The sand filter should be sized by comparing the organic and hydraulic loading rates. The pipes distributing the effluent to the bed should be placed on 18-inch laterals.

2.7 RSF Design Calculations

The results for the RSF design calculations are list in below in Table 3.

Table 3. RSF Design Calculations

RSF Design		
Flow	5183	gpd
# of doses	48	per day
Recirculation Ratio	5	: 1
Total Volume Pumped	31095	gpd
Total Pump Run Time	240	min
Pump Flowrate	130	gpm
Filter Bed Sizing		
Organic Loading Rate	9.6	lbs BOD5/day
Hydraulic Loading Rate	10.0	gpd/ft^2
Surface Area of Filter Bed	518	ft^2
Media Type	Gravel	
Depth	30	in
Length	32.2	ft
Width	16.1	ft
Number of Laterals	11	
Detention Time	1	day

2.7.1 Design Equations

The equations used for designing the RSF are listed in APPENDIX A.1, equations A-12 through A-18.

2.7.2 Septic Tank Design

The septic must have a minimum volume of 8,368 gallons. The largest pre-constructed septic tank available is 5,025 gallons. This requires that 2 or septic tanks be operated in parallel in order to meet TDEC design requirements.

2.7.3 Recirculation Tank Design

The daily wastewater flow for the rest area is 5,183 gpd. Therefore, the recirculation tank must have a minimum volume of 5,183 gallons. The largest pre-constructed recirculation tank available is 5,025 gallons. This will require that 2 or more recirculation tanks will have to be operated in parallel.

2.7.4 Dosing frequency

Dosing must be performed on a timed basis and can be adjusted at any time during operation to meet the needs based on the effluent the system receives. For instance, during seasons of low flow, the dosing can be as little as once per hour. Dosing can be performed as much as twice per hour during seasons when flows are higher. The design results in Table 3 are based on 48 doses per day. This decision was made so that the system would be adequately sized to handle peak demand. The only requirements that pertain to dosing is that all of the effluent has to be treated in 24 hours and that doses be spaced enough to allow the filter to drain and reaerate.

2.7.5 Recirculation Ratio

The recirculation ratio is a measure of how much flow treated water is recirculated back through the system with the effluent and can be adjusted depending on the effluent flowing through the system. Recirculation ratios normally range from 3:1 to 5:1. A recirculation ration of 5:1 means that there are 5 parts recirculated flow with 1 part of forward effluent flow. TDEC requires sufficient evidence be provided if a system should need to operate on a ratio outside of this range. The recirculation ratio is adjusted in conjunction with the dosing frequency. For instance, if the system is operation on 48 doses per day on a 5:1 recirculation ratio the recirculation pumps will run for 5 minutes. The effluent would drain through the filter and the filter would have some reaeration during the next 25 minutes and then the cycle would repeat. It is important to know that recirculation should still be conducted on its appropriate interval during periods of extremely low flow, or perhaps no flow, so that the bacteria treating the water in the sand filter is kept alive. Its suggested that the system will need 3-5 days once it is up and running to build up a sufficient number of bacteria to treat the water.

2.7.6 Recirculation Pump Design

The total amount of water pumped, the pump run time, and flowrate required by the recirculation pump was determined using equations A-12, 13, and 14. During peak demand, the pump for this system will pump 31,095 gpd and will run for 3 hours. The pump would have to be capable of pumping water at a rate of 130 gpm.

2.7.7 Filter Bed Sizing

TDEC acknowledges that the initial performance of a new RSF will not be known until it is in operation. The design calculations for the system should be done again once the system is operating in order to make changes necessary to ensure the system operates correctly. The organic loading rate was determined using equation A-16. The BOD_5 content used in the original calculation was determined from raw wastewater strength samples that James E. Pretzel obtained from Tennessee rest areas during a study he performed. The organic loading rate is 9.6 lbs. of BOD_5 per day. Using table 15.1 in TDEC's RSF design manual (see Figure 2), for an organic loading rate greater to or equal to 10 lb. BOD_5/1000 ft^2, the hydraulic loading is 10-15 gpd/ft^2, filter depth should be 24-30 inches deep, and the filter media should be composed of gravel or similar media type with an effective grain size that ranges from 0.6-1 cm in diameter.

Table 15.1 Suggested Design Parameters for Granular Media Filter			
Design Parameter	Effective Size (D_{10})	Depth	Design Value
Filter media			
Sand or other, similar granular media	1.5-2.5 mm (Uniformity Coefficient = 1-3)	24-30 inches	3-5 gpd/ft^2 (hydraulic loading - forward flow) < or = 6.2 lb BOD_5/1000 ft^2/day organic loading
Gravel or other, similar granular media	0.6-1 cm diameter	24-30 inches	10-15 gpd/ft^2 (hydraulic loading - forward flow) < or = 10 lb BOD_5/1000 ft^2/day organic loading
Underdrain media	#57 stone	12-18 inches	

Figure 2. TDEC's RSF Suggested Design Parameters

The organic loading rate for the rest area is 9.6 lb. BOD5/day. The decision was made to design the filter for 10 lb. BOD5/day to help prevent issues from overloading the system. The surface area for the sand filters is 518 ft^2. TDEC requires that 2 sand filters be constructed so that the system doesn't have to be shut down for maintenance.

2.8 Potable Water Supply

The design of the potable water supply system was performed using the 2012 International Plumbing Code, AASHTO's Guide for Development of Rest Areas on Major Arterials and Freeways, 2012 International Fire Code and information provided by Millington Water Treatment Plant (MWTP). AASHTO's Guide for Development of Rest Areas on Major Arterials and Freeways was used to determine the number of rest area users that would result from the traffic flows. Traffic data was provided by Dr. Osman. According to the node combination in Table 4. TDOT Traffic Data that TDOT decided to use, the 30-year extrapolated data (beginning in year 2010) indicates that 35,150 vehicles are expected to use the I-69 corridor in year 2030. The designed rest area will only serve southbound traffic and therefor will be designed for approximately 17,500 vehicles.

Table 4. TDOT Traffic Data

Roadway Segment		*Analysis Years*	
		Year 2010 (ADT)*	*Year 2030 (ADT*)*
Existing Condition			
From	*To*		
SR 385	SR 59	C (33148)	F (49784)
SR 59	SR 87	B (25470)	D (40750)
SR 87	SR 19	B (19440)	D (38880)
SR 19	SR 88	B (18050)	C (27075)
SR 88	SR 104	A (15080)	B (19120)
SR 104	SR 78	B (23200)	C (30160)
US 51 Bypass	I-155 via SR 78	D (38620)	F (61790)
SR 78	US 412 via I-155	A (19120)	C (36330)
No Build W/ I-69 Traffic from SIUs 7 and 9			
From	*To*		
SR 385	SR 59	C (33148)	F (58584)
SR 59	SR 87	B (25470)	E (49550)
SR 87	SR 19	B (19440)	D (47680)
SR 19	SR 88	B (18050)	C (35875)
SR 88	SR 104	A (15080)	C (27920)
SR 104	SR 78	B (23200)	D (38960)
US 51/Bypass 3	I-155 via SR 78	D (38620)	F (70590)
SR 78	US 412 via I-155	A (19120)	C (45130)
Build Alternatives by Node			
From	*To*		
A (SR 385)	**B** (South of SR 59)	A (19288)	B (35150)
B (South of SR 59)	**D** (South of Hatchie River)	A (15280)	B (29730)
D (South of Hatchie River)	**K** (North of Hatchie River	A (25470)	B (49550)
E (SR 87)	**G** (Unionville Road)	A (10410)	A (21940)
K (North of Hatchie River)	**E** (SR 87)	A (15280)	B (29730)
K (North of Hatchie River)	**W** (SR 87)	A (16560)	B (32210)
G (Unionville Road)	**H** (I-155)	A (13920)	B (24975)
G (Unionville Road)	**Y** (SR 210)	A (11435)	B (22125)
J (SR 385)	**S** (Brighton-Clopton Road)	A (18963)	B (34388)
S (Brighton-Clopton Road)	**T** (SR 59)	A (19650)	C (36860)
S (Brighton-Clopton Road)	**C** (SR 59)	A (20110)	C (37530)
T (SR 59)	**U** (North of SR 54)	A (17600)	B (33580)
U (North of SR 54)	**V** (North of Hatchie River)	A (16560)	B (32210)
V (North of Hatchie River)	**W** (SR 87)	A (16560)	B (32210)
V (North of Hatchie River)	**E** (SR 87)	A (15920)	B (30970)
W (SR 87)	**Y** (SR 210)	A (11424)	B (23146)
Y (SR 210)	**Z** (I-155)	A (30137)	C (59957)

Figure 3 determines the number of fixtures that will be needed to accommodate the rest area users. It was determined that the rest area will need 20 toilets for the women's restroom. The men's restroom will need 13 fixtures composed of toilets and urinals. The rest area will also have 4 sinks in each restroom, 1 service sink, and 1 water fountain.

Restroom Stalls	T_1=A*UV*B*PF*P*UHF	T=Total Toilets	33		
		A= 1 way Design Year ADT	17575		
		UV= 1.3 Restroom users per vehicle	1.3		
		B= .15= Ratio of Design hourly volume to ADT	0.15	T_2	33
	or	PF= 1.8= Peak Factor	1.8		
		P= Total % of traffic stopping at rest area	0.16	T_3= A*P*.0117	33
		UHF= 30= Restroom users per hour per fixture	30		
		based on 2 min cycle			
	T_2=(S*1.3*1.5*1.8*P)/30				
	W= T * .6	W= Number of women's toilets		W=	20
	M= T * .4	M= Total number of men's toilets & urinals		M=	13

Figure 3. Fixture Requirements

2.8.1 International Plumbing Code Preliminary Requirements

There are currently no existing utilities located near the rest area site location. Potable water supply lines will have to constructed to the site from the nearest available utility. The water distribution system will have to connect MWTP's supply main located on West Union Rd. (Refer to which is roughly 1.25 miles south of the site location. The minimum daily service pressure, as provided by MWTP, in the area is 72 psi. The piping system will be constructed of ductile iron pipe to keep consistent with the type of material that MWTP currently uses in their systems. The water supply line will be constructed parallel to I-69 until it reaches the site location. This is done in order to minimize the water supply line from being located in the surrounding farm land, provide access to future expansion in the area, and to minimize the length of pipe needed to reach the site. The total developed length of the pipe is 6,748 ft.

2.8.2 Demand load

Chapter 6 of the 2012 International Plumbing Code (IPC) was used to determine the building water demand based off the number of fixtures the rest area needs.

Figure 4 of the IPC provides flowrates for different types of fixtures. For this project, fixtures of the flushometer type was chosen as they prevent vandalism by the plugging of toilets. The water supply demand was computed by summing all the flowrates of the fixtures listed in Figure 4. The total flowrate is 771 gpm with a minimum delivery pressure of 35 psi. (see Figure 5) The flowrate determined by Figure 6 assumes that all fixtures are being used at the same time. In order to account for a more realistic design flowrate, the IPC adjusts the building demand by converting the flowrates into Water Supply Fixture Units (w.s.f.u.'s), listed in Figure 6, the w.s.f.u.'s for the rest area is 315.

Figure 7 provides a list of flowrates associated with given w.s.f.u.'s. By linearly interpolating, Figure 8 shows the building demand is now 111 gpm.

FIXTURE SUPPLY OUTLET SERVING	FLOW RATE[a] (gpm)	FLOW PRESSURE (psi)
Bathtub, balanced-pressure, thermostatic or combination balanced-pressure/thermo-static mixing valve	4	20
Bidet, thermostatic mixing valve	2	20
Combination fixture	4	8
Dishwasher, residential	2.75	8
Drinking fountain	0.75	8
Laundry tray	4	8
Lavatory	2	8
Shower	3	8
Shower, balanced-pressure, thermostatic or combination balanced-pressure/thermo-static mixing valve	3	20
Sillcock, hose bibb	5	8
Sink, residential	2.5	8
Sink, service	3	8
Urinal, valve	12	25
Water closet, blow out, flushometer valve	25	45
Water closet, flushometer tank	1.6	20
Water closet, siphonic, flushometer valve	25	35
Water closet, tank, close coupled	3	20
Water closet, tank, one piece	6	20

Figure 4. Table 604.3 from 2012 IPC

Water Distribution System Design (Section 604.3)					Minmum Sizes of Fixture Water Supply Pipes (Section 604.5)
Fixture Supply Outlet	# of fixtures	Flowrate (gpm)	Total Flowrate (gpm)	Flow Pressure (psi)	Minimum Pipe Size (in)
Drinking Fountain	2	0.75	1.5	8	3/8
Residential Sink	8	2.5	20	8	1/2
Service Sink	1	3	3	8	1/2
Urinal Valve	8	12	96	25	3/4
Water Closet Flushometer Siphonic	26	25	650	35	1
Totals			770.5		

Figure 5. Total Flowrate

Load Values Assigned to Fixtures						
Fixture	Type	Load Values (wsfu)			Number of Fixtures	wsfu*# fixtures
		Cold	Hot	Total		
Water Closet	Public Flushometer Valve	10	0	10	26	260
Urinal Valve	3/4" Flushometer Valve	5	0	5	8	40
Drinking Fountain	3/8" valve	0.25	0	0.25	2	0.5
Residential Sink	Compared to Res. Kitchen sink	1	1	1.4	8	11.2
Service Sink	Faucet	2.25	2.25	3	1	3
				Total (wsfu)		315

Figure 6. WSFU Adjustment

SUPPLY SYSTEMS PREDOMINANTLY FOR FLUSHOMETER VALVES		
Load	Demand	
(Water supply fixture units)	(Gallons per minute)	(Cubic feet per minute)
275	104.5	13.96956
300	108.0	14.43744
400	127.0	16.97736

Figure 7. 2012 IPC Table 103

Building Demand		
Load (wsfu)	gpm	ft^3/min
315	111	14.8
Linear Interpolation		
wsfu	gpm	ft^3/min
300	108	14.43744
400	127	16.97736
315	110.85	14.81843

Figure 8. WSFU Linear Interpolation

2.8.3 Fire Code Requirements

The 2012 International Fire Code (IFC) was used to determine the fire requirements for the building. According the IFC, a Type A-3 building with a floor size ranging from 0-12,700 ft^2 requires a fire flow of 1,500 gpm at a pressure of 20 psi. One fire hydrant is needed for the rest area and should be located within 250 feet of the building.

2.8.4 Design Calculations

The design load of the building was established by comparing the fire requirements with the building load demand. The fire flow requirement is the controlling factor for supply flowrate to the site and the building demand controls the delivery pressure. The site was designed to supply 1,500 gpm at a pressure of 35 psi. The supply system was design using the methods and procedures found in Mott & Untener's 7th edition Applied Fluid Mechanics. MWTP utilizes pipe sizes in the range between 8-20 inches. Design calculations were performed for each of the pipe sizes in the given range and then the best option was selected based on the results. The water supply design results are listed in APPENDIX A.1. The equations used to design the water supply lines can be found in APPENDIX A.1.

Design Variables

Variables used in equations A-1 through A-11 are defined as follows:

Table 5. Definition of Variables

Q = Flow rate (gallon per minute, gpm) v = Velocity (ft/s) A = Area (ft) I.D.= Inside Diameter (ft) NR = Reynold's Number	f = Turbulent Flow Friction Factor υ = Kinematic Viscosity of Water (ft^2/s) ε = Pipe Roughness Coefficient L = Length (ft) K = Resistance Coefficient	g = Force Due to Gravity (ft/s^2) h_L = Head Loss (ft) P = Pressure (lb/in^2 or psi) z = Elevation γ = Unit Weight of Water (lb/ft^3)

Design Equations

Bernoulli's general energy equation (Refer to equation A-11) is the primary equation governing the design. All other equations were used to calculate the inputs, such as major and minor losses, needed to complete the energy equation. Equation A-11 was rearranged to solve for the amount of head needed to be supplied by a booster pump if a booster pump was needed. Each variable required by equation A-11 will individually discussed in detail. Most of the variables in the equation are needed for two separate points in the system. Point 1 is defined as branch main tie-in location located on West Union Rd. Point 2 is defined as the building tie-in location located on the site.

Velocity

Equation A-1 was rearranged to solve for velocity in the system. This could be done because the required flowrate and the areas of listed pipe ranges are known. The piping does not change in size at any point in the system. Therefore, the velocities at both locations are equal. Velocity is used in more than one equation.

Head Loss due to friction

Head loss due to friction was computed using equation A-4. Solving equation, A-4 requires calculating a friction factor and a Reynold's Number for the pipe. The friction factor was solved using equation A-3. The friction factor depends on the inside diameter of the pipe, roughness coefficient of the pipe (dependent upon pipe material and unit system being used), and the Reynold's number. The Reynold's number is a measurement used to determine whether a fluid is in laminar or turbulent flow. Equation A-2 was used to determine the Reynold's number and depends on inside pipe diameter, kinematic viscosity of the fluid and the velocity of the fluid in the system. The design temperature used for the kinematic viscosity was taken at 32^{o} Fahrenheit as this when water is least viscous.

Head Loss due to fittings

The water supply line fittings consist of butterfly valves, 90^{o} elbows, and tees. Butterfly valves and gate valves were considered for use in the system because these types have the lower head losses compared to other valves. Gate valves has less head losses compared to butterfly valves but were not chosen as they have handles that required multiple turns to close and the handle on these valves could fail easily due to corrosion. Equation A-5 was used to calculate the head loss for each type of fitting. The inputs are velocity, resistance coefficient, and gravity. Equations A-6 through A-9 were used to compute resistance coefficient, instead of converting the fittings to equivalent lengths of pipes, was chosen as this is a are more conservative approach.

Head Supplied by Pump

As previously mentioned, the primary equation used for designing the water supply system is the general energy equation. All the variables in the general energy equation are known except for the head supplied by the booster pump. Equation A-11 was rearranged to solve for the amount of head the pump will need to supply. By observing the values listed in the column h_A pump in APPENDIX A.3, a booster pump is not needed for pipe sizes larger than a 12-inch pipe. Therefore, a 12-inch pipe is the chosen pipe size that the supply lines will be constructed with.

2.9 Wastewater Summary

2.9.1 RSF Summary

The recirculating sand filter is designed to treat 5,183 gpd of wastewater. An attempt was made to get the closest possible design strength of wastewater that the system would receive. After the system is in operation, samples of influent and effluent will have to be taken so the system performance can be measured. Adjustments will have to be made if the actual influent is considerably stronger than the initial wastewater strength estimate. The RSF system is to be equipped with components that allows for adjustments to be made to the number of doses per day and the recirculation ratio. It is required that the number of doses per day stay in the between 24-48 doses per day. The recirculation ratio must remain between 3:1and 5:1. TDEC requires evidence to be submitted if it is determined that the system needs to operate outside of this range. The recirculation tanks are required to have 2 pumps so that any one pump can be maintenance without the system shutting down. The system must be in operation for 3-5 days, depending on the amount of flow the system experiences, before full treatment of the water is performed as bacteria needs time to build up in the sand. Once the system is in operation, dosing should be performed at least once per hours, even if there is zero flow through the system, in order to keep the population of bacteria treating the water alive.

2.9.2 Potable Water Supply Summary

The site location for the I-69 rest area is located in Shelby County, TN. Currently there are no existing water supply systems located in the area. There nearest water municipality in the area is the Millington Water Treatment Plant. Water Supply lines will be constructed and routed to the site by connecting to the MWTP supply main located on West Union Rd. Farmland encompasses the land between the site and water connection. The water supply line will be constructed alongside the I-69 corridor so that the impact on the farmland is minimized. The water supply line shall be buried a minimum of 14 inches below ground level. This ensures that the top of the 12-inch pipe is below the 8-inch frost line in West TN. However, 901 Design recommends that the pipe be buried 36-48 inches below ground level in order to prevent digging type farm equipment from damaging the pipeline. The pipeline will be constructed using a 12-inch pipe. This is to eliminate the need of installing a booster pump in the supply system.

CHAPTER 3. STRUCTURAL

This chapter will discuss the various aspects that goes into the design for a one-story building that will be used as a rest area along the projected I-69. The work to be discussed will include:

- Load combinations that were developed and which load combinations will control for the design of the structure.
- The load path and how it transitions throughout the frame of the structure.
- The methods/procedures that were implemented along with the logic for making any decisions, such as determining span spacing, placing structural bracing, implementing pin vs moment connections, etc.
- The interpretation of the analytical process.
- An overall summary that provides a listing of assigned members to the structural frame.

3.1 Structural Design Process

3.1.1 Preliminary Structure and Floorplan

A preliminary structure was first designed (refer to drawings S.B.1 through S.B.3) before structural members were analyzed. The preliminary structure was assigned structural members and therefore will be analyzed with load combinations developed for this project. Load combinations have been calculated and more details will be provided in section 3.3.

The design of the building and bathroom floor plan took the *International Building Code* (IBC) of 2012 ("Searchable platform for building codes, IBC" n.d.), *International Plumbing Code* (IPC) of 2012 ("Searchable platform for building codes, IPC" n.d.), *International Fire Code* (IFC) of 2012 ("Searchable platform for building codes, IFC" n.d.), and the 2010 *Americans with Disabilities Act* (ADA) ("Searchable platform for building codes, ADA" n.d.) standards into consideration to develop the preliminary structure and bathroom floorplan designs (refer to drawing S.B.9).

The building has been classified according to section 503 of the IBC. 901 Design determined that for this project, the building is classified as follows:

- Group: A-3
- Type of Construction: Type V – B

With the above classification, the building cannot exceed a maximum height of 40 feet or a maximum area of 6,000 ft^2.

From section 1021 of the IBC, the number of exits needed for the building are two. The building may have more but at a minimum, need two exits. The preliminary structure reflects this criterion.

Calculations were done in accordance with the AASHTO book *Guide for Development of Rest Areas on Major Arterials and Freeways* (American Association of State Highway and Transportation Officials 2001) to determine how many urinals and water closets are needed for the bathrooms (refer to Figure 50 in APPENDIX B.10). A total ADT of 35,150 was used and then halved to reflect only the south-bound traffic (given during the TDOT presentation on September 17, 2018).

The bathroom floor plan utilized the IBC, IPC, and ADA to determine dimensions. Aisle widths are in accordance with section 1017.3 from the IBC. Aisles must not be less than 36 inches. Locations for the water closets are in accordance with section 604.2 from the ADA. The centerline of the water closet shall be 17 inches minimum and 19 inches maximum from the side wall. Clearances around the water closets are in accordance with section 604.3 from the ADA. Clearance around a water closet shall be 60 inches minimum measured perpendicular from the side wall and 56 inches minimum measured perpendicular from the rear wall. Wheelchair accessible water closets conform to section 604.8.1.1 of the ADA. Wheelchair accessible compartments shall be 60 inches wide minimum measured perpendicular to the side wall, and 59 inches deep minimum for wall-hung water closets measured perpendicular to the rear wall. Partitions for urinals and water closets are in accordance with section 405.3.1 from the IPC. A minimum of 15 inches is needed from centerline of urinal or water closet to adjacent partitions or walls. There shall be not less than 21 inches of clearance in front of the water closet or urinal. Water closet compartments shall be not less than 30 inches in width and not less than 56 inches in depth for wall-hung water closets. The bathroom floorplan meets this criterion and therefore, a preliminary structure was designed to accommodate the floorplan developed (refer to drawings S.B.1 through S.B.3).

3.1.2 Roof Design

The design for the roof will consist of 7 W6X9 steel beams that run 52 ft in length and will sit atop the trusses of the structure (refer to drawing S.B.5). The roof beams will be the first contact support for the metal roof that will sit atop the roof beams. The roof beams will be set 9.17 ft apart from one another (refer to drawing S.B.5). This spacing should allow enough support to the load being applied to the roof which will transfer to the roof beams, allowing for the maximum deflection to be less than the building requirements stated in the *Steel Design* (Segui, William n.d.) book used to determine deflection.

The selection of W6x9 members was determined using the *Steel Construction Manual* (American Institute of Steel Construction 2017) Table 6-2. The maximum moment allowed by a W6X9 member is 9.8 k-ft, thus controlling the selection. The maximum moment occurring in the critical beam is 7.47 k-ft (refer to Figure 29 in APPENDIX B.6).

3.1.3 Truss Design

Trusses were selected for aesthetic purposes, allowing the roof to be pitched so that natural lighting may be utilized through the truss members. The client wants design features that will allow self-sustainability. Utilizing the truss as windows, and leaving the interior of the building exposed, will allow for natural light to shine in the building. This feature should help reduce lighting costs.

The design of the truss is to help support the roof beams. The vertical components of the truss are aligned to support the roof beams, set at 9.17 ft apart from one another. This will allow the load to be directly transferred from the roof beams to the vertical supports of the truss. The bracing components of the truss are placed in compression to support the vertical components of the truss (refer to the configuration in Figure 36 in APPENDIX B.8).

The truss will be designed using double channel C15X50 with a 3/8 in plate between for connections. A large member is needed to support the 55 ft span of the truss, thus C15X50 was chosen for all members of the truss.

3.1.4 Column Design

The columns for the structure will be W14X48. There will be 5 columns on either side of the structure, spaced at 13 ft. The columns will be supporting the trusses of the frame. The column members were determined by checking flange and web slender compressions (refer to calculations in Figure 44 in APPENDIX B.9). A large enough member was chosen to satisfy criteria allowing for non-slender members. The *Steel Construction Manual* (American Institute of Steel Construction 2017), Table 6-2 was referenced to determine adequate steel members that would satisfy shear, moment, and defection criteria.

3.1.5 Bracing and Connections

The structure was designed without considering bracing from lateral wind loads. Due to time, an analysis was not performed for bracing; however, refer to Figure 51 and Figure 52 in APPENDIX B.11 to see the configuration for the bracing that requires an analysis. The bracing would need to be implemented in the structure of the building to provide the necessary moment support for the frame, without the bracing, the structure would fail due to large moments created from the wind pressure. The configuration for the connections of the bracing for the roof system can be seen in drawing S.B.8.

The analysis done for the structure was based on pin supports on either end of the column, however, after reviewing the analysis it was determined that a fixed connection from the truss to the column would provide the necessary moment support for the structure (refer to drawing S.B.7). Due to time constraints, the analysis was not completed for the correct configuration of the column.

3.2 LEED Considerations

Per the client’s request, one aspect taken into consideration when determining the building material was increasing the LEED rating for the structure. Structural steel is the premier green construction material. It's high recycled content and recycling rate exceed those of any other construction material. Under LEED 2009 and V4 criteria, structural steel receives maximum credit for its contribution to the overall rating for a structure, due in large part to its recycled content, recycling rate and transparency. Structural steel produced in the United States contains 93% recycled steel scrap, on average. At the end of a building's life, 98% of all structural steel is recycled back into new steel products, with no loss of its physical properties. As such, structural steel isn't just recycled but "multi-cycled," as it can be recycled again.

3.3 Load Combinations

The load combinations can be found in APPENDIX B.1 through APPENDIX B.5. The load combinations were developed with the use of the *Minimum Design Loads for Buildings and other Structures*. The design of this structure accounts for the following loads: Dead, Live, Live Roof, Snow, and Wind. The following sections will provide more detail on how each load was determined.

3.3.1 Wind

When determining the wind load, there are two different methods to choose from. The method selected for this structure was the directional procedure (Structural Engineering Institute 2006). Refer to Figure 13 in APPENDIX B.2. The more conservative approach was selected to minimize risk during the design process.

The basic wind speed in Memphis is 115 mph (refer to Figure 14. in APPENDIX B.2). The wind directionality factor K_d is 0.85 (Figure 15. in APPENDIX B.2). Both the surface roughness and exposure category are classified as “C” (Figure 16 in APPENDIX B.2). The topographic factor K_{zt} is 1.0 (Figure 17 in APPENDIX B.2). The gust factor G is 0.85 (Figure 18 in APPENDIX B.2).

The velocity pressure exposure coefficient, K_z (for ground level) and K_h (height at 22.5 feet which is the mid-point of the roof truss height), was determined using table 27.3-1 (refer to Figure 19. in APPENDIX B.2). Linear interpolation was used to obtain K_h.

The velocity pressure exposure values (q_z and q_h, ground level and mid-truss level respectively) can be seen in the wind load calculations excel spreadsheet (refer to APPENDIX B.2). The equation used to determine the values was given in the *Minimum Design Loads for Buildings and other Structures* and can be seen in the spreadsheet (refer to Figure 12 in APPENDIX B.2).

External pressure coefficients (C_p) were determined for both the roof and the side walls of the structure. Figure 20. in APPENDIX B.2 was used in determining the various values for C_p.

Using the values described in this section, a table of pressures was developed for the many different wind loading cases the structure will be subjected to (refer to Figure 12 in APPENDIX B.2). These pressures will be applied to the specific tributary areas on the structure for design purposes.

3.3.2 Snow

The value for the snow loading pressure can be found in Figure 21 in APPENDIX B.3. Figure 22 in APPENDIX B.3, was used to determine the ground snow load. The minimum snow load for low-slope roofs, P_m, was determined using Figure 23 and Figure 24 in APPENDIX B.3. The snow pressure developed from this procedure will be applied to the specific tributary areas on the structure for design purposes.

3.3.3 Live

The live loading pressures (live load and live roof load) were determined using Figure 26 in APPENDIX B.4. These pressures will be applied to the specific tributary areas on the structure for design purposes.

3.3.4 Dead

The dead load values were determined for various tributary areas as well as an overall total dead load for the entire frame, which consists of the roof dead load as well as the dead load from the internal steel members. The total dead load for framing can be found in APPENDIX B.5 refer to Figure 27. However, when applying the load combinations seen in Figure 11 from APPENDIX B.1, the dead load was set to 0 because when the analysis was performed, SAP2000 was used. When using SAP2000, entering specific steel members and running the analysis will account for the dead load condition.

3.4 Load Path

The load path is the direction in which each consecutive load will pass through connected members. The sequence commences at the highest point of the structure working all the way down to the footing system, ultimately transferring the total load of the structure to the foundation. This section will detail the load path of the structure to be designed.

The path begins on the roof of the structure. To support the entire loading of the structure and other loads that the roof will be subjected to, a roofing system needs to be developed. This roofing system will consist of 7 beams that run the length of the structure, sitting on top of the 5 trusses used to construct the frame. This can be seen in drawing S.B.4.

Once the load is transferred from the roof to the beams that support the roof, the load will transition into the trusses of the structure. As mentioned before, there will be 5 trusses that support the structure. The trusses will take the bulk of the loading and will need to be designed appropriately. The load then continues its path and transitions into the columns of the structure. As can be seen in drawing S.B.5, the structure will consist of 10 columns. Finally, the load will transition into the foundation of the building.

3.5 Analysis

This section will discuss the logic and methods used during the analysis of the structure. SAP2000 was used as a tool for the analysis of the design process. All calculations and SAP2000 figures can be found in the appendix (refer to APPENDIX B.6).

3.5.1 Roof Beams

The calculations for the roof beam analysis can be found in Figure 28 and Figure 29 of APPENDIX B.6. To determine which beam is most critical, the tributary area must first be established. Figure 28, shows how the tributary areas were developed. Because the roof is symmetrical, ½ the roof will be analyzed (this half will incorporate the worst-case wind loading conditions). As seen in the calculations, T_2 and T_3, are the greater values for the tributary area. Therefore, beams 2 and 3 (B_2 and B_3) will be recognized as the most critical beams and the SAP2000 figures (Figure 31 - Figure 35) will reflect these beams.

The loading combination condition which controls the design parameters for these critical beams can be seen in Figure 11 of APPENDIX B.1. The value for dead load in that spreadsheet is set at 0 because SAP2000 will apply the weight of the specified beam material when conducting the analysis. SAP2000 was utilized to run an analysis with the specified loading conditions which was applied to the critical roof beam members. The shear, moment, and deflections are shown in Figure 31 - Figure 35 of APPENDIX B.6.

Once the data has been obtained for these critical beams, the values were checked to verify whether the beams were sufficient to withstand the loading condition. The use of *Steel Design* (Segui, William n.d.) and the *Steel Construction Manual* (American Institute of Steel Construction 2017) were used to verify conditions. These values have been verified and are sufficient to use (refer to Figure 29 in APPENDIX B.6).

3.5.2 Trusses

The calculations for the truss analysis can be found in Figure 36 - Figure 39 of APPENDIX B.8. To determine which truss is most critical, the reactions from all roof beams were calculated using SAP2000 (refer to Figure 30 and Figure 33 in APPENDIX B.6). The greatest reactions occur in trusses 2 and 4, as can be identified in the drawing from the calculation done in Figure 28 of APPENDIX B.6. The configuration shown in Figure 36 of APPENDIX B.8 was analyzed using SAP2000. A complete listing of axial forces within the truss can be found in Figure 40 of APPENDIX B.8. The column titled F_{SAP} lists the axial force values for the corresponding numbered member of the truss (refer to Figure 42 in APPENDIX B.8).

The compression members were verified using the Euler buckling model (refer to Figure 41 of APPENDIX B.8). As for the tension members (refer to Figure 37 of APPENDIX B.8) the *Steel Construction Manual* was referenced to determine if the truss was sufficient in the tension members. From the calculations and spreadsheet used for compression and tension verification, the analyzed truss is sufficient for the structure.

3.5.3 Columns

The calculations for the column analysis can be found in Figure 43 - Figure 47 in APPENDIX B.9. The most critical column was analyzed and if proven to be sufficient, then the other columns will be sufficient as well. Columns 2, 4, 7, and 9 were identified as most critical (refer to Figure 43 in APPENDIX B.9). Buckling, slender compression, shear, moment, and deflection were assessed to determine if the column was adequate for the structure.

To determine if the column satisfied the buckling criteria, the *Steel Construction Manual* was referenced, specifically equation E3-2 of the manual (refer to refer to Figure 43 in APPENDIX B.9).

SAP2000 was used to analyze the column for max shear, moment, and deflection. Refer to the configuration shown in Figure 44 in APPENDIX B.9. The load that is applied to the column was determined using the tributary area of the exterior wall that will rest upon the columns. The calculation for the tributary wall can be found in Figure 44 in APPENDIX B.9. The wind pressure (refer to Figure 12 in APPENDIX B.2) was applied to the tributary area and converted to a distributed load which was applied to the column to analyze. The results of the column analysis can be seen in Figure 49 in APPENDIX B.9. Checking these values against the *Steel Construction Manual* Table 6-2 will verify the structural members satisfy the shear, moment, and deflection criteria.

3.6 Structural Summary

The structure will consist of a roofing system (refer to drawing S.B.5), truss members (refer to drawing S.B.6), and columns to support the loading conditions developed for this project. The roofing system will be made up of W6X9 steel members. There will be 7 roof beams that run 52 ft in length and will be connected to the truss members of the structure. There will be 5 trusses to support the roofing system and will be made up of double channels, C15X50, with a 3/8 in plate in between for connections. Each truss will be connected to a W14X48 column on either end of the truss. There will be a total of 10 W14X48 columns to support the trusses. Refer to drawing S.B.4 for the complete configuration of the structure.

CHAPTER 4. GEOTECHNICAL

4.1 Introduction

The geotechnical scope of work, for the I-69 rest area, consisted of a sub-surface soil investigation and a foundation design for the building. An alternative analysis was performed for the interim report to determine which type of foundation would be chosen for the building. The highest scoring foundation of the alternative analysis was chosen for the final design. The interim report also included a boring plan that specified boring locations, depths, and lab tests that would be required to obtain the necessary soil parameters for design of the foundation. The following sections will discuss the soil investigation results, the field and laboratory tests performed, the results obtained from the tests, and the necessary earthwork required to build the foundation. A discussion of the recommended foundation will follow which will include the structural design specifications of the foundation.

4.2 Field Investigation

The boring plan submitted in the interim report, specified that there will be 4 borings located at each corner of the rest area building. The borings will go to a depth of 20 feet beneath the ground surface. Soil samples were recovered by performing the Standard Penetration Test (SPT), and the use of Shelby tubes. Refer to APPENDIX C.1 to view the submitted boring plan.

4.3 Laboratory Testing

The soil samples recovered from the soil borings were classified by lab tests specified in the interim report boring plan. The tests include in-situ water content test, sieve analysis and Atterberg limits test. The in-situ water content test is a measure of the soils water content in field conditions. The water content is essential for computing the soils dry unit weight (γ_{dry}) and void ratio (e_o). The sieve analysis obtains the soils gradation and the Atterberg Limits obtains the soils liquid and plastic limits. Both the gradation and Atterberg limits are necessary for the Unified Soil Classification System (USCS). Additional laboratory testing includes the one-dimensional consolidation test, and the unconfined compressive strength test. Both previously stated tests were performed using undisturbed soil samples recovered by Shelby tubes. The consolidation test allows the calculation of the compression index (C_c), and the recompression index (C_r or C_s) which are necessary to calculate soil settlement. The Unconfined compressive strength test will be performed to measure the undrained shear strength (S_u) of normally consolidated and slightly over consolidated cylindrical specimens of cohesive soil. The undrained shear strength (S_u) obtained from the unconfined compressive test is used to estimate the bearing capacity of spread footings and other structures when placed on deposits of cohesive soil. The completion of the previously described tests allows the engineer to size a foundation based on bearing capacity and settlement.

4.4 Discussion of Field and Laboratory Test Results

Test results obtained from the field and laboratory test are shown in the boring logs located in APPENDIX C.2. The four boreholes show there are two different soil strata that are located underneath the building foundation. The soil stratum closest to the surface is brown clayey silt (CL-ML), and the soil stratum below the previously mentioned is mottled brown and tan silty clay (CL). The boreholes located at the southeast and southwest corners of the building indicate a 10 ft. thickness of each soil stratum. This stratum combination will be referred to in later sections as combination 1. The boreholes located at the northeast and northwest corners of the building indicate the thickness of 5 ft. for the brown clayey silt, and 15 ft. for the mottled brown and tan silty clay. This stratum combination will be referred to in later sections as combination 2. The variation in strata thicknesses is an indication for possible differential settlement and must be addressed in the foundation design. The ground water level (GWL) is located 18.5 ft. below the ground surface and is deep enough to not have an impact on the foundation design. The soil parameters used for designing the foundation are shown in APPENDIX C.4. The unit weights (γ_{moist}) were determined for each stratum by taking the average value for each of the two soil strata. The soils N-values were computed by summing the last two increments obtained from the Standard Penetration Test (SPT). The N-values were used to get the soils effective friction angle (ø’). The diagram used to obtain the effective friction angle is from the EPRI soil manual located in APPENDIX C.5. The effective friction angles from each soil stratum was then averaged to get one value per soil strata. The void ratio was computed by performing a phase relationship. The phase relationships were based off the computed average unit weights and the average water content for each soil stratum.

4.5 Foundation Recommendation

An alternative analysis was performed in the interim report that examined three different types of foundations. These foundations include a slab on grade, continuous wall spread footing, and a deep foundation. The slab on grade foundation rated highest for ease of constructability, time to complete construction, overall construction cost, and required site preparation work. The following sections will summarize how the dimensions of the slab on grade foundation was determined and the structural design of the foundation.

4.5.1 Foundation Summary

The slab on grade foundation was sized by performing a primary consolidation analysis and a bearing capacity analysis. Elastic settlement will not be considered due to the foundation preparation work that will be discussed in section *4.6.2*. The slab on grade foundation is unique for the slab and the supporting beams being cast together in one concrete placement. The surface area of slab will not be considered for the settlement or bearing capacity calculation. The supporting beams dimensions will be the only structure analyzed for settlement and bearing capacity. Only analyzing the beams will result in minor forces acting on were the slab and beams meet. Previously stated in section 4.4, half of the building will sit on combination 1 soil strata and the other half will sit on combination 2 soil strata. For this situation, the entire foundation was analyzed as if it were placed on each soil combination independently. Analyzing each soil strata combination separately will give insight on any possible differential settlement.

4.5.2 Building Loads

The foundation settlement and bearing capacity calculation were analyzed using the Allowable Strength Design (ASD) loads provided by the structural engineer. The ASD loads reflect the weight of the frame including the roof, live loads, and vertical forces due to wind. The ASD load that will be applied to the foundation is 2.316 kips (231,600 lbs.).

4.5.3 Settlement

Settlement of the foundation was analyzed using the primary consolidation formulas indicated in APPENDIX C.6. The soil stratum closest to the surface is an over consolidated clay and was evaluated using the over consolidated settlement equation. The lower soil stratum is normally consolidated and was analyzed using the normally consolidated settlement equation. The 2:1 method was used to find the change in stress at the center of each clay stratum applied by the load of the building. The total settlement for both clay strata in Combination 1 is 0.225 in. This settlement value results in a safety factor of 4.44. The total settlement for both clay strata in combination 2 is 0.279 in. This settlement value results in a safety factor of 3.58. These resulting values represent 9 in. wide beams that are 19 in. in depth. The allowable settlement for the structure is 1 in., so each scenario satisfies the allowable settlement requirements. The foundations supporting beams are laid out in a grid pattern that is similar to grade beams. Grade beams are placed to resist differential settlement. With these circumstances, differential settlement will not be a concern and will not be evaluated due to the slight variance in settlement between combination 1 and combination 2.

4.5.4 Bearing Capacity

The building foundation will be placed on fine grain soils. For this reason, the foundation was analyzed using effective stress analysis (ESA) and total stress analysis (TSA). The Terzaghi's bearing capacity equations used for ESA and TSA are shown in APPENDIX C.7. The most conservative value between ESA and TSA was used to determine if the foundation beams were sized appropriately. Using the dimensions stated in section *4.5.3*, the foundation will transfer 708.2 psf. to the soil directly beneath the foundation. The ESA value was shown to be the more conservative value. The cohesion parameter in the ESA equation was assumed to be zero to represent the worst-case scenario. With a safety factor of 4, the allowable bearing capacity for the soil is 5138.8 psf. This resulting value shows the soil will be more than adequate for supporting the building and foundation.

4.6 Preliminary Earth Work

4.6.1 Site Clearing

The site of the I-69 rest area currently sits on farmland that contains corn crops. Before the construction of the building foundation starts, the area must be cleared. The existing vegetation will be removed and replaced with more stable materials. The clearing of vegetation is imperative to reduce the chances of increased settlement.

4.6.2 Site Compaction

The foundation site will be compacted after the vegetation has been cleared. The compaction will ensure the foundation will not fail due to immediate settlement. For this design a pre-compression technique will be used. This involves pre-loading the soil where the foundation will be placed. The loading force will be applied by soil brought in from an offsite location. To get a load comparable to the weight of the building, 243 cubic yards of soil will be placed were the foundation will be built. The applied soil load will be left in place for 1 month and removed before construction begins.

4.6.3 Cut and Fill

The first 6 in. of soil will be removed to ensure all vegetation roots and top soil will not compromise the foundation. The total cut for the slab foundation is 55 cubic yards. This cut will be filled with ¾ in. crushed stone that will act as the slab's drainage layer. Water underneath the slab can induce unwanted stresses on the slab during freeze thaw cycles. The purpose of the crushed stone is to keep water from collecting directly underneath the slab to mitigate the effects of the freeze thaw cycles. The crushed stone will be compacted to a range of 95-100% compaction. With the drainage layer placed, the trenching for the beams will be completed. The total cut for the beam trenches is 26 cubic yards. This value represents all 8 of the foundations supporting beams.

4.7 Water Proofing & Forming

Once the beam trenching is complete, the exterior beam forming will be constructed. The forms will be constructed out of plywood sheets that are braced at the top and bottom. The plywood bracing will be secured to wooden stakes driven into the ground. Forming will only be necessary for the outside perimeter of the exterior beams. With exterior beam forms in place, the waterproof membrane will be installed. WRI specifies that either 6 mil poly or hot-mopped asphalt impregnated felt is used for weatherproofing. The weatherproofing should be lapped adequately to act as one continuous sheet under the entire slab. This design will use hot-mopped asphalt impregnated felt because it is less susceptible to being damaged during the installation process.

4.8 Structural Design

International Building Code (IBC) 2009 requires the design for all slab on grade foundations to follow the Wire Reinforcement Institutes (WRI) design guidelines. The calculations and figures shown in APPENDIX C.8, display the WRI methods used to size the slab and beam reinforcing.

4.8.1 Concrete

WRI design manual requires the compressive strengths for concrete slab on ground foundations to have a minimum of 2500 psi at 28 days. This design reflects the use 2500 psi concrete.

4.8.2 Beam Reinforcement

The moments for the beams in the long and short directions of the foundation were calculated following the WRI design guidelines. The moment generated in the beams in the long direction is 79.78 k-ft. The moment generated in the beams in the short direction is 82.57 k-ft. These moments were used to size the rebar that will be located in the top and bottom of the slabs supporting beams. Additional reinforcing is needed where the exterior beams tie into the interior beams. For the exterior beam tie in's, the reinforcement is sized from the reinforcement that will be counteracting the moments. The larger bar size between the top and bottom beam reinforcement will be used for the tie in reinforcement. The exterior beam tie ins are detailed in APPENDIX C.9.The beam reinforcing summary is shown below.

Long Direction Beams

- 4 – 9" x 20" x 56' beams, reinforced with 2 #4 bars on bottom, and 2 #3 bars on top.

Short Direction Beams

- 4 – 9" x 18" x 53' beams, reinforced with 2 #5 bars on bottom, and 2 #4 bars on top.

Stirrups

- All beams will have #3 bar stirrups placed at 21" OC.

4.8.3 Slab Reinforcement

The slab thickness for this design will be 4 in. This is the minimum thickness recommended by WRI. The slab will be reinforced by welded wire reinforcing. The benefits of using welded wire reinforcing is that it will save on labor cost, and construction time. Using Figure 11. in APPENDIX C.8, the required area of steel per linear foot of this slab was determined. The required area of steel per linear foot is 0.05 sq.in./LF. The required area will be satisfied by using W5 welded wire reinforcement. The American Concrete Institute (ACI) specifies a 2 in. lap between welded wire reinforcing is required.

4.9 Summary

The foundation for the I-69 rest area will be a slab on grade design. The results of the settlement and bearing capacity analysis show that the soil will support the foundation with minimal settlement and soil deformation. Refer to drawings S.C.1 - S.C.4 for beam and slab reinforcement detail.

CHAPTER 5. TRANSPORTATION

This chapter will discuss all the elements of design pertaining to the Transportation section. This discussion includes the explanation of the elements, rationale for the design, related literature and official requirements which govern the design. The elements of design are listed as follows:

- Entrance Ramp
- Exit Ramp
- Car Parking Area
- Truck Parking Area
- Inner Parking Roadway
- Signage and Marking
- Miscellaneous Item
- Self-Sustaining Building: A Truck Smart Parking Approach

All the designs are based on the specification given by the Tennessee Department of Transportation (TDOT) Standard Roadway Design Guidelines (TDOT 2017). Refer to the guidelines of TDOT located in APPENDIX D.10 to APPENDIX D.17. If the information from TDOT is not sufficient, the guidelines given by the American Association of State Highway and Transportation Officials (AASHTO) in the book *A Policy on Geometric Design of Highways and Streets* (herein referred to as Green Book) (AASHTO 2011) will be consulted. The related information located in the Green Book are shown in the calculations of APPENDIX D.

5.1 Design of Entrance Ramp to the Rest Area

The design of the ingress ramp can be considered like the design of a single lane free flow terminal freeway exit. The term free flow terminal freeway exit refers to the section located adjacent to the through traffic highway which facilitates the diverging traffic at a specified flat angle (AASHTO 2011). The design can be categorized further as either multilane or single lane. With the given information from TDOT of the demanding traffic flow, as specified by the 30 years projected average annual daily traffic, 901 Design determines that a single lane ingress ramp would be enough to handle such traffic. With only one lane necessary for diverging traffic into the rest area, a taper-type exit is chosen because of the following reasons:

- It is applicable for one-lane ramp only
- It coincides with the driver’s preferred path of diverging

- It requires fewer resources in terms of cost, time of construction, and human labor compared to parallel type
- It is suitable for low traffic volume

Section 10.9.6 of the book *A Policy on Geometric Design of Highways and Streets* gives specific guidelines about the design of a free flow terminal taper type exit ramp of which the ingress ramp design is based on (AASHTO 2011). The following information discusses each element of design that is applicable for the ingress ramp. Refer to APPENDIX D.5 at page 117 for the calculations of the entrance ramp.

5.1.1 Design Speed

The design speed of the ingress ramp can be determined based on the existing highway design speed. AASHTO (2011) gives guidance on determining ramp design speed based on the type of ramp configuration and adjacent highway speed in Table 10-1: Guide values for Ramp Design Speed as Related to Highway Design Speed. Refer to APPENDIX D.5 at page 117 for the relationship. The ingress ramp can be categorized as a ramp for right turns with a low diverging angle. Therefore, the upper range of ramp design speed is applicable in this scenario. Because the highway design speed is 70 mph as specified by TDOT, the ramp design speed is determined to be 60 mph. This ramp design speed is necessary for the calculation of the length for the deceleration lane and the value of entrance ramp speed limit sign.

5.1.2 Deceleration Lane

The deceleration lane should provide enough length for vehicles especially large trucks to safely decelerate from the current highway speed to the speed limit of the parking lot. The length of deceleration lane is a function of which variables are the design speed limit of the existing highway and the design speed limit at the end of the ingress ramp or the parking area speed limit. These two design speeds are calculated to be 70 mph and 20 mph respectively. AASHTO (2011) gives guidance on determining the length of deceleration in Table 10-5: Minimum Deceleration Lengths for Exit Terminals with Flat Grades of Two Percent or less. Refer to APPENDIX D.3 at page 115 for the calculation for the length of deceleration lane. From this table, a minimum length of 570 ft is required for the deceleration lane and 901 Design determines the length of deceleration lane be 580 ft. The guidance for measuring the length of deceleration lane is as followed: "The length available for deceleration may be assumed to extend from a point where the right edge of the tapered wedge is about 12 ft from the right edge of the right through lane to the point of initial curvature of the ramp" (AASHTO 2011). Refer to the drawing S.D.3 for the details dimension of the deceleration lane.

5.1.3 Diverging Angle, Cross Slope, and Diverging Area

The diverging angle of the tapered entrance ramp should be in the range of 2 to 5 degree (AASHTO 2011). The choice of the diverging angle will affect the distance from the existing highway to the rest area and the total length of the entrance ramp needed to achieve such distance. 901 Design chooses the upper limit of 5 degrees to maximize the distance from the parking area to the existing highway and minimize the length of the entrance ramp which ultimately yields a more safe and economical design. The area of diverging is specified from the start of the right edge of the tapered wedge to the painted nose of the gore area. With a diverging angle of 5 degrees and a width of a driveway of 16 ft for entrance ramp, 901 Design specifies this distance to be 183.6 ft which is sufficient for drivers to diverge safely.

The entrance ramp road width is a function of which variables are the following elements: traffic condition, radius on the inner edge of the pavement, and type of curb/shoulder (AASHTO 2011). First, the rest area serves a high proportion of trucks and recreational vehicles. Therefore, the number of large vehicles is high enough to govern the design and can be classified as traffic condition C. Second, the entrance ramp is designed as a tangent ramp. Third, an 8 ft shoulder ramp are provided on the right edge of the pavement. From these statistics, a 14 ft entrance ramp width is recommended (AASHTO 2011). Refer to APPENDIX D.5 at page 117 for the calculation of road width. In addition, TDOT (2017) suggests a 16 ft driveway for entrance one-lane ramp. Refer to APPENDIX D.11 at page 123 for this guidance. Because TDOT's driveway width guidance is larger than the Green Book limit and 901 Design's prior local guidance, a 16 ft entrance ramp width is selected.

The taper entrance ramp cross slope shall be consistent to the adjacent highway (AASHTO 2011). The proposed I-69 has a constant 2% downslope toward the right shoulder as specified by TDOT. In order to maintain a slope of 2% toward the edge of the right pavement measured relative to the road alignment, the slope recommended for construction of the diverging area is slightly different from the normal 2%. Refer to drawing S.D.4 for the construction guideline of this diverging area and APPENDIX D.5 at page 117 for the calculation of the cross slope.

5.1.4 Superelevation

According to the specification of the highway cross section provided by TDOT (2017), the normal slope of this proposed I-69 highway is 2% in the 24 ft driveway downward to the shoulder. This slope value is also applied to the deceleration lane. On the other hand, TDOT (2017) also specifies for inner roadway parking cross-section with a normal crown of 2% downslope from the centerline toward the curb and gutter. Refer to APPENDIX D.10 and APPENDIX D.12 at page 122 and 124 for TDOT guidelines for these two cross-sections. The deceleration lane is connecting these two cross-sections. In order to accommodate this difference in the driveway slope, a superelevation runout and runoff is needed. AASHTO (2011) gives guidance on developing a superelevation profile based on design speed, initial and target slopes. Refer to the APPENDIX D.2 at page 113 for calculation of the superelevation profile and drawing S.D.4 for detailed dimensions of the superelevation profile.

5.1.5 Road Cross Section and Widening

TDOT (2017) specifies the deceleration lane width to be 16 ft with a 6 ft shoulder on the left side and an 8 ft shoulder on the right side and the inner parking roadway width to be 22 ft. In order to accommodate the difference in road width, a widening section is needed. AASHTO (2011) suggests a tapering/widening ratio of 1:35 for a critical section such as the highway entrance ramp. However, because the widening area within this project is located at the end of the entrance ramp and can be considered less critical, a widening ratio of 1:30 is utilized. Refer to the drawings S.D.3 and S.D.4 for detailed dimensions of road cross-sections and widening area.

5.1.6 Entrance Ramp Gore Area

AASHTO (2011) specifies the term gore nose as the conjunction area between diverging ramp shoulder and the existing highway ramp shoulder. The width of the gore is specified to be at least 2 ft and located 2 ft away from the diverging ramp and 12 ft away from the existing highway. The recovery area of the gore is defined as the tapering of the pavement measured from the gore nose (AASHTO 2011). AASHTO (2011) gives guidance on determining the ratio based on the Table 10-2: Minimum Length of Taper Beyond an Offset Nose. With a highway design speed of 70 mph which yields a tapering ratio of 35, a 12 ft highway shoulder, and a 6 ft ramp shoulder, the highway and ramp pavement taper lengths are calculated to be 420 ft and 70 ft respectively. The landscaping area shall be located 12 ft away from the edge of pavement of existing highway and 6 ft away from the edge of pavement of entrance ramp and a landscaping nose dimension is specified to be 6 ft (AASHTO 2011). From these dimensions, 901 Design calculates the distance from shoulder gore nose to the landscaping nose to be 132.6 ft. Refer to the drawing S.D.3 for details of the gore area and APPENDIX D.6 at page 118 for calculations of it.

5.2 Design of Exit Ramp from the Rest Area

The design of the exit ramp from the rest area is like the single lane entrance ramps. 901 Design utilizes the design of parallel entrance ramp because it provides the following advantages:

- It provides a safer way of merging traffic compared to taper type entrance ramp.
- It provides sufficient sight distance for both on-coming highway and merging traffic.
- It provides longer merging area compared to taper type entrance ramp which facilitates the process of merging to the Interstate I69 for vehicles from the rest area.

AASHTO (2011) gives guidance on the design of parallel entrance ramps in Section 10.9.6 of the Green Book. The exit ramp is divided into 3 different elements for different purposes which are given the name Exit Ramp 1, Exit Ramp 2, and Exit Ramp 3 respectively. Refer to the drawing S.D.1 and S.D.2 for the geometric division of these exit ramp.

5.3 Design of Exit Ramp 1

The Exit Ramp 1 consists of two tangent T1 and T2 and one curve C1. Tangent T1 is designed for the following purposes. First, it provides an easement for trucks leaving the truck parking area. WSDOT (2012) recommends at least 100 ft of easement alignment beyond truck parking area. Combined with the inner parking roadway Road 3.2, Tangent T1 yields a 200 ft for the truck easement. Second, it provides an easement for the merging of cars from Road 3.1 into one roadway Exit Ramp 1. Third, it provides sufficient area to taper the road width from 22 ft to 16 ft. 901 Design selects the ratio of tapering to be 1:30 because of the less critical nature of the section in order to shorten the length. Refer to the drawing S.D.6 for detailed dimensions of the tapering area.

Curve C1 is designed to facilitate the change in direction from the rest area toward the existing highway and the superelevation runout length due to the difference in cross slope between inner roadway (2% normal crown) and acceleration lane (superelevated to 12% downslope). A parking lot design speed of 20 mph is used for calculating the radius for curve C1, refer to APPENDIX D.1 at page 110 for detail calculations of the horizontal alignment. In addition to Curve C1, Tangent T2 provides additional runout length because the difference in cross slope between the parking area and Exit Ramp 2 is considerably high that the required runout length exceed the length of curve C1. Refer to drawing S.D.6 and APPENDIX D.2 at page 113 for detailed dimensions and calculations of this superelevation profile.

5.4 Design of Exit Ramp 2

The Exit Ramp 2 consists of only a Curve C2. “A curve with a radius of 1,000 ft or higher can be considered as an acceleration length” (AASHTO 2011). In order to minimize the total alignment length while simultaneously providing enough length for vehicle acceleration, 901 Designs specifies the Curve C2 to have a radius of 1,100 ft in order to achieve both objectives of facilitating the change in direction (at least two curves are required for traffic merging from the rest area to I-69) and providing sufficient acceleration length. The superelevation from the previous Exit Ramp 1 will be carried onto the Exit Ramp 2 and completed at the station of 0+94 ft. Refer to drawing S.D.7 and APPENDIX D.2 at page 113 for the dimensions and calculations of the superelevation. TDOT (2017) gives guidelines for the cross-section of superelevated exit ramp which in this project is specified to be a 16 ft driveway with 8 ft shoulder on the higher side and 6 ft shoulder on the lower side. This configuration is slightly different compared to the entrance ramp and further attention is needed for the construction of it.

5.4.1 Gore Area

Compared to the entrance ramp, there are fewer requirements for the gore of the Exit Ramp 2. The gore nose is constructed as the nose of landscaping area with a width of 2 ft separating the 12 ft shoulder of the Interstate and the 6 ft shoulder of the exit ramp (AASHTO 2011). Refer to the drawing S.D.8 for the detail dimension of the gore area.

5.4.2 Tapering Section

The Exit Ramp 2 width starts at 16 ft driveway from the beginning point. However, a 12 ft width at the end of the exit is desirable to facilitate the uniformity in width with the acceleration lane connected to it. AASHTO (2011) requires a tapering ratio of 1:35 for this section given the critical nature of it. Refer to the drawing S.D.8 for specific dimensions of the tapering section.

5.5 Design of Exit Ramp 3

5.5.1 Length of Parallel Deceleration Lane

The Exit Ramp 3 consists of a parallel acceleration traffic lane adjacent to the existing highway, so traffic can safely merge into. This acceleration lane combined with the Curve C2 in Exit Ramp 2 shall yield a total length long enough to sufficiently facilitate the act of merging for incoming traffic. This length is a function of which variables are the initial design speed of the ramp which is 20 mph and the final design speed which is 70 mph I-69 design speed. AASHTO (2011) gives guidance on determining the length for acceleration based on initial and final design speed in Table 10-3. Minimum Acceleration Lengths for Entrance Terminals with Flat Grades of Two Percent or less. Based on this table, the minimum acceleration length is 1,520 ft and 901 Design specifies this length to be 1,576 ft. Refer to the drawing S.D.9 and APPENDIX D.4 at page 116 for the dimensions and calculations of the acceleration lane.

5.5.2 Tapering Area

A minimum of 300 ft in length of a tapering area beyond the parallel acceleration lane is recommended which is sufficient for a design speed up to 70 mph. However, since the acceleration lane's length is larger than 1,300 ft a uniform taper ratio of 50:1 to 70:1 is suggested (AASHTO 2011). Therefore, a ratio 50:1 is selected for this design. Refer to the drawing S.D.10 and APPENDIX D.4 at page 116 for detail dimensions and calculations of the Exit Ramp 3.

5.5.3 Superelevation

The superelevation is located at the beginning of the Exit Ramp 3. It facilitates the transition for a 12% superelevated cross section of Exit Ramp 2 due to the nature of a high-speed curve for accelerating and normal 2% downslope of I-69. However, because the highway I-69 right side edge of pavement's elevation is fixed, the adjacent left side of the parallel acceleration lane is also fixed at that elevation. Therefore, the superelevation is classified as rotating about the outside edge and its profile control is the left side edge of the parallel acceleration lane. Refer to drawing S.D.10 and APPENDIX D.2 at page 113 for the superelevation profile.

5.6 Design of Car Parking

The alternative analysis within the Interim Report has concluded the characteristics of the car parking lot, which optimize the safety and economical aspect, as follow: a 70-degree angular parking, homogeneous one-way traffic, and parking along the curbside layout. The total number of parking spaces are 140 and it is divided into three sections located around the main building area, each has 60, 80, and 60 car parking spaces respectively. Refer to the drawing S.D.11 for the layout of the car parking lot. The aspect of the design of the car parking lot based on these characteristics is listed in the following sections.

5.6.1 Parking Stall Dimension

The car parking lot of the rest area can be categorized as a high turnover rate parking area because vehicle operators spend less usage time of the facilities, which yields a shorter length of time between pulling in and pulling out of parking lot, compared to other types of buildings such as office building or school. Therefore, the parking stall should be designed in such a way that it facilitates the easiness of pulling into and out of the parking lot. ITE (1994) defines the term parking class which measure this easiness of maneuvering within car parking lot. Because of this high turnover rate nature of the rest area parking lot, a parking class of A is required and the parking stall shall be designed in such manner to achieve this standard of parking class A (ITE 1994). ITE (1994) gives guidance of the parking stall dimension based on the desirable parking class and parking angle in Table 12.10: Parking Module Layout Dimension Guidelines in the Guidelines for parking facility location and design book. 901 Design specifies the design cars to be Large Passenger Cars for a conservative approach. Refer to APPENDIX D.8 at page 120 for the dimensions of the design car. Therefore, the car parking shall satisfy the following constraints:

- Provides a minimum stall width of 9 ft
- Provides a stall depth to interlock of 17.5 ft
- Provides an aisle width of 22 ft

In addition to the guidance of ITE (1994) on determining the aisle width, the minimum aisle width based on a desirable parking angle as calculated by the Ricker Equation (Ricker 1957) will be compared to double check if any modification is necessary. The aisle width is an important aspect of the car parking lot because it facilitates the turning movement into the parking stall. The aisle width shall be large enough so that its turning radius is larger than the required minimum turning radius as defined by AASHTO (2011). Other parking stall dimensions can be mathematically derived from the stall width and stall depth to interlock. Refer to APPENDIX D.8 at page 120 and drawing S.D.11 for dimensions and calculations of the car parking lot.

5.6.2 Accessible Parking Requirement

Car parking area shall provide a certain number of accessible car parking spaces and van accessible car parking spaces based on the aggregate sum of car parking spaces as defined in the 2010 ADA Standards for Accessible Design (herein referred as ADA Standards) (Department of Justice 2010). Refer to APPENDIX D.7 at page 119 for calculations of the number of accessible parking space. With a car parking of 140 lots, 901 Design determines that 6 accessible car parking lots and 1 van accessible car parking lot are required. Two accessible car parking lots will share the same accessible aisle which then leads to a perpendicular curb ramp heading to the sidewalk. The aisle width for car and van accessible parking aisle are 5 ft and 8 ft respectively. Refer to drawing S.D.15 for dimensions and details of the accessible parking lot. The first two car accessible parking lots are located in the middle of Car Parking lot 1, an accessible car parking paired with a van accessible parking are located in the middle of Car Parking 2, and the last two car accessible parking spots are located in Car Parking 3. Because the main building is located in the middle area of the main area, this layout minimizes the average distances from the accessible parking lot to the main area to facilitate the movement of disabling individual

5.6.3 Cross Section

The aisle has a driveway of 22 ft with a normal crown 2% downslope from centerline toward the curb and gutter to facilitate drainage as recommended by WSDOT (2012). The left side of the aisle is extended to 20.5 ft to accommodate the car parking stall. Refer to the drawing S.D.14 for the cross-section of the car parking area.

5.7 Design of Truck Parking

From the calculations in the Interim Report, a total of 35 truck parking spaces are sufficient for the rest area. The design vehicle is specified as an Interstate Semi Trailer WB-20 or WB-67 truck. A truck parking angle of 30-degrees is desirable because it facilitates the parking practice of pulling in and through for large trucks (PADOT n.d.). The angular parking accounts for only 30 truck parking spaces for the driver with low turnover rate. The remaining 5 parking spaces will be designed as truck aisle parking for the driver with high turnover rate. This area also includes a fire truck lane. The exit of the truck parking lot is extended by 100ft to provide a superelevation runoff because of the difference in cross slope (2% downslope of truck parking versus 2% normal crown of inner roadway). Refer to drawing S.D.12 for the layout and dimensions of truck parking.

5.7.1 Truck Parking Stall Dimension

The design of the truck parking stall dimension is consulted by the guidelines provided by the PADOT (n.d.) and the WSDOT (2012). The larger dimension within one element of design between the two institutions is selected for a conservative approach since the rest area is mainly used by long-distance truck drivers. After the comparison between the two guidelines, the following dimensions for truck parking is determined:

- A Parking of angle of 30-Degrees
- An entrance/exit road width of 22 ft
- A Stall width of 15 ft
- A Stall length of 100 ft

The truck aisle parking stall shall have a dimension of 135 ft long and 16ft wide (AASHTO 2001). This also includes a fire lane for in case of accidents. These 5-truck parking aisles will be located parallel to the aisle and adjacent to the main building area.

5.7.2 Turning Radius for Truck

In addition, aisles will be located at the beginning and end of the truck parking area to provide spaces for drivers who decide to pull directly through the parking area. The angle between the aisle and the pavement shall be chamfered to accommodate the movement of turning for long trucks. WSDOT (2012) gives guidance on determining the radius for this chamfered section. The radii of the chamfering sections for entrance and exit are 85 ft and 100 ft respectively. In addition, the aisle width shall be big enough so that its turning radius is larger than the minimum required turning radius for the WB-67 truck. These radii and chamfered area radius are double-checked with the minimum turning radius as defined by AASHTO (2011). Refer to the drawing S.D.12 for the description of the chamfered area and APPENDIX D.8 at page 120 for the calculation of truck turning radius.

5.7.3 Cross Section

The truck parking area cross section consists of multiple parts. The first part is the entrance aisle with a width of 22 ft and a normal crown 2% downslope from the centerline. The second part is the main truck parking area of 50 ft in width and 2% successive downslope from the entrance aisle width. The third part is the exit aisle of 22 ft in width and also a 2% successive downslope from the parking area. The cross-section design of the truck parking area is consulted by the guidance given by WSDOT (2012).

5.7.4 Fire lane Requirement

The truck parking area also provides spaces for a fire truck in case of an accident should occur. Space shall be large enough to accommodate a fire truck with a dimension of 47 ft long, 8 ft wide, and a curb-to-curb turning radius of 40 ft (University of Houston 2014). Therefore, the truck parking is designed to have a length of 135 ft, a width of 16 ft and it is located on the sidewalk side of the truck parking area. Refer to the drawing S.D.12 for dimensions of the area.

5.8 Design of Inner Roadway

Inner roadway within the parking lot shall be designed to facilitate the traffic flow within the parking lot which can be considered homogenous, slow, steady, and low in volume. In addition, because there is a lot of pedestrian movement within the rest area, the design speed shall be set low enough to provide a safe and friendly environment for pedestrians. 901 Design consults the school speed zone as developed by TDOT (2018) in Guidance on Setting Speed Limits to set the parking speed limit of 20 mph.

The alignment shall not have any sudden turning angle because it would pose a potential hazard for drivers and disrupt the homogeneous circulation of traffic. Where turning is necessary, it shall be provided with a curve to smoothly guide the vehicle through corners. A superelevated curve is desirable especially in high-speed roadways (AASHTO 2011). Because the speed limit of the parking lot is only 20 mph, a superelevated horizontal alignment is not necessary and a normal crown of 2% slope is sufficient. The turning radius can be determined based on the design speed and the slope of superelevation (AASHTO 2011). An assumption that cross sections are superelevated to the 2% slope is made during the calculation of the turning radius. Refer to APPENDIX D.1 at page 110 for calculation of horizontal alignments.

The inner roadway consists of multiple alignments. An intersection is defined as the conjunction between two alignments. The angle created by the pavement edge of two alignments shall be chamfered to provide sufficient space, so the vehicle can diverge or merge safely. The radius of the chamfered area is often referred to as a curb-return radius which facilitates the turning movement of passenger cars. This radius shall be in the range of 15-40 ft as shown in APPENDIX D.17 at page 129 (TDOT 2017). In addition, the radius will be checked with a minimum design turning radius in APPENDIX D.8.

5.9 Miscellaneous Item

5.9.1 Curb and gutter

Curb and gutter are mainly used in the inner roadway. It can be considered less expensive while still providing most of the utilities of a shoulder such as defining clear driveway section and providing drainage capability. 901 Design consults several layouts of curb and gutter (TDOT 2017) and a 6 in combined curb and gutter is selected. Refer to the drawing in S.D.18 and APPENDIX D.13 at page 125 for the dimension of curb and gutter.

5.9.2 Signage

Signage is used to guide the circulation of traffic within the parking lot and prohibit unintended traffic movement. Refer to drawing S.D.17 and S.D.16 for the location and detailing of these signs. The following Table 6. Signage Description and Usage shows the description and usage of these sign.

Table 6. Signage Description and Usage

Road	Sign Name	Description	Usage
Interstate I-69	D5-1	Rest Area in 1 mile (next rest area in 100 miles)	Notice drivers of the upcoming rest area and the distance to the adjacent one if they decide to skip it
Interstate I-69	D5-1a	Rest Area Next Right	Second notice for driver
Interstate I-69	D5-2a	Rest Area	Guidance on the direction of diverging to the rest area
Entrance Ramp	W13-3	Ramp 60MPH	Notice driver of ramp design speed limit
Entrance Ramp	R8-3a	No Parking	Avoid parking on shoulders of trucks which creates a potential hazard for incoming traffic
Road 1.1	W1-2	Road Curve Sign	Notice of upcoming changing in directions
Road 1.1	D1-2d	Car/Truck Destination Guide	Split car and truck traffic into their respective place
Car Parking 1	R2-1	20MPH Speed Limit	Set speed limit for car parking area
Car Parking 1,2,3	R7-8	Accessible Parking	Define accessible car parking stall
Car Parking 2	R7-8a	Van Accessible	Define accessible van parking stall
Road 3.2	R1-2	Yield Sign	Caution car drivers of merging into the existing truck's exit aisle
Interstate I-69	W4-1	Merging Sign	Caution the upcoming interstate traffic of merging vehicle
Interstate I-69	W4-2	Lane end	Caution traffic of upcoming tapering area

In addition, the pavement marking of the gore merging and diverging area are designed based on the recommendation of TDOT (2017). Refer to APPENDIX D.16 at page 128 for the specification for pavement marking in these areas.

5.9.3 Sidewalk

Sidewalks are used to facilitate the movement of pedestrians from the parking lot toward the main building. 901 Design consults several layouts of sidewalks (TDOT 2017). A 6 ft in width and 1.5% downslope for drainage is selected for the rest area. In addition, the layout of the sidewalk within the rest area is designed in such a way that it balances two objectives of minimizing pedestrian walking distances and minimizing total construction length of the sidewalk. Refer to the drawing S.3 for the layout of the sidewalk. In addition, perpendicular curb ramps are located adjacent to the accessible parking lot to minimize the travel distance of wheelchair individuals.

Refer to APPENDIX D.14 at page 126 for the dimensions of perpendicular curb ramp as specified by TDOT (2017).

5.9.4 Level of Service of Weaving, Merging, and Diverging

Weaving refers to an act of crossing other traffic paths/lanes in order to get to the desired location along the length of the facility. This type of movement is commonly seen in ramp interchanges and may cause potential disruption to the traffic. The rest area is located near interchanges Wilkinsville and West Union Road. The length between the interchanges of Wilkinsville road and the entrance ramp of the rest area is 1,740 ft. The length between the interchanges of West Union road and the exit ramp of the rest area is 1,795 ft. With a projected 30-year annual daily traffic of 35,000 veh/day, this length may not be sufficient to facilitate the weaving movement of traffic. The term Level of Service is an assessment criterion developed by Transportation Research Board (2010) to quantitatively defines the performance of a certain section of an interstate such as interchanges. The measurement is based on several factors such as the geometric of the roadways (number of lanes, road width), incoming flow, and traffic characteristic (speed, the percentage of truck…). A Level of Service F determines that the facility is in a congested condition and therefore is not desirable. Refer to the APPENDIX D.9 at page 121 for the studies of the level of service. 901 Design determines that the number of lanes for proposed I-69 is not enough for the projected 30-year traffic.

5.10 Self-Sustaining Truck Parking: An ITS Smart Park Approach

5.10.1 Introduction

The truck parking demand along interstates is immense within recent years due to the following reasons. First, the traffic traveling along interstates has been growing in recent years as recorded by the 11% increase of average annual amount of travel per Interstate Lane-mile from the year 2000 to 2014. Vehicle travel miles, which is also a parameter representing the travel demand, increases by 14% within the same period. Within this increase of traffic, the category of freight traffic experiences the sharpest growth of 29% more vehicles (Mohamed Osman, Ph.D., P.E. 2018). Second, corporations are now forcing tighter delivery schedules which as a result, forces truck drivers to travel longer distances. Third, drivers must stop, park, and take a rest after an extended period of driving because the federal government regulates the hours of driving. The truck parking demand is so heavy that it exceeds the capacity of some certain rest areas. Some fatigue related accidents are associated with the inadequacy of truck parking spaces. If there are not enough parking spaces, drivers will park on the shoulder of entrance or exit ramp which is extremely dangerous, illegal and creates potentially fatal crashing hazards. This section introduces a technology using sensors and computer algorithms to inform the drivers of remaining truck parking spaces at a certain point of time. This technology will herein be referred as Smart Park. Smart Park allows truck driver to track real time available parking spaces so they can plan on the most appropriate rest areas among several options. This will avoid the condition of one rest area being overcrowded while the other rest areas do not operate at their full capacity. Smart Park is an effort to achieve the criteria of Self-Sustaining Building and Intelligent Transportation System as requested by the owners. The Smart Park can operate without human interference once the facilities are installed, the rest area can be considered self-sustaining.

Disclaimer: The scope of the Smart Park project is immense and this report will only cover the basic elements. In addition, the civil site layout is designed to facilitates the implementation of Smart Park. This means that there are reserved spaces to install the facilities needed for this technology in the future.

5.10.2 Methodology and Implementation Approach of Smart Park

The procedure for implementing Smart Park consists of two phases which are discussed in the following sections.

Phase 1: Asserting the current condition of commercial vehicle parking trends

The objective of phase 1 is to determine whether this rest area location is worth implementing Smart Park based on historic data. Phase 1 will follow the following steps:

Step 1: Identify interstates segment of consideration

The portion of the interstate I-69 from West Union Road to Walker Avenue is selected for the study of asserting the traffic condition.

Step 2: Collecting data

Personnel at the rest area will record the peak number of truck parking within a day of the rest area. The number of truck parking shall be categorized as legal or illegal. Legal parking refers to the parking of truck at the dedicated truck parking stall. Illegal parking refers to the parking of truck at unauthorized areas such as entrance ramp and exit ramp. The aggregate sum of legal parking and illegal parking is the number of truck parking. In addition, the ratio of number of truck parking over the rest area's capacity, herein referred to as utilization ratio, will be recorded. This utilization ratio asserts whether the facility is overcrowded. A value of less than 1 indicates that the facility is operating as normal and Smart Park will not be necessary. A value greater than 1 indicates that the facility is overcrowded, and the implementation of Smart Park is necessary.

Phase 2: Implementing Smart Park

After the confirmation that the rest area is eligible for the implementation of Smart Park in phase 1, phase 2 will install the facilities needed into the rest area. The facilities can be categorized as either hardware and software. The following sections discuss the description of these hardware as well as its mechanism and installation.

The hardware facilities consist of sensor nodes, relay nodes, and an on-site data collector. Sensor nodes are imbedded underground of each truck parking stall to detect whether there is a truck over it. A relay node will then collect the data from adjacent sensor nodes. The relay nodes are often located above the pavement with a different in elevation of 10 ft or more. Refer to the drawing S.D.20 for the installation of the sensor and relay nodes. A data center located on-site will connect the relay node's information and transfer/archive it to the cloud database.

The software consists of a cloud database storing historic data of truck parking at various point of time. In general, truck drivers prefer the number of truck parking spaces at near future, such as 15 minutes from the movement they request the information, over the real time number of truck parking spaces. Truck drivers usually plan their schedule before pulling into a rest area and the real time data does not provide the necessary information which is the number of parking spaces when they get there in a short period of time. Therefore, a prediction model based on the historic data is developed to interpolate the number of truck parking space at some certain point in the near future.

This prediction model can also be categorized as a software facility. The algorithm used in this model is The Kalman filter. The historic data are the initial points of which the interpolation model is based on. In the beginning, the algorithm may not be accurate due to the limited data. However, as more data is collected, the prediction model will be automatically updated, and the mechanism can be described as a feedback loop. If the real time data (as recorded by the hardware facilities) deviates from the predicted data, adjustment will be made to the prediction model.

5.10.3 Conclusion of Smart Park

Smart Park technology, if implemented correctly, can reduce the potential circumstances of overcrowded rest area. It helps to avoid the parking along the entrance and exit ramp in the case of truck parking demand exceeds the rest area’s capacity.

5.11 Pavement Design

5.11.1 Design ESAL's

Refer to the following Table 7 for the design variables used in this section:

Table 7. Design Variables

ADT = Average Daily Traffic
T = Percent Trucks
T_f = Truck Factor
D = Direction Distribution
L = Lane Distribution

The design ESAL's was determined using equation (Pavement-1). The ADT was determined from the traffic data provided from Dr. Osman. The truck percentage was determined using line B2 in Figure 9. The truck factor was determined by the composition of the types of truck classes. It assumes that the rest area will see wide array of trucks and buses. The direction and lane distribution factors were set to 1. This is because there is only one lane in and out of the rest area and the rest area is serving only southbound traffic. The results for the design ESAL's are listed in Figure 10.

Traffic Data	A= 1-way, design year,ADT		17575
	B= Ration of design hourly volumes to ADT		
	B1 Cars, Generally=15%	%*A=	2636
	B2 Trucks, when ADT < 12,500 = .15, when ADT > 12,500 = .1	%*A=	1758
	C-Traffic Composition in percent (from counts or estimates below)		
	C1 Cars (generally 75-89% of total traffic)	%*B1=	1977
	C2 Cars with trailers or RV's (generally 4-9% of total traffic)	%*B1=	105
	C3 Trucks (generally 7-16% of total traffic)	%*B2=	123
	D= Vehicles per hour stopping at rest area		
	D1	%*C1	237
	a- Near commercial or metro area, 9%		
	b- Typical rural route, 12%		
	c- Information and Welcome Centers, 9-15%		
	D2 Cars with trailers, 9-15%	%*C2	16
	D3 Trucks, 9-15%	%*C3	18

Figure 9. Traffic

Design ESALs =	365(ADT)(T)(Tf)(D)(L)(G)
(ADT) =	17575
(T) =	0.15
(Tf) =	1.1
(D) =	1
(L) =	1
(G) =	1.00
Design ESALs =	1,058,454

Figure 10. Design ESAL's

5.11.2 Layer Thicknesses

The pavement was designed using PAIKY's Pavement Design Table shown in Table 8. This table is based on the AASHTO 1993 pavement design equation. The table's design is based on an 80% reliability. The soil for the site has a CBR of 5. The 8 million design ESAL column with ADT< 24,000 was used for design as this column is the only one that meets both design parameters for the site. The asphalt surface should be 1.25 inches. However, the minimum lift thickness for asphalt wearing course is 1.5 inches. The asphalt base should be 7.5 inches. The lift thickness range for base is 4-6 inches. This requires the pavement to be constructed with a 4-inch lift and a 3.5-inch lift. The aggregate layer is 6 inches and can be constructed with one lift. The pavement requires a tac coat layer after the first 4-inch base layer and one after the 3.5-inch layer.

Table 8. PAIKY Design Table

PAIKY Pavement Design Table (AASHTO 1993)
Heavy Duty Traffic Applications

Traffic Characteristics	Autos (92%), Single Unit Trucks (5%), and Combination Trucks (3%)		
Estimate ESALs	2,000,000	4,000,000	8,000,000
Average Daily Traffic	< 6,000	< 12,000	< 24,000
	CBR Value = 1.0 (Soil Stabilization Recommended)		
Asphalt Surface Thickness (in)	1.25	1.25	1.25
Asphalt Base Thickness (in)	9.50	10.00	11.00
Aggregate Thickness (in)	8.00	10.00	12.00
	CBR Value = 2.0 (Soil Stabilization Recommended)		
Asphalt Surface Thickness (in)	1.25	1.25	1.25
Asphalt Base Thickness (in)	7.50	9.00	10.00
Aggregate Thickness (in)	6.00	6.00	6.00
	CBR Value = 3.0		
Asphalt Surface Thickness (in)	1.25	1.25	1.25
Asphalt Base Thickness (in)	6.50	7.50	9.00
Aggregate Thickness (in)	6.00	6.00	6.00
	CBR Value = 4.0		
Asphalt Surface Thickness (in)	1.25	1.25	1.25
Asphalt Base Thickness (in)	6.00	7.00	8.00
Aggregate Thickness (in)	6.00	6.00	6.00
	CBR Value = 5.0		
Asphalt Surface Thickness (in)	1.25	1.25	1.25
Asphalt Base Thickness (in)	5.50	6.50	7.50
Aggregate Thickness (in)	6.00	6.00	6.00
	CBR Value = 6.0		
Asphalt Surface Thickness (in)	1.25	1.25	1.25
Asphalt Base Thickness (in)	5.00	5.50	6.50
Aggregate Thickness (in)	6.00	6.00	6.00
	CBR Value = 7.0		
Asphalt Surface Thickness (in)	1.25	1.25	1.25
Asphalt Base Thickness (in)	4.50	5.50	6.50
Aggregate Thickness (in)	6.00	6.00	6.00
	CBR Value = 8.0		
Asphalt Surface Thickness (in)	1.25	1.25	1.25
Asphalt Base Thickness (in)	4.00	5.00	6.00
Aggregate Thickness (in)	6.00	6.00	6.00
	CBR Value = 9.0		
Asphalt Surface Thickness (in)	1.25	1.25	1.25
Asphalt Base Thickness (in)	3.75	4.75	5.75
Aggregate Thickness (in)	6.00	6.00	6.00
	CBR Value = 10.0		
Asphalt Surface Thickness (in)	1.25	1.25	1.25
Asphalt Base Thickness (in)	3.50	4.50	5.50
Aggregate Thickness (in)	6.00	6.00	6.00
	CBR Value = 11.0		
Asphalt Surface Thickness (in)	1.25	1.25	1.25
Asphalt Base Thickness (in)	3.25	4.25	5.25
Aggregate Thickness (in)	6.00	6.00	6.00

5.12 Summary

Chapter 5 discusses the transportation design of the rest area. The following are the element of design: entrance ramp, exit ramp, car parking, truck parking, and inner roadway. These designs are based on the guidance of the Green Book and TDOT Standard Roadway Specification. In addition, a level of service analysis of the segment concludes that the facility may not provide the sufficient infrastructure for the 30-year projected traffic. Chapter 5 also includes an overview for a self-sustaining solution for the truck parking area, which is Smart Park. The technology has a potential of avoiding overcrowded rest areas, avoiding illegal truck shoulder parking on entrance and exit ramps, and providing truck drivers valuable information of near future truck parking supply. In addition, this chapter also discusses the design of pavement based on the design table PAIKY Pavement Design Table (Refer to Table 8) developed by AASHTO.

CHAPTER 6. WATER RESOURCES

6.1 Introduction

This chapter will discuss the plan 901 Design has for tackling the drainage of the rest area on the proposed I-69. The work to be discussed in the following sections will include:

- The drainage characteristics and peak discharge of the area before development.
- The design storm chosen as the basis for all calculations.
- The change in drainage characteristics after development.
- The drainage plan for required runoff
- Any extra drainage plans.

6.2 Pre-Development Drainage

The sub-surface soil investigation was conducted after the interim report was submitted. It gave information regarding the soil strata found on the site. The design storm was chosen by TDOT's standards for ditch design. The table from TDOT's drainage manual is Table 9 in APPENDIX E.1. This information was used in calculations to help understand the drainage characteristics of the land pre-development.

6.2.1 Soil Types

The test results from the sub-surface soil field tests and laboratory tests are the first indicator of what the land characteristics are like. There are two different soil strata found on the site, brown clayey silt and tan silty clay. The Tennessee Department of Transportation (TDOT) has a drainage manual, and it is the guideline for creating the drainage for this rest area. In APPENDIX E.1 there are three tables to help understand the hydrologic conditions of the soil. These three tables came from the TDOT Drainage Manual. Table 10 in APPENDIX E.1 is for deciding the hydrologic soil group, and this is where knowing the existing soil types is important. Table 11 and Table 12 in APPENDIX E.1 are used to decide the curve number for the site. The soil retention pre-development is 1.24-inches.

6.2.2 Runoff

The runoff curve number is an empirical parameter used in hydrology for predicting direct runoff or infiltration from rainfall excess. The curve number is 89 for pre-development. According to TDOT, the design storm that needs to be used is the 50-year rain fall event. The 50-year, 24-hour rainfall depth is 7.41 inches. The 3-day 50-year rainfall event is a rain fall depth of 22.23

inches. In APPENDIX E.1 the CN method was used to determine the pre-development runoff. The pre-development runoff is 20.81-inches deep for the 8-acre area.

6.3 Post-Development Drainage

6.3.1 Effects by Development

When developing an area, there are certain aspects such as pavement and buildings that create impervious areas. The impervious areas create more runoff and no place for it to go. Each area has its own curve number to help determine the runoff. For the parking lots, the soil classification is still D and using Table 12 in APPENDIX E.1 it shows that the curve number is 98. The building area gets the same curve number. The soil retention capacity decreased post-development by 0.39-inches.

6.3.2 Runoff

In order to calculate the drainage post-development, the composite curve number has to be calculated. In APPENDIX E.1 all of the CN method calculations and equations are shown for the post-development runoff. The equations used are the CN composite equation, the soil retention capacity equation, and the runoff equation.

6.4 Drainage Plan

According to TDOT's Drainage Manual it is required that either the first inch of runoff for the entire site or the runoff from a 3-day, 50-year storm event, whichever is greater. To be conservative in the design, the 3-day, 50-year storm even was chosen.

6.4.1 Ditch Design

The ditch design aspect of runoff is mostly to be conservative with the design. The ditches run along the existing interstate. They are for any overflow of the retention areas and may rarely be used for the design storm runoff. In APPENDIX E.2 the ditch design equations and calculations can be seen. All the design of the ditches was dictated by the codes in chapter 5 of TDOT's drainage manual.

6.4.2 Retention Design

To achieve the criteria required by TDOT, three retention areas were designed to collect runoff from the impervious areas. The inlets in the parking lots were designed to slope toward the nearest retention pond at a 1% slope. The retention areas are all located on the outside of each of the three parking areas. In APPENDIX E.3 the retention sizing and equations for piping can be seen. The retention areas have been designed to handle the whole amount of runoff for the design storm.

6.5 Other Drainage Plans

Although TDOT only requires that the 50-year, 3-day storm, the design of the retention areas can handle more than the design storm required. Since there was a lot of unused land for this rest area, we decided to maximize the size of the retention areas and connect them to the ditches for overflow. This will contain storm events greater than the design storm.

6.6 Summary

The water resources design consists of 3 retention areas on the outside of the 3 parking areas and ditches. The ditches run along the whole interstate section in the rest area land. There are 3 drains on each parking area that drain into the retention areas. Extra space was created to hold more than the amount of runoff for the design storm.

CHAPTER 7. COST ESTIMATES

The cost estimates developed for this projected used the various RSMeans data books associated with each aspect of the project. The following sections provide details regarding the development of the estimated cost, both construction and design. Please refer to 0and APPENDIX F.6for a detailed breakdown of each cost associated with the project.

7.1 Environmental Cost Estimate

7.1.1 Potable Water Supply

The total cost of constructing the potable water supply lines is $93,000. An itemized cost list can be found in APPENDIX F.1. The cost estimate was done using RSMeans catalog. The total cost was adjusted by the local Memphis factor. The

7.1.2 Recirculating Sand Filter

Cost analysis for the recirculating sand filter was not performed. 901 Design was unable to detail all of the internal components of the system.

7.2 Structural Cost Estimate

The estimated cost for the items related to the structural design utilized *Assemblies Cost Data* and *Building Construction Cost Data* from the RSMeans collection that was provided by the University of Memphis Civil Engineering Department. The cost associated with the structural estimate includes: internal steel members, external non-load bearing concrete walls, curtain windows, bolts, welding plates, and roof material.

Many line items include overhead and profit into the pricing. For those items without overhead and profit, 901 Design will incorporate their own 10% overhead and profit into the final price. Refer to APPENDIX F.2 for details concerning the structural cost estimate. The final estimated structural cost, after adjusting for the local city index, is $176,500.00.

7.3 Geotechnical Cost Estimate

The estimated cost for the items related to geotechnical design utilized *Heavy Construction Cost Data* from the RSMeans collection that was provided by the University of Memphis Civil Engineering Department. The cost associated with the geotechnical estimate includes: surveys, geotechnical investigations, concrete forming, concrete accessories, reinforcement bars, fabric and grid reinforcing, cast in place concrete, concrete cutting, clearing and grubbing, excavation and fill. The costs displayed in APPENIX F.4 includes material, labor, mobilization, and material hauling costs. The final cost including the local adjustment is $88,173.00.

7.4 Transportation Cost Estimate

The total pavement cost is $934,000. The estimate was prepared using the RSMeans catalog. Included in the estimate is 1 lift of asphalt surface course, 2 lifts of asphalt base, 1 lift of aggregate base and 2 layers of tac coat. The area of paved surfaces is 17,627 yd^2. Refer to APPENDIX F.4 for the calculations.

7.5 Water Resources Cost Estimate

The estimated cost for the items related to the drainage design utilized *Heavy Construction Cost Data* from the RSMeans collection that was provided by the department. The cost estimate includes all thing required for creating the drainage design such as excavation, parking lot inlets, pipes, material hauling, and more. Refer to APPENDIX F.5 for the water resources detailed cost estimate. The final estimated water resources cost is $555,500.00 after the local adjustment.

7.6 Estimated Design Cost

901 Design strives to work 12 hours per week per individual. The hourly rate 901 Design charges for design work is $100/hour. This hourly rate was discussed and developed during a lecture for Senior Design, advised by Dr. Arellano. Refer to APPENDIX G.1 for details concerning the hours associated with each individual and their hours spent on the project. The final cost for design is $69,500.00.

CHAPTER 8. SUMMARY

This report presents the design of a rest area that is to be constructed along the proposed I-69 interstate. This report provides design recommendations from the various civil engineering aspects associated with the project. The following chapter summarizes each aspect of the project, providing an overall summary of the design work performed by 901 Design. Refer to drawing S.3 to see the overall site layout proposed for the project.

8.1 Wastewater Treatment

8.1.1 Potable Water Supply

The site location for the I-69 rest area is located in Shelby County, TN. Currently there are no existing water supply systems located in the area. There nearest water municipality in the area is the Millington Water Treatment Plant. Water Supply lines will be constructed and routed to the site by connecting to the MWTP supply main located on West Union Rd. Farmland encompasses the land between the site and water connection. The water supply line will be constructed alongside the I-69 corridor so that the impact on the farmland is minimized. The water supply line shall be buried a minimum of 14 inches below ground level. This ensures that the top of the 12-inch pipe is below the 8-inch frost line in West TN. However, 901 Design recommends that the pipe be buried 36-48 inches below ground level in order to prevent digging type farm equipment from damaging the pipeline. The pipeline will be constructed using a 12-inch pipe. This is to eliminate the need of installing a booster pump in the supply system.

8.1.2 Recirculation Sand Filter

The recirculating sand filter is designed to treat 5,183 gpd of wastewater. An attempt was made to get the closest possible design strength of wastewater that the system would receive. After the system is in operation, samples of influent and effluent will have to be taken so the system performance can be measured. Adjustments will have to be made if the actual influent is considerably stronger than the initial wastewater strength estimate. The RSF system is to be equipped with components that allows for adjustments to be made to the number of doses per day and the recirculation ratio. It is required that the number of doses per day stay in the between 24-48 doses per day. The recirculation ratio must remain between 3:1and 5:1. TDEC requires evidence to be submitted if it is determined that the system needs to operate outside of this range. The recirculation tanks are required to have 2 pumps so that any one pump can be maintenance without the system shutting down. The system has to be in operation for 3-5 days, depending on the amount of flow the system experiences, before full treatment of the water is performed as bacteria needs time to build up in the sand. Once the system is in operation, dosing should be performed at least once per hours, even if there is zero flow through the system, in order to keep the population of bacteria treating the water alive.

8.2 Structural Summary

The structure will consist of a roofing system (refer to drawing S.B.5), truss members (refer to drawing S.B.6), and columns to support the loading conditions developed for this project. The roofing system will be made up of W6X9 steel members. There will be 7 roof beams that run 52 ft in length and will be connected to the truss members of the structure. There will be 5 trusses to support the roofing system and will be made up of double channels, C15X50, with a 3/8 in plate in between for connections. Each truss will be connected to a W14X48 column on either end of the truss. There will be a total of 10 W14X48 columns to support the trusses. Refer to drawing S.B.4 for the complete configuration of the structure.

The client also asked for this project to meet LEED requirements. Structural steel is the premier green construction material. It's high recycled content and recycling rate exceed those of any other construction material. Under LEED 2009 and V4 criteria, structural steel receives maximum credit for its contribution to the overall rating for a structure, due in large part to its recycled content, recycling rate and transparency. Structural steel produced in the United States contains 93% recycled steel scrap, on average. At the end of a building's life, 98% of all structural steel is recycled back into new steel products, with no loss of its physical properties. As such, structural steel isn't just recycled but "multi-cycled," as it can be recycled again.

8.3 Geotechnical Summary

The foundation site will undergo a pre-loading supplied by 243 cubic yards of soil that will last for 1 month. The soil used for the pre-loading phase will be removed before construction begins. The foundation will be a slab on grade design. The slab dimensions are 56’x53’x4”. The dimensions of the 4 beams in the short direction will be 53’x9”x18”. The short beams will be reinforced with 2 #5 rebar on bottom and 2 #4 rebar on top. The dimensions of the 4 beams in the long direction will be 56’x9”x20”. The long beams will be reinforced with 2 #4 rebar on bottom and 2 #3 rebar on top. The exterior beam tie ins will require additional reinforcing. The 4 corner beam tie ins will require an addition of 8 #5 sticks of rebar and 8 #4 sticks of rebar. The 8 T beam tie ins will require an addition of 16 #5 sticks of rebar. The slab will be reinforced with 6’x6’ W5 welded wire reinforcing with a 2” lap. The slab will be placed on a 6” drainage layer of ¾” crushed stone compacted to 95%. Between the slab and drainage layer will be hot-mopped asphalt impregnated felt weatherproofing.

8.4 Transportation Summary

The transportation has completed the design of the following elements: entrance/exit ramp, car parking lot, truck parking lot, and the analysis of Level of Service. The analysis and design are based on the guidance of the book A Policy on Geometric Design of Highways and Streets by AASHTO (2011), the Tennessee Department of Transportation Standard Drawing Library, and the Highway Capacity Manual 2010. The pavement design is discussed in the next paragraph.

The pavement consists of a 1.5-inch asphalt surface layer, a 7.5-inch asphalt base layer, and a 6-inch aggregate base layer. The pavement requires a design to support 1,000,000 ESAL’s with an ADT of 17,500. The only design that meets both criteria in the design tables is to design for 8,000,000 ESAL’s with an ADT ranging between 12,000-24,000 vehicles. The pavement is over designed in terms of loading design needs but the thickness of each layer for the over design is only a couple of inches, so it doesn’t impact cost that much. In fact, it can be negligible when considering the accuracy of construction.

8.5 Water Resources Summary

The water resources design consists of 3 retention areas on the outside of the 3 parking areas and ditches. The ditches run along the whole interstate section in the rest area land. There are 3 drains on each parking area that drain into the retention areas. Extra space was created to hold more than the amount of runoff for the design storm.

8.6 Cost Estimate Summary

The total cost associated with the project is $1,916,000.00. This includes both the design work and construction costs. Please refer to 0and APPENDIX G.1 for details concerning cost estimates. Please note the estimates have been rounded to the appropriate values in accordance with the RSMeans data information.

CHAPTER 9. BIBLIOGRAPHY

AASHTO. (2001). *Guide for development of rest areas on major arterials and freeways*.

AASHTO. (2011). *A Policy on Geometric Design of Highways and Streets*. American Association of State Highway and Transportation Officials.

American Association of State Highway and Transportation Officials (Ed.). (2001). *Guide for development of rest areas on major arterials and freeways*. American Association of State Highway and Transportation Officials, Washington, D.C.

American Institute of Steel Construction. (2017). *Steel construction manual*.

Department of Justice. (2010). “2010 ADA Standards for Accessible Design.” 279.

ITE. (1994). “Guidelines for parking facility location and design.” Institute of Transportation Engineers.

PADOT. (n.d.). “Chapter 9 Safety Rest Areas and Welcome Centers PADOT.”

Ricker, E. R. (1957). “TRAFFIC DESIGN OF PARKING GARAGES.”

“Searchable platform for building codes, ADA.” (n.d.). *UpCodes*, <https://up.codes/viewer/ada> (Oct. 18, 2018a).

“Searchable platform for building codes, IBC.” (n.d.). *UpCodes*, <https://up.codes/viewer/tennessee/ibc-2012> (Oct. 18, 2018b).

“Searchable platform for building codes, IFC.” (n.d.). *UpCodes*, <https://up.codes/viewer/tennessee/ifc-2012> (Oct. 19, 2018c).

“Searchable platform for building codes, IPC.” (n.d.). *UpCodes*, <https://up.codes/viewer/tennessee/ipc-2012> (Oct. 19, 2018d).

Segui, William. (n.d.). *Steel Design*.

Structural Engineering Institute (Ed.). (2006). *Minimum design loads for buildings and other structures*. ASCE standard, American Society of Civil Engineers/Structural Engineering Institute, Reston, VA.

TDOT. (2017). “Standard Roadway Design Guidelines.” <https://www.tn.gov/tdot/roadway-design/standard-drawings-library.html> (Nov. 19, 2018).

TDOT. (2018). “Guidance on Setting Speed Limits.” 65.

Transportation Research Board. (2010). “Highway Capacity Manual 2010.” <http://hcm.trb.org/?qr=1> (Nov. 25, 2018).

University of Houston. (2014). “Campus Design Guidelines and Standards, Section 9.0 Parking Lot Design Standards.”

WSDOT. (2012). “Design Manual WSDOT- July 2012.”

Etzel, J. (1981). *Treatment of Sanitary Wastes at Interstate Rest Areas*. Purdue University, West Lafayette, IN.

Mott, R. L., and Untener, J. A. (2015). *Applied fluid mechanics*. Pearson, Boston.

“Sand Filters.” (n.d.). <https://engineering.purdue.edu/~frankenb/NU-prowd/sand.htm> (Nov. 19, 2018).“Technology_Fact_Sheet_Recirculating_Media_Filters_EPA.pdf.” (n.d.).

Snowden, W. (1981). *Design of Slab on Ground Foundation.* Wire Reinforcement Institute, Hartford, CT.

Das, B. M., and Sivakugan, N. (2017). *Fundamentals of Geotechnical Engineering*. Pearson, Boston.

APPENDIX A. WASTEWATER TREATMENT

APPENDIX A.1 Potable Design Equations

$$Q = vA \qquad \text{A-1}$$

$$N_R = \frac{vD}{\upsilon} \qquad \text{A-2}$$

$$f = \frac{0.25}{\left[log\left(\frac{1}{3.7\left(\frac{D}{\varepsilon}\right)}+\frac{5.74}{N_R{}^{0.9}}\right)\right]^2} \qquad \text{A-3}$$

$$h_{L\,friction} = f * \frac{L}{D} * \frac{v^2}{2g} \qquad \text{A-4}$$

$$h_{L\,fittings} = K_{fitting}\left(\frac{v^2}{2g}\right) * quantity_{fittings} \qquad \text{A-5}$$

$$K_{valve} = 45f \qquad \text{A-6}$$

$$K_{90} = 50f \qquad \text{A-7}$$

$$K_{T\,through\,run} = 20f \qquad \text{A-8}$$

$$K_{T\,through\,branch} = 60f \qquad \text{A-9}$$

$$h_{fire\,hydrant} = \frac{P}{\gamma_{water}} \qquad \text{A-10}$$

$$\frac{P_1}{\gamma_{water}} + \frac{v_1{}^2}{2g} + z_1 + h_{pump} - \Sigma h_L = \frac{P_2}{\gamma_{water}} + \frac{v_2{}^2}{2g} + z_2 \qquad \text{A-11}$$

$$Volume_{total\,pumped} = flow + (R.R.* flow) \qquad \text{A-12}$$

$$Run\,Time_{pump} = \#\,of\,doses * R.R. \qquad \text{A-13}$$

$$Flowrate_{pump} = \frac{Volume_{total\,pumped}}{Run\,Time_{pump}} \qquad \text{A-14}$$

$$Loading\,Rate_{organic} = Flow(mgd) * BOD * 8.34 \qquad \text{A-15}$$

$$S.A._{filter\,bed} = \frac{Flow}{Loading\,Rate_{Hydraulic}} \qquad \text{A-16}$$

$$Length_{filter\,bed} = 2 * Width_{filter\,bed} \qquad \text{A-17}$$

$$Width_{filter\,bed} = \sqrt{\frac{S.A._{filter\,bed}}{2}} \qquad \text{A-18}$$

APPENDIX A.2 Potable Water Results

Q=	3.3425	ft^3/s
L=	6748	ft
ε=	8.00E-04	ft
g=	32.2	ft/s
ν=	1.89E-05	ft^2/s

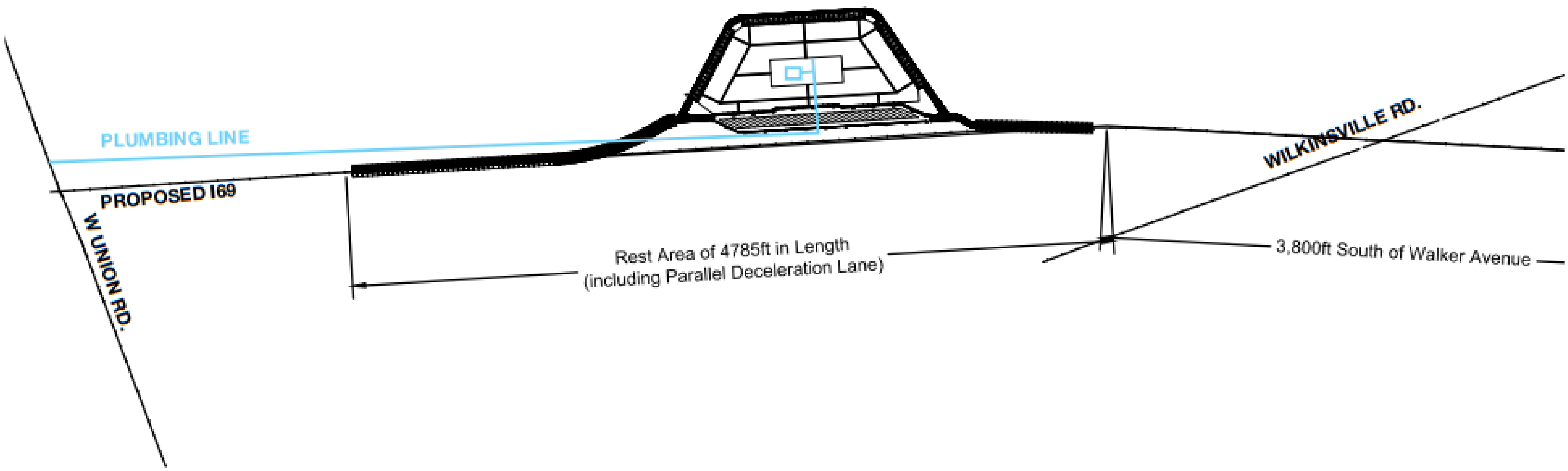
Table 8.2 Mott & Utnert

$h_{A\ pump}$ (ft)	P_1 (psi)	ϒ (lb/ft^3)	v_1 (ft/s)	z_1 (ft)	P_2 (psi)	v_2 (ft/s)	z_2 (ft)	g (ft/s^2)
209.2	72	62.4	8.64	273	35	8.64	303	32.2
40.6			5.61			5.61		
-9.8			3.92			3.92		
-27.7			2.91			2.91		
-35.5			2.22			2.22		
-39.2			1.75			1.75		
-41			1.42			1.42		

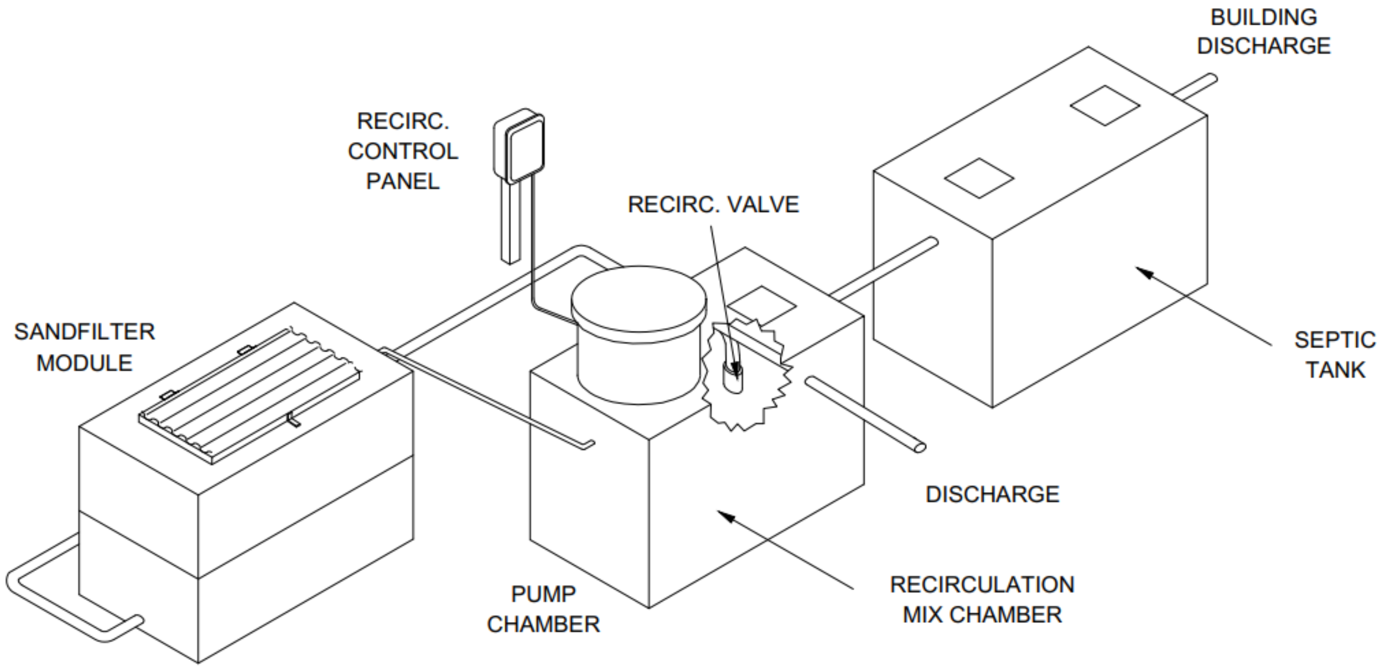
Size	Flow Area (ft^2)	Velocity (ft/s)	I.D. (ft)	NR	*f*	$h_{L\ friction}$ (ft)	K_{valve}
8	0.387	8.64	0.702	321050	0.021	237	0.956
10	0.596	5.61	0.871	258627	0.021	77.9	0.925
12	0.854	3.92	1.043	216092	0.02	31	0.905
14	1.15	2.91	1.213	186540	0.02	14.5	0.893
16	1.505	2.22	1.384	162633	0.02	7.4	0.886
18	1.905	1.75	1.558	144638	0.02	4.1	0.883
20	2.348	1.42	1.729	130229	0.02	2.4	0.882

$h_{L\ Valve}$ (ft)	$K_{90\ fittings}$	$h_{L\ 90\ fittings}$ (ft)	$K_{T\ fittings}$	$h_{L\ T\ fittings}$ (ft)	$h_{L\ fire\ hydrant}$ (ft)
6.66	1.06	4.93	3.82	4.44	11.54
2.71	1.03	2.01	3.7	1.81	
1.29	1.01	0.96	3.62	0.86	
0.7	0.99	0.52	3.57	0.47	
0.41	0.98	0.3	3.54	0.27	
0.25	0.98	0.19	3.53	0.17	
0.17	0.98	0.12	3.53	0.11	

APPENDIX A.3 Plumbing Plan



APPENDIX A.4 RSF System



RECIRCULATING SAND FILTER SYSTEM

SCOPE: HOUSEHOLD SEWAGE WILL FLOW BY GRAVITY THROUGH A TREATMENT UNIT, TYPICALLY A SEPTIC TANK, TO THE SANDFILTER FEED TANK. FROM THE SANDFILTER FEED TANK THE EFFLUENT IS PUMPED TO THE SANDFILTER WHERE, AFTER TREATMENT, IT FLOWS BY GRAVITY TO A RECIRC VALVE WHERE ALL OF THE EFFLUENT WILL BE SENT BACK TO THE RECIRCULATION TANK AND THE OVERFLOW TO DISPOSAL.
IF SURFACE DISCHARGE, CHLORINATE THE EFFLUENT AND FLOW BY GRAVITY THROUGH A SERIES OF CHLORINE CONTACT CHAMBERS. THE FINAL CONTACT CHAMBER WILL ALSO ACT AS A SAMPLE MODULE. FROM THE CONTACT CHAMBERS THE EFFLUENT WILL FLOW BY GRAVITY TO A DISCHARGE POINT.

APPENDIX B. STRUCTURAL

APPENDIX B.1 Load Combination for Most Critical Roof Beam

Loads (kips)	
D	0
L	0
L_r	10
S	4.85
W	11.12
E	0

Building Dimensions	
Length (ft)	55
Width (ft)	52
Height (ft)	18.00

Roof Area (ft^2)	
A_r	2860

Wall Area (ft^2)	
A_{north}	990
A_{south}	990
A_{east}	936
A_{west}	936

Tributary Area (ft^2)	
T1:T3	242
T2	485

LRFD (kips)		
1.4D	1	0.00
1.2D + 1.6L + 0.5 (L_r or S or R)	2	4.85
1.2D + 1.6(L_r or S or R) + (L or 0.5W)	3	21.08
1.2D + 1.0W + L + 0.5(L_r or S or R)	4	15.97
1.2D + 1.0E + L + 0.2S	5	0.97
0.9D + 1.0W	6	11.12
0.9D + 1.0E	7	0.00

ASD (kips)		
D	1	0.00
D + L	2	0.00
D + (L_r or S or R)	3	9.70
D + 0.75L + 0.75(L_r or S or R)	4	7.28
D + (0.6W or 0.7E)	5	6.67
D + 0.75L + 0.75(0.6W) + 0.75(L_r or S or R)	6a	12.28
D + 0.75L + 0.75(0.7E) + 0.75S	6b	3.64
0.6D + 0.6W	7	6.67
0.6D + 0.7E	8	0.00

Symbols	
A_k	Load or load effect arising from extra ordinary event A
D	Dead load
D_i	Weight of ice
E	Earthquake load
F	Load due to fluids with well-defined pressures and max. heights
F_a	flood load
H	Load due to lateral earth pressure, ground water pressure, or pressure of bulk materials
L	Live Load
L_r	Roof Live Load
R	Rain Load
S	Snow Load
T	Self-Straining Load
W	Wind Load
W_i	Wind-on-ice determined in accordance with Chapter 10

Figure 11. Combination Loads for Most Critical Roof Beam Member

APPENDIX B.2 Wind Load

Basic Wind Speed (See Figure 26.5-1A) (Risk Category II. See Snow Loads)		
V	115	mph

Wind Directionality Factor (Section 26.6, Table 26.6-1)		
K_d	0.85	

Exposure Category (See Section 26.7	
Surface Roughness	C
Exposure Category	C

Topographic Factor (See Section 26.8)		
K_{zt}	1.0	

Gust Effect Factor (See Section 26.9)		
G	0.85	

Enclosure Classifcation (See Section 26.10)
Partially Enclosed

Velocity Pressure Exposure (Table 27.3-1)		
$q_z = 0.00256K_zK_{zt}K_dV^2$		
q_z	24.46	
q_h	26.48	

Internal Pressure Coefficient (See Section 26.11 , Table 26.11-1)		
GC_{pi}	0.55	Towards
GC_{pi}	-0.55	Away

Velocity Pressure Exposure Coefficient (See Table 27.3-1)		
K_z	0.85	
K_h	0.92	

External Pressure Coefficient (Wall) (See Figure 27.4-1)		
C_p	0.8	Windward use q_z
C_p	-0.5	Leeward use q_h
C_p	-0.7	Side use q_h

External Pressure Coefficient (Roof) (See Figure 27.4-1)		
C_p	-0.77	Smaller Windward C_p
C_p	-0.25	Larger Windward C_p
C_p	-0.46	Leeward use q_h

P value roof windward		
p = qGCp-qi(GCpi)		
p	-5.64	Larger C_p
p	-17.29	Smaller C_p

Structure Flexible/Rigid (See Section 29.9.3)		
$n_a = 22.2/h^{\wedge}(0.8)$		
n_a	1.69	Rigid

Wind Design Pressure			
$p = qGC_p$			psf
Windward (South Wall) use q	p	16.63	psf
Leeward (North Wall) use q_h	p	-11.25	psf
Side Walls (East/West Wall)	p	-15.75	psf
Windward (South Roof)	p	-5.64	psf
	p	-17.29	psf
Leeward (North Roof)	p	-10.27	psf

Symbols	
V	Basic wind speed obtained from Figure. 26.5-1A in mph.
K_d	Wind directionality factor in Table 26.6-1
K_{zt}	Topographic factor as defined in Section 26.8
G	Gust-effect factor
q_z	Velocity pressure evaluated at height z above ground, in psf
q_h	Velocity pressure evaluated at height z=h, in psf.
Gc_{pi}	Product of internal pressure coefficient and gust-effect factor to be used in determination of wind loads for buildings
K_z	Velocity pressure exposure coefficient evaluated at height z.
K_h	Velocity pressure exposure coefficient evaluated at height z=h
C_p	External pressure coefficient to be used in determination of wind loads for buildings.
p	Design pressure to be used in determination of wind loads for buildings, in psf
n_a	Approximate lower bound natural frequency (Hz) from Section 26.9.2

Figure 12. Wind Load Spreadsheet

Comparison of Directional and Envelope Wind Load Provisions of ASCE 7

Christopher A. Trautner, P.E., S.M.ASCE[1]; and Rasko P. Ojdrovic, P.E., M.ASCE[2]

Abstract: ASCE 7-10 allows the design of the main wind force resisting system (MWFRS) of buildings with a mean roof height of less than 18.3 m (60 ft) by using either the directional procedure of Chapter 27 or the envelope procedure of Chapter 28 (sometimes referred to as the all heights and low-rise procedures, respectively). These two procedures were developed based on research that used very different methodologies to develop enveloped wind loads. As a result, the two methods may predict very different wind loads and subsequent structural behavior. This paper presents motivation for the research and a comparison of the structural demands calculated by using the two procedures; identifies some situations for which the low-rise procedures may give unconservative MWFRS member loads, proposes changes to the provisions, and identifies avenues for future research. **DOI: 10.1061/(ASCE)ST.1943-541X.0000868.**

Author keywords: Wind loads; Structural design; Structural failures; Standards and codes; Wind effects.

Figure 13. Directional Procedure Selection

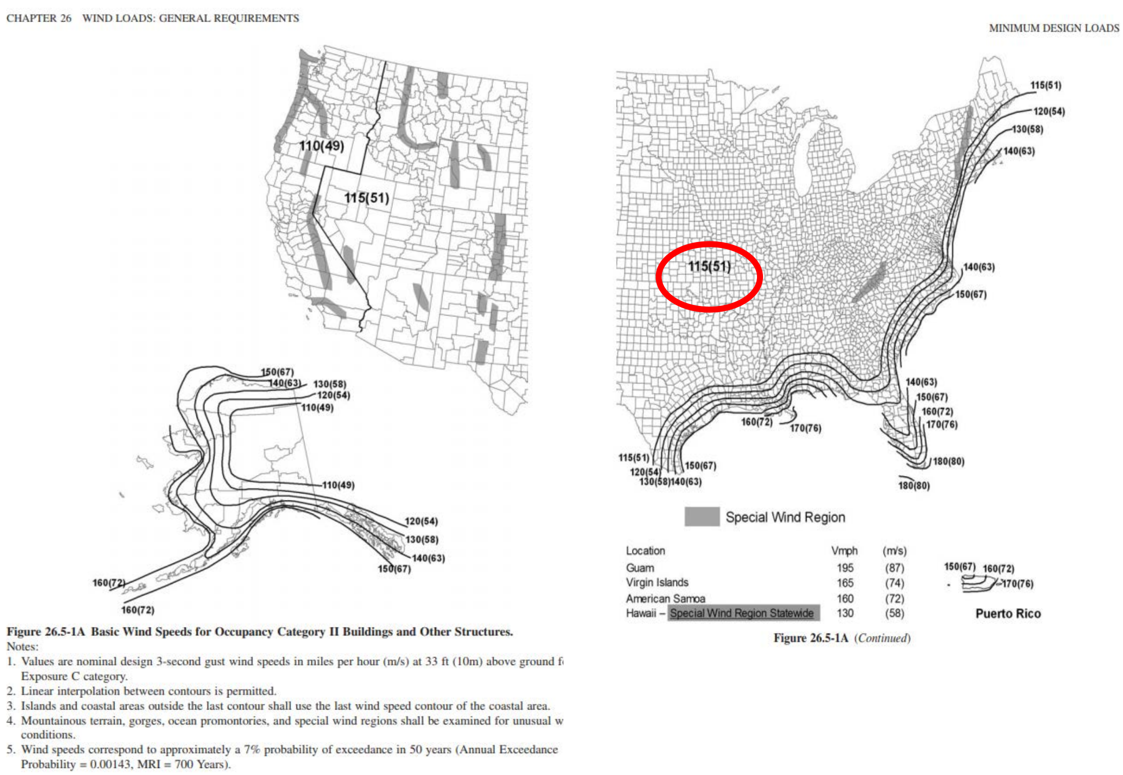


Figure 14. Wind Speed

Wind Directionality Factor, K_d

Table 26.6-1

Structure Type	Directionality Factor K_d*
Buildings	
Main Wind Force Resisting System	0.85
Components and Cladding	0.85
Arched Roofs	0.85
Chimneys, Tanks, and Similar Structures	
Square	0.90
Hexagonal	0.95
Round	0.95
Solid Freestanding Walls and Solid Freestanding and Attached Signs	0.85
Open Signs and Lattice Framework	0.85
Trussed Towers	
Triangular, square, rectangular	0.85
All other cross sections	0.95

*Directionality Factor K_d has been calibrated with combinations of loads specified in Chapter 2. This factor shall only be applied when used in conjunction with load combinations specified in Sections 2.3 and 2.4.

Figure 15. Wind Directionality Factor K_d

Surface Roughness B: Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.

Surface Roughness C: Open terrain with scattered obstructions having heights generally less than 30 ft (9.1 m). This category includes flat open country and grasslands.

Surface Roughness D: Flat, unobstructed areas and water surfaces. This category includes smooth mud flats, salt flats, and unbroken ice.

Figure 16. Surface Roughness

26.9.1 Gust-Effect Factor: The gust-effect factor for a rigid building or other structure is permitted to be taken as 0.85.

Figure 18. Gust-Effect Factor

26.8.2 Topographic Factor

The wind speed-up effect shall be included in the calculation of design wind loads by using the factor K_{zt}:

$$K_{zt} = (1 + K_1K_2K_3)^2 \qquad (26.8\text{-}1)$$

where K_1, K_2, and K_3 are given in Fig. 26.8-1.

If site conditions and locations of structures do not meet all the conditions specified in Section 26.8.1 then $K_{zt} = 1.0$.

Figure 17. Topographic Factor

Main Wind Force Resisting System – Part 1	All Heights
Velocity Pressure Exposure Coefficients, K_h and K_z	
Table 27.3-1	

Height above ground level, z		Exposure		
ft	(m)	B	C	D
0-15	(0-4.6)	0.57	0.85	1.03
20	(6.1)	0.62	0.90	1.08
25	(7.6)	0.66	0.94	1.12
30	(9.1)	0.70	0.98	1.16
40	(12.2)	0.76	1.04	1.22
50	(15.2)	0.81	1.09	1.27
60	(18)	0.85	1.13	1.31
70	(21.3)	0.89	1.17	1.34
80	(24.4)	0.93	1.21	1.38
90	(27.4)	0.96	1.24	1.40
100	(30.5)	0.99	1.26	1.43
120	(36.6)	1.04	1.31	1.48
140	(42.7)	1.09	1.36	1.52
160	(48.8)	1.13	1.39	1.55
180	(54.9)	1.17	1.43	1.58
200	(61.0)	1.20	1.46	1.61
250	(76.2)	1.28	1.53	1.68
300	(91.4)	1.35	1.59	1.73
350	(106.7)	1.41	1.64	1.78
400	(121.9)	1.47	1.69	1.82
450	(137.2)	1.52	1.73	1.86
500	(152.4)	1.56	1.77	1.89

Notes:

1. The velocity pressure exposure coefficient K_z may be determined from the following formula:

   For 15 ft. $\leq z \leq z_g$ — For $z < 15$ ft.

   $K_z = 2.01\,(z/z_g)^{2/\alpha}$ — $K_z = 2.01\,(15/z_g)^{2/\alpha}$

2. α and z_g are tabulated in Table 26.9.1.
3. Linear interpolation for intermediate values of height z is acceptable.
4. Exposure categories are defined in Section 26.7.

Figure 19. Velocity Pressure Exposure Coefficients, K_h and K_z

Main Wind Force Resisting System – Part 1		All Heights
Figure 27.4-1 (cont.)	External Pressure Coefficients, C_p	**Walls & Roofs**
Enclosed, Partially Enclosed Buildings		

Wall Pressure Coefficients, C_p

Surface	L/B	C_p	Use With
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥4	-0.2	
Side Wall	All values	-0.7	q_h

Roof Pressure Coefficients, C_p, for use with q_h

Wind Direction	Windward									Leeward		
	Angle, θ (degrees)									Angle, θ (degrees)		
	h/L	10	15	20	25	30	35	45	≥60#	10	15	≥20
Normal to ridge for θ ≥ 10°	≤0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.01 θ	-0.3	-0.5	-0.6
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01 θ	-0.5	-0.5	-0.6
	≥1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01 θ	-0.7	-0.6	-0.6

Wind Direction	h/L	Horiz distance from windward edge	C_p
Normal to ridge for θ < 10 and Parallel to ridge for all θ	≤ 0.5	0 to h/2	-0.9, -0.18
		h/2 to h	-0.9, -0.18
		h to 2 h	-0.5, -0.18
		> 2h	-0.3, -0.18
	≥ 1.0	0 to h/2	-1.3**, -0.18
		> h/2	-0.7, -0.18

*Value is provided for interpolation purposes.

**Value can be reduced linearly with area over which it is applicable as follows

Area (sq ft)	Reduction Factor
≤ 100 (9.3 sq m)	1.0
250 (23.2 sq m)	0.9
≥ 1000 (92.9 sq m)	0.8

Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. Linear interpolation is permitted for values of *L/B*, *h/L* and θ other than shown. Interpolation shall only be carried out between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.
3. Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures and the roof structure shall be designed for both conditions. Interpolation for intermediate ratios of h/L in this case shall only be carried out between C_p values of like sign.
4. For monoslope roofs, entire roof surface is either a windward or leeward surface.
5. For flexible buildings use appropriate G_f as determined by Section 26.9.4.
6. Refer to Figure 27.4-2 for domes and Figure 27.4-3 for arched roofs.
7. Notation:
   B: Horizontal dimension of building, in feet (meter), measured normal to wind direction.
   L: Horizontal dimension of building, in feet (meter), measured parallel to wind direction.
   h: Mean roof height in feet (meters), except that eave height shall be used for θ ≤ 10 degrees.
   z: Height above ground, in feet (meters).
   G: Gust effect factor.
   q_z, q_h: Velocity pressure, in pounds per square foot (N/m^2), evaluated at respective height.
   θ: Angle of plane of roof from horizontal, in degrees.
8. For mansard roofs, the top horizontal surface and leeward inclined surface shall be treated as leeward surfaces from the table.
9. Except for MWFRS's at the roof consisting of moment resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.

#For roof slopes greater than 80°, use C_p = 0.8

Figure 20. External Pressure Coefficients, Cp

APPENDIX B.3 Snow Load

Ground Snow Load from Fig. 7-1		
P_g	10	psf

Snow Load		
S	28600	pounds

θ	11.1	degrees

Minimum Snow Load for Low-Slope roofs (θ is less than 15 degrees, and P_g </= 20 psf) $P_m=I_sP_g$		
I_s	1.0	Table 1.5-1 and 1.5-2
P_m	10	psf

Symbols	
C_e	Exposure Factor as determined from Table 7-2
C_s	Slope Factor as determined from Fig. 7-2
C_t	Thermal factor as determined from Table 7-3
h	Vertical separation distance in feet (m) between the edge of a higher roof including any parapet and the edge of a lower adjacent roof excluding any parapet
h_b	height of balanced snow laod determined by dividing p_s, by γ, in ft (m)
h_c	clear height from top of balanced snow load to (1) closest point on adjacent upper roof, (2) top of parapet, or (3) top of a projection on the roof, in ft (m)
h_d	height of snow drift, in ft (m)
h_o	height of bobstruction above the surface of the roof, in ft (m)
I_s	importance factor as prescribed in Section 7.3.3. Tables 1.5-1 and 1.5-2 (Shown below)
l_u	length of the roof upwind of the drift, in ft (m)
P_d	maximum intensity of drift surcharge load, in lb/ft^2
P_f	snow load on flat roofs ("flat"=roof slope </= 5 degrees), in lb/ft^2
P_g	ground snow load as determined from Fig. 7-1 and Table 7-1; or a site-specific analysis, in lb/ft^2
P_m	minimum snow load for low-slope roofs, in lb/ft^2
P_s	sloped roof (balanced) snow load, in lb/ft^2
s	horizontal separation distance in feet between the edges of two adjacent buildings
S	roof slope run for a rise of one
θ	roof slope on the leeward side, in degrees
w	width of snow drift, in ft
W	horizontal distnce from eave to ridge, in ft
γ	snow density, in lb/ft^3 as determined from Eq. 7.7-1

Figure 21. Snow Load Spreadsheet

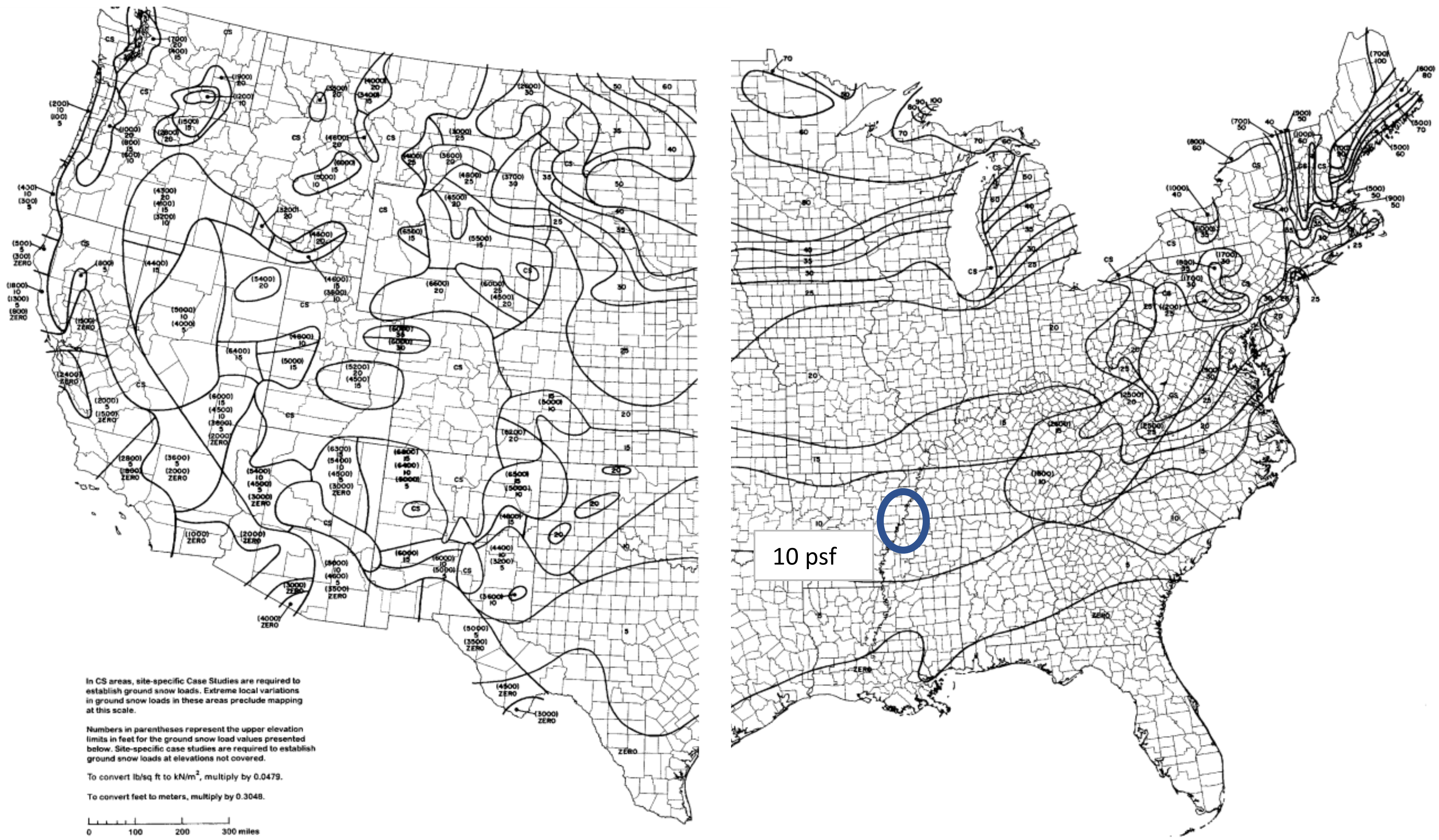


FIGURE 7-1 Ground Snow Loads, P_g, for the United States (Lb/Ft²).

FIGURE 7-1. (*Continued*)

Figure 22. Ground Snow Load Pressure, P_g

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads[a]

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.25	1.00	1.25
IV	1.20	1.25	1.00	1.50

[a]The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it is dependent on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

Figure 23. Importance Factors

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent a low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life.	III
Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure.	
Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where their quantity exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.	
Buildings and other structures designated as essential facilities.	IV

Figure 24. Risk Category of Buildings

APPENDIX B.4 Live Load

From Table 4.1		
Uniform LL Lobby	100	psf
Uniform LL Roof	20	psf
Lobby/Bathroom	286000	lb
Roof	57200	lb
Total Live Load	343200	lb

Live Load shall not be reduced in Assemblies

From Table 4.1, there is only a uniform live load and not a concentrated live load for lobbies

Figure 25. Live Load Spreadsheet

Occupancy or Use	Uniform psf (kN/m^2)	Conc. lb (kN)
Apartments (see Residential)		
Access floor systems		
Office use	50 (2.4)	2,000 (8.9)
Computer use	100 (4.79)	2,000 (8.9)
Armories and drill rooms	150 (7.18)[a]	
Assembly areas and theaters		
Fixed seats (fastened to floor)	60 (2.87)[a]	
Lobbies	100 (4.79)[a]	
Movable seats	100 (4.79)[a]	
Platforms (assembly)	100 (4.79)[a]	
Stage floors	150 (7.18)[a]	
Balconies and decks	1.5 times the live load for the occupancy served. Not required to exceed 100 psf (4.79 kN/m^2)	
Catwalks for maintenance access	40 (1.92)	300 (1.33)
Corridors		
First floor	100 (4.79)	
Roofs		
Ordinary flat, pitched, and curved roofs	20 (0.96)[n]	
Roofs used for roof gardens	100 (4.79)	
Roofs used for assembly purposes	Same as occupancy served	
Roofs used for other occupancies	[o]	[o]
Awnings and canopies		
Fabric construction supported by a skeleton structure	5 (0.24) nonreducible	300 (1.33) applied to skeleton structure

Figure 26. Uniform Pressure for Live Loading

APPENDIX B.5 Dead Load

Weight of internal members					
			lb/item	total	
2C15X50	Trusses	5	16902	84510	lb
W6X9	Roof Beam	7	468	3276	lb
14X48	Columns	10	864	8640	lb
				96426	

Roof			
		ft^2	lb
Area		2860	1859

Total Dead Load for Framing		
Sum of all DL	98285	lb

Figure 27. Dead Load Spreadsheet for Entire Frame to Support

APPENDIX B.6 Structural Analysis – Roof Beams

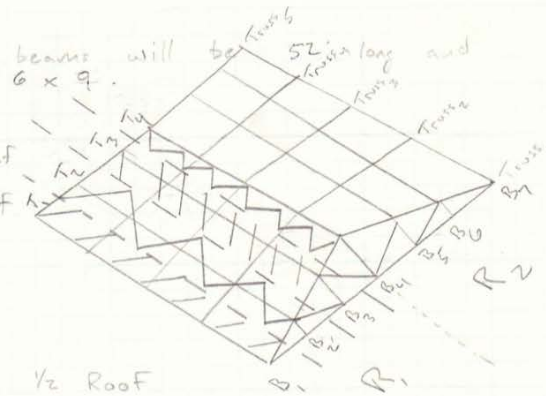
11/2/18 | ROOF BEAM | Thusius | 901 Design | 1/2

The roof consists of 5 be 7 that th roof are will support the loads that th roof are subjected to.

★ Each of the 7 beams will be 52' long and made up of W 6 x 9.

R_1 = South side Roof

R_2 = North side Roof



Tributary Areas on 1/2 Roof

$T_1 = 52' \times 4.66' = T_4 = 242.3\ ft^2$

$T_2 = 9.32' \times 52' = T_3 = 484.6\ ft^2$

★ Because the Roof is symmetric, only half will be analyzed.

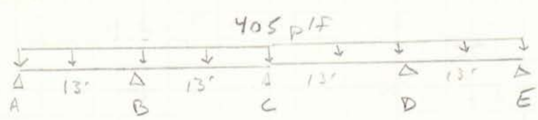
★ Also, only the middle beams will be analyzed on the 1/2 of the roof due to $T_2 > T_1 : T_3$

BEAM 2 + 3 WILL BE SAME

$(B_2 + B_3)$ GREATER WIND EFFECT ON R_1 than R_2

LRFD Case 3 Controls (SEE Roof Loads SPREADSHEET)

Total Load on B_2 = 21.08 kips (Uniform Loading)



A992
W 6 x 9

$A_y = E_y = 2.12^k$ $B_y = D_y = 6.15^k$ $C_y = 5.01^k$ kips (SEE DIAGRAMS)

Max Moment @ $C = 7.47$ k-ft

Figure 28. Roof Beam Analysis, P.1

11/2/18 | ROOF BEAM | Thusius | 901 Design | 2/2

CHECK

Deflection: $\Delta_L = \frac{13'}{360} \times 12 = 0.43''$ (SEE NEXT PAGE)

$\Delta_L = 0.27'' < 0.43''$ ok

Shear: Max shear = 3.27 k (SEE SHEAR DIAGRAM NEXT PAGE)

Table 6-2 (Steel Man)

$\phi_v V_n = 41.6\,k > 3.27\,k$ ok

Moment: Max Moment 7.47 k-ft (SEE MOMENT DIAGRAM NEXT PAGE)

Table 6-2

$\phi_b M_n = 9.8\ k\text{-ft} > 7.47\ k\text{-ft}$ ok

Roof BEAMS

★ SELECT W 6 x 7 For EACH MEMBER

★ Most Critical ½ of Roof was Analyzed

★ Most Critical Beam was analyzed

★ Because the most critical beam on the most critical portion of the roof is ok all members will be ok.

BEAMS 1 + 4 can be analyzed by SAP to Find joint reactions to use for truss Analysis

A992 W6x9

$A_y = E_y = 1.08^k$

$B_y = D_y = 3.13^k$

$C_y = 2.55^k$

Load $= 10.52^k$ / Beam

Figure 29. Roof Beam Analysis, P.2

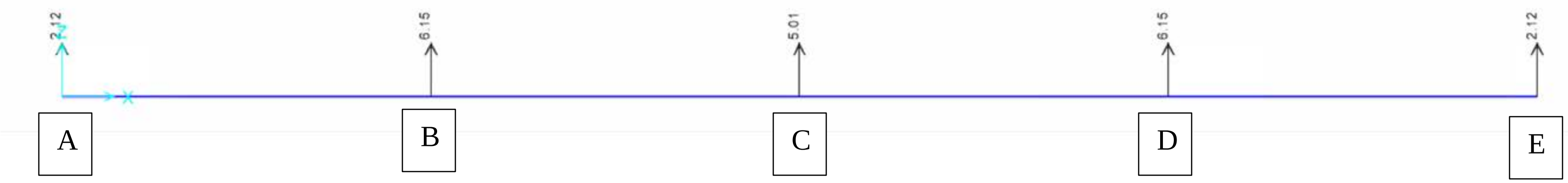


Figure 30. Reactions along Critical Roof Beam Member

Figure 32. Span AB Critical Roof Beam Member

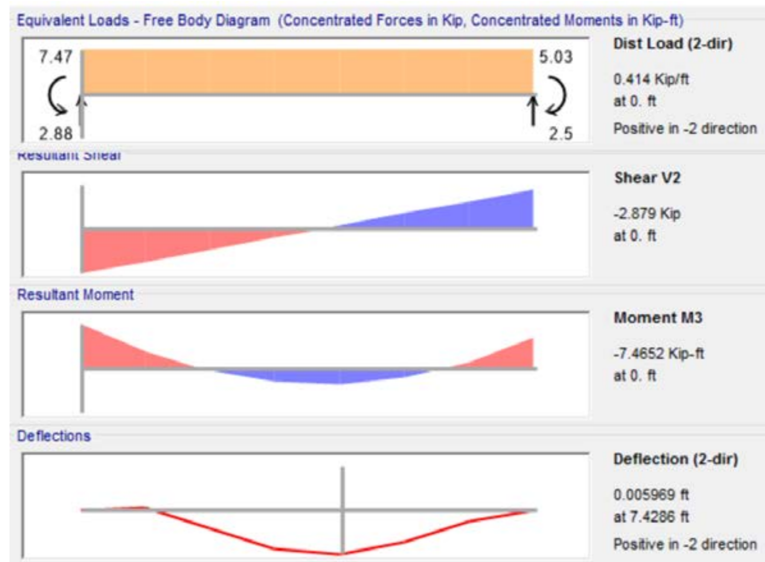


Figure 31. Span BC Critical Roof Beam Member

APPENDIX B.7 Reactions along Critical Roof Beam Member

Figure 35. Span CD Critical Roof Beam Member

Figure 34. Span DE Critical Roof Beam Member

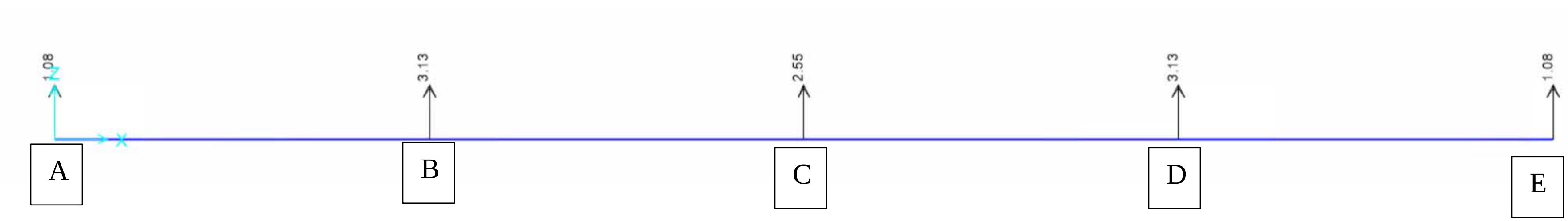


Figure 33. Reactions along Remaining Roof Beam Members

APPENDIX B.8 Structural Analysis – Trusses

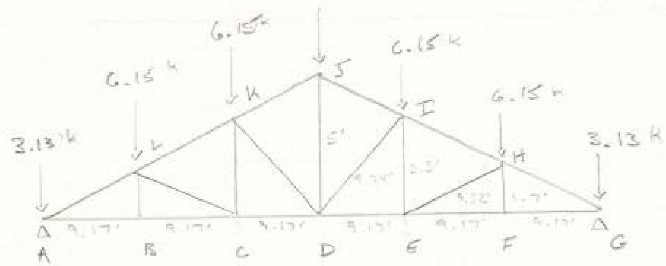
11/4/18	TRUSS ANALYSIS	Thusius	901 Design	1/4

Truss Analysis will be preformed for the truss that is subjected to the greatest loads. The loads were determined from the beam analysis.

☆ Most Critical Condition

★ Double Channel 2C15 X 50 X 3/8

2x3.13k = 6.26 K



☆ Truss 2 + 4 are most critical

☆ Loads are from B_2 (B_y) = 6.15 k

B_1 (B_y) = 3.13 k

☆ Truss Members are made up of 2C15x50

☆ Max Load = 6.26 k @ Middle

☆ SAP2000 will be utilized to verify strength in members

☆ SEE SAP RESULTS + SPREADSHEET

☆ EQ. For Buckling - Compression

$$P_{cr} = \frac{\pi^2 E I}{(KL)^2} \qquad k = 1.0 \quad (\text{Pin connections})$$

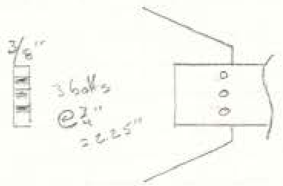
☆ Compression Check SEE SPREADSHEET

Figure 36. Truss Analysis, P.1

11/4/18	Truss Analysis	Thusius	901 Design	4/4

→ 3/4" diameter

* Assuming 3 Bolts Per Member



2C15x50 x 3/8" Members   A325 Bolts

A992 Steel

$F_y = 50$ ksi $F_u = 65$ ksi

Tension Check   Steel Man   Ch. D   16-28/29

yielding $A_g = 2 \times 14.7 in^2 = 29.4 in^2$

$P_n = 50 ksi \times 29.4 in^2 = 1470$ kips   (nominal Strength)

$\phi_t P_n = 0.9 \times 1470$ kips = 1323 kips   (yielding strength)

Rupture $A_n = A_g - A_{holes} = 29.4 in^2 - \frac{3}{8}(\frac{3}{4}) \times 3$ holes $= 28.6 in^2$

$P_n = 65 ksi \times 28.6 in^2 = 1859$ kips

$\phi_t P_n = 0.75 \times 1859$ kips = 1394 kips   (Rupture)

* Yielding Controls

LRFD Design Strength for tension

Members in truss cannot exceed

1323 kips

Largest Tension Member, From SAP, is 16.3k < 1323k

Tension + Compression Members OK

∴ Trusses are OK

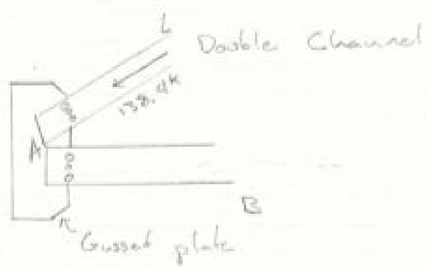
Figure 37. Truss Analysis, P.2

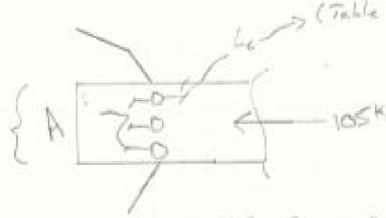
11/6/18 | Truss Connections | Thusius | 901 Design | 3/4

★ Worst Joint will be analyzed (Joint A + G of Truss 2 + 4)

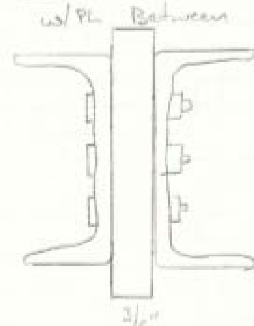
★ $F_{AL} = F_{GH} = 104.7^k$

★ All connections will be designed in trusses to satisfy $F = 105^k$

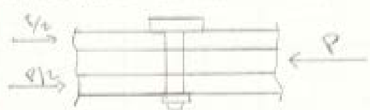




2C 15x50x3/8
Double Channel
w/ PL Between



★ Double Shear ($P = 105^k$)



★ Using 3 bolts @ 7/8"

Table 7-1 Steel Manual - Avail Shear Strength in Bolts

7/8", Group A, N, D

$\phi r_n = 48.7^k$ / bolt

$\therefore \frac{138.4^k}{48.7^k/\text{bolt}} = 2.84 \rightarrow$ 3 Bolts For Truss Connections

Min
$L_e = 1"$ Table J3.4
$s = 3d = 2.25$

$\therefore 15" \geq 2 \times 2.25 + 3 \times \frac{7}{8} + 2 \times 1$ OK

Figure 38. Truss Analysis, P.3

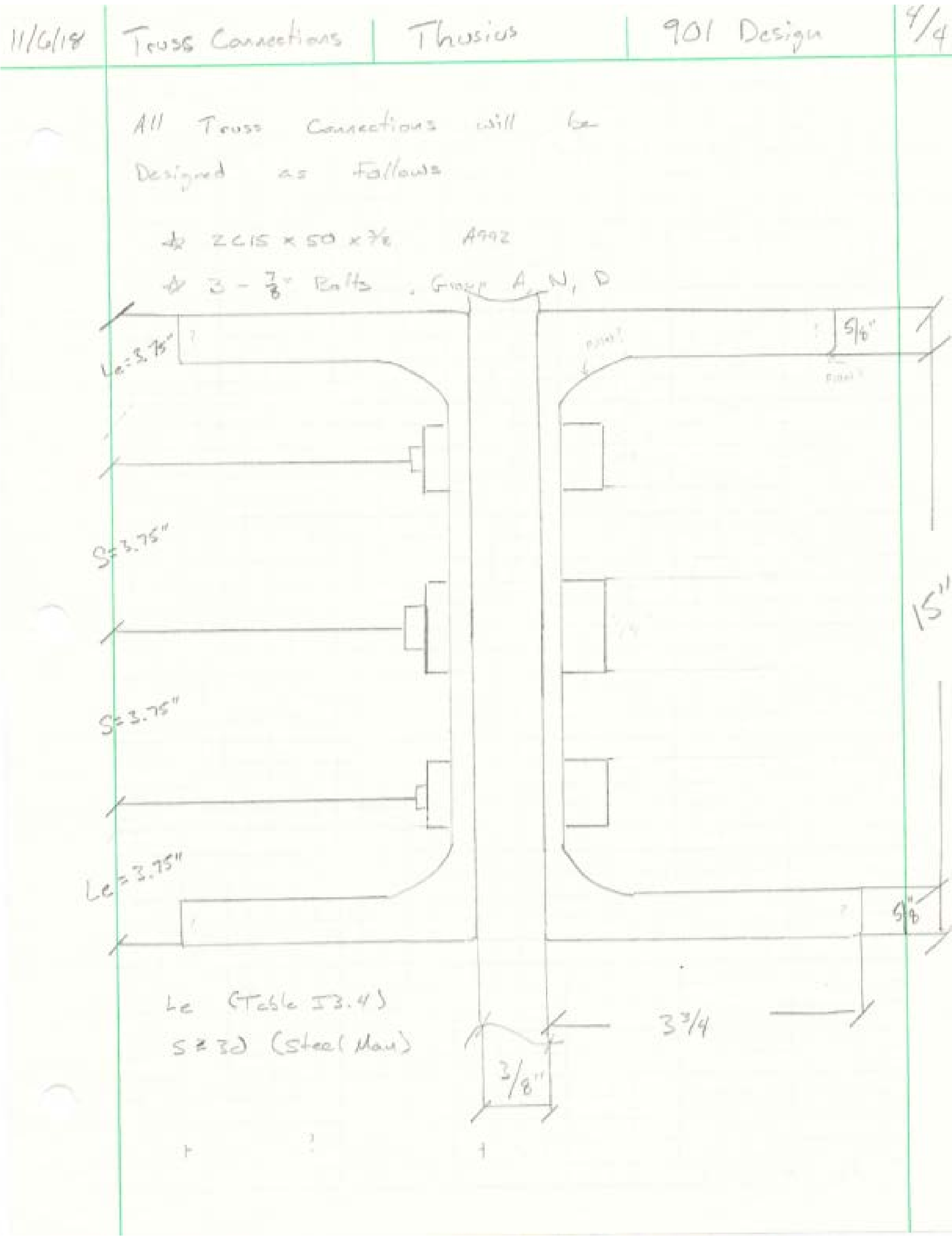


Figure 39. Truss Analysis, P.4

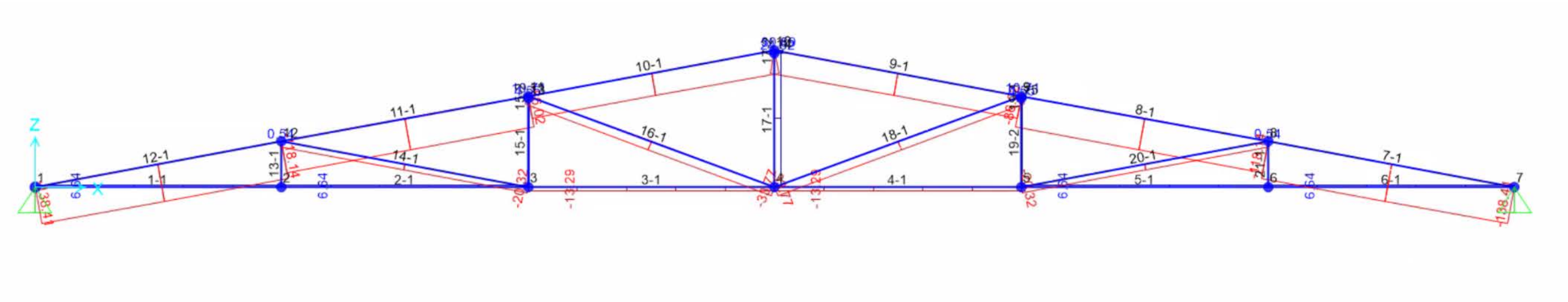


Figure 40. Axial Forces in Critical Truss

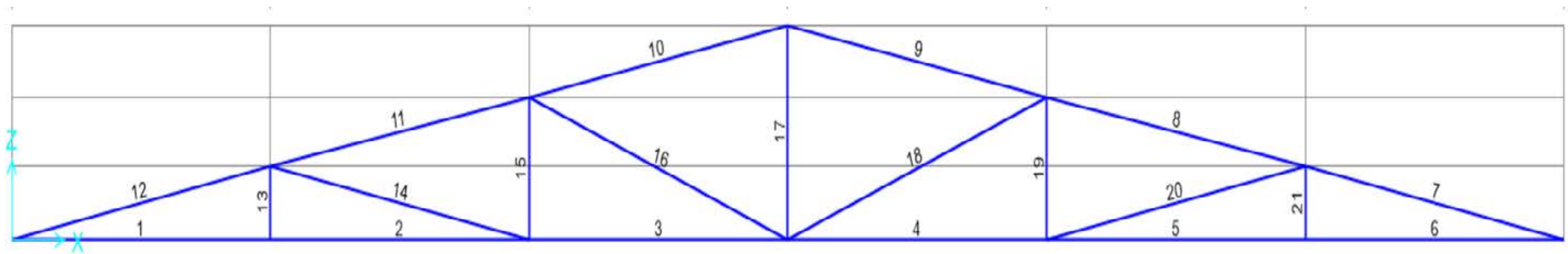


Figure 41. Truss Members Numbered for Compression Spreadsheet

Table: Element Forces - Frames

	L(in.)	F_{SAP} (kip)	P_{cr}	P(kips)
1	110.00	5.82	Tension Member SEE TENSION CALCS	
2	110.00	5.82	Tension Member SEE TENSION CALCS	
3	110.00	-11.64	-520393.54	44707.35
4	110.00	-11.64	-520393.54	44707.35
5	110.00	5.82	Tension Member SEE TENSION CALCS	
6	110.00	5.82	Tension Member SEE TENSION CALCS	
7	111.84	-89.85	-503414.95	5602.84
8	111.84	-72.10	-503414.95	6982.18
9	111.84	-54.20	-503414.95	9288.10
10	111.84	-54.20	-503414.95	9288.10
11	111.84	-72.10	-503414.95	6982.18
12	111.84	-89.85	-503414.95	5602.84
13	20.40	0.14	Tension Member SEE TENSION CALCS	
14	111.84	-17.70	-503414.95	28441.52
15	39.60	3.46	Tension Member SEE TENSION CALCS	
16	116.88	-18.80	-460935.35	24517.84
17	60.00	19.64	Tension Member SEE TENSION CALCS	
18	116.88	-18.80	-460935.35	24517.84
19	39.60	3.46	Tension Member SEE TENSION CALCS	
20	111.84	-17.70	-503414.95	28441.52
21	20.40	0.14	Tension Member SEE TENSION CALCS	

2C15x50x3/8

E=	29,000,000	psi
I=	22	$in.^4$
2C15x40x3/8		

Applied Load

$P_{applied}$(kips)= 19.76

Critical Load in member

$P_{allowable}$(kips)= 5602.84

Check if $P_{allowable}$>$P_{applied}$

O.K.

$$P_{cr} = \frac{\pi^2 EI}{KL^2}$$

Where K is equal to 1.0 for pinned connections.

Figure 42. Compression Check Spreadsheet

APPENDIX B.9 Structural Analysis – Columns

Column Analysis | Thusius | 901 Design | 1/5

* Total of 10 columns
* W14 x 48
*

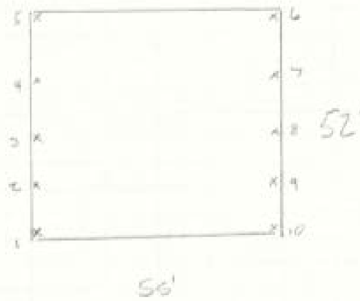


Diagram of colums

* Colums 2, 4, 7, 9 are most critical
* IF 2, 4, 7, 9 are ok then 1, 3, 5, 6, 8, 10 will be ok
* Load = 28.82 kips on 2, 4, 7, 9

$P_u = 28.82$ kips

A992

$E = 29{,}000{,}000$

$F_y = 50$ ksi

$L_e = 18' \times 12 = 216''$

$r = 1.91''$ $A_g = 14.1 \text{ in}^2$

Buckling

- Check Flexural Buckling w/out slender elements

$\frac{L_c}{r} > 4.71\sqrt{\frac{E}{F_y}}$

Steel Manual

$\frac{216}{1.91} = 113.1 > 113.4$ USE EQ E3-2

$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) F_y$ $F_e = \frac{\pi^2 \, 29{,}000}{146^2} = 13.6$ ksi

$F_{cr} = 10.7$ ksi

Figure 43. Column Analysis, P.1

Column Analysis | Thusius | 901 Design | 2/3

$F_{cr} = 10.7$ ksi

$P_n = F_{cr} \cdot A_g$

$P_n = 10.7$ ksi $\times 14.1$ in^2 = 151 kips

$P_u = 28.82$ kips

$\therefore \; P_u \leq P_n$ OK

Check Slender Compression

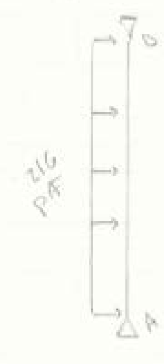
Flange $\lambda = \frac{b_f}{2t_f} = 6.75'' < \lambda_r = 0.56\sqrt{\frac{29000}{60}} = 13.5$

$\lambda < \lambda_r$ Flange is nonslender

Web $\lambda = \frac{h}{t_w} = 33.6 < \lambda_r = 1.49\sqrt{\frac{29000}{60}} = 35.9$

$\lambda < \lambda_r$ Web is non slender

CHECK SHEAR / MOMENT / DEFLECTION



Wind load - Spreadsheet Calcs

Tributary Wall Area = 234 ft^2

Max Wind Pressure = 16.63 psf

Wind load = 3891 lb

Max Shear: 2.42^k < $\frac{\text{Check}}{154}$ kips OK

Moment: 0.27 k-ft < 208 k-ft OK (Table 6-2)

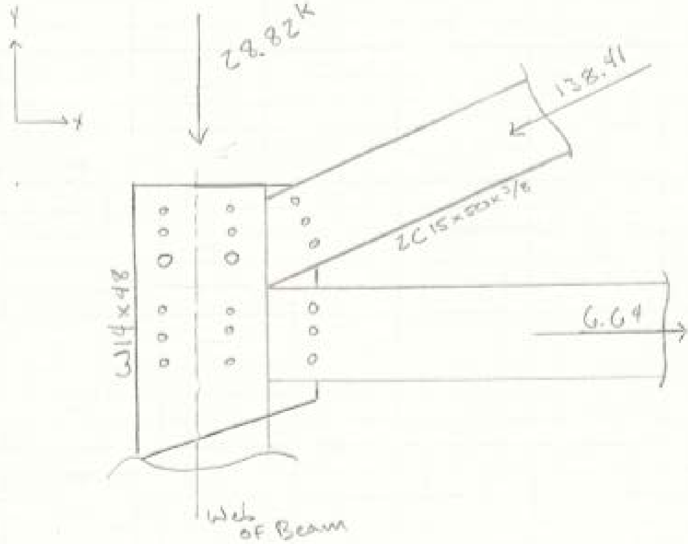
Deflection: 0.04'' < $\frac{18' \times 12}{360} = 0.6''$ OK

Therefore, All Column will be W14x48

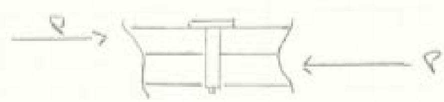
Figure 44. Column Analysis, P.2

11/8/18	Column — Truss Connections	Thusius	901 Design	3/5

* The most critical column – Truss will be analyzed. All other connections will be modeled after most critical case.



* Single Shear



* Using 2 Bolts @ 3/4"

Table 7-1 Steel Manual – Avail Shear strength in Bolts

3/4", Group A, N, S

$\phi r_n = 17.9$ kips / bolt

$\therefore \frac{28.82^K}{17.9^K/B} = 1.61 \rightarrow 2$ Bolts

Min Spacing $L_e = 1\frac{1}{8}"$ Table J3.4

$s = 3d = 2.63"$

Figure 45. Column Analysis, P.3

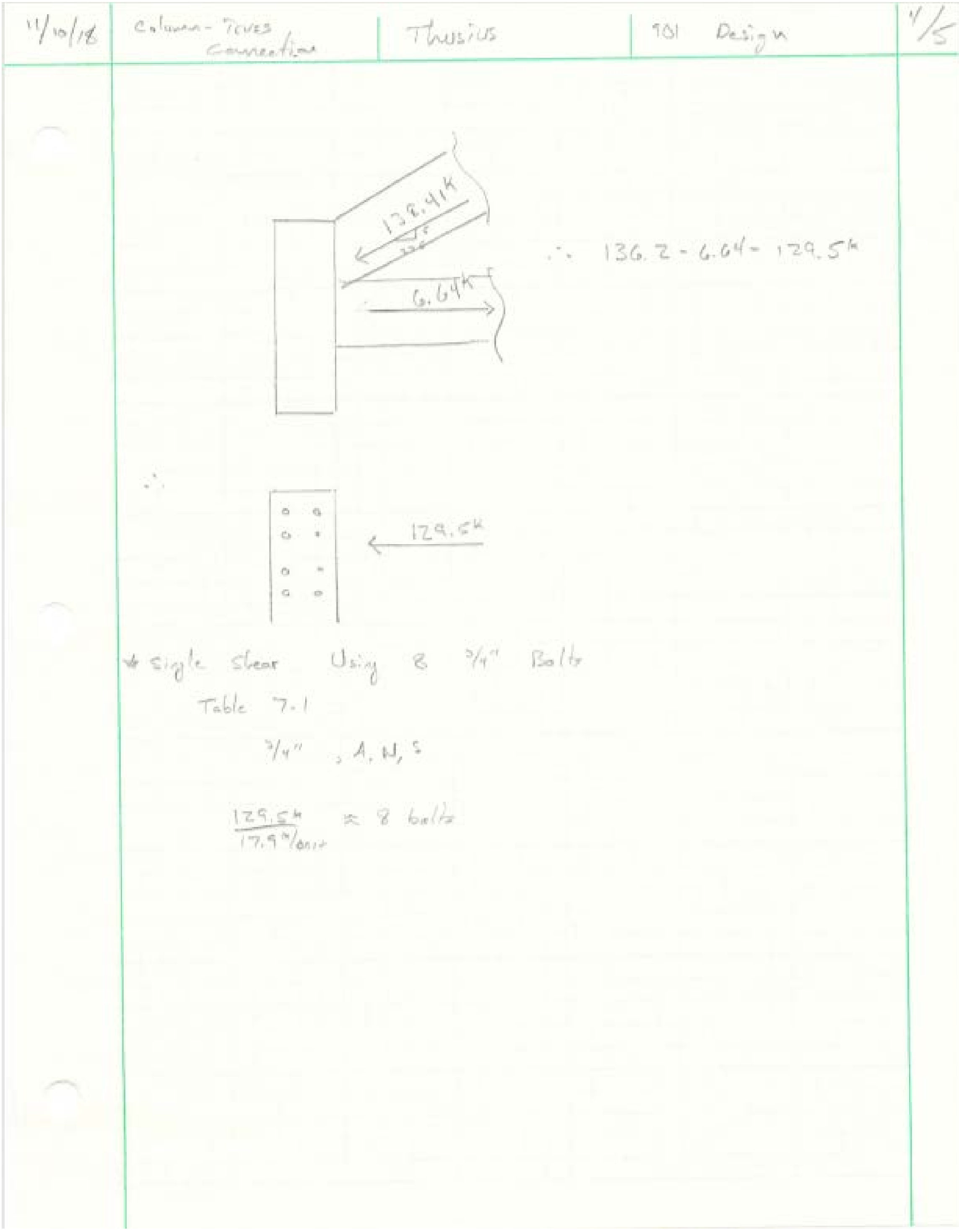


Figure 46. Column Analysis, P.4

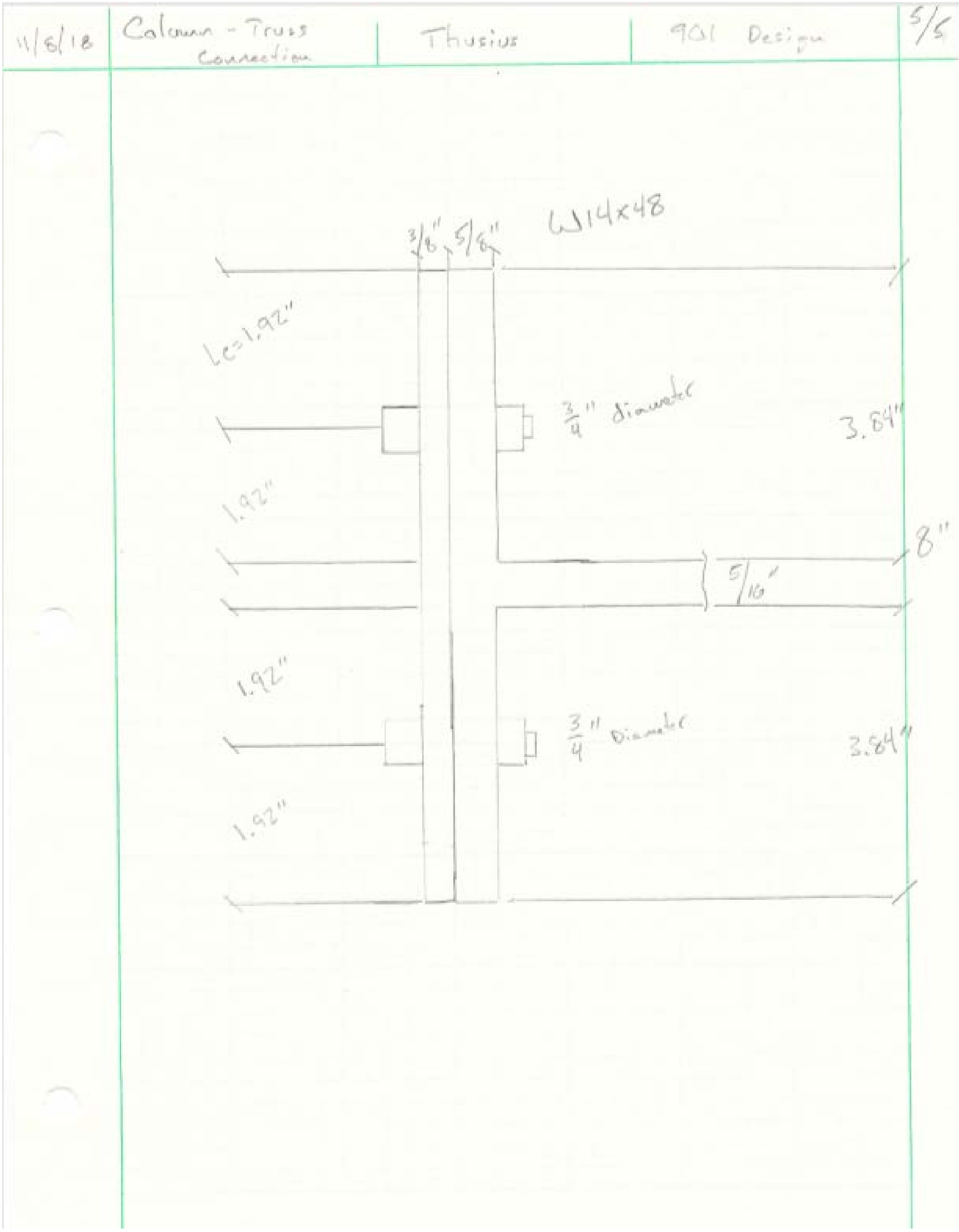


Figure 47. Column Analysis, P.5

Figure 48. Horizontal Column Reactions

Figure 49. Max Shear, Moment, Deflection in Critical Column

APPENDIX B.10 Rest Room Water Closet Calculation

Restroom Stalls	T_1=A*UV*B*PF*P*UHF	T=Total Toilets	32.90		
		A= 1 way Design Year ADT	17575.00	T_1	
		UV= 1.3 Restroom users per vehicle			
		B= .15= Ratio of Design hourly volume to ADT		T_2	32.90
	or	PF= 1.8= Peak Factor			
		P= Total % of traffic stopping at rest area	0.16	T_3= A*P*.0117	32.90
		UHF= 30= Restroom users per hour per fixture based on 2 min cycle			
	T_2=(S*1.3*1.5*1.8*P)/30				
	W= T * .6	W= Number of women's toilets		W=	19.74
	M= T * .4	M= Total number of men's toilets & urinals		M=	13.16

Figure 50. Rest Room Water Closet Calculation

APPENDIX B.11 Wind Load Bracing

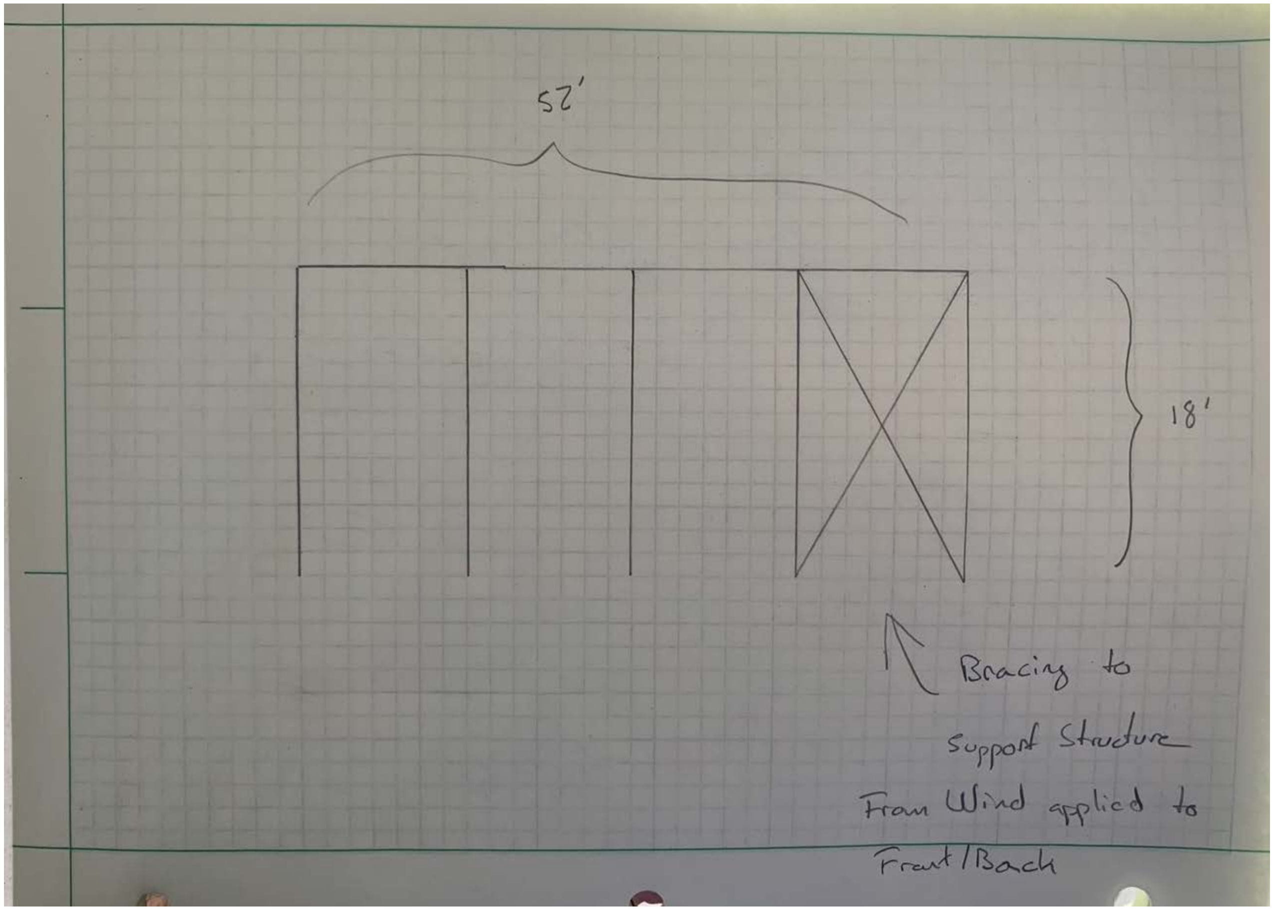


Figure 51. Wind Load Bracing – Sides of Building

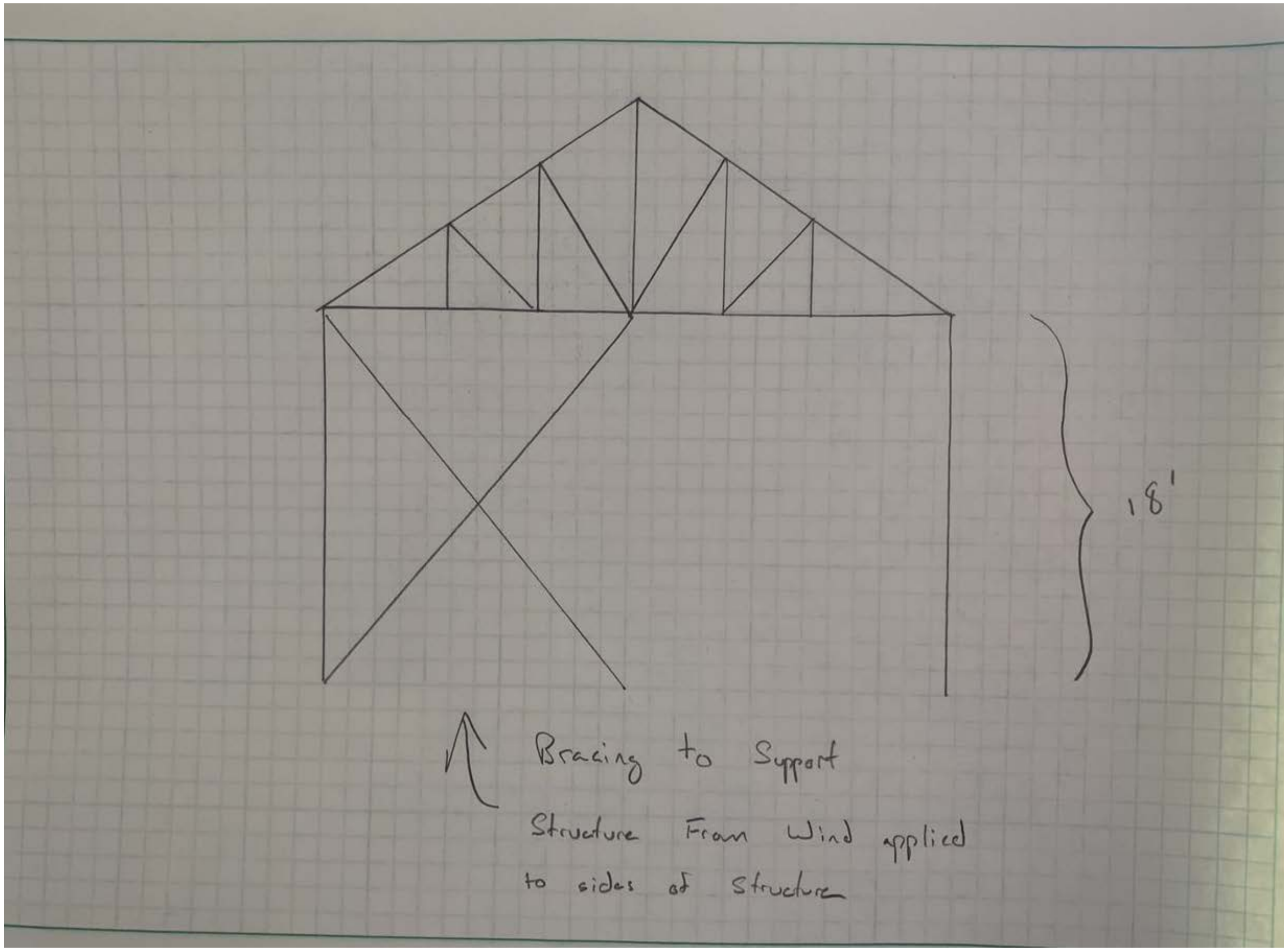


Figure 52. Wind Load Bracing – Front and Back

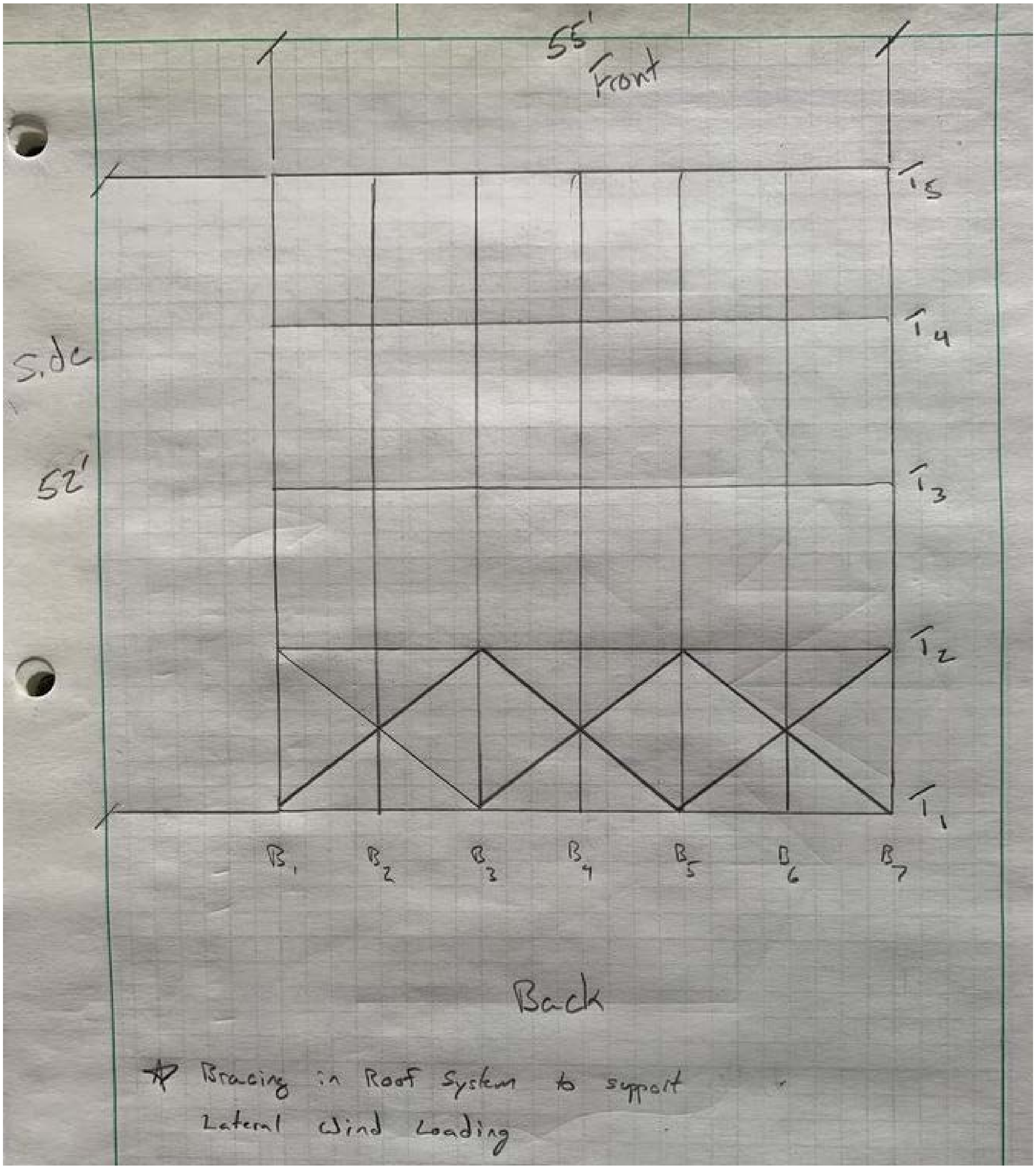


Figure 53. Wind Load Bracing – Roof System

APPENDIX C. GEOTECHNICAL

APPENDIX C.1 Boring Location Plan

THE UNIVERSITY OF MEMPHIS

CIVL 4199 – CIVIL ENGINEERING SENIOR DESIGN

Boring Location Plan:

I-69 Proposed Rest Area

901 Design

Date Submitted: October 19, 2018

Prepared by:

Kendall Lee Brown
Huan Hoang Ngo
Mark Anthony Rippy
Stephen Carl Thusius
Jana Marie East Moss

Prepared for:

Dr. David Arrellano
The University of Memphis
Department of Civil Engineering
Memphis, TN 38152

Available Subsurface Information

A site visit was made on September 17, 2018. The information collected from the site visit is that the location is existing farm land and has minimal elevation change. The site is private property, so observations could only be made from the shoulder of Wilkinsville Road. Information on the Soil surface was available on the Tennessee Virtual Archive (TeVA). TeVA's website displays a Shelby County Tennessee soil map of 1916. The map specifies the primary surface soils that are present around the proposed construction site location. These soils are shown to be predominately silt loam and Memphis silt loam. Additional information pertaining to the subsurface soil was found on the Web Soil Survey website. The data displayed below corresponds to the proposed construction site location.

Typical Subsoil Profile	
Depth	**Soil Type**
0 to 7 inches	Silt Loam
7 to 28 inches	Silt Loam
28 to 50 inches	Silt Loam
50 to 60 inches	Silt Loam

Table 1. Typical Soil Profile

Preliminary Model of Subsurface

The subsurface model displayed below (Figure 1. Typical Soil Profile) corresponds to the information gathered from Web Soil Survey. The first 5 ft. of soil consist of silt loam. The location has an annually fluctuating ground water level that varies between 1 ft. to 2 ft 4 in. in depth. Silt soils are not ideal for shallow foundations and will most likely need to be cut and filled with more stable material. Silt soil has a tendency to retain moisture and drains poorly. The retention of water causes the silty soil to expand, pushing against a foundation and weakening it, making it not ideal for support. However, Loam is the ideal soil type. Typically, it's a combination of sand, silt and clay. Loam is great for supporting foundations because of its evenly balanced properties, especially how it maintains water at a balanced rate. Loam is a good soil for supporting a foundation and should allow the engineer to design a shallow foundation. The laboratory testing results will determine if the silt loam near the surface will need to be cut and filled with new soil.

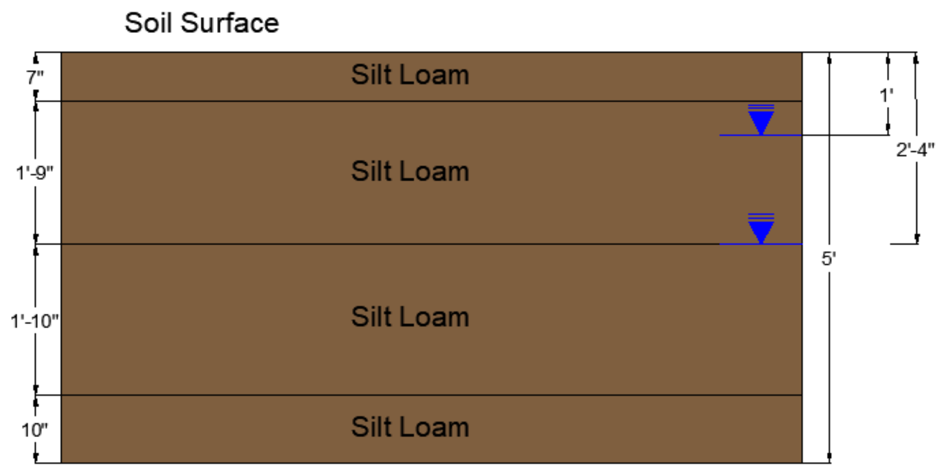


Figure 1. Typical Soil Profile

Required Soils Needed for Design and Construction

With the proposed site being in Shelby County Tennessee, sand's, silt's, and clays are all possible subgrade soils. A slab or continuous wall foundation was originally planned for this building. This plan is possible if lab tests conclude the existing soil can support a shallow foundation. If the lab tests conclude the soil is not capable of supporting the shallow foundation, the location must undergo preliminary earth work before the foundation could be constructed. Preliminary earth work would involve removing the undesirable soil and replacing it with the appropriate soil type necessary to meet the foundation's needs. If the silt loam soil is shown through laboratory testing to be an unstable soil and earth work/cut and fill is greater than a depth of 10 ft., the excessive preparation work may make a shallow foundation unappealing. If the situation occurs, where the sub soil is inferior in bearing capacity and settlement, a deep foundation will need to be considered. Firm clays, loam, or sand near the soil surface would be ideal for a shallow/continuous wall foundation.

Proposed Boring Location Plan

The construction site for the proposed I-69 rest area has been chosen. However, the layout for the building and parking lot has not been finalized. For this reason, the boreholes for this project will be located at the corners of the proposed building. Its recommended that more boreholes be placed for the parking lots and any other proposed structures. For this project it will be assumed that the rest of the site layout will reflect the same soil strata recovered in the building boreholes. The spacing was chose based off the Table 2. Bore Spacing shown below.

Table 12.2 Approximate Spacing of Boreholes (Das)	
Type of project	Spacing (m)
Multistory building	10 – 30
One-story industrial plants	20 – 60
Highways	250 – 500
Residential subdivisions	250 – 500
Dams and dikes	40 – 80

Table 2. Bore Spacing

The type of construction for the I-69 rest area is similar to a Multistory building. This spacing will result in a detailed subsurface investigation for the proposed building, see the attached map (Figure 2. Boring Locations) for borehole locations. There will be a total number of 4 boreholes for the construction site. the boreholes will be placed 5 ft. away from the corners of the proposed building location. After all soil sample are recovered, the 4 boreholes for the proposed building subsoil investigation will be backfilled with grout. Prior to soil investigation boring, surveyors will be hired to locate and stake the proposed borehole locations.

<u>Boring Depths</u>

The depth of boreholes will be calculated according to Sowers and Sowers (1970). The calculations in the table below represent two types of buildings. Both calculations will be examined, and the most practical borehole depth will be chosen.

$D_b=3S^{0.7}$	(for light steel or narrow concrete buildings)	Equation (12.1) Das
$D_b= 6S^{0.7}$	(for heavy steel or wide concrete buildings)	Equation (12.2) Das

Table 3. Boring Depth Equations

Where

D_b = depth of boring (m)

S = number of stories

The borehole depth for light steel buildings results in a depth of 3 meters (9.84 ft.). The borehole depth for heavy steel buildings results in a depth of 6 meter (19.69 ft.). If the light steel calculation was chosen for the borehole depth, assuming Web Soil Survey's data is correct, the engineer would only gain information on the next 5 ft. of subsoil. There will be large stresses placed on the soil from the building and the tractor trailer parking lot. For this reason, the borehole depth for the grid will comply with the heavy steel building calculation. The depth of the boreholes confined to the grid will be 20 ft. in depth. The boreholes that are placed for the building will have locations that diverge from the grid and will go down to deeper depths. The building boreholes will have a minimum depth of 20 ft. If firm soil is not found in the first 20 ft., the borings shall continue until firm ground is reached. The deeper depth of the building boreholes is meant to protect the building from any unexpected soil layers that could increase the settlement.

Field Tests

Field testing will be performed to gain information on the subsoil's friction angle (ø'), unit weight (γ), and ground water level. The test that will be completed in the field is the Standard Penetration Test (SPT). The SPT samples will be recovered every 1.5 meters (5 ft.). If soil sample recovery is unsuccessful due to a granular type of soil, it is advised that a spring core catcher be placed inside the split spoon sampler. The results of the SPT will give the soils N-value that will allow the engineer to determine the soils unit weight (γ), and friction angle (ø'). When cohesive soil is encountered, Soil samples will be recovered using thin walled tubes/Shelby tubes. Like the SPT, the Shelby tube samples will be recovered every 1.5 meters (5 ft.) when applicable. The unit weight of the soil and the ground water level are necessary for calculating the effective stress (σ'_o) of the subsoil. The Shelby tubes will allow the lab to receive undisturbed soil samples for testing consolidation, and undrained shear strength.

Laboratory Tests

The lab tests will allow the engineer to obtain the remaining soil parameters that are necessary to size the building foundation based on settlement and bearing capacity. The tests to be performed in the laboratory will include the in-situ water content test, sieve analysis, Atterberg limits, consolidation test, and the unconfined compressive test. All tests will be executed in compliance with ASTM specifications. The in-situ water content test is necessary for the engineer to understand the natural subsoil conditions that will influence the soils strength, settlement, and bearing capacity. A sieve analysis will also be completed to attain information on the subsoil particle gradation. The soil samples will also be tested for Atterberg Limits. The Atterberg limits test will allow the computation of the subsoils Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI). With Sieve Analysis and Atterberg Limits tests completed, the recovered subsoil samples will then be assigned the appropriate soil classification. Disturbed soil samples recovered from the SPT will suffice for in-situ water content, sieve analysis, and Atterberg Limit tests. The one-dimensional consolidation test, and the unconfined compressive strength test will both be performed using the soil samples recovered by Shelby tubes. The consolidation test will quantify both the ultimate amount of settlement and the time rate of settlement in the soil layers. Using laboratory derived parameters, field settlement behavior of the soil layer can be predicted. The results from the consolidation test will allow the calculation of the compression index (C_c), recompression index (C_r), and void ratio (e_o). The Unconfined compressive strength test will be performed to measure the unconfined compressive strength (qu) and undrained shear strength (su) of normally consolidated and slightly over consolidated cylindrical specimens of cohesive soil. The information attained from the unconfined compressive test is used to estimate the bearing capacity of spread footings and other structures when placed on deposits of cohesive soil. The completion of the previously described tests will allow the engineer to size a foundation based on bearing capacity and settlement.

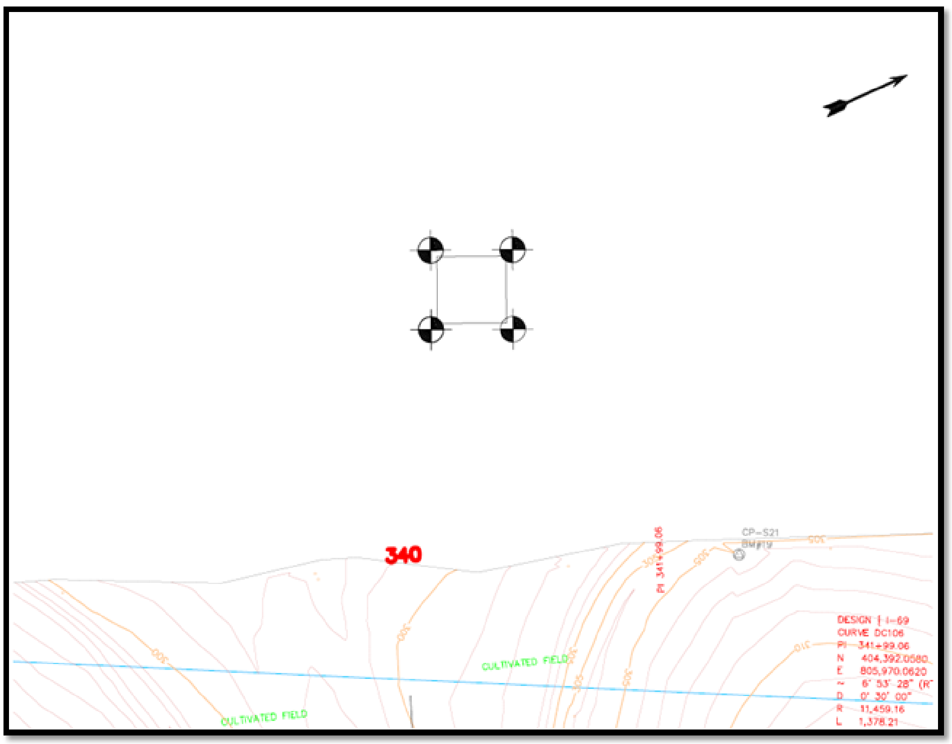


Figure 2. Boring Locations

APPENDIX C.2 Boring Logs

Southeast Borehole 1

Sample Interval		Sample Type	SPT Values			N	ø'	Water Content (%)	Unconfined Compressive Strength (psf)	Sample Description	USCS	LL	PL	PI	Unit Weight (pcf)	Cc	Cr	OCR
(ft)	(ft)																	
1	2.5	SS	16	21	23	44	40	15		brown clayey silt	(CL-ML)							
3.5	5	SS	15	20	22	42	40	16		brown clayey silt								
6	7.5	ST				0			6500	brown clayey silt		21	14	7	115	0.11	0.06	2.0
8.5	10	SS	15	18	18	36	38	16		brown clayey silt								
11	12.5	SS	21	22	23	45	40	15		mottled brown and tan silty clay	(CL)							
13.5	15	ST							7600	mottled brown and tan silty clay		33	14	19	124	0.15		1
18.5	20	SS	19	20	21	41	40	15		mottled brown and tan silty clay								

Southwest Borehole 2

Sample Interval		Sample Type	SPT Values			N	ø'	Water Content (%)	Unconfined Compressive Strength (psf)	Sample Description	USCS	LL	PL	Unit Weight (pcf)	Cc	Cr	OCR
(ft)	(ft)																
1	2.5	ST						14	5200	brown clayey silt	(CL-ML)			116			
3.5	5	SS	12	17	19	36	38	15		brown clayey silt							
6	7.5	SS	15	13	16	29	37	18		brown clayey silt							
8.5	10	SS	12	15	15	30	37	16		brown clayey silt							
11	12.5	SS	18	19	20	39	39	14		mottled brown and tan silty clay							
13.5	15	SS	17	20	20	40	39	13		mottled brown and tan silty clay							
18.5	20	SS	16	17	18	35	38	14		mottled brown and tan silty clay							

Table 4. Combination 1 Bore Logs

Northeast Borehole 3

Sample Interval		Sample Type	SPT Values			N	ø'	Water Content (%)	Unconfined Compressive Strength (psf)	Sample Description	USCS	LL	PL	PI	Unit Weight (pcf)	Cc	Cr	OCR
(ft)	(ft)																	
1	2.5	SS	12	17	19	36	38	15		brown clayey silt	(CL-ML)							
3.5	5	ST						16	6500	brown clayey silt		21	14	7	115			
6	7.5	SS	9	14	16	30	37	21		mottled brown and tan silty clay								
8.5	10	ST	10	13	13	26	35	20	7000	mottled brown and tan silty clay	(CL)	32	13	19	122			
11	12.5	SS	16	17	18	35	38	18		mottled brown and tan silty clay								
13.5	15	SS	15	18	18	36	38	17		mottled brown and tan silty clay								
18.5	20	SS	14	15	16	31	37	18		mottled brown and tan silty clay								

Northwest Borehole 4

Sample Interval		Sample Type	SPT Values			N	ø'	Water Content (%)	Unconfined Compressive Strength (psf)	Sample Description	USCS	LL	PL	Unit Weight (pcf)	Cc	Cr	OCR
(ft)	(ft)																
1	2.5	SS	13	18	20	38	39	14		brown clayey silt	(CL-ML)						
3.5	5	SS	12	17	18	35	38	15	6300	brown clayey silt							
6	7.5	SS	11	14	19	33	37	19	7100	mottled brown and tan silty clay	(CL)						
8.5	10	SS	12	15	15	30	37	18		mottled brown and tan silty clay							
11	12.5	SS	18	19	20	39	39	16		mottled brown and tan silty clay							
13.5	15	SS	17	20	20	40	39	15		mottled brown and tan silty clay							
18.5	20	SS	16	17	18	35	38	16		mottled brown and tan silty clay							

Table 5. Combination 2 Bore Logs

APPENDIX C.3 Soil Profiles

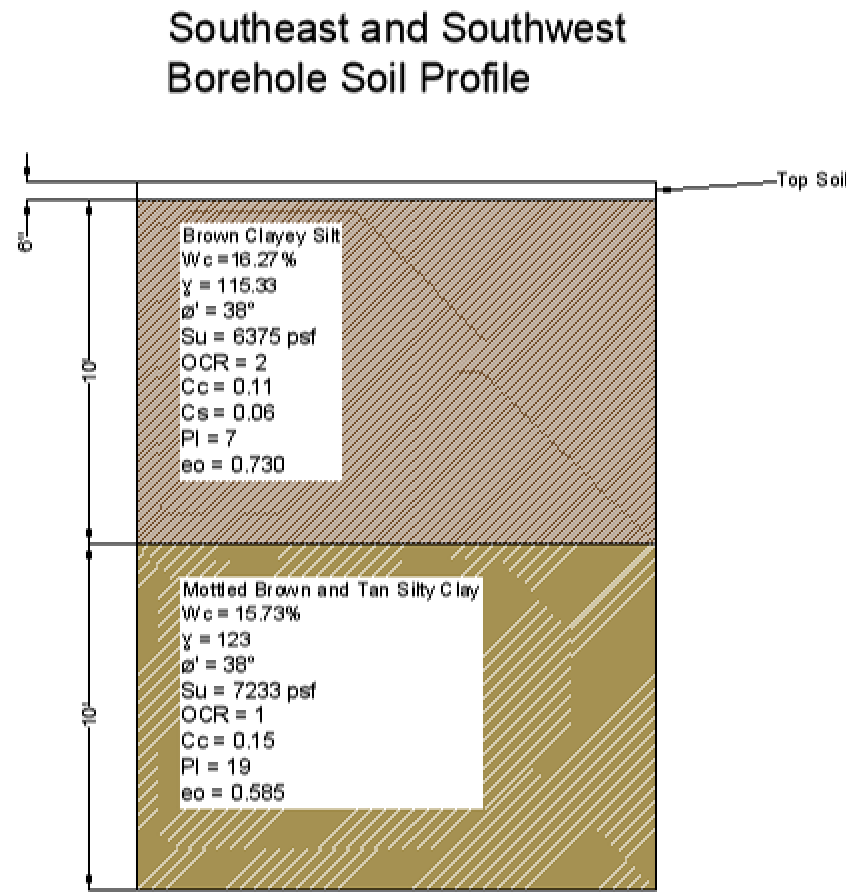


Figure 3. Combination 1

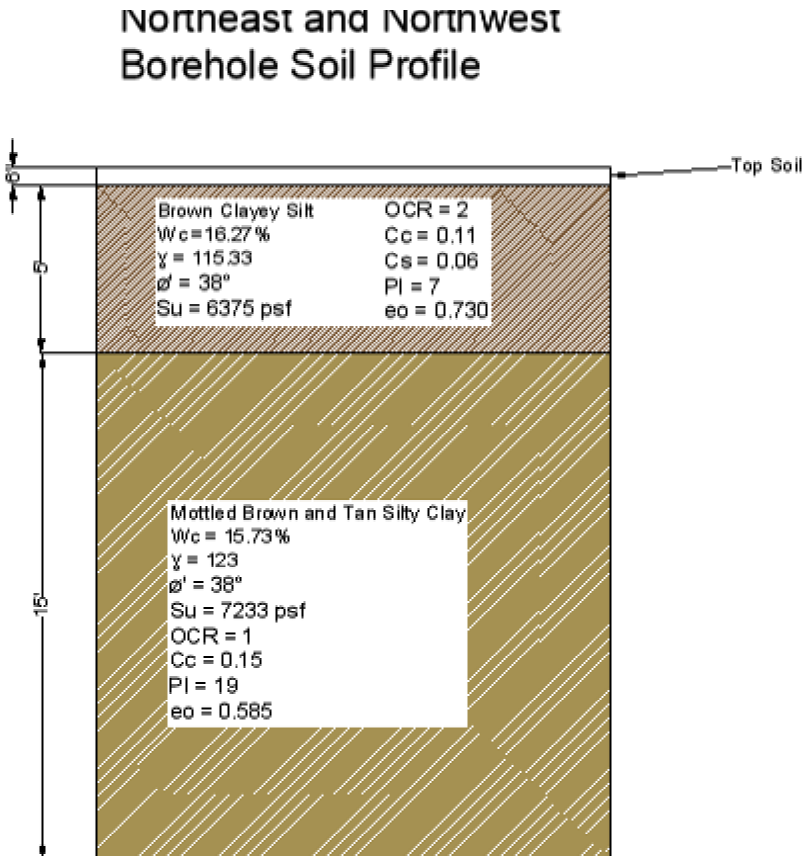


Figure 4. Combination 2

APPENDIX C.4 Soil Parameters

Soil Properties					
Layer 1			Layer 2		
brown clayey silt			mottled brown and tan silty clay		
Wc =	16.27	%	Wc =	15.73	%
γ_{moist} =	115.33	pcf	γ_{moist} =	123	pcf
ø' =	38	degrees	ø' =	38	degrees
B1 (H) =	10	ft	B1 (H) =	10	ft
B2 (H) =	10	ft	B2 (H) =	10	ft
B3 (H) =	5	ft	B3 (H) =	15	ft
B4 (H) =	5	ft	B4 (H) =	15	ft
Su =	6375	psf	Su =	7233	psf
OCR =	2		OCR =	1	
Cc =	0.11		Cc =	0.15	
Cs =	0.06		PI =	19	
PI =	7		e_o =	0.585	
e_o =	0.730				

Table 6. Soil Parameters

APPENDIX C.5 EPRI Soil Manual Friction Angle Chart

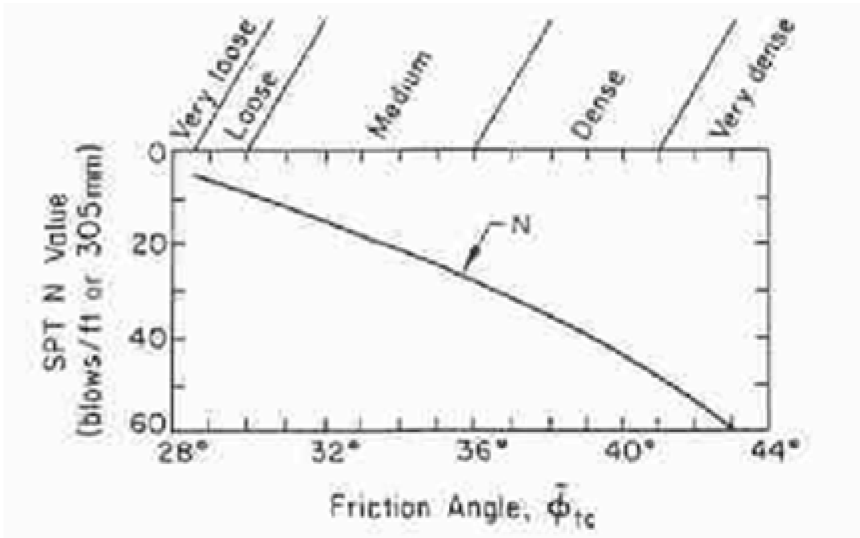


Figure 5. N-value and Friction Angle

APPENDIX C.6 Settlement Equations

2:1 Method - $\Delta\sigma = Q/((B+z)(L+z))$

Over consolidated clay - $S_p = ((C_sH)/(1+e_o))\log((\sigma'_o+\Delta\sigma')/(\sigma'_o))$

Normally consolidated clay - $S_p = ((C_cH)/(1+e_o))\log((\sigma'_o+\Delta\sigma')/(\sigma'_o))$

Combination 1 - Change in Stress 2:1 Method

Layer 1			Layer 2		
P =	231600	lbs	P =	231600	lbs
B =	0.75	ft	B =	0.75	ft
L =	436	ft	L =	436	ft
Z =	4.208333	ft	Z =	13.41667	ft
Δσ' =	106.1071	psf	Δσ' =	36.37657	psf

Combination 1 - Settlement

Layer 1			Layer 2		
Df =	19	in	Df =	19	in
Cc =	0.11		Cc =	0.15	
Cs =	0.06		H =	10	ft
H =	8.417	ft	e0 =	0.585	
e0 =	0.730		σ'o =	1730	psf
σ'o =	667.97	psf	Δσ' =	36.377	psf
σ'c =	1335.94	psf	Sp =	0.009	ft
Δσ' =	106.11	psf	(Eq. 9.16)	0.001	in
σ'o + Δσ' =	774.08	psf			
Sp =	0.019	ft	Sp Total =	0.225	in
(Eq. 9.18)	0.224	in	SF =	4.444534	

Combination 2 - Change in Stress 2:1 Method

Layer 1			Layer 2		
P =	231600	lbs	P =	231600	lbs
B =	0.75	ft	B =	0.75	ft
L =	436	ft	L =	436	ft
Z =	1.708333	ft	Z =	10.91667	ft
Δσ' =	215.235	psf	Δσ' =	44.41864	psf

Combination 2 - Settlement

Layer 1			Layer 2		
Df =	19	in	Df =	19	in
Cc =	0.11		Cc =	0.15	
Cs =	0.06		H =	15	ft
H =	3.417	ft	e0 =	0.585	
e0 =	0.730		σ'o =	1499.167	psf
σ'o =	379.64	psf	Δσ' =	44.419	psf
σ'c =	759.28	psf	Sp =	0.018	ft
Δσ' =	215.24	psf	(Eq. 9.16)	0.001	in
σ'o + Δσ' =	594.87	psf			
Sp =	0.023	ft	Sp Total =	0.279	in
(Eq. 9.18)	0.277	in	SF =	3.585978	

APPENDIX C.7 Bearing Capacity Equations

Effective Stress Analysis (ESA) – $q_u = c'N_c + qN_q + \frac{1}{2}\gamma BN_\gamma$

Total Stress Analysis (TSA) – $qu=5.7S_u+q$

Building load		
Q =	231600	lbs
B =	0.75	ft
L =	436	ft
A =	327	sf
FS =	4	
q =	708.2569	psf
	531.1927	lb/lf

Strip Foundation						
ESA				TSA		
	ø' =	38		Su =	6500	psf
	c' =	0		q =	278.71417	psf
	$q = \gamma D_f =$	278.7222	psf	$q_u =$	37328.714	psf
	γ =	115.33	pcf	$q_{all} =$	9332.1785	psf
	B =	0.75	ft			
	$N_c =$	77.5				
	$N_q =$	61.55				
	$N_\gamma =$	78.61				
	$q_u =$	20555.24	psf			
	$q_{all} =$	5138.809	psf			

APPENDIX C.8 WRI Structural Design of Slab on Grade

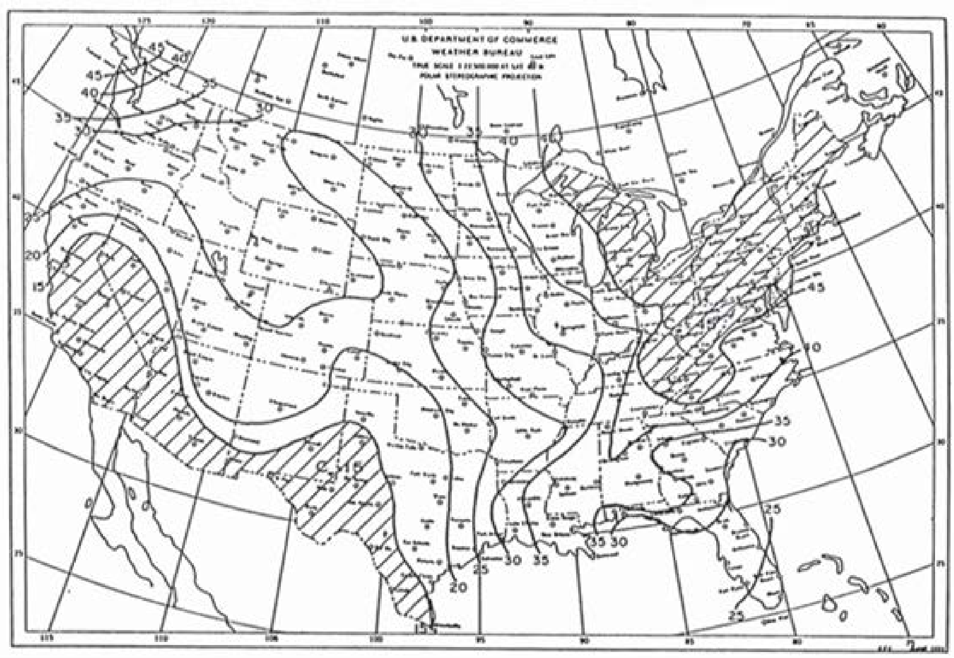


Figure 6. Climatic Rating (Cw) Chart

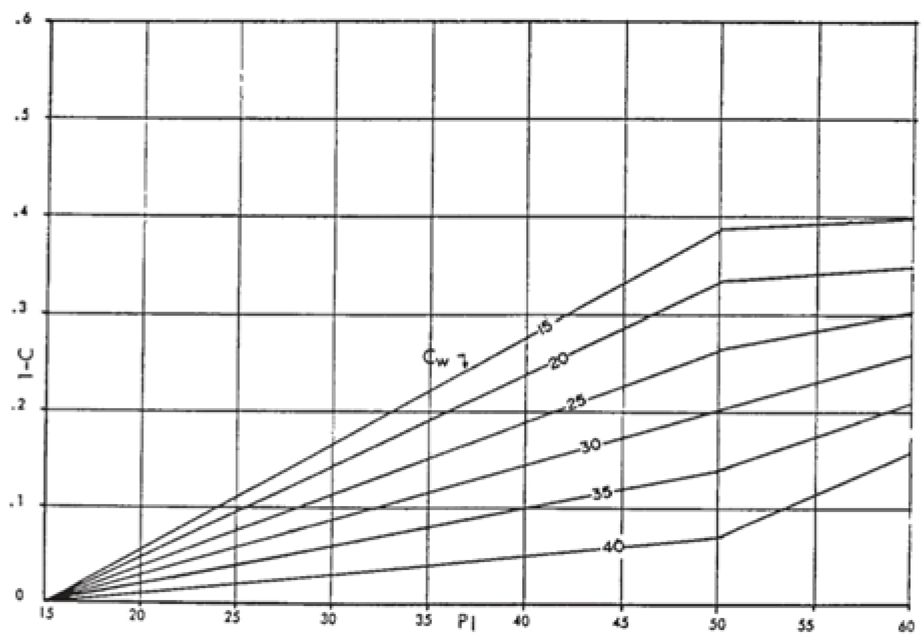


Figure 7. PI vs (1-C)

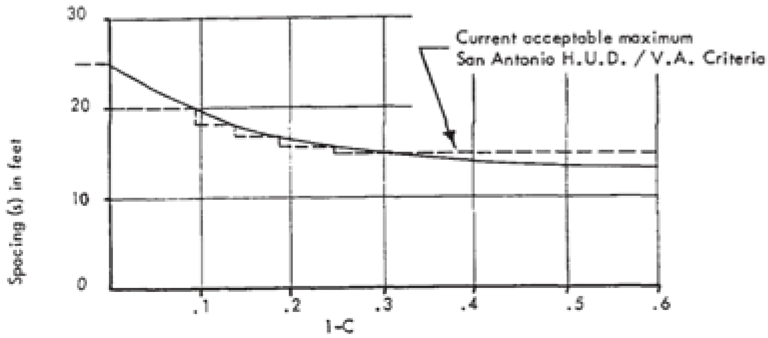


Figure 8. (1-C) vs Max Beam Spacing

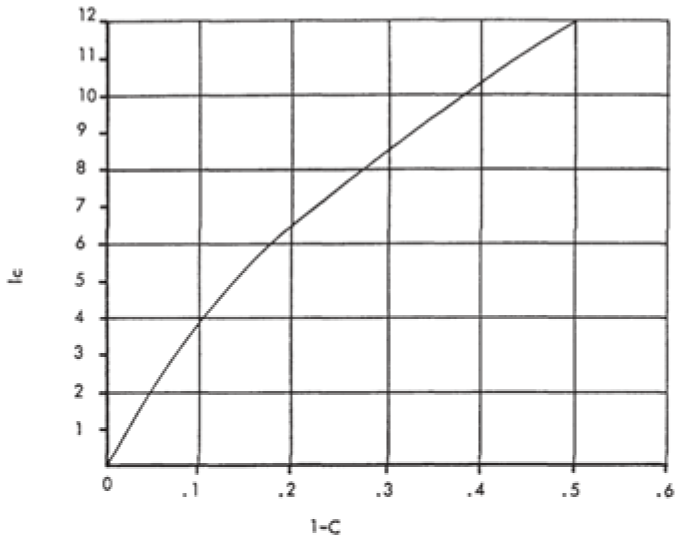


Figure 9. (1-C) vs Cantilever Length (lc)

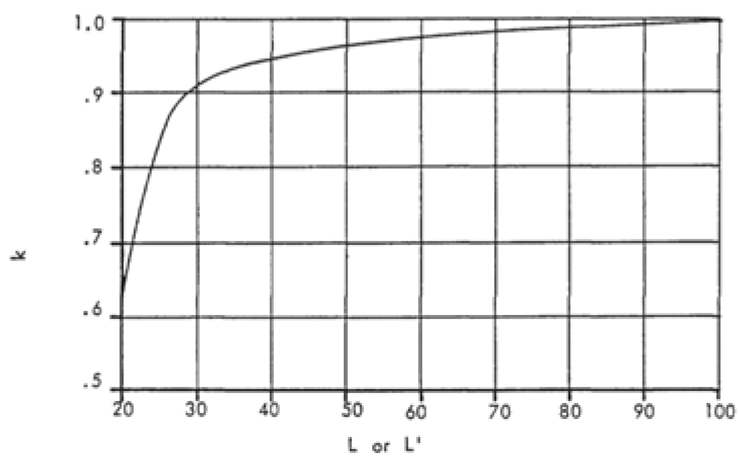


Figure 10. L or L' vs k

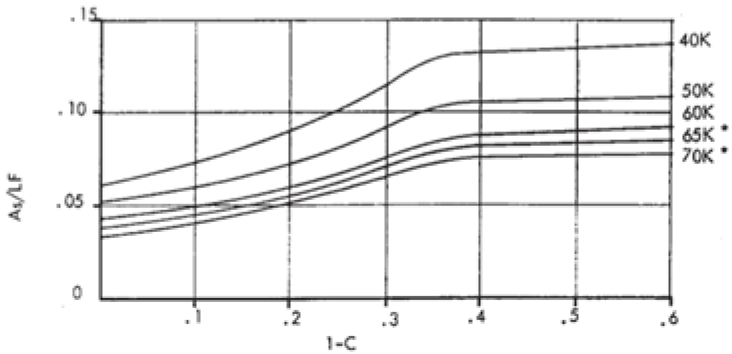


Figure 11. (1-C) vs As/LF

	Number of Beams		
	Effective PI =	13	
	f'c =	2500	
	Climate rating Cw =	30	
	Slope =	0	
	Unit Weight =	200	lbs/SF
Fig. 15	1-C =	0	
Fig. 17	S =	20	
Fig. 12	I_c =	4	
Fig. 13	K_l =	0.97	
	K_s =	0.96	
	$K_l I_c$ =	3.88	
	$K_s I_c$ =	3.84	
	N_l =	4	
	N_s =	4	

Slab Dimensions			* add 1ft
L =	56	ft	
L' =	53	ft	

Total Beam Width		
Assume beam Widths =	9	in
B_L =	36	in
B_S =	36	in
Geometry of building causes 5 beams		
B_L =	27	in

Long and Short Moments		
M_L =	79.78832	kf
M_S =	82.57536	kf

Beam Depths		
d_L =	20	in
d_S =	18	in

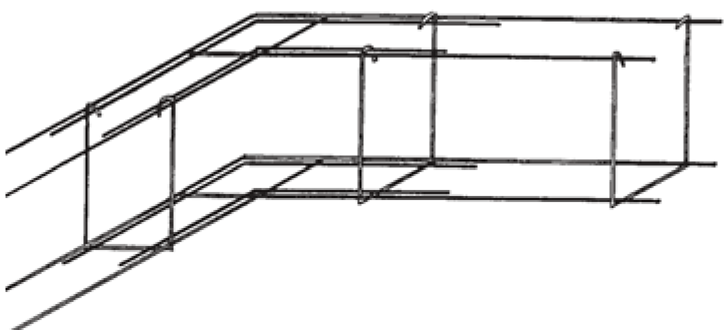
Solve for bottom steel in **LONG** direction					
Assume: 8 #5 bars			Assume: 8 #4 bars		
fy =	60000		fy =	60000	
As =	2.48	sq.in.	As =	1.6	sq.in.
b =	212	in	b =	212	in
a =	0.330	in	a =	0.213	in
Assume: lever arm for positive rienforcing = d-3			Assume: lever arm for positive rienforcing = d-3		
Mu =	210.8		Mu =	136	
M =	131.75		M =	85	
Check:	SAFE		Check:	SAFE	

Solve for top steel in **LONG** direction								
Assume lever arm for negative rienforcing = d-4			Assume: 8 #4 bars			Assume: 8 #3 bars		
			fy =	60000		fy =	60000	
Flange total =	176	in	As =	1.6	sq.in.	As =	0.88	sq.in.
Asfy =	3.85	k/lf	Mu =	128	kf	Mu =	70.4	kf
d-4 =	16	in	m =	80.0	kf	m =	44	kf
M =	75.3	kf	Check:	SAFE		Check:	SAFE	
Moment to be	32.7	kf						

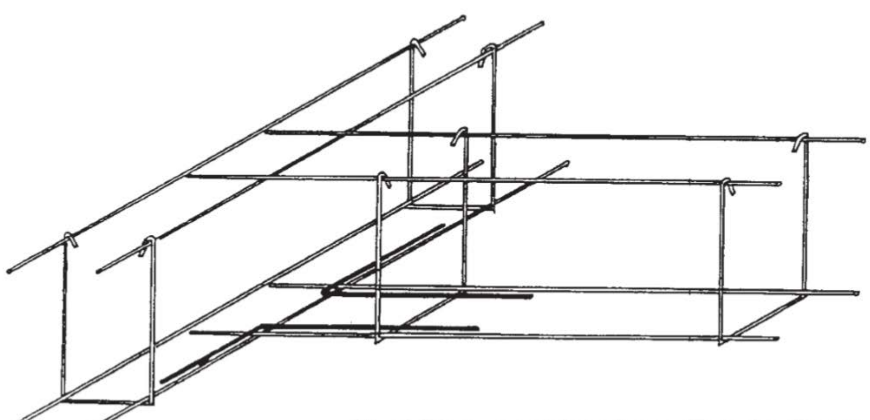
Solve for bottom steel in **SHORT** direction					
Assume: 8 #5 bars			Assume: 8 #4 bars		
fy =	60000		fy =	60000	
As =	2.48	sq.in.	As =	1.6	sq.in.
b =	212	in	b =	212	in
a =	0.330	in	a =	0.213	in
Assume: lever arm for nrgative rienforcing = d-3			Assume: lever arm for positive rienforcing = d-3		
Mu =	186		Mu =	120	
M =	116.25		M =	75	
Check:	SAFE		Check:	NO GOOD	

Solve for top steel in **SHORT** direction								
Assume lever arm for negative rienforcing = d-4			Assume: 8 #3 bars			Assume: 8 #4 bars		
			fy =	60000		fy =	60000	
Flange total =	176	in	As =	0.88	sq.in.	As =	1.6	sq.in.
Asfy =	3.85		Mu =	61.6	kf	Mu =	112	kf
d-4 =	14		m =	38.5	kf	m =	70	kf
M =	65.9		Check:	NO GOOD		Check:	SAFE	
Moment to be	41.4 kf							

APPENDIX C.9 Exterior Beam Tie Ins



Steel Placement at Corners



Steel Placement Interior to Exterior Beam

APPENDIX D. TRANSPORTATION

APPENDIX D.1 Horizontal Alignment Studies

Horizontal Alignment Studies

Table 1

Design Speed (mi/h)	f
10	0.38
15	0.32
20	0.26
25	0.23
30	0.20

Table 2

US Customary	
Design speed (mph)	Limiting superelevation rate (%)
15	8
20	8
25	10
30	11
35	11
40	11
45	12

General Input:

- Design speed: $V = 20$ mph
- Design superelevation: $e_{des} = 2\%$*
- Turning angle

*Note: Due to the low speed and volume of the inner roadway, a normal crown of 2% downslope is used

General Output

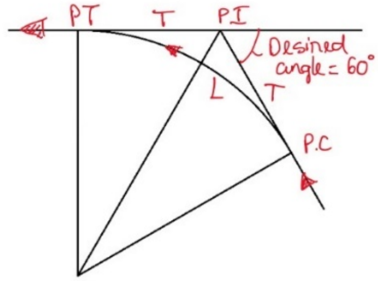
- Length of curve: L
- length of tangent: T
- Radius: R

L, T → Good for construction and survey

General parameter

Table 1 ⇒ $f = 0.26$;

Table 2 ⇒ $e_{max} = 8\%$



Road 2.1

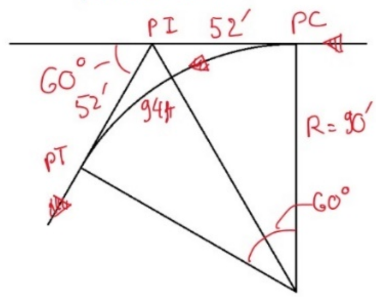
Radius:

$$R_{min} = \frac{v^2}{15(e_{des} + f_s)} = \frac{20^2}{15(8\% + 0.26)} = 78.43\text{ ft}$$

↳ Choose $\boxed{90\text{ ft}}$

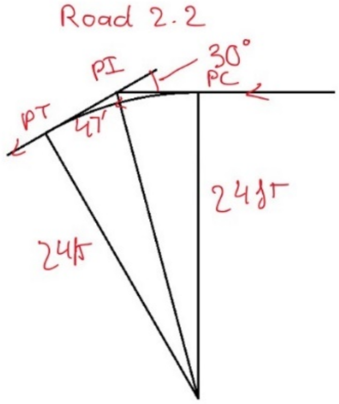
$$\left.\begin{aligned} L &= 100\,\Delta/D \\ D &= 5729.8/R \end{aligned}\right\} \Rightarrow L = 100 \times \frac{60}{5729.8/90} = \boxed{94\text{ ft}}$$

$$T = 90 \times \tan\frac{60}{2} = \boxed{52\text{ ft}}$$



Road 2.2

Same as Road 2.1

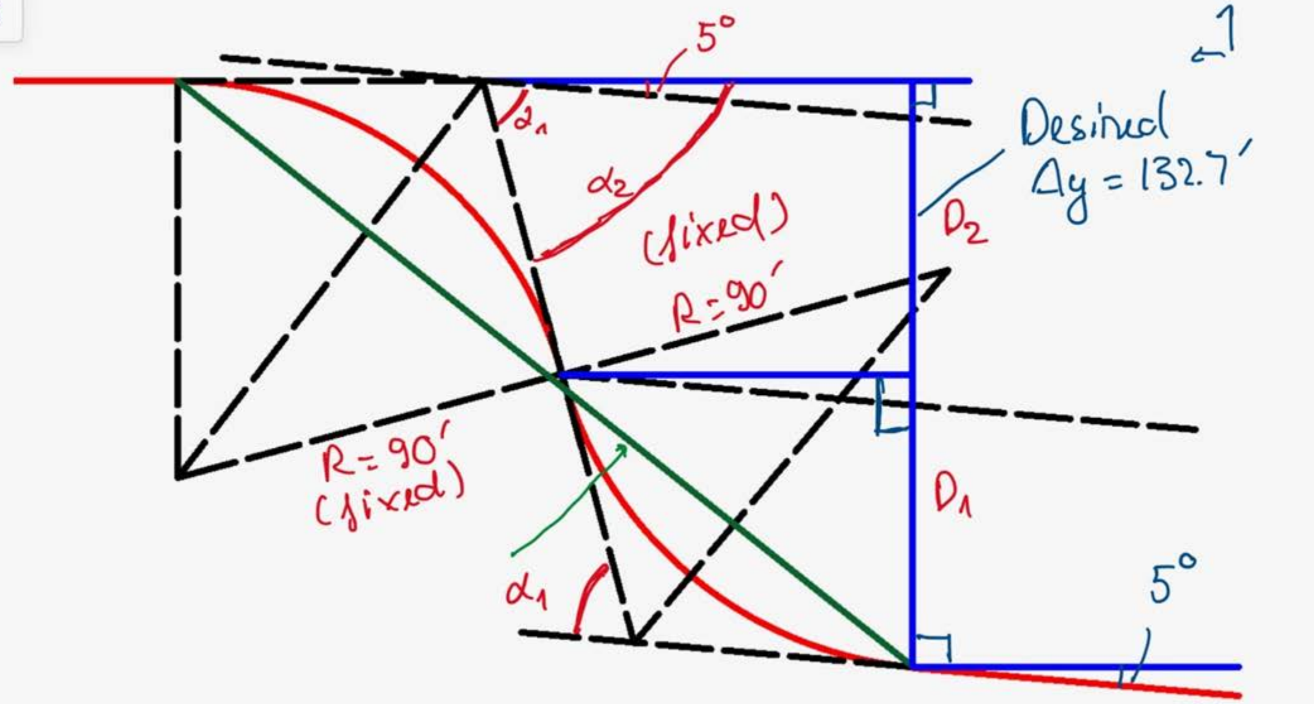


Exit Ramp 1 Curve C1

Same Radius $R = 90$ ft (Same v)

$$L = 100\,\Delta/D = 100 \times \frac{30}{5729.8/90} = 47\text{ ft}$$

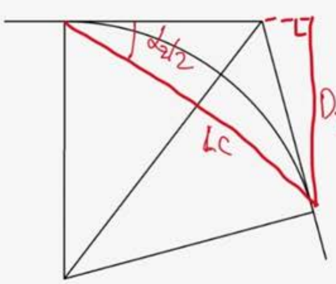
$$T = R\tan\Delta/2 = 90 \times \tan\frac{60}{2} = 24\text{ ft}$$



This curve is designed last because it consists least legal constraint to it compared to other design elements (such as exit ramp 2). The design criteria is to provide two consecutive curve that guide the traffic safely from entrance ramp to the parking lot area instead of taking the direct path (greenline) which is too dangerous due to sudden changes in angle. The radius of the curve is the same as inner roadway which is 90ft. In summary:

Input: R = 90ft; Achieve in a northing difference of 132.7ft; Tangent between curves is not desirable
Output: Turning angle alpha1 and alpha2

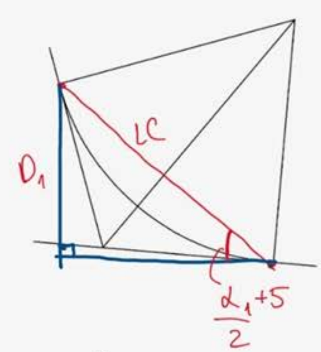
Calculate D_2



$$LC = 2R\sin\frac{\alpha_2}{2}$$

$$\Rightarrow D_2 = LC\sin\frac{\alpha_2}{2} = 2R\sin^2\frac{\alpha_2}{2}$$

Calculate D_1



$$LC = 2R\sin\frac{\alpha_1}{2}$$

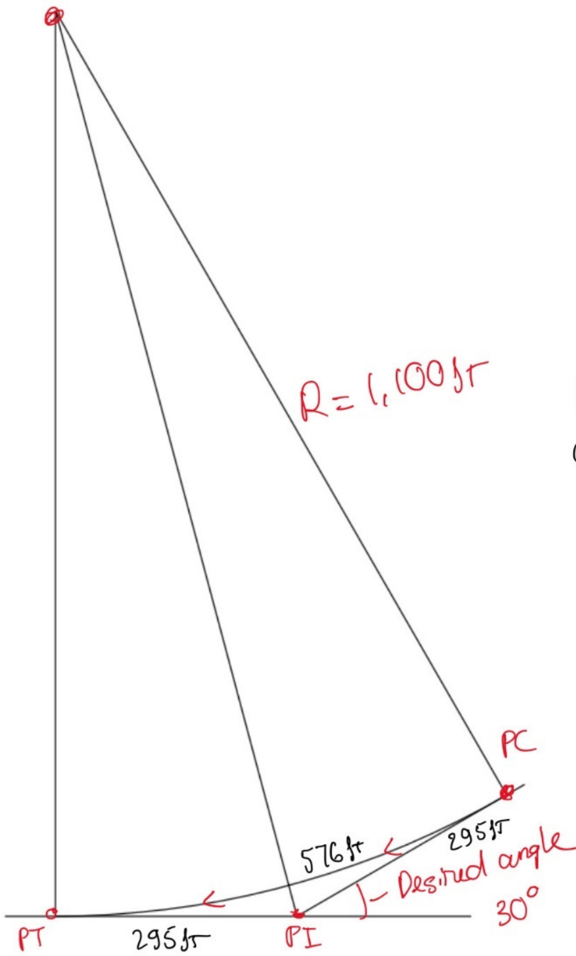
$$D_1 = LC\sin\left(\frac{\alpha_1}{2}+5\right)$$

$$\Rightarrow D_1 = 2R\sin\frac{\alpha_1}{2}\sin\left(\frac{\alpha_1}{2}+5\right)$$

We have $D_1 + D_2 = 2\overset{90'}{R}\left(\sin^2\frac{\alpha_2}{2} + \sin\frac{\alpha_1}{2}\sin\left(\frac{\alpha_1}{2}+5\right)\right) = 132.7 \ (1)$

$\alpha_2 = \alpha_1 + 5 \ (2)$

From (1), (2) $\Rightarrow$ $\boxed{\begin{array}{l}\alpha_1 = 70 \\ \alpha_2 = 75^\circ\end{array}}$



Ramp design speed: $V = 60$ mph $\Rightarrow f_s = 0.12$

Maximum superelevation: $e_{max} = 12\%$

(low volume; gravel roads

$$\Rightarrow R_{min} = \frac{V^2}{15(e_{max} + f_s)} = \frac{60^2}{15(12\% + 0.12)} = 1{,}000 \text{ ft}$$

* Note: 901 Design decides to use this curve as part of the acceleration lanes. AASHTO (2011) specifies only a curve with R > 1000 ft can facilitate the acceleration of merging vehicles.

Conclusion: $e_{des} = 12\%$; $R = 1{,}100$ ft.

Length of Curve:

$$L = 100\,\Delta/D = 100\,\frac{30}{5729.8/1100} = \boxed{576 \text{ ft}}$$

This attribute to acceleration lane

Tangent:

$$T = R \tan\frac{\alpha}{2} = 1{,}100 \text{ ft} \times \tan\frac{30}{2} = \boxed{295 \text{ ft}}$$

APPENDIX D.2 Superelevation Studies

US Customary

$$L_r = \frac{(wn_1)e_d}{\Delta}(b_w) \qquad (3\text{-}25)$$

where:

- L_r = minimum length of superelevation runoff, ft;
- Δ = maximum relative gradient, percent;
- n_1 = number of lanes rotated;
- b_w = adjustment factor for number of lanes rotated;
- w = width of one traffic lane, ft (typically 12 ft);
- e_d = design superelevation rate, percent

US CUSTOMARY

Number of Lanes Rotated, n_l	Adjustment Factor, b_w[a]	Length Increase Relative to One-lane Rotated (=$n_l b_w$)
1	1.00	1.0
1.5	0.83	1.25
2	0.75	1.5
2.5	0.70	1.75
3	0.67	2.0
3.5	0.64	2.25

US Customary

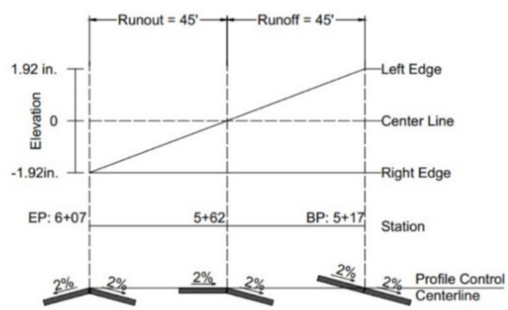
Design speed (mph)	Maximum relative gradient (%)	Equivalent maximum relative slope
15	0.78	1:128
20	0.74	1:135
25	0.70	1:143
30	0.66	1:152
35	0.62	1:161
40	0.58	1:172
45	0.54	1:185
50	0.50	1:200
55	0.47	1:213
60	0.45	1:222
65	0.43	1:233
70	0.40	1:250
75	0.38	1:263
80	0.35	1:286

US Customary

$$L_t = \frac{e_{NC}}{e_d} L_r \qquad (3\text{-}26)$$

where:

- L_t = minimum length of tangent runout, ft;
- e_{NC} = normal cross slope rate, percent;
- e_d = design superelevation rate, percent;
- L_r = minimum length of superelevation runoff, ft



Entrance Ramp in S.D.4

Given: located at the end of deceleration lane

Design speed: $V = 20$ mph

Design Superelevation: $e_d = 2\%$

Normal crown: $e_{nc} = 2\%$

Width: $w = 16$ ft

\# of lanes rotated: $n_1 = 1$

Solution

$V = 20$ mph $\Rightarrow$ Maximum gradient $\Delta = 0.74$

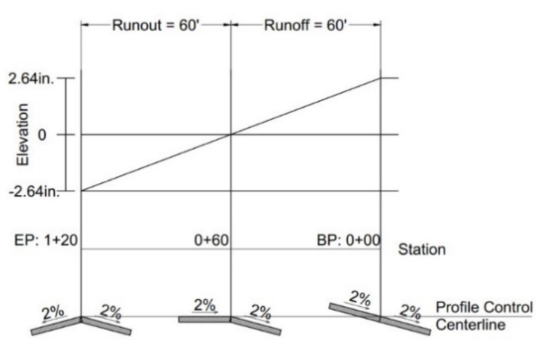
$n_1 = 1 \Rightarrow b_w = 1$

Run off

$$L_r = \frac{w n_1 e_d b_w}{\Delta} = \frac{16 \times 1 \times 2 \times 1}{0.74} = 43 \text{ ft} \Rightarrow \text{Choose } \boxed{45'}$$

Runout

$$L_t = \frac{e_{nc}}{e_d} L_r = \frac{2}{2} \times 45 = \boxed{45'}$$



Road 3.2 in S.D.13

Given

$V = 20$mph

$e_{des} = 2\%$

$e_{nc} = 2\%$

$W = 22$ ft

$n_1 = 1$

Solution

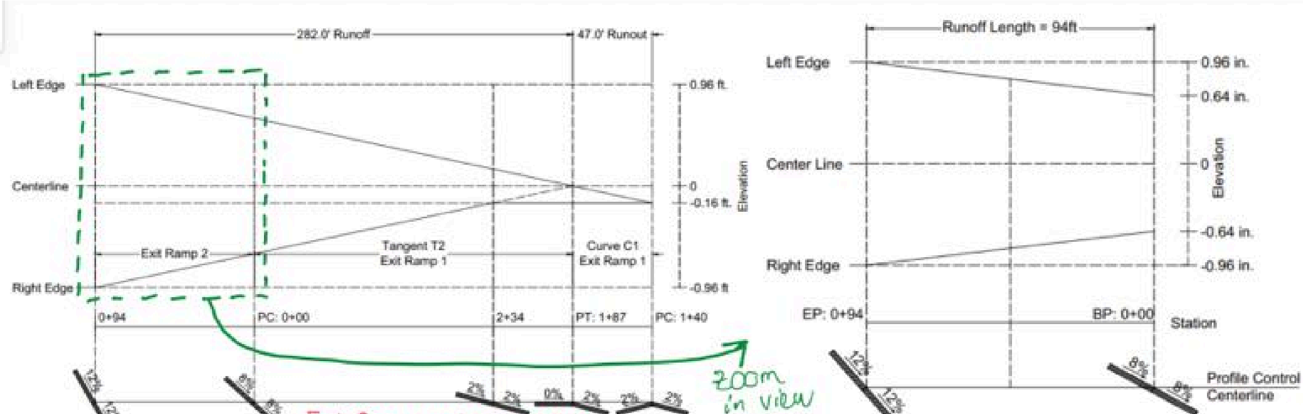
$V = 20$mph $\Rightarrow \Delta = 0.74$

$n_1 = 1 \Rightarrow b_w = 1$

Runoff

$$L_r = \frac{w n_1 e_d b_w}{\Delta} = \frac{22 \times 1 \times 2 \times 1}{0.74} = 59.4 \Rightarrow \text{Choose } 60'$$

Runout: $L_t = \frac{e_{nc}}{e_d} L_r = \frac{2}{2} \times 60 = 60'$



Exit Ramp 1 and 2 in S.D.8

Design speed: $V = 20$ mph (This section is not part of the acceleration lane and is still considered as inner guiding roadway)

$\Rightarrow \quad \Delta = 0.74$

$n_1 = 1 \Rightarrow b_w = 1; \quad e_{des} = 12; \quad e_{NC} = 2$

Runoff: $L_r = \dfrac{w n_1 e_d b_w}{\Delta} = \dfrac{16 \times 1 \times 12 \times 1}{0.74} = 260$ ft

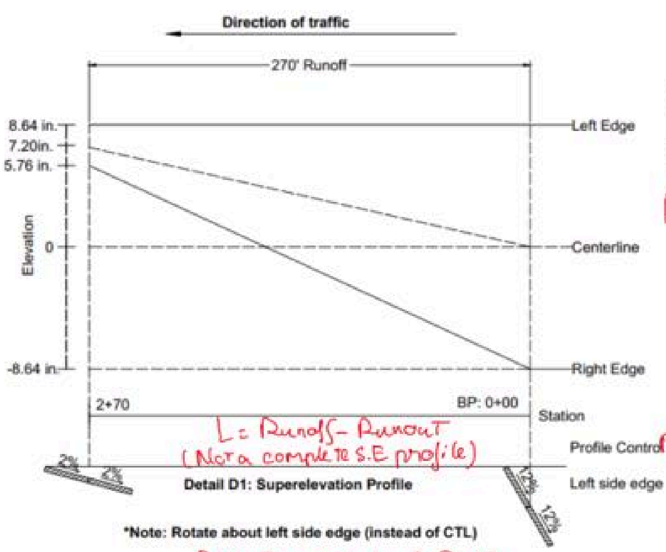
Runout: $L_t = \dfrac{e_{NC}}{e_d} L_r = \dfrac{2}{12} \times 260 = 43$ ft

However: • Curve C_1 has an arc length $L = 47$ ft

• For convenience in construction: Curve C_1 lies entirely on Runout area

} Choose Runout $L_t = 47'$

$\Rightarrow$ Revised Runoff: $L_r = \dfrac{e_d}{e_{NC}} L_t = \dfrac{12}{2} \times 47 = 282'$



Detail D1: Superelevation Profile

*Note: Rotate about left side edge (instead of CTL)

Exit Ramp 3 in S.D. 10

$V = 60$ mph $\Rightarrow \Delta = 0.45$

$e_{des} = 12 \qquad w = 12$ ft

Runoff: $L_r = \dfrac{w n_1 b_w e_d}{\Delta} = \dfrac{12 \times 1 \times 1 \times 12}{0.45} = 320$ ft

Runout: $L_t = \dfrac{e_{NC}}{e_{des}} L_{off} = \dfrac{2}{12} \times 320 = 53$ ft

$\Rightarrow L =$ Runoff − Runout $= 320 - 53 = 267$

$\hookrightarrow$ Choose 270 ft

APPENDIX D.3 Entrance Deceleration Lane Studies

US Customary										
		Deceleration length, L (ft) for design speed of exit curve, $V'N$ (mph)								
		Stop condition	15	20	25	30	35	40	45	50
Highway design speed, V (mph)	Speed reached, V_a (mph)	For average running speed on exit curve, V'_a (mph)								
		0	14	18	22	26	30	36	40	44
30	28	235	200	170	140	—	—	—	—	—
35	32	280	250	210	185	150	—	—	—	—
40	36	320	295	265	235	185	155	—	—	—
45	40	385	350	325	295	250	220	—	—	—
50	44	435	405	385	355	315	285	225	175	—
55	48	480	455	440	410	380	350	285	235	—
60	52	530	500	480	460	430	405	350	300	240
65	55	570	540	520	500	470	440	390	340	280
70	58	615	590	570	550	520	490	440	390	340
75	61	660	635	620	600	575	535	490	440	390

V = design speed of highway (mph)
V_a = average running speed on highway (mph)
$V'N$ = design speed of exit curve (mph)
V'_a = average running speed on exit curve (mph)

Highway design speed

$V = 70$ mph

$\Rightarrow V_a = 58$ mph

Inner Roadway speed:

$V_n = 20$ mph

$\Rightarrow V'_a = 18$ mph

$\Rightarrow$ Deceleration Lane length:

$L = 570$ ft

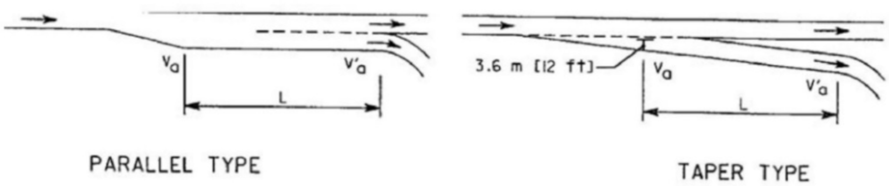
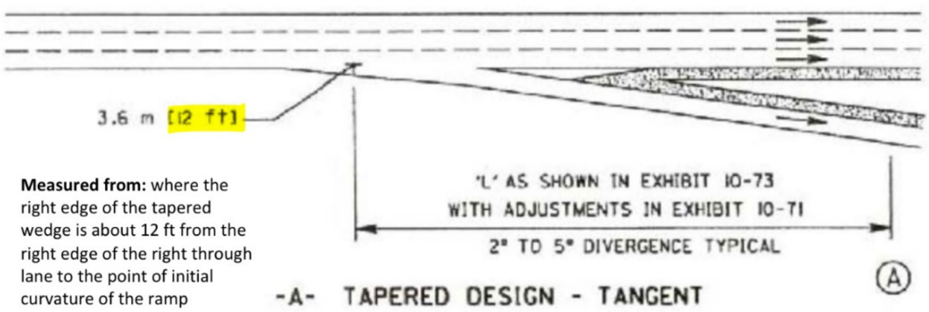
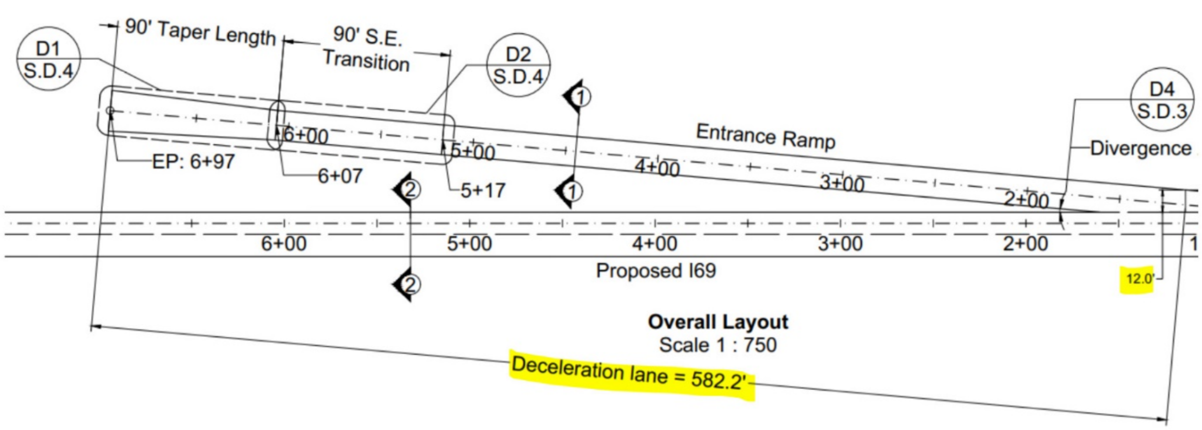


Exhibit 10-73. Minimum Deceleration Lengths for Exit Terminals with Flat Grades of Two Percent or Less



Measured from: where the right edge of the tapered wedge is about 12 ft from the right edge of the right through lane to the point of initial curvature of the ramp

To: First Point of Curvature



APPENDIX D.4 Exit Acceleration Lane Studies

Inner Roadway:

Design speed = 20 mph => v'_a = 18 mph

Highway:

Design speed = 70 mph => v_a = 53 mph

Acceleration lane: $\boxed{L = 1520\ ft}$

US 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	1340	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.

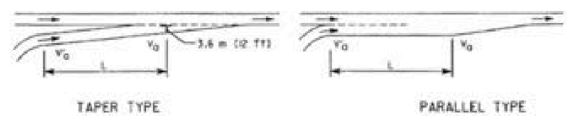
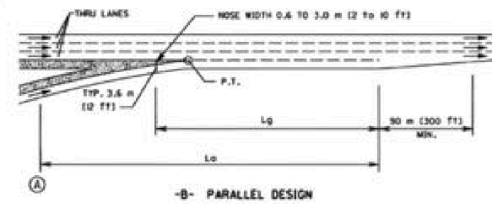


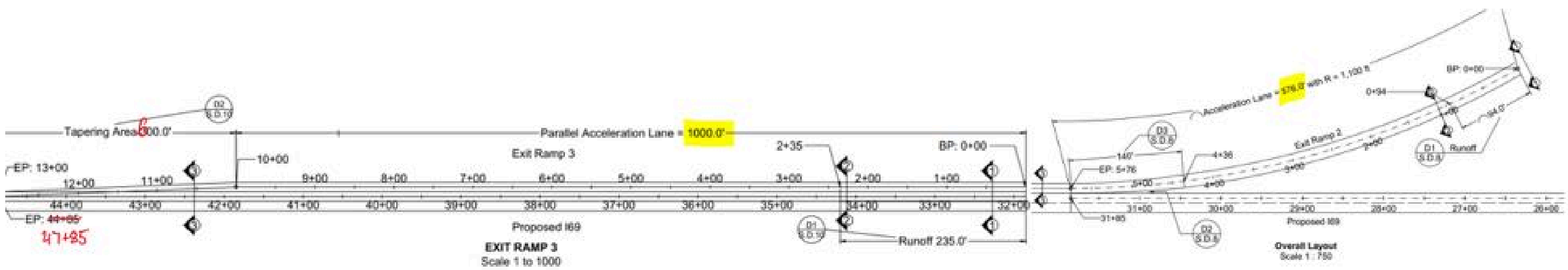
Exhibit 10-70. Minimum Acceleration Lengths for Entrance Terminals with Flat Grades of Two Percent or Less



NOTES:

1. La IS THE REQUIRED ACCELERATION LENGTH AS SHOWN IN EXHIBIT 10-70 OR AS ADJUSTED BY EXHIBIT 10-71.
2. POINT (A) CONTROLS SPEED ON THE RAMP. La SHOULD NOT START BACK ON THE CURVATURE OF THE RAMP UNLESS THE RADIUS EQUALS 300 m (1000 FT) OR MORE.
3. Lg IS REQUIRED GAP ACCEPTANCE LENGTH. Lg SHOULD BE A MINIMUM OF 90 TO 150 m (300 TO 500 FT) DEPENDING ON THE NOSE WIDTH.
4. THE VALUE OF La OR Lg, WHICHEVER PRODUCES THE GREATER DISTANCE DOWNSTREAM FROM WHERE THE NOSE EQUAL 0.6 m (2 FT), IS SUGGESTED FOR USE IN THE DESIGN OF THE RAMP ENTRANCE.

Exhibit 10-69. Typical Single-Lane Entrance Ramps



Exit Ramp 2: Provide 576 ft of Acceleration lane (R = 1,100 ft > 1000)

Exit Ramp 3: Provide 1000 ft =>

} => A total of 1,576 ft > 1,520 ft

Taper: Because the acceleration lane is long: 1 = 1520 > 1300 ft => A 50:1 Taper is recommended

$$\frac{50}{1} = \frac{L}{\text{Road Width}} = \frac{1}{12} \Rightarrow L = 600\ ft.$$

APPENDIX D.5 Entrance Ramp Studies

US 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~~	$\boxed{50}$	55
Lower range (50%)	15	18	20	23	25	28	30	30	35	40
Corresponding minimum radius (ft)	see Exhibits 3-25 through 3-29									

Section 1. Design Speed

The entrance ramp is defined as a Ramp for right turns => Upper range speed is used. : $V_{des} = 60$ mph

US Customary									
	Pavement width (ft)								
Radius on inner edge of pavement	Case I One-lane, one-way operation—no provision for passing a stalled vehicle			Case II One-lane, one-way operation—with provision for passing a stalled vehicle			Case III Two-lane operation—either one-way or two-way		
	Design traffic conditions								
R (ft)	A	B	C	A	B	C	A	B	C
50	18	18	23	20	26	30	31	36	45
75	16	17	20	19	23	27	29	33	38
100	15	16	18	18	22	25	28	31	35
150	14	15	17	18	21	23	26	29	32
200	13	15	16	17	20	22	26	28	30
300	13	15	15	17	20	22	25	28	29
400	13	15	15	17	19	21	25	27	28
500	12	15	15	17	19	21	25	27	28
Tangent	12	14	$\boxed{14}$	17	18	20	24	26	26
Width modification regarding edge treatment									
No stabilized shoulder	None			None			None		
Sloping curb	None			None			None		
Vertical curb:									
one side	Add 1 ft			None			Add 1 ft		
two sides	Add 2 ft			Add 1 ft			Add 2 ft		
Stabilized shoulder, one or both sides	Lane width for conditions B & C on tangent may be reduced to 12 ft where shoulder is 4 ft or wider			Deduct shoulder width; minimum pavement width as under Case I			Deduct 2 ft where shoulder is 4 ft or wider		

Note: A = predominantly P vehicles, but some consideration for SU trucks.
B = sufficient SU vehicles to govern design, but some consideration for semitrailer combination trucks.
C = sufficient bus and combination-trucks to govern design.

Section 2. Ramp pavement width.

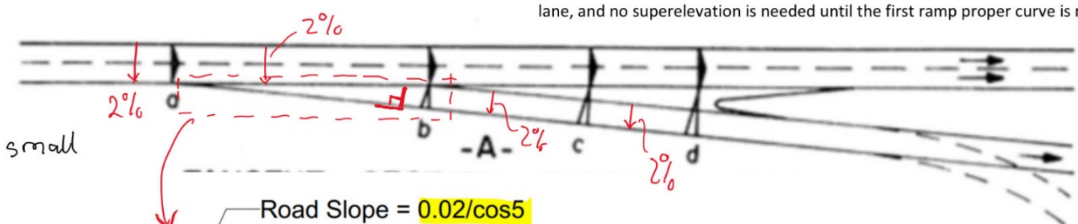
There are sufficient bus / truck to govern the design => Traffic condition C

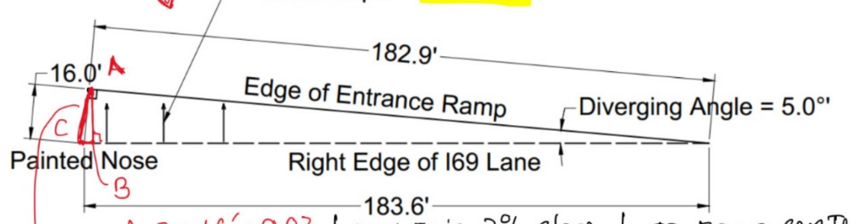
- Entrance ramp is Tangent (5° diverging angle) => Tangent
- Shoulder provided on both sides

=> Recommended width: 14 ft => Choose 16 ft

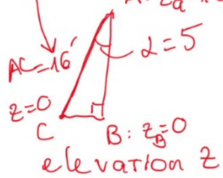
Section 3: Cross Slopes.

"A tapered exit from a tangent section with the first ramp curve falling beyond the design deceleration length. The normal cross slope is projected onto the auxiliary lane, and no superelevation is needed until the first ramp proper curve is reached"



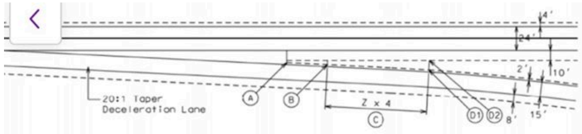


A: $z_a = 16' \times 0.02$ (maintain 2% slope ⊥ to ramp centerline



Road slope in direction perpendicular to I69 center line : $Slope = \frac{z_a}{AB} = \frac{AC \times 2\%}{AC \cos\alpha} = \frac{0.02}{\cos 5}$

APPENDIX D.6 Entrance Ramp Gore Studies



EXIT RAMP GORE AREA

(A) Exit Ramp Gore begins where the edge of the ramp shoulder establishes a two (2) foot knockout with the edge of the mainline shoulder.

(B) Establish a point of intersection with a line four (4) feet parallel to the edge of the ramp shoulder that intersects the mainline shoulder.

(C) Determine the length of gore beyond the point of intersection from AASHTO Green Book Exhibit 10-61.

Example: Design speed of approach highway = 75 mph.
Z = length, in feet, of the nose taper per foot of nose offset = 37.5'
Therefore, the end of the gore is 37.5 x 4 = 150' beyond the point of intersection.

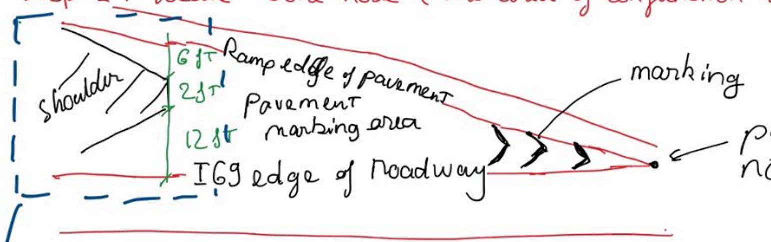
Note: The width at the end of the gore will vary between the edge of the ramp shoulder and mainline shoulder depending on the deflection of the ramp and any existing curves on mainline.

(D) Show station and offset, (1) & (2), at the end of the exit ramp gores on the "Ramp Gore Layout" included in the plans.

LEGEND
——— Driving Edge
- - - - Shoulder Edge

US Customary	
Design speed of approach highway (mph)	Length of nose taper (Z) per unit width of nose offset
30	15.0
35	17.5
40	20.0
45	22.5
50	25.0
55	27.5
60	30.0
65	32.5
70	35.0
75	37.5

Step 1: locate Gore nose (The area of conjunction between 2 shoulder)



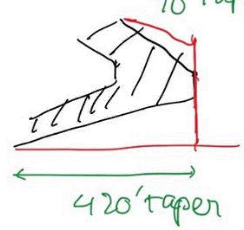
can be located by Autocad = 228.6'

Step 2: Calculate pavement taper

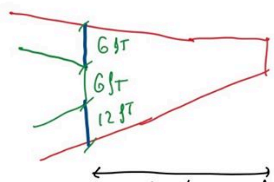
Highway speed V = 70mph => 1:35 pavement taper ratio

I69: L_{Taper} = Offset × 35 = 12' × 35 = 420'

Ramp: L_{Taper} = Offset × 35 = 2' × 35 = 70'

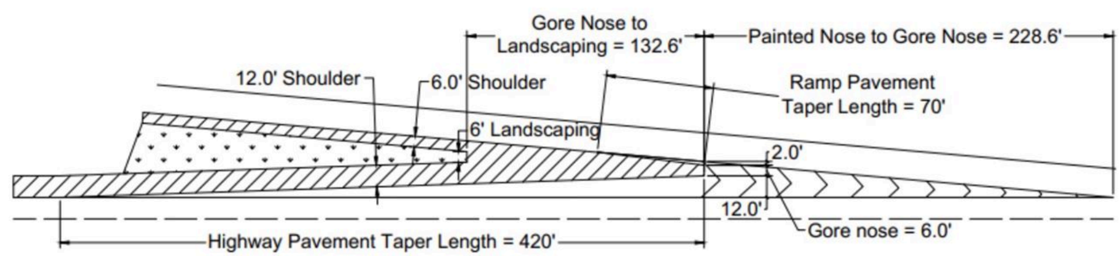


Step 3: Locate landscaping area



Can be located via AutoCAD

The landscaping cross section consists of:
- 6 ft shoulder
- 6 ft Landscaping nose
- 12 ft shoulder



Details D4: Gore Detail

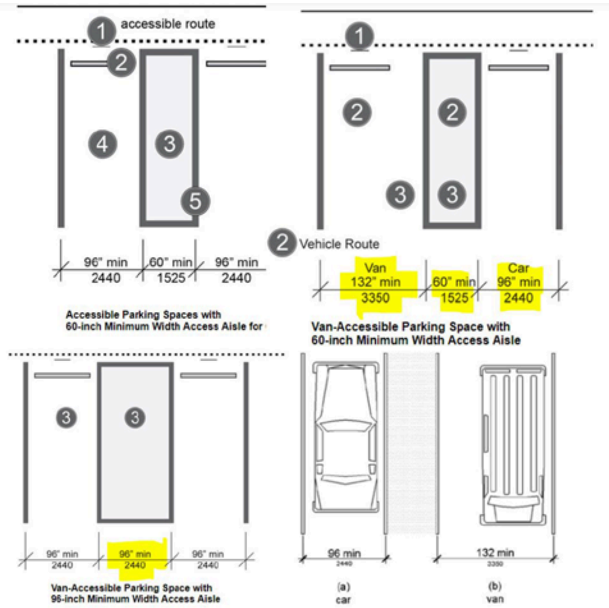
APPENDIX D.7 Accessible Parking Studies

Minimum Number of Accessible Parking Spaces

2010 Standards (208.2)

Total Number of Parking Spaces Provided in Parking Facility (per facility)	(Column A) Minimum Number of Accessible Parking Spaces (car and van)	Mininum Number of Van-Accessible Parking Spaces (1 of six accessible spaces)
1 to 25	1	1
26 to 50	2	1
51 to 75	3	1
76 to 100	4	1
101 to 150	5	1
151 to 200	6	1
201 to 300	7	2
301 to 400	8	2
401 to 500	9	2
500 to 1000	2% of total parking provided in each lot or structure	1/6 of Column A*
1001 and over	20 plus 1 for each 100 over 1000	1/6 of Column A*

*one out of every 6 accessible spaces



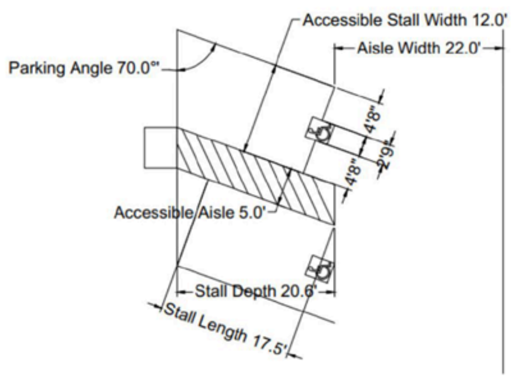
Conclude: 6 accessible parking including 1 van accessible

2x Pair: 1 car + 1 car

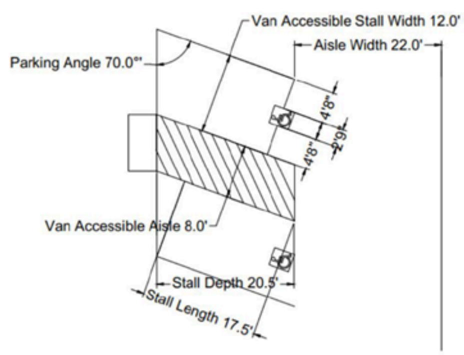
1x Pair: 1 car + 1 van

Target: • Car accessible stall width = Van accessible stall width = 12ft

• Car accessible aisle = 5ft   Van accessible aisle = 8ft



Pair of 2 car accessible parking
Scale 1 to 150



Pair of 1 car and 1 van accessible parking
Scale 1 to 150

APPENDIX D.8 Turning Radius Studies

Passenger Car (P)

Interstate Semi Trailer (WB-67)

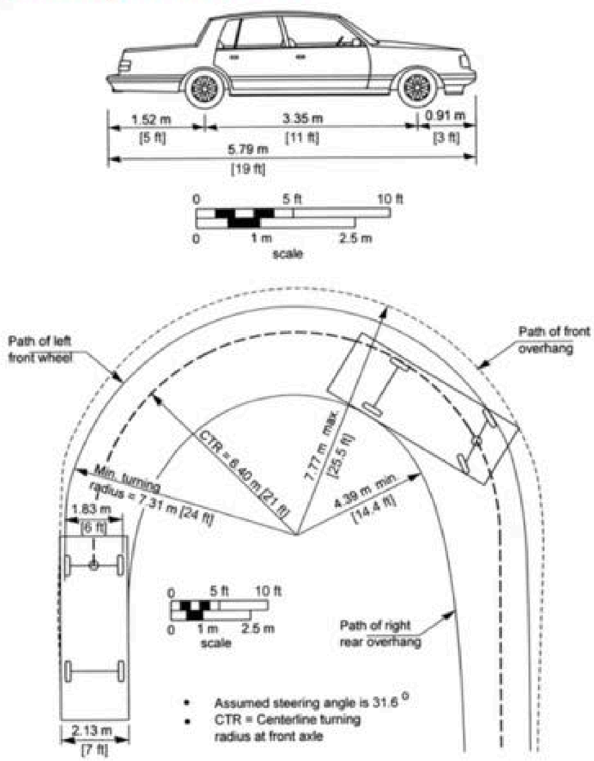


Exhibit 2-3. Minimum Turning Path for Passenger Car (P) Design Vehicle

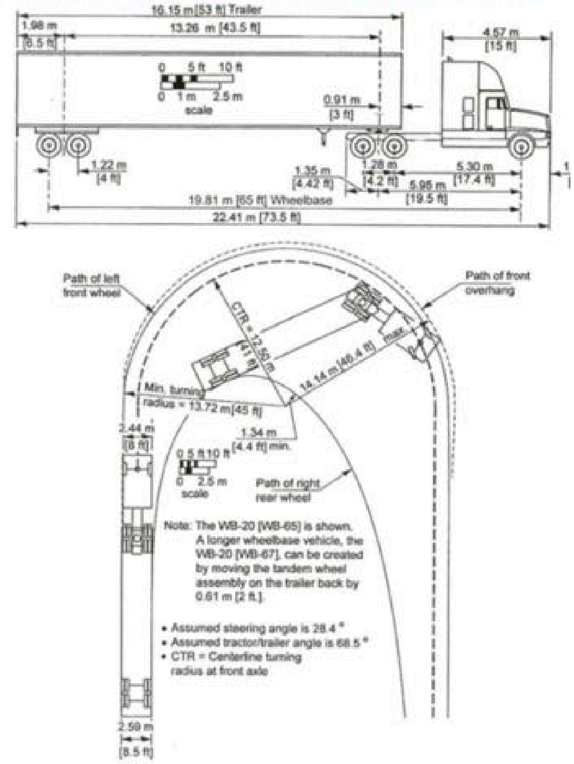


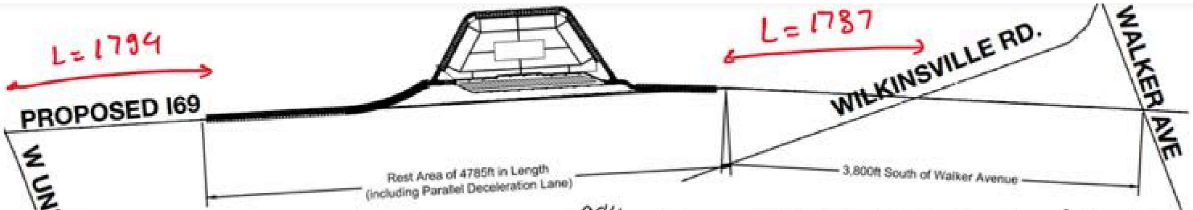
Exhibit 2-16. Minimum Turning Path for Interstate Semitrailer (WB-20 [WB-65 and WB-67]) Design Vehicle

	Car	Truck (WB-67)
Minimum Turning Radius (outside)	24	45
Centerline turning radius	21	41
Minimum Inside Radius	14.4	4.4

* Dimension shown in Set of drawings.

Road	Drawing Shee	Station	Description	Centerline Radius	Outside curb Radius	Inside curb radius	Car	Truck (WB-6
Road 1.1	S.D.13	0+00-1+00	Curve	90	101	79	Good	Good
Road 1.1	S.D.13	1+00-2+00	Curve	90	101	79	Good	Good
Road 1.1-1.2	S.D.14	3+00/0+00	Intersection	25	36	25	Good	Good
Road 2.1	S.D.13	0+00-0+94	Curve	90	101	79	Good	Good
Road 2.2	S.D.13	0+00-0+94	Curve	90	101	79	Good	Good
Road 3.1-3.2	S.D.14	1+20/1+20	Intersection	25	36	25	Good	Good
Exit Ramp 1	S.D.5	1+40-1+87	Curve	90	101	79	Good	Good
Exit Ramp 2	S.D.7	0+00-5+76	High Speed Turns	N/A	N/A	N/A	N/A	N/A
Truck Parking Entrance	S.D.12	N/A	Inner Road	95	105	85	Good	Good
Truck Parking Exit	S.D.12	N/A	Inner Road	107.5	115	100	Good	Good
Car Parking Stall	S.D.11	N/A	Inner Parking	22	26.5	17.5	Good	N/A
Truck Parking Stall	S.D.11	N/A	Inner Parking	45	52.5	37.5	N/A	Good

APPENDIX D.9 Level of Service Studies



$FFS = 75.4 - f_{LW} - f_{LC} - 3.22\,TRD^{0.84} = 65$ mph ($f_{LW} = 0.0$, $f_{LC} = 0.0$, $TRD = 4R/\text{mile}$)

$DDHV = AADT \times K \times D = 3427$ veh/h ($AADT = 35150$ veh/d (TDOT info · 30yr), $K = 0.15$ (Rural), $D = 0.65$ (Rural))

$f_{hv} = \dfrac{1}{1+P_T(E_T-1)+P_R(E_R-1)} = 50/53$ ($P_T = 10\%$, $E_T = 1.5$, $P_R = 0$)

Lane Width (ft)	Reduction in Free-Flow Speed, f_{LW} (mi/h)
≥12	0.0
11	1.9
10	6.6

Right Shoulder Lateral Clearance (ft)	Reduction in Free-Flow Speed, f_{LC} (mi/h)			
	Lanes in One Direction			
	2	3	4	≥5
≥6	0.0	0.0	0.0	0.0

Upgrade (%)	Length (mi)	E_T: Percentage of Trucks and Buses (%)								
		2	4	5	6	8	10	15	20	≥25
<2	All	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
>2–3	0.00–0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5

Grade (%)	Length (mi)	E_R: Percentage of RVs (%)								
		2	4	5	6	8	10	15	20	≥25
≤2	All	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
[illegible]	0.00–0.50	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Level of Service	Weaving Areas: On Freeways	Weaving Areas: On Multilane Highways or C-D Roadways	Merge or Diverge Areas: On Freeways, Multilane Highways, or C-D Roadways
	Density Range (pc/mi/ln)		
A	0–10	0–12	0–10
B	>10–20	>12–24	>10–20
C	>20–28	>24–32	>20–28
D	>28–35	>32–36	>28–35
E	>35	>36	37 >35

Passenger car equivalent

$\Rightarrow V_{pc} = \dfrac{V}{PHF \times N \times f_{hv} \times f_p} = \dfrac{3427}{0.85 \times 2 \times 50/53 \times 1}$

$= 2595$ pc/h/ln Speed = 65 mph

$\Rightarrow$ Density : $k = \dfrac{\text{Vol}}{\text{speed}} = \dfrac{2595}{70} = 37$ pc/mi/ln

$\Rightarrow$ Merging Demand > capacity

• Check for weaving: Shortest length to adjacent ramp L = 1740 ft (to Wilkinsville)

Given 3400 → 300, 300, 100; convert to passenger car equivalent → 4460, 395, 395, 130

10% Truck

0% RV Level

PHF = 0.85 (Rural)

FFS = 65 mph

ID = 0.8 IT/mile

$f_p = 0.95$ (commuter from other area)

$V_{weav} = 395 + 395 = 790$

$V_{nw} = 4460 + 130 = 4590$

$V = 5350 \Rightarrow V/n = 2690$ pc/h/ln

$VR = V_w/V_{nw} = 0.17$

$LC_{min} = LC_{fr} \times V_{fr} + LC_{rf} \times V_{rf} = 1 \times 395 + 1 \times 395$

$L_{max} = 5728(1+VR)^{1.6} - 1566\,N_{wl} = 5798 > 1740$ ($VR = 0.17$)

Capacity $c_{IWL} = c_{IFL} - 438.2(1+VR)^{1.6} + 0.0765\,L_s + 119.8\,N_{wl}$ ($v = 65 \rightarrow 2350$, $VR = 0.17$, $L_s = 1740$)

$= 2040$ pc/h/ln

$c_{W1} = 2040 \times 2 \times 50/53 \times 0.95 = 3657$ veh/h

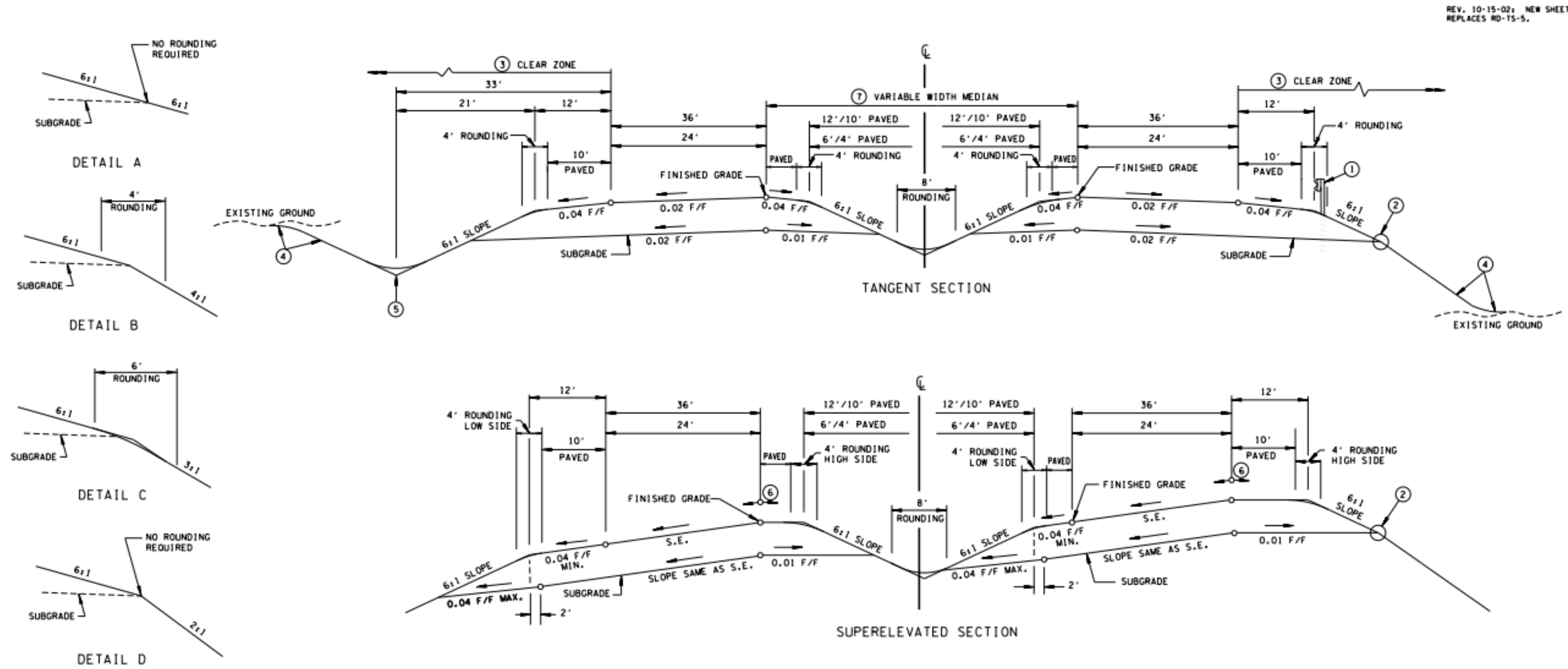
$c_{IW} = \dfrac{2400}{0.17} = 14118$ $c_{W2} = c_{IW} f_{hv} f_p = 12653$ veh/h

Demand:

$v = V/PHF = 4100/0.85 = 4824 > 3657$

↳ The facility may not in good condition 30 years to come

APPENDIX D.10 TDOT Design Freeways with Depressed Medians



FOOTNOTES

1. SEE GUARDRAIL STANDARD DRAWINGS FOR TYPICAL GUARDRAIL PLACEMENT.
2. SEE DETAILS A, B, C, OR D FOR ROUNDING.
3. CLEAR ZONE WIDTH SHALL BE DETERMINED FROM STANDARD DRAWING RD01-S-12. SEE THE "ROADSIDE DESIGN GUIDE," AASHTO, 2002, FOR FURTHER INFORMATION ON CLEAR ZONES.
4. SEE STANDARD DRAWINGS RD01-S-11 AND RD01-S-11B FOR FILL AND CUT SLOPE TABLES, ROUNDING ON TOP OF CUT SLOPES AND TOE OF FILL SLOPES, AND SPECIAL ROCK CUT TREATMENT.
5. SEE STANDARD DRAWING RD01-S-11A FOR ROUNDING OF ROADSIDE DITCH SLOPES.
6. THE SLOPES OF THE SHOULDER AND ROADWAY PAVEMENT SHALL NOT EXCEED AN ALGEBRAIC DIFFERENCE OF 0.07 FOOT PER FOOT.
7. MINIMUM MEDIAN WIDTH IS TO BE 48 FEET WHEN USING 6 FEET WIDE INSIDE SHOULDERS. MINIMUM MEDIAN WIDTH IS TO BE 64 FEET WHEN USING 12 FOOT WIDE INSIDE SHOULDERS.
8. GRADES ONE PERCENT STEEPER THAN THE VALUE SHOWN MAY BE USED FOR EXTREME CASES IN URBAN AREAS WHERE DEVELOPMENT PRECLUDES THE USE OF FLATTER GRADES AND FOR ONE-WAY DOWNGRADES EXCEPT IN MOUNTAINOUS TERRAIN.
9. ALTHOUGH THE SELECTED DESIGN SPEED ESTABLISHES THE LIMITING VALUES OF CURVE RADIUS AND MINIMUM SIGHT DISTANCE THAT SHOULD BE USED IN DESIGN, THERE SHOULD BE NO RESTRICTION ON THE USE OF FLATTER HORIZONTAL CURVES OR GREATER SIGHT DISTANCES WHERE SUCH IMPROVEMENTS CAN BE PROVIDED AS A PART OF AN ECONOMICAL DESIGN (SEE PAGE 69).

GENERAL NOTES

A. FOR SPECIFIC CONDITIONS NOT COVERED ON THIS SHEET, REFERENCE SHOULD BE MADE TO "A POLICY ON GEOMETRIC DESIGN OF HIGHWAYS AND STREETS," AASHTO, 2001.
B. PAGE NUMBERS REFERRED TO ON THIS DRAWING ARE FROM "A POLICY ON GEOMETRIC DESIGN OF HIGHWAYS AND STREETS," AASHTO, 2001, UNLESS OTHERWISE NOTED.
C. REFERENCE SHOULD ALSO BE MADE TO THE "ROADSIDE DESIGN GUIDE," AASHTO, 2002.
D. DESIRABLE RIGHT-OF-WAY IS SLOPE LINE PLUS TWENTY FEET.
E. ALL NEW AND REHABILITATED BRIDGES SHALL BE DESIGNED FOR HS-20 LOADING. THE MINIMUM CLEAR WIDTH FOR NEW AND REHABILITATED BRIDGES SHALL BE EQUAL TO THE FULL WIDTH OF THE APPROACH ROADWAY, CURB-TO-CURB OR FULL SHOULDER WIDTH AS APPLICABLE.
F. FOR EXISTING BRIDGES TO REMAIN IN PLACE, THEY SHOULD HAVE ADEQUATE STRENGTH AND A WIDTH AT LEAST EQUAL TO THE WIDTH OF THE TRAVELED WAY PLUS 2 FEET CLEARANCE ON EACH SIDE. BRIDGES SHOULD BE CONSIDERED FOR ULTIMATE WIDENING OR REPLACEMENT IF THEY DO NOT PROVIDE AT LEAST 3 FEET CLEARANCE ON EACH SIDE OR ARE NOT CAPABLE OF HS-20 LOADINGS. AS AN INTERIM MEASURE, ALL BRIDGES THAT ARE LESS THAN FULL WIDTH SHOULD BE CONSIDERED FOR SPECIAL NARROW BRIDGE TREATMENTS SUCH AS SIGNING AND PAVEMENT MARKING.
G. FOR INTERSTATES, SEE THE CURRENT EDITION OF AASHTO'S "A POLICY ON DESIGN STANDARDS-INTERSTATE SYSTEM"

TABLE II. FREEWAY - DESIGN STANDARDS (9)

DESIGN STANDARDS (FOR GIVEN DESIGN SPEED)		DESIGN SPEEDS (MPH) 50	55	60	65	70	
MINIMUM RADIUS (FT.) 0.08 MAX. S.E.		760	965	1205	1485	1820	SEE PAGE 145
MINIMUM STOPPING SIGHT DISTANCE (FT.)		425	495	570	645	730	SEE PAGE 112
MINIMUM "K" VALUE	CREST VERTICAL CURVE	84	114	151	193	247	SEE PAGE 274
	SAG VERTICAL CURVE	96	115	136	157	181	SEE PAGE 280
MAXIMUM GRADES % (8)	LEVEL TERRAIN	4	4	3	3	3	SEE PAGE 510
	ROLLING TERRAIN	5	5	4	4	4	
	MOUNTAINOUS TERRAIN	6	6	6	5	5	
SUPERELEVATION		SEE STANDARD DRAWINGS RD01-SE-2 AND RD01-SE-3					

TABLE I. MINIMUM DESIGN SPEEDS FOR FREEWAYS (SEE PAGE 507)

LOCATION	MINIMUM DESIGN SPEED (MPH)
URBAN	50
RURAL	70
MOUNTAINOUS	50

MINOR REVISION -- FHWA APPROVAL NOT REQUIRED.

STATE OF TENNESSEE
DEPARTMENT OF TRANSPORTATION

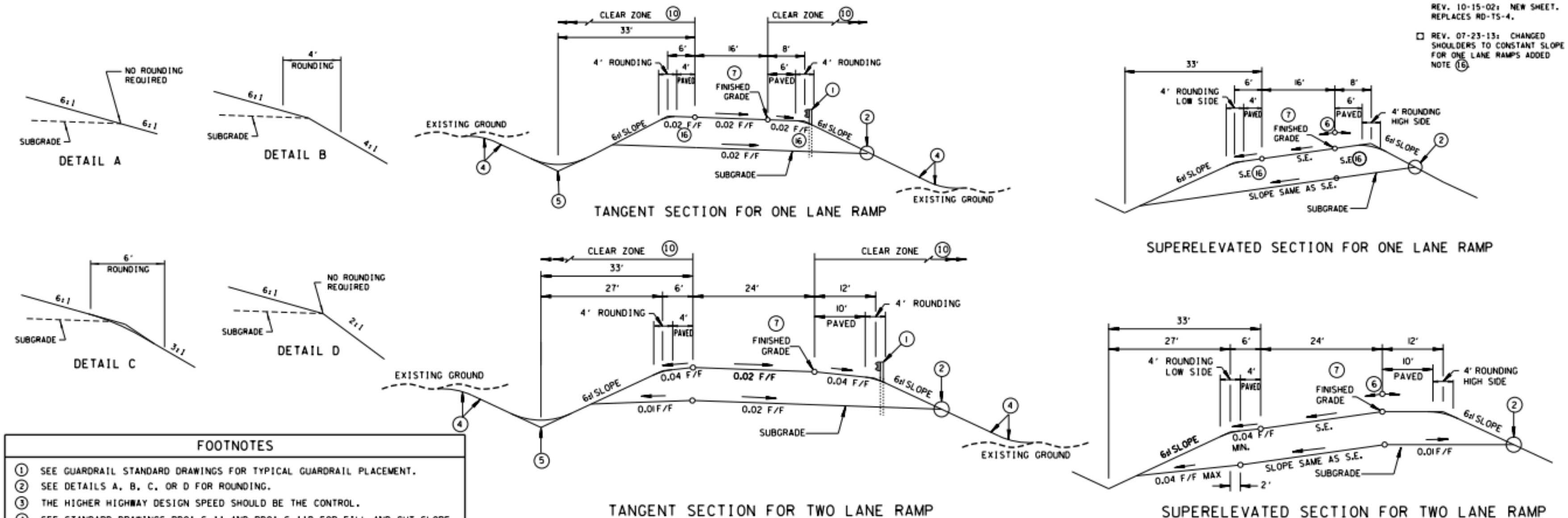
DESIGN STANDARDS FREEWAYS WITH DEPRESSED MEDIANS

10-15-02 RD01-TS-5

APPENDIX D.11 TDOT Design Standards 1 & 2 Lane Ramps

REV. 10-15-02: NEW SHEET. REPLACES RD-TS-4.

□ REV. 07-23-13: CHANGED SHOULDERS TO CONSTANT SLOPE FOR ONE LANE RAMPS ADDED NOTE (16).



FOOTNOTES

(1) SEE GUARDRAIL STANDARD DRAWINGS FOR TYPICAL GUARDRAIL PLACEMENT.

(2) SEE DETAILS A, B, C, OR D FOR ROUNDING.

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

(4) SEE STANDARD DRAWINGS RD01-S-11 AND RD01-S-11B FOR FILL AND CUT SLOPE TABLES, ROUNDING ON TOP OF CUT SLOPES AND TOE OF FILL SLOPES, AND SPECIAL ROCK CUT TREATMENT.

(5) SEE STANDARD DRAWING RD01-S-11A FOR ROUNDING OF ROADSIDE DITCH SLOPES.

(6) THE SLOPES OF THE SHOULDER AND ROADWAY PAVEMENT SHALL NOT EXCEED AN ALGEBRAIC DIFFERENCE OF 0.07 FOOT PER FOOT.

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

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

(9) 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.

(10) CLEAR ZONE WIDTH SHALL BE DETERMINED FROM STANDARD DRAWING RD01-S-12. SEE THE "ROADSIDE DESIGN GUIDE," AASHTO, 2002 FOR FURTHER INFORMATION.

(11) 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.

(12) 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 192).

(13) MAY BE 2% OR 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.

(14) FOR DESIGN SPEEDS GREATER THAN 45 MILES PER HOUR, USE VALUES FOR OPEN HIGHWAY CONDITIONS.

(15) SUPERELEVATION ASSUMED OF 8%.

(16) 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 SUPER ELEVATION 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 SUPER ELEVATION RATE.

GENERAL NOTES

(A) FOR SPECIFIC CONDITIONS NOT COVERED ON THIS SHEET, REFERENCE SHOULD BE MADE TO "A POLICY ON GEOMETRIC DESIGN OF HIGHWAYS AND STREETS," AASHTO, 2001.

(B) PAGE NUMBERS REFERRED TO ON THIS DRAWING ARE FROM "A POLICY ON GEOMETRIC DESIGN OF HIGHWAYS AND STREETS," AASHTO, 2001, UNLESS OTHERWISE NOTED.

(C) REFERENCE SHOULD ALSO BE MADE TO THE "ROADSIDE DESIGN GUIDE," AASHTO, 2002.

(D) SECTIONS DRAWN LOOKING IN DIRECTION OF TRAVEL.

NOT TO SCALE

TABLE I. GUIDE VALUES FOR RAMP DESIGN SPEED AS RELATED TO HIGHWAY DESIGN SPEED (SEE PAGE 830)

(3) (8) HIGHWAY DESIGN SPEEDS, MPH	30	35	40	45	50	55	60	65	70	
(11) RAMP DESIGN SPEED, MPH										
UPPER RANGE (85%)	25	30	35	40	45	48	50	55	60	SEE PAGE 830
MIDDLE RANGE (70%)	20	25	30	35	35	40	45	45	50	
LOWER RANGE (50%)	15	20	20	25	25	30	30	30	35	
CORRESPONDING MINIMUM RADIUS (FT)	230	310	450 (15)	565 (15)	(14)	(14)	(14)	(14)	(14)	SEE PAGE 201

TABLE II. STOPPING SIGHT DISTANCE FOR RAMPS (SEE PAGE 112) (9)

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 192) (12)

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

TABLE IV. RECOMMENDED DESIGN GRADES ON RAMPS (SEE PAGES 832-833)

DESIGN SPEED (MPH)	UPGRADE	DOWNGRADE (13)
15-25	6-8%	6-8%
25-30	5-7%	5-7%
40	4-6%	4-6%
45 OR GREATER	3-5%	3-5%

□ MINOR REVISION -- FHWA APPROVAL NOT REQUIRED.

STATE OF TENNESSEE
DEPARTMENT OF TRANSPORTATION

DESIGN STANDARDS
1 & 2 LANE
RAMPS

10-15-02 RD01-TS-4

APPENDIX D.12 TDOT Design Standards for Local Roads and Street

REV. 10-15-02: NEW SHEET. REPLACES RD-TS-1.

REV. 2-5-16: REMOVE DETAIL A AND DETAIL B.

DESIGN LOADING: ALL NEW AND REHABILITATED BRIDGES SHALL BE DESIGNED FOR HS-20 LOADING.

FOR NEW ROUTE CONSTRUCTION OR ROUTE RECONSTRUCTION PROJECTS:

THE MINIMUM CLEAR WIDTH FOR NEW BRIDGES SHALL BE EQUAL TO THE FULL WIDTH OF THE APPROACH ROADWAY (CURB-TO-CURB OR FULL SHOULDER WIDTH AS APPLICABLE).

TABLE I. MINIMUM CLEAR ROADWAY WIDTHS AND DESIGN LOADINGS FOR NEW AND RECONSTRUCTED BRIDGES (SEE PAGE 390)

DESIGN ADT (VEH/DAY)	DESIGN LOADING	MINIMUM CLEAR ROADWAY WIDTH OF BRIDGE (1)
UNDER 400	HS-20	TRAVELED WAY + 4 FT. (2 FT. EACH SIDE)
400 TO 2,000	HS-20	TRAVELED WAY + 6 FT. (3 FT. EACH SIDE)
OVER 2,000	HS-20	APPROACH ROADWAY WIDTH

TABLE II. MINIMUM STRUCTURAL CAPACITIES AND MINIMUM ROADWAY WIDTHS FOR EXISTING BRIDGES TO REMAIN IN PLACE (SEE PAGE 390) (3)

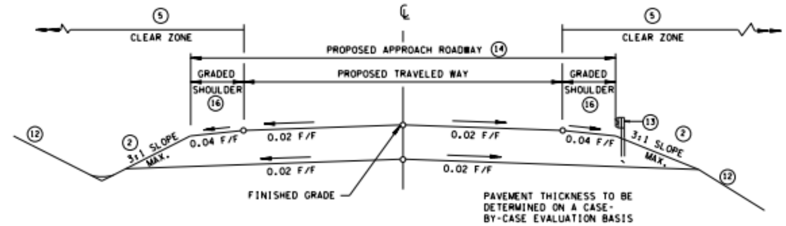
DESIGN ADT (VEH/DAY)	DESIGN LOADING (STRUCTURAL CAPACITY)	MINIMUM CLEAR ROADWAY WIDTH (FT) (4)
0 TO 50	H-15	20
50 TO 250	H-15	20
250 TO 1,500	H-15	22
1,500 TO 2,000	H-15	24
OVER 2,000	H-15	28

TABLE III. MINIMUM DESIGN SPEEDS FOR LOCAL RURAL ROADS

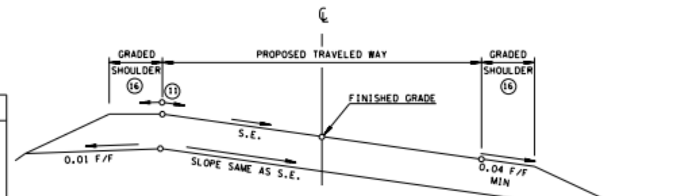
TYPE OF TERRAIN	DESIGN SPEED (MPH) FOR SPECIFIED DESIGN ADT (VEH/DAY)					
	UNDER 50	50-250	250-400	400 TO 1,500	1,500 TO 2,000	2,000 AND OVER
LEVEL	30	30	40	50	50	50
ROLLING	20 (6)	30	30	40	40	40
MOUNTAINOUS	20 (6)	20 (6)	20 (6)	30	30	30

TABLE IV. LOCAL ROADS AND STREETS - DESIGN STANDARDS (8)

DESIGN STANDARDS (FOR GIVEN DESIGN SPEED)		DESIGN SPEEDS (MPH)										MINIMUM WIDTH OF SHOULDERS FOR ALL SPEEDS (FEET) (SEE PAGE 388)
		15	20	25	30	35	40	45	50	55	60	
MINIMUM WIDTH OF TRAVELED WAY IN RURAL AREAS (FEET) (SEE PAGE 388)	DESIGN ADT UNDER 400	18	18	18	18	18	18	20	20	22	22	4 (7)
	DESIGN ADT 400 - 1,500	20 (7)	20 (7)	20 (7)	20 (7)	20 (7)	20 (7)	22	22	22	22	5 (7) (9)
	DESIGN ADT 1,500 - 2,000	20	22	22	22	22	22	22	22	24 (10)	24 (10)	6
	DESIGN ADT OVER 2,000	22	24 (10)	24 (10)	24 (10)	24 (10)	24 (10)	24 (10)	24 (10)	24 (10)	24 (10)	8
MINIMUM RADIUS (FEET) 0.04 MAX. S.E.		70	125	205	300	420	565	730	930	1190	1505	SEE PAGE 145
MINIMUM RADIUS (FEET) 0.06 MAX. S.E.		65	115	185	275	380	510	660	835	1065	1340	
MINIMUM RADIUS (FEET) 0.08 MAX. S.E.		60	105	170	250	350	465	600	760	965	1205	
MAXIMUM RURAL GRADES %	LEVEL TERRAIN	9	8	7	7	7	7	7	6	6	5	SEE PAGE 386
	ROLLING TERRAIN	12	11	11	10	10	10	9	8	7	6	
	MOUNTAINOUS TERRAIN	17	16	15	14	14	13	12	10	10		
MINIMUM STOPPING SIGHT DISTANCE (FEET)		80	115	155	200	250	305	360	425	495	570	SEE PAGE 385
MINIMUM "K" VALUE	CREST VERTICAL CURVE	3	7	12	19	29	44	61	84	114	151	
	SAG VERTICAL CURVE	10	17	26	37	49	64	79	96	115	136	
MINIMUM PASSING SIGHT DISTANCE (FEET)			710	900	1090	1280	1470	1625	1835	1985	2135	SEE PAGE 386
MINIMUM "K" VALUE FOR CREST VERTICAL CURVE			180	289	424	585	772	943	1203	1407	1628	
SUPERELEVATION		SEE STANDARD DRAWINGS RD01-SE-2 AND RD01-SE-3										



TANGENT SECTION



SUPERELEVATED SECTION

TYPICAL CROSS-SECTIONS

GENERAL NOTES

(A) FOR SPECIFIC CONDITIONS NOT COVERED ON THIS SHEET, REFERENCE SHOULD BE MADE TO "A POLICY ON GEOMETRIC DESIGN OF HIGHWAYS AND STREETS," AASHTO 2001.

(B) FOR URBAN DESIGN GUIDANCE AND CRITERIA, REFERENCE IS MADE TO "A POLICY ON GEOMETRIC DESIGN OF HIGHWAYS AND STREETS," AASHTO, 2001, PAGES 393 TO 408.

(C) PAGE NUMBERS REFERRED TO ON THIS DRAWING ARE FROM "A POLICY ON GEOMETRIC DESIGN OF HIGHWAYS AND STREETS," AASHTO, 2001.

(D) REFERENCE IS ALSO MADE TO THE "ROADSIDE DESIGN GUIDE," AASHTO, 2011.

(E) FOR CORNER SIGHT DISTANCE AT RURAL INTERSECTIONS SEE PAGES 654 THROUGH 681. ALSO STANDARD DRAWING SD-SERIES.

(F) IF NO ABOVE GROUND UTILITIES ARE INVOLVED, MINIMUM RIGHT-OF-WAY SHALL BE TRAVELED WAY PLUS CLEAR ZONE (MINIMUM OF 10 FEET EACH SIDE).

(G) IF ABOVE GROUND UTILITIES ARE INVOLVED, MINIMUM RIGHT-OF-WAY SHALL BE SUFFICIENT TO ACCOMMODATE THE UTILITIES OUTSIDE THE CLEAR ZONE.

(H) DESIRABLE RIGHT-OF-WAY IS SLOPE LINES PLUS TEN FEET.

FOOTNOTES

(1) WHERE THE APPROACH ROADWAY WIDTH (TRAVELED WAY PLUS SHOULDERS) IS SURFACED, THAT SURFACE WIDTH SHOULD BE CARRIED ACROSS THE STRUCTURE.

(2) 4:1 SLOPE FOR 40 MILES PER HOUR OR GREATER WITH A DESIGN ADT OF 1,000 OR GREATER OR ANY LOCATION GUARDRAIL IS USED.

(3) THESE STRUCTURES SHOULD BE ANALYZED INDIVIDUALLY, TAKING INTO CONSIDERATION THE CLEAR WIDTH PROVIDED, TRAFFIC VOLUMES, REMAINING LIFE OF THE STRUCTURE, PEDESTRIAN VOLUMES, SNOW STORAGE, DESIGN SPEED, ACCIDENT RECORD, AND OTHER PERTINENT FACTORS.

(4) CLEAR WIDTH BETWEEN CURBS OR RAILS, WHICHEVER IS THE LESSER. MINIMUM CLEAR WIDTHS THAT ARE TWO FEET NARROWER MAY BE USED ON ROADS WITH FEW TRUCKS. IN NO CASE SHALL THE MINIMUM CLEAR WIDTH BE LESS THAN THE APPROACH TRAVELED WAY WIDTH.

(5) THE CLEAR ZONE WIDTH SHALL BE DETERMINED FROM STANDARD DRAWING RD01-S-12. SEE THE "ROADSIDE DESIGN GUIDE," AASHTO, 2002, FOR FURTHER INFORMATION ON CLEAR ZONES.

(6) EFFORTS SHOULD BE MADE TO SELECT A DESIGN SPEED GREATER THAN 20 MILES PER HOUR. SEE PAGE 384 FOR FURTHER INFORMATION.

(7) FOR ROADS IN MOUNTAINOUS TERRAIN WITH A DESIGN YEAR ADT OF 0 TO 600 VEHICLES PER DAY AND THE DESIGN SPEED IS GREATER THAN OR EQUAL TO 15 MILES PER HOUR AND LESS THAN OR EQUAL TO 40 MPH, USE 18 FEET TRAVELED WAY WIDTH AND 2 FEET SHOULDER WIDTH.

(8) ALTHOUGH THE SELECTED DESIGN SPEED ESTABLISHES THE LIMITING VALUES OF CURVE RADIUS AND MINIMUM SIGHT DISTANCE THAT SHOULD BE USED IN DESIGN, THERE SHOULD BE NO RESTRICTION ON THE USE OF FLATTER HORIZONTAL CURVES OR GREATER SIGHT DISTANCES WHERE SUCH IMPROVEMENTS CAN BE PROVIDED AS A PART OF AN ECONOMICAL DESIGN (SEE PAGE 69).

(9) MAY BE USED TO ACHIEVE A MINIMUM ROADWAY WIDTH OF 30 FEET FOR DESIGN SPEEDS GREATER THAN 40 MILES PER HOUR.

(10) WHERE THE WIDTH OF THE TRAVELED WAY IS SHOWN AS 24 FEET, THE WIDTH MAY REMAIN AT 22 FEET ON RECONSTRUCTED HIGHWAYS WHERE ALIGNMENT AND SAFETY RECORDS ARE SATISFACTORY.

(11) THE SLOPES OF THE SHOULDER AND ROADWAY PAVEMENT SHALL NOT EXCEED AN ALGEBRAIC DIFFERENCE OF 0.07 FOOT PER FOOT.

(12) SEE STANDARD DRAWINGS RD01-S-11 (CASE II) AND RD01-S-11B FOR DESIRABLE SLOPES & NOTE REGARDING GEOLOGICAL RECOMMENDATIONS.

(13) SEE S-PL-6 FOR GUARDRAIL PLACEMENT.

(14) PROPOSED APPROACH ROADWAY WIDTH WILL NOT BE LESS THAN EXISTING WIDTH.

(15) WHEN GUARDRAIL IS PLACED BEHIND CURB AND GUTTER, THE SLOPING CURB HEIGHT MUST BE 4 INCHES OR LESS.

(16) SHOULDER SURFACE TREATMENT TO BE SPECIFIED BY THE DESIGN DIVISION'S PAVEMENT DESIGN SECTION. DESIGNERS SHOULD REFER TO THE DESIGN GUIDELINES FOR PAVEMENT REQUEST PROCEDURES. WHEN SHOULDERS ARE PAVED AND GRADED SHOULDER WIDTH IS 6 FEET OR GREATER, THE SHOULDER SHOULD BE PAVED THE GRADED SHOULDER WIDTH MINUS TWO FEET. WHEN SHOULDERS ARE PAVED AND THE GRADED SHOULDER WIDTH IS LESS THAN 6 FEET, THE SHOULDER SHOULD BE PAVED THE WIDTH OF THE GRADED SHOULDER.

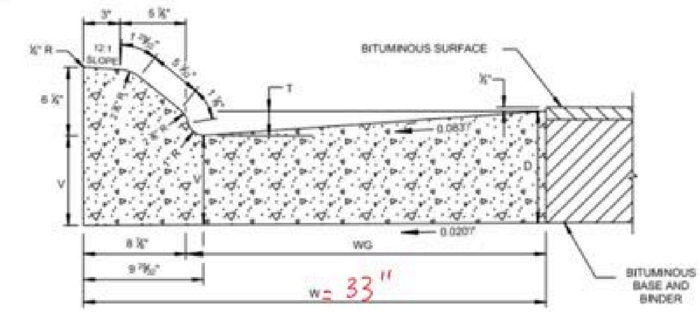
☐ MINOR REVISION -- FHWA APPROVAL NOT REQUIRED.

STATE OF TENNESSEE
DEPARTMENT OF TRANSPORTATION

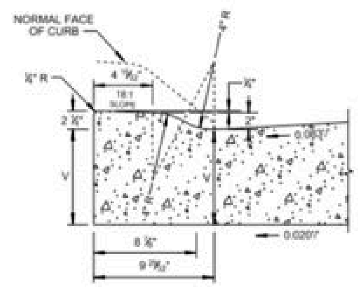
DESIGN STANDARDS FOR LOCAL ROADS AND STREETS

10-15-02 | RD01-TS-1

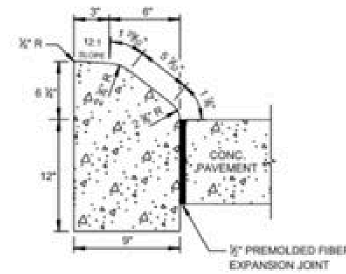
APPENDIX D.13 TDOT Curb and Gutter



6" SLOPING CONCRETE COMBINED CURB AND GUTTER



LOWERED CONCRETE CURB



6" SLOPING DETACHED CONCRETE CURB

6" SLOPING CONCRETE COMBINED CURB AND GUTTER TABLE					
TYPE	TOTAL WIDTH (W) IN INCHES	WIDTH OF GUTTER (WG) IN INCHES	VERTICAL DROP (T) IN INCHES	VERTICAL DEPTH (D) OF GUTTER	VERTICAL DEPTH (V) OF GUTTER AT FLOW LINE
6-33	33	24⅜	2	AS NOTED ON TYPICAL X-SECTIONS	D + 1.52"
6-39	39	30⅜	2⅜		D + 1.90"
6-45	45	36⅜	3		D + 2.28"

VERTICAL DEPTH (V) MUST ALWAYS EXCEED SIX (6) INCHES.

LOWERED CONCRETE CURB NOTES	
L1	TO BE BUILT AS COMBINED CURB AND GUTTER, DETACHED CURB OR INTEGRAL CURB AS NOTED ON THE PLANS OR AS DIRECTED BY THE ENGINEER.
L2	FOR DETACHED CURB, OMIT RADIUS AT FLOW LINE.

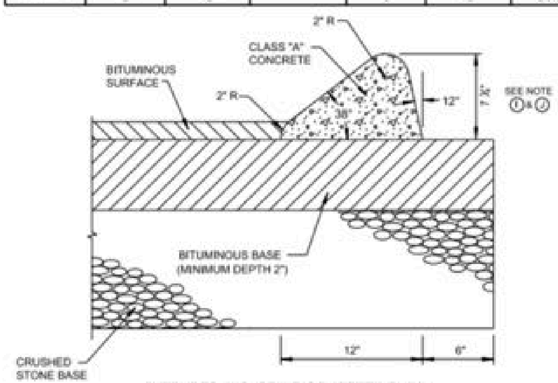
QUANTITIES FOR DETACHED CURB	
HEIGHT OF CURB	CUBIC YARD PER LINEAR FOOT
LOWERED CURB	0.03099
6" SLOPING	0.03841

QUANTITIES FOR COMBINED CURB AND GUTTER

HEIGHT OF CURB	DEPTH (D) OF GUTTER IN INCHES	TOTAL WIDTH (W) IN INCHES	CUBIC YARDS PER LINEAR FOOT	DEPTH (D) OF GUTTER IN INCHES	TOTAL WIDTH (W) IN INCHES	CUBIC YARDS PER LINEAR FOOT	DEPTH (D) OF GUTTER IN INCHES	TOTAL WIDTH (W) IN INCHES	CUBIC YARDS PER LINEAR FOOT	DEPTH (D) OF GUTTER IN INCHES	TOTAL WIDTH (W) IN INCHES	CUBIC YARDS PER LINEAR FOOT	DEPTH (D) OF GUTTER IN INCHES	TOTAL WIDTH (W) IN INCHES	CUBIC YARDS PER LINEAR FOOT
LOWERED CURB	8	33	0.06362	9	33	0.07211	10	33	0.08061	11	33	0.08909	12	33	0.09757
	8	39	0.07748	9	39	0.08751	10	39	0.09754	11	39	0.10758	12	39	0.11763
	8	45		9	45	0.10385	10	45	0.11543	11	45	0.12701	12	45	0.13857
6" SLOPING	8	33	0.07060	9	33	0.07909	10	33	0.08757	11	33	0.09606	12	33	0.10455
	8	39	0.08446	9	39	0.09449	10	39	0.10452	11	39	0.11455	12	39	0.12458
	8	45		9	45	0.11083	10	45	0.12240	11	45	0.13398	12	45	0.14555

LEGEND

- D = VERTICAL DEPTH OF GUTTER
- T = VERTICAL DROP IN GUTTER FROM FRONT EDGE TO FACE OF CURB
- V = VERTICAL DEPTH OF GUTTER AT FLOW LINE
- W = TOTAL WIDTH OF COMBINED CURB AND GUTTER
- WG = WIDTH OF GUTTER



EXTRUDED 6" SLOPING CONCRETE CURB

GENERAL NOTES

Ⓐ FOR SPECIFICATIONS SEE "STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION" OF THE TENNESSEE DEPARTMENT OF TRANSPORTATION, SECTION 702 - CEMENT CONCRETE CURB, GUTTER AND COMBINED CURB AND GUTTER.

Ⓑ THE FRONT FACE OF THE CONCRETE CURBS FOR ALL DEGREES OF CURVATURE SHALL CONFORM TO THE CONTOUR OF THE CURVE AND NO CHORD SECTIONS WILL BE PERMITTED.

Ⓒ CONCRETE EXPANSION JOINT MATERIAL IS TO BE 3/4 " PRE-MOLDED FIBER IN ACCORDANCE WITH SECTION 905 - JOINT MATERIALS OF THE STANDARD SPECIFICATIONS.

Ⓓ EXPANSION JOINTS ARE TO BE PLACED AS FOLLOWS:

1. AT TANGENT POINTS OF CIRCULAR CUBS.
2. BETWEEN CURBS AND ABUTTING RIGID OBJECTS.
3. AT OTHER PLACES WHERE STRESSES MAY DEVELOP.
4. TO LINE UP WITH PAVEMENT JOINTS WHERE THE ADJACENT PAVEMENT IS CONCRETE.
5. THE MAXIMUM SPACING IS TO BE 100 FEET.

Ⓔ CONTRACTION JOINTS ARE TO BE SPACED AT 10 FEET. THE SPACING OF 10 FEET MAY BE REDUCED FOR CLOSURES, BUT NOT LESS THAN 6 FEET.

Ⓕ EDGES OF JOINTS SHALL BE FINISHED ON ONE-QUARTER INCH RADII.

Ⓖ ALL COST OF JOINTS SHALL BE INCLUDED IN THE UNIT PRICE BID FOR CONCRETE CURBS AND CONCRETE CURBS AND GUTTERS.

Ⓗ THE UNIT PRICE BID FOR CONCRETE CURB, CONCRETE CURB AND GUTTER AND CONCRETE PAVEMENT WILL INCLUDE ANY CIRCULAR SECTION REQUIRED TO BE BUILT CONFORMING TO SECTIONS SHOWN ON THIS SHEET.

Ⓘ WHERE CONCRETE MEDIAN PAVEMENT IS POURED BEHIND EXTRUDED 6" CONCRETE CURB, IT MAY BE POURED MONOLITHICALLY WITH THE CURB.

Ⓙ THE EXTRUDED 6" SLOPING CONCRETE CURB IS TO BE USED ONLY IN SPECIAL CONDITIONS SUCH AS LOW SPEED LOW VOLUME LOCAL STREETS, AS A TEMPORARY MEASURE TO CONTROL TRAFFIC FLOW OR WHEN TIEING TO SIMILAR CURBS ON SUBDIVISION STREETS OR IN PARKING LOTS.

Ⓚ PAYMENT WILL BE AS FOLLOWS:

ITEM NO. 702-01 CONCRETE CURB, PER CUBIC YARD.

ITEM NO. 702-01.01 EXTRUDED SLOPING CURB, PER LINEAR FOOT.

ITEM NO. 702-03 CONCRETE COMBINED CURB AND GUTTER, PER CUBIC YARD.

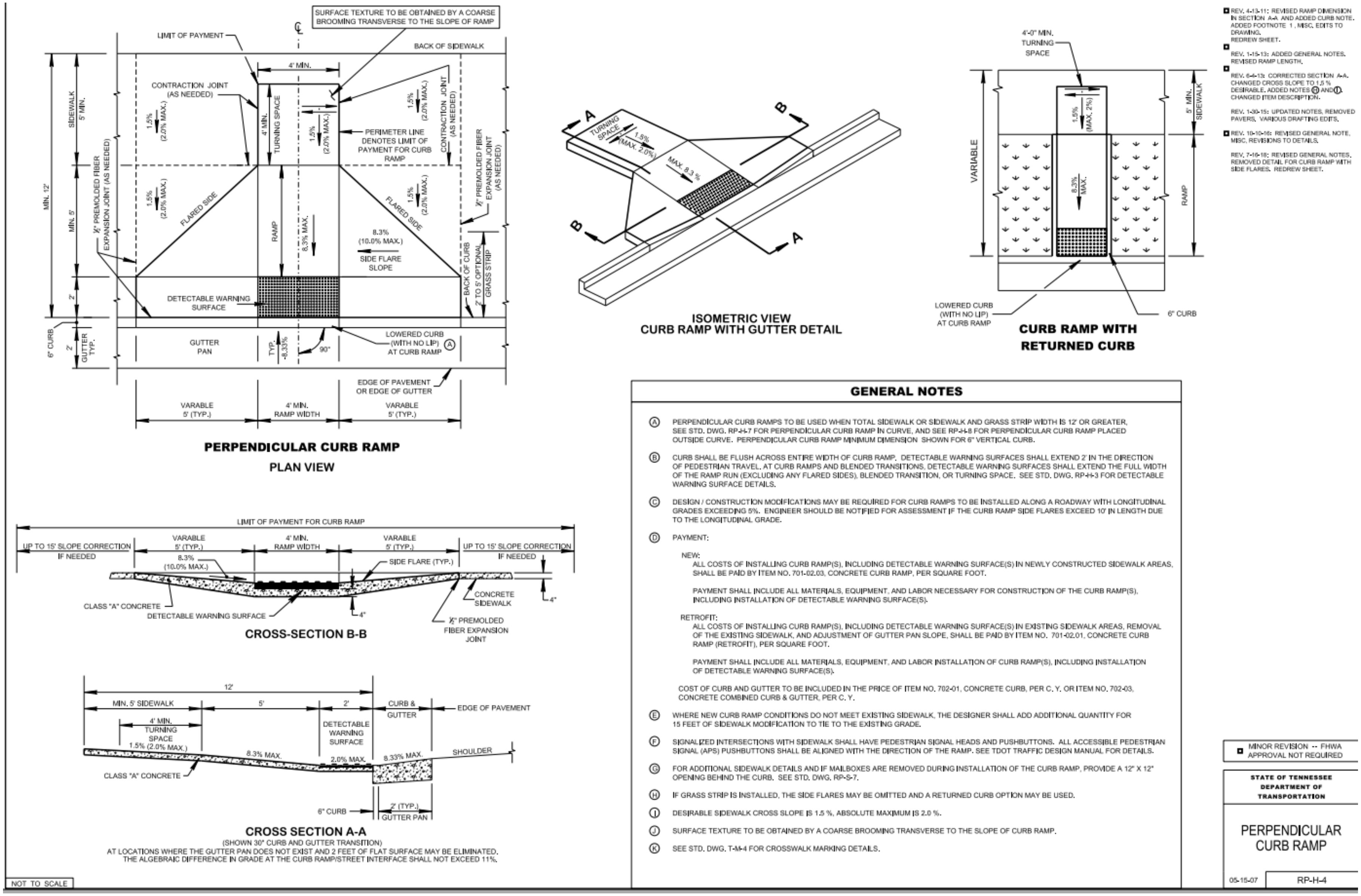
NOT TO SCALE

STATE OF TENNESSEE DEPARTMENT OF TRANSPORTATION

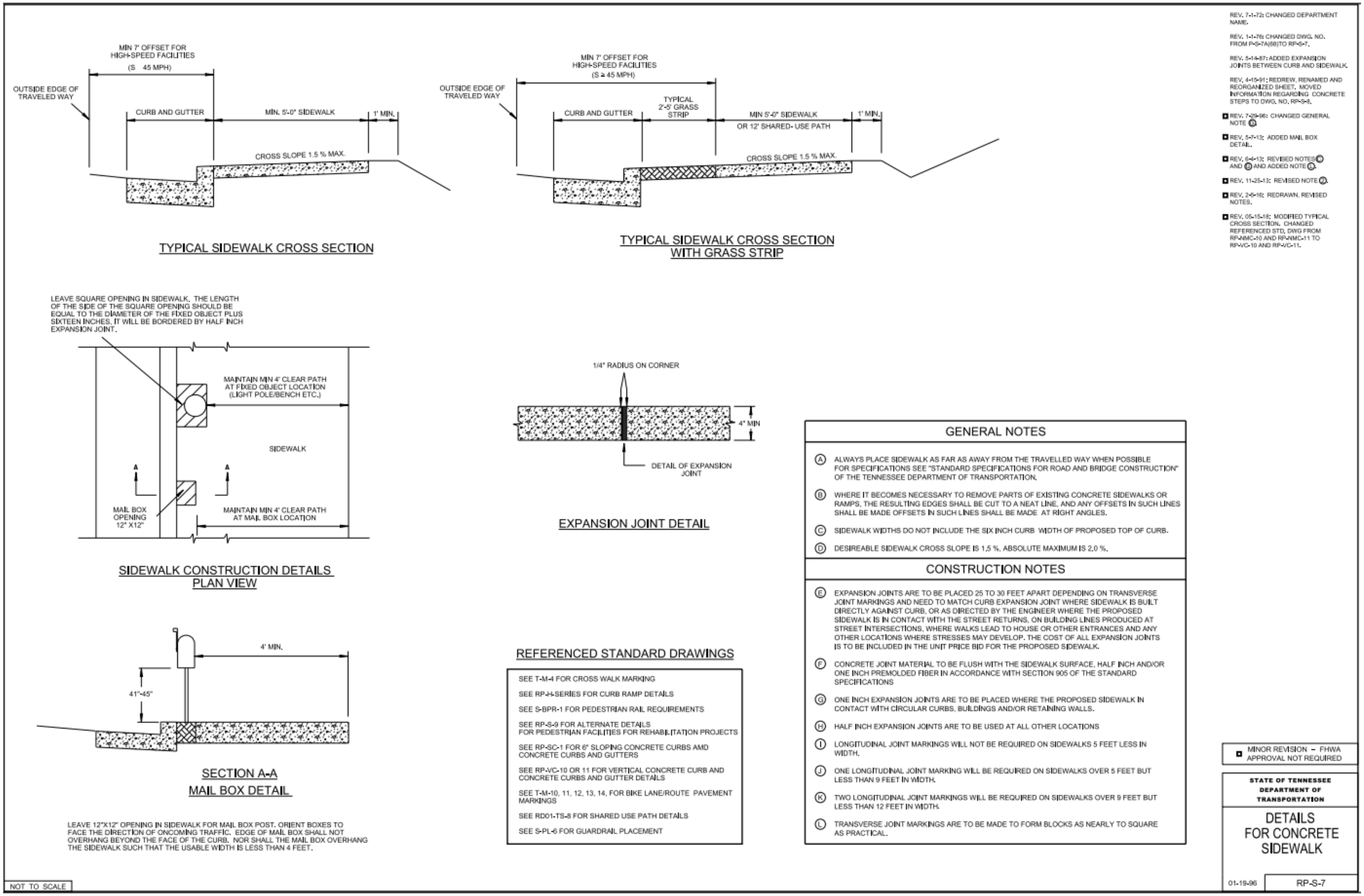
6" SLOPING CONCRETE CURBS AND CONCRETE CURBS AND GUTTERS

05-15-18 | RP-SC-1

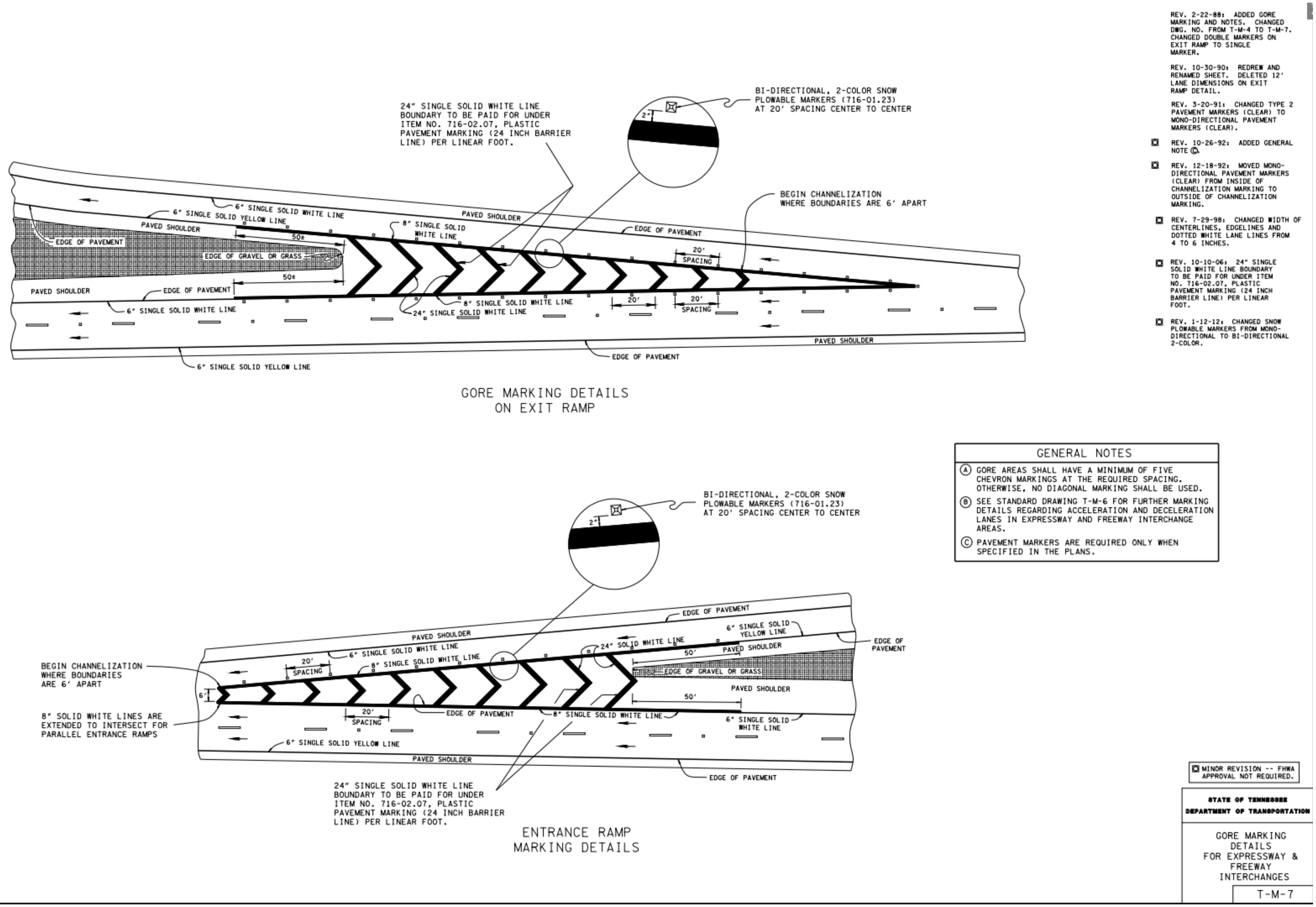
APPENDIX D.14 TDOT Perpendicular Curb Ramp



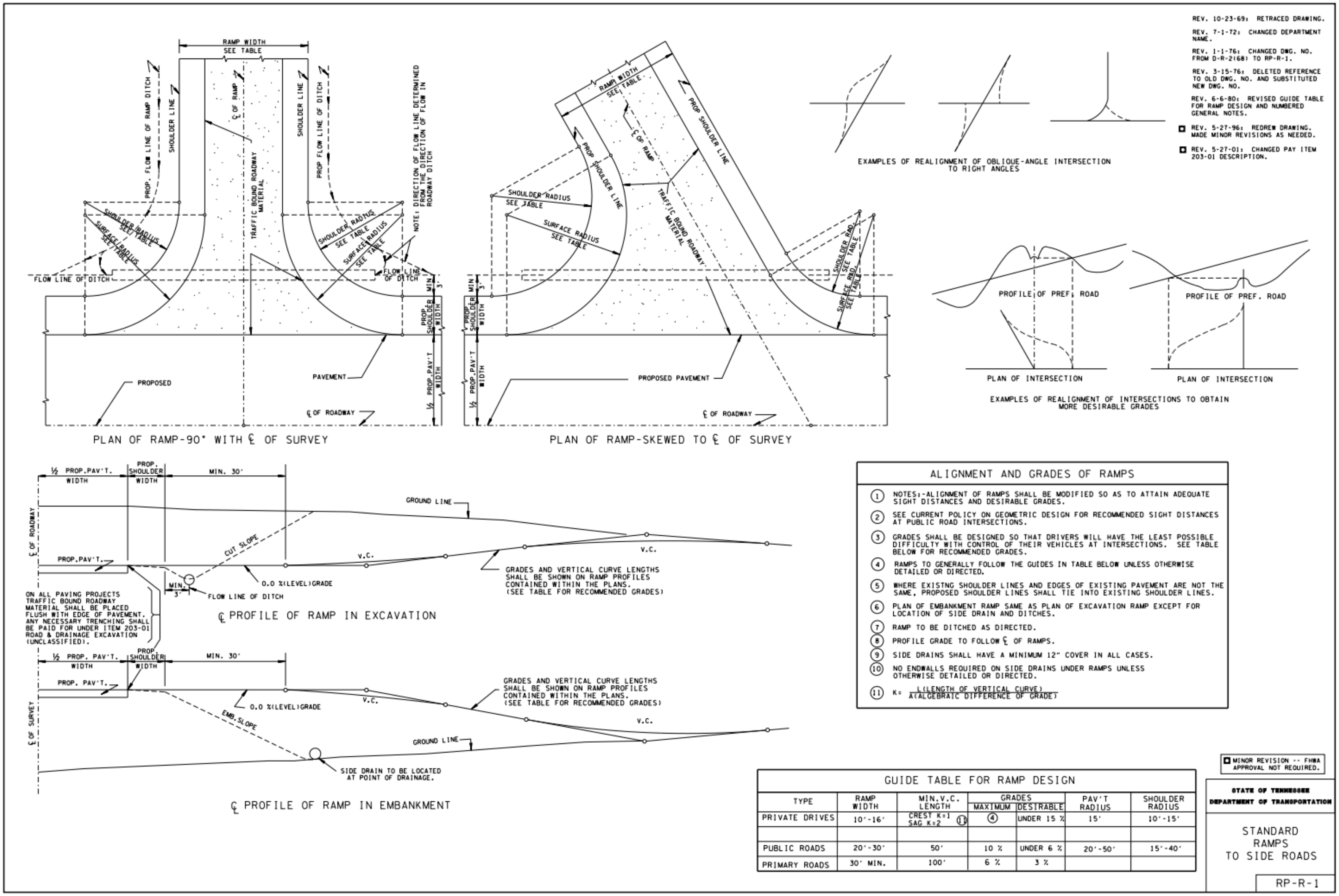
APPENDIX D.15 TDOT Concrete Sidewalk



APPENDIX D.16 TDOT Gore Marking and Details



APPENDIX D.17 TDOT Intersection Curb Return



REV. 10-23-69: RETRACED DRAWING.
REV. 7-1-72: CHANGED DEPARTMENT NAME.
REV. 1-1-76: CHANGED DWG. NO. FROM D-R-2(68) TO RP-R-1.
REV. 3-15-76: DELETED REFERENCE TO OLD DWG. NO. AND SUBSTITUTED NEW DWG. NO.
REV. 6-6-80: REVISED GUIDE TABLE FOR RAMP DESIGN AND NUMBERED GENERAL NOTES.
REV. 5-27-96: REDREW DRAWING. MADE MINOR REVISIONS AS NEEDED.
REV. 5-27-01: CHANGED PAY ITEM 203-01 DESCRIPTION.

ALIGNMENT AND GRADES OF RAMPS

1. NOTES:-ALIGNMENT OF RAMPS SHALL BE MODIFIED SO AS TO ATTAIN ADEQUATE SIGHT DISTANCES AND DESIRABLE GRADES.
2. SEE CURRENT POLICY ON GEOMETRIC DESIGN FOR RECOMMENDED SIGHT DISTANCES AT PUBLIC ROAD INTERSECTIONS.
3. GRADES SHALL BE DESIGNED SO THAT DRIVERS WILL HAVE THE LEAST POSSIBLE DIFFICULTY WITH CONTROL OF THEIR VEHICLES AT INTERSECTIONS. SEE TABLE BELOW FOR RECOMMENDED GRADES.
4. RAMPS TO GENERALLY FOLLOW THE GUIDES IN TABLE BELOW UNLESS OTHERWISE DETAILED OR DIRECTED.
5. WHERE EXISTNG SHOULDER LINES AND EDGES OF EXISTING PAVEMENT ARE NOT THE SAME, PROPOSED SHOULDER LINES SHALL TIE INTO EXISTING SHOULDER LINES.
6. PLAN OF EMBANKMENT RAMP SAME AS PLAN OF EXCAVATION RAMP EXCEPT FOR LOCATION OF SIDE DRAIN AND DITCHES.
7. RAMP TO BE DITCHED AS DIRECTED.
8. PROFILE GRADE TO FOLLOW ℄ OF RAMPS.
9. SIDE DRAINS SHALL HAVE A MINIMUM 12" COVER IN ALL CASES.
10. NO ENDWALLS REQUIRED ON SIDE DRAINS UNDER RAMPS UNLESS OTHERWISE DETAILED OR DIRECTED.
11. $K = \frac{L\text{(LENGTH OF VERTICAL CURVE)}}{A\text{(ALGEBRAIC DIFFERENCE OF GRADE)}}$

GUIDE TABLE FOR RAMP DESIGN

TYPE	RAMP WIDTH	MIN.V.C. LENGTH	GRADES		PAV'T RADIUS	SHOULDER RADIUS
			MAXIMUM	DESIRABLE		
PRIVATE DRIVES	10'-16'	CREST K=1 SAG K=2 (11)	(4)	UNDER 15 %	15'	10'-15'
PUBLIC ROADS	20'-30'	50'	10 %	UNDER 6 %	20'-50'	15'-40'
PRIMARY ROADS	30' MIN.	100'	6 %	3 %		

APPENDIX E. WATER RESOURCES

APPENDIX E.1 Runoff Tables, Calcs, and Equations

Pre-Development

	Area	Soil Classification	CN Number
Land:	348480	D	89
		Soil Retention Cap (in)=	1.24
		Runoff (in)=	20.81

Post-Development

	Area (ft^2)	Soil Classification	CN Number
Building:	2881	D	98
Pavement:	232002.9	D	98
Grass:	113596.1	D	80
		CN composite=	92.13
		Soil Retention Cap (in)=	0.85
		Runoff (in)=	21.24

Figure 54. Soil, CN, Runoff

Composite CN Eqn
$\mathrm{CN} = \frac{\sum_i^N A_i CN_i}{\sum_i^N A_i}$
where,
Ai= area
Cni= curve number

Soil Retention Cap. Eqn
$S = \frac{100}{CN} - 10$
where,
S= Soil Retention Capacity
CN= Composite Curve Number

Runoff Depth Eqn
$R = \frac{(P-0.2S)^2}{P+0.8S}$
where,
R= Runoff Depth
P= Precipitation

Table 9. Hydrologic Design Criteria

	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

Table 10. Hydrologic Soil Group

Hydrologic Soil Group (HSG)	Soil Textures
A	Sand, loamy sand, or sandy loam
B	Silt loam or loam
C	Sandy clay loam
D	Clay loam, silty clay loam, sandy clay, silty clay, or clay

Table 11. Soil CN Number for Agricultural Land

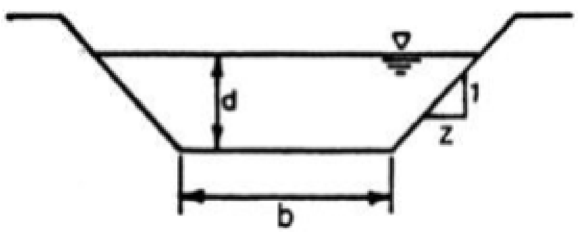
Cover Description[a]		Hydrologic Condition[c]	CN for Soil Group			
Cover type	Treatment[b]		A	B	C	D
Fallow	Bare soil	-----	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C +CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

Table 12. CN Number for Urban Areas

Cover Type and Hydrologic Condition Fully Developed Urban Areas (vegetation established):[a]		CN for Soil Group			
		A	B	C	D
Open space (lawn, parks, golf courses, cemeteries, etc.)[c]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of -way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts:[b]					
Commercial and business	85% average impervious area	89	92	94	95
Industrial	72% average impervious area	81	88	91	93
Residential districts by average lot size:[b]					
1/8 acre or less (town houses)	65% average impervious area	77	85	90	92
1/4 acre	38% average impervious area	61	75	83	87
1/3 acre	30% average impervious area	57	72	81	86
1/2 acre	25% average impervious area	54	70	80	85
1 acre	20% average impervious area	51	68	79	84
2 acres	12% average impervious area	46	65	77	82
Developing urban areas:					
Newly graded areas (pervious areas only, no vegetation)[d]		77	86	91	94
For idle lands, CN's are determined using cover types similar to those in Table 4A-3					

APPENDIX E.2 Ditch Design

Trapezoidal Ditch Design	
Depth (ft)	3
Area (sqft)	28.5
Top Width (ft)	17
Hydraulic Depth (ft)	1.7
Wetted Perimeter (ft)	18.2
Hydraulic Radius (ft)	1.6



b=	2	ft
d=	3	ft
z=	2.5	ft
T=	17	ft

Area (A)
$A = bd + zd^2$
where,
b= bottom width
d= depth
z= slope

Wetted Perimeter (P)
$\mathrm{P}= b + 2d\sqrt{z^2 + 1}$
where,
b= bottom width
d= depth
z= slope

Hydraulic Radius (R)
$R = \dfrac{bd + zd^2}{b + 2d\sqrt{z^2 + 1}}$
where,
b= bottom width
d= depth
z= slope

Top WIdth (T)
$A = b + 2zd$
where,
b= bottom width
d= depth
z= slope

Figure 55. Equations used in Water Resources Design

APPENDIX E.3 Retention Design

Pipes From Parking Areas								
Area	Pipe #	k (constant)	C	I (in/hr)	Q (ft^3/sec)	D (ft)	Calculated diameter (in)	Pipe diameter (in)
1017.36	1A	1.008	0.85	0.30875	269.129	6.9013	0.575108	18
1017.36	2A	1.008	0.85	0.30875	269.129	6.9013	0.575108	18
1017.36	3A	1.008	0.85	0.30875	269.129	6.9013	0.575108	18
1017.36	1B	1.008	0.85	0.30875	269.129	6.9013	0.575108	18
1017.36	2B	1.008	0.85	0.30875	269.129	6.9013	0.575108	18
1017.36	3B	1.008	0.85	0.30875	269.129	6.9013	0.575108	18
1017.36	1C	1.008	0.85	0.30875	269.129	6.9013	0.575108	18
1017.36	2C	1.008	0.85	0.30875	269.129	6.9013	0.575108	18
1017.36	3C	1.008	0.85	0.30875	269.129	6.9013	0.575108	18

*Parking A is to the left of building

*Parking B is to the right of building

*Parking C is behind the building

Figure 56. Piping Calculations

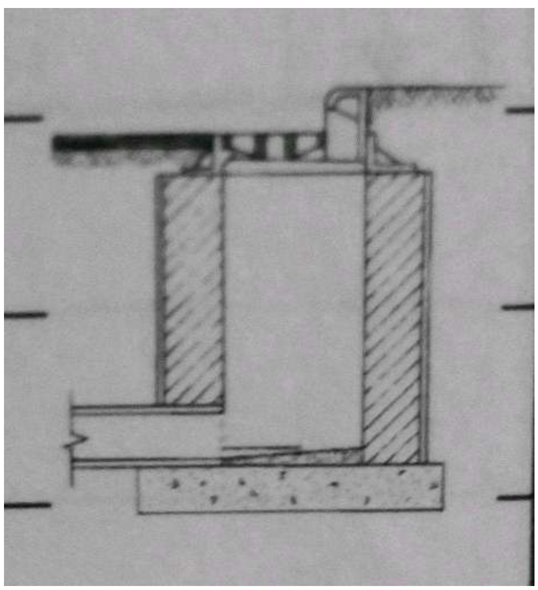


Figure 57. Parking Lot Inlets

Retention Area A Dimensions		
depth=	4	ft
width=	10	ft
length=	403	ft

* Retention A is to the left of the building

Retention Area B Dimensions		
depth=	4	ft
width=	10	ft
length=	403	ft

* Retention B is to the right of the building

Retention Area C Dimensions		
depth=	4	ft
width=	40	ft
length=	603	ft

* Retention A is behind the building

Figure 58. Retention Areas Dimensions

APPENDIX F. COST ESTIMATES

APPENDIX F.1 Environmental Cost Estimate

Potable Water Supply							
Material	Spec. Type	Length (ft)	QTY	Unit	Total	Total Cost	Local Adjustment
Trench Excavating & Backfill	16"W by 24" Deep	6748	1-4' deep	C.Y.	$ 1.3	$ 28,000	$ 27,000
Ductile Iron Water Supply	12" Mechanical Joint	6748	18	L.F.	$ 119.0	$ 44,000	$ 43,000
Elbows (90)	12"		4	Ea.	$ 1,300.0	$ 5,000	$ 5,000
Tee's	12"		3	Ea.	$ 2,250.0	$ 7,000	$ 65,000
Butterfly Valves	12"		6	Ea.	$ 2,200.0	$ 13,000	$ 13,000
					Total	$ 98,000	$ 93,000

APPENDIX F.2 Structural Cost Estimate

Table 13. Assemblies Cost Data

Assemblies Cost Data							
			Cost				
System Line	Quantity	Unit	Mat.	Inst.	Total	Total $ 10% O&P	Total $
B3010 130 0900	2907	S.F.	$0.85	$1.45	6686.10	$7,354.71	$7,350.00
B2010 110 3250	2304	S.F.	$1.33	$4.33	13040.64	$14,344.70	$14,300.00
B2020 220 1000	1567	S.F.	$7.55	$8.15	24601.90	$27,062.09	$27,100.00

Table 14. Building Construction Cost Data

Building Construction Cost Data								
			Cost					
System Line	Quantity	Unit	Mat.	Labor	Equip	Total Incl O&P	Total	Total $
05120 640 0100	364	L.F.	$6.30	$3.36	2.36	$15.30	$5,569.2	$5,575.00
05120 640 2340	180	L.F.	$37.00	$2.52	1.77	$47.50	$8,550.00	$8,550.00
			Material $/lb	Labor $/lb	Equip $/lb	Total $/lb		
Online lookup	84520	lb	$1.25	$0.24	0.13	$1.62	$136,922.40	$137,000.00
05100 560 2200	12	Cwt	$35.50			$41.50	$498.00	$500.00
05090 420 0200	80	Cwt	$0.64	$2.48		$5.15	$412.00	$410.00
05090 420 0365	495	Cwt	$1.10	$2.70		$6.05	$2,994.75	$3,000.00

Table 15. Line Descriptions

System Line	Description
B3010 130 0900	Preformed Metal Roofing - Steel, Galvanized 29 ga.
B2010 110 3250	Liteblock - Closest Cost in Assembly Book is …
B2020 220 1000	Exterior Glass Curtain Walls
05120 640 0100	Roof Beams W6x9
05120 640 2340	Columns are W14x48…book only has W14x53
Online lookup	C15x50
05100 560 2200	3/8" Plates
05090 420 0200	3/4" Bolts 2" long
05090 420 0365	7/8" Bolts 3" long

Table 16. Total Estimated Building Costs

Total Cost	
Without City Index	$204,000.00
With MEM Index	$176,500.00

APPENDIX F.3 Geotechnical Cost Estimate

Table 17. Geotechnical Estimated Cost Data

Site Surveys	Crew	Daily Output	Labor Hours	Unit	Material	Labor	Equipment	Total	Total Incl O&P	Unit Total	Total Cost
Topographical surveying	A-7	3.3	7.273	Acre	20	375	16.6	411.6	615	8	$ 4,920

Geotechnical Investigations	Crew	Daily Output	Labor Hours	Unit	Material	Labor	Equipment	Total	Total Incl O&P	Unit Total	Total Cost
Borings, initial field stake out &determination of elevations	A-6	1	16	day		750	55	805	1200	1	1200
Drawings showing boring details				day		335		335	425	1	425
mobilization and demobilization	B-55	4	6	day		229	271	500	650	1	650
Case borings 2-1/4" diameter	B-56	55.5	0.432	LF	14	16.5	19.5	50	62	80	4960
											$ 7,235

Foundation	Crew	Daily Output	Labor Hours	Unit	Material	Labor	Equipment	Total	Total Incl O&P	Unit Total	Total Cost
3/4 Stone Drainage Layer				cy	34			34	37.5	121	4538
Water proofing				cy	12.7			12.7	14	115	1610
Forms in place footings	C-1	350	0.091	LF	0.34	4.08		4.42	6.65	218	1450
Welded Wire Reinforcement	2 Rodm	27	0.593	CSF	31.5	31		62.5	83	85	7055
Beam Reienforcing labor	4 Rodm	3	10.667	ton	970	560		1530	1950	0.5966	1163
#3 Rebar					152			152	167	0.084224	14
#4 Rebar					76			76	83.5	0.291248	24
#5 Rebar					38			38	42	0.221116	9
Stirrups					152			152	167	0.140436	23
concrete	C-14A	35.87	5.799	CY	216	273	21	510	680	48.75	33150
saw cut control joints 1"	C-27	2000	0.008	LF	0.04	0.36	0.08	0.48	0.66	109	72
anchor bolts for collums	1 carp	24	0.333	Ea.	18.6	15.65		34.25	44.5	40	1780
											$ 50,889

Earth Work	Crew	Daily Output	Labor Hours	Unit	Material	Labor	Equipment	Total	Total Incl O&P	Unit Total	Total Cost
Site Clearing	B-11A	1.5	10.667	Acre		470	925	1395	1750	8	14000
Top Soil Stripping	B-10B	2300	0.005	Cy		0.24	0.6	0.84	1.03	54.95	56.5985
Excavating/Trenching	B-11C	150	0.107	BCY		4.7	2.43	7.13	9.85	80.95	797.3575
Pre compaction 242 CY of soil includes	B-10M	735	0.016	LCY		0.76	1.89	2.65	3.23	242	781.66
hauling cost				LCY	15			15	18	242	8712
Pre compaction soil removal	B-10M	735	0.016	LCY		0.76	1.89	2.65	3.23	242	781.66
											$ 25,129
										Total Cost :	$ 88,173

APPENDIX F.4 Transportation Cost Estimate

Road	Length (ft)	Left Side of Pavement	Driveway (ft)	Right Side of Pavement
Entrance Ramp	698	6ft shoulder	16	8ft shoulder
Road 1.1	398	6in. Curb and Gutter	22	6in. Curb and Gutter
Road 1.2	203	6in. Curb and Gutter	22	6in. Curb and Gutter
Car Parking 1	407.2	Not Applicable	22	6in. Curb and Gutter
Car Parking 2	602.1	Not Applicable	22	6in. Curb and Gutter
Car Parking 3	407.2	Not Applicable	22	6in. Curb and Gutter
Road 2.1	94	6in. Curb and Gutter	22	6in. Curb and Gutter
Road 2.2	94	6in. Curb and Gutter	22	6in. Curb and Gutter
Road 3.1	120	6in. Curb and Gutter	22	6in. Curb and Gutter
Road 3.2	150	6in. Curb and Gutter	22	6in. Curb and Gutter
Exit Ramp 1	375	6in. Curb and Gutter	16	6in. Curb and Gutter
Exit Ramp 2	576	8ft shoulder	16	6ft shoulder
Exit Ramp 3	1600	Not Applicable	12	6ft shoulder

Pavement Cost								
Material	Lift Thickness	# Lifts	QTY	Unit	Total	Total Cost	Local Adjustment	Area of Road (S.Y.)
Asphalt Surface Course	1.5"	1	17627	S.Y.	$ 8.60	$ 152,000	$ 145,000	17627
Tac Coat		2	17627	S.Y.	$ 0.58	$ 21,000	$ 20,000	
Asphalt Base	4"	2	17627	S.Y.	$ 19.68	$ 694,000	$ 664,000	
Aggregate Base	4"-6"	1	17627	S.Y.	$ 6.20	$ 109,000	$ 105,000	
					Total	$ 976,000	$ 934,000	

APPENDIX F.5 Water Resources Cost Estimate

Earthwork Cost Data										
						Cost				
System Line	Description	Daily Output	Unit	Mat	Labor Hrs	Labor	Equip	Total (Inc O&P)	Total $	Total $
2000040	Hauling	100	CY	26968.89	0.08	$ 1.92	$ 3.70	$ 7.00	$ 5.62	$ 188,782.22
3007600	Compaction	840	CY	26968.89	0.14	$ 0.40	$ 0.13	$ 0.76	$ 0.53	$ 20,496.36
4002420	Excevation	100	CY	39973.81	0.12	$ 3.34	$ 3.99	$ 9.50	$ 7.33	$ 379,751.15
5050010	Backfill	1000	CY	338.25	0.12	$ 0.33	$ 0.74	$ 1.32	$ 1.07	$ 446.49

Pipe and Drain Cost Data										
						Cost				
System Line	Description	Daily Output	Unit	Mat	Labor Hrs	Labor	Equip	Total (Inc O&P)	Total $	Total $
1002100	Corrugated Metal	200	LF	10.25 (9)	0.24	$ 5.80	$ 0.90	$ 21.50	$ 16.95	$ 193.50
2001700	Catchbasin Precast	10	EA	114 (9)	2.4	$ 59.50	$ 18.10	$ 238.00	$ 191.60	$ 2,142.00

Total Estimated Building Costs (Including O&P Cost)	
Without City Index	$ 591,811.72
With MEM City Index	$ 554,527.58

APPENDIX F.6 Total Cost Estimate

Table 18. Total Project Cost

Total Cost	
Environmental	$93,000.00
Structural	$176,500.00
Geotechnical	$88,000.00
Transportation	$934,000.00
Water Resources	$555,000.00
Design	$69,500.00
Total Project Cost	$1,916,000.00

APPENDIX G. PROJECT MANAGEMENT

APPENDIX G.1 Timesheet

901 Design			901 Design						
			Final Cost						
			Name						
	Date	Day	Huan Hoang Ngo	Mark Anthony Rippy	Kendall Lee Brown	Stephen Carl Thusius	Jana Marie East Moss	Project Total	Total Cost
PROJECT TOTAL	Week 1		0	0	0	4.5	0	4.5	$ 450.00
	Week 2		0	0	0	5	0	5	$ 500.00
	Week 3		0	0	7	13	0	20	$ 2,000.00
	Week 4		14	13	7	17.5	11	62.5	$ 6,250.00
	Week 5		16	17	12.5	18	16	79.5	$ 7,950.00
	Week 6		16	13	11	11	9	60	$ 6,000.00
	Week 7		16	21	15.5	19.5	16	88	$ 8,800.00
PROJECT TOTAL	Week 8		10	13.5	12	8	15	58.5	$ 5,850.00
	Week 9		14	11	12.5	17	12	66.5	$ 6,650.00
	Week 10		17	0	12.5	16.5	11	57	$ 5,700.00
	Week 11		22	8	21.5	17	16	84.5	$ 8,450.00
	Week 12		12	20	12	12	12	68	$ 6,800.00
	Week 13		8	8	8	8	8	40	$ 4,000.00
	Total		145	124.5	131.5	167	126	694	$ 69,400.00

Figure 59. Final Design Hours and Cost